UPDATED REPORT OF GEOTECHNICAL INVESTIGATION SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT CITY OF SAN DIEGO

Submitted to:

RICK ENGINEERING COMPANY
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Prepared By:

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AGE Project No. 190 GS-18-D

March 8, 2019
(Updated August 15, 2019)

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Mr. Kevin Gibson, P.E.
Project Manager
Rick Engineering Company
5620 Friars Road
San Diego, CA

Subject: **UPDATED REPORT OF GEOTECHNICAL INVESTIGATION**
SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT
CITY OF SAN DIEGO
AGE Project No. 190 GS-18-D

Dear Mr. Gibson:

Allied Geotechnical Engineers, Inc. is pleased to submit the accompanying updated report to present the findings, opinions, and recommendations of a geotechnical investigation that was performed to assist Rick Engineering Company with their design of the subject project.

We appreciate the opportunity to be of service on this project. If you have any questions regarding the contents of this report or need further assistance, please feel free to contact our office.

Sincerely,

ALLIED GEOTECHNICAL ENGINEERS, INC.

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SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT
CITY OF SAN DIEGO

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1.0 INTRODUCTION

Allied Geotechnical Engineers, Inc. (AGE) is pleased to submit this report to present the findings, opinions, and recommendations of a geotechnical investigation conducted to assist Rick Engineering Company (Rick Engineering) with their design of the South Mission Beach Green Infrastructure Project for the City of San Diego (City). The investigation was performed in conformance with AGE's proposal dated July 11, 2018 (revised July 24, 2018), and the subconsultant agreement entered into by and between Rick Engineering and AGE on November 1, 2018. This report has been updated to incorporate the results of a subsurface geotechnical investigation performed by Southern California Soil and Testing, Inc. (SCS&T), dated April 16, 2019 and to provide additional recommendations to mitigate the groundwater bouyant uplift forces on the proposed storm drain pipelines.

This report has been prepared for the exclusive use of Rick Engineering and its design team and the City in their design of the project as described herein. The information presented in this report is not sufficient for any other uses or the purposes of other parties.

2.0 SITE AND PROJECT DESCRIPTION

The project alignments Map is shown on Figure 1. Based on a review of the 60% submittal plans prepared by Rick Engineering Company, undated, it is our understanding that the scope of the proposed project will include the following:

- design and construction of approximately 88 feet of concrete lined channel;
- design and construction of approximately 6,253 feet of storm drain pipelines;
- design and construction of approximately 142 feet of encased storm drain;
- design and construction of 16 feet of culvert; and
- design and construction of associated headwalls, inlets, connectors, cleanouts, outlets, tidegates and weep sumps.

The proposed project alignments extend along public right-of-ways in the South Mission Beach area of San Diego. The proposed pipelines will consist of 18-, 30-, 36- and 48-inch diameter reinforced concrete pipes (RCP). It is anticipated that the proposed pipelines will be installed using conventional cut-and-cover construction method with cover thickness on the order of 2 to 13 feet above the pipe crown.

Existing improvements along the project alignments include a mix of residential and commercial developments as well as Mission Beach and Mission Bay Park. The topography along the project alignments varies from level to very gently sloping with elevations which vary from sea level to approximately 13 feet above mean sea level (msl).

3.0 OBJECTIVE AND SCOPE OF INVESTIGATION

The objectives of this investigation were to characterize the subsurface conditions along the project alignments and to develop geotechnical recommendations for use in the design of the currently proposed project. The scope of our investigation included several tasks which are described in more detail in the following sections.

3.1 Information Review

This task involved a review of readily available information pertaining to the project study area, including the preliminary project plans, as-built utility maps, topographic maps, published geologic literature and maps, and AGE's in-house references. In addition AGE also perform a review of a report of Geotechnical Investigation prepared by Souther California Soil & Testing for the Mission Beach Water and Sewer Replacement City of San Diego Task #17CD03, dated April 16, 2019.

3.2 Geotechnical Field Exploration

The field exploration program for this project was performed on February 11 and 12, 2019. A total of four (4) soil borings, four (4) infiltration test holes, and two (2) pavement corings were performed at the approximate locations shown on Figures 2 through 5. In addition, AGE attempted to perform infiltration testing inside an existing weep sump located on the west side of Mission Boulevard, at the entrance of an alley located between Brighton Court and Capistrano Place. The soil borings were advanced to depths ranging from 15 feet to 16.5 feet below the existing ground surface (bgs). The infiltration test holes were hand-augured to depths ranging from 36 inches to 63 inches bgs. A brief description of the location and depth, pavement sections, groundwater level, and subsurface conditions encountered in the borings and infiltration test holes is presented in Table 1 on the next page. A more detailed description of the excavation and sampling activities, and logs of the soil borings are presented in Appendix A.

Table 1

Summary of Subsurface Conditions

Boring & Test Hole ID	Location	Depth (Feet)	Existing Pavement Section	Subsurface Conditions	Estimated Groundwater Depth/ Elevation (Feet bgs/feet msl)
B-1	Mission Bay beach, approximately 10 feet east of Bayside Walk at intersection with San Fernando Place.	16.5	N/A	Hydraulic fill to 10 feet and old paralic deposits to the maximum depth of exploration.	11/-3.7
B-2	Southbound Mission Boulevard, approximately 40 feet south of San Fernando Place and 4 feet west of the center median.	15	4" A.C. over 8" P.C.C. underlain by 6" miscellaneous base.	Old paralic deposits to the maximum depth of exploration.	4.25/+2.0
C-2	Southbound Mission Boulevard, approximately 40 feet south of San Fernando Place and 12 feet east of the curb.	N/A	4.5" A.C. over 9.5" P.C.C. Unable to differentiate base materials.	N/A	N/A
B-3	Mission Bay beach, approximately 20 feet east of Bayside Walk at intersection with Coronado Court.	16.5	N/A	Hydraulic fill to 10 feet and old paralic deposits to the maximum depth of exploration.	4/+1.2
B-4	Southbound Mission Boulevard, approximately 60 feet south of Brighton Court and 4 feet west of the center median.	15	4.5" A.C. over 7.5" P.C.C. underlain by 4" miscellaneous base.	Old paralic deposits to the maximum depth of exploration.	3.25/+1.75

Table 1 (continued)

Summary of Subsurface Conditions

Boring & Test Hole ID	Location	Depth (Feet)	Existing Pavement Section	Subsurface Conditions	Estimated Groundwater Depth/ Elevation (Feet bgs/feet msl)
C-4	Northbound Mission Boulevard, approximately 60 feet south of Brighton Court and 12 feet west of the curb.	N/A	6" A.C., 6" P.C.C., 2" miscellaneous base.	N/A	N/A
P-1	Lawn area approximately 30 feet east of Mission Boulevard and 240 feet north of San Fernando Place.	62"	N/A	Four inches of topsoil underlain by old paralic deposits to the maximum depth of exploration.	3'/+2.7'
P-2	Tree planter on east side of Mission Boulevard approximately 20 feet north of Deal Court.	36"	N/A	Twelve inches of topsoil underlain by old paralic deposits to the maximum depth of exploration.	3'/+1.9'
P-3	Tree planter on west side of Mission Boulevard approximately 15 feet north of Balboa Court.	48"	N/A	Twelve inches of topsoil underlain by old paralic deposits to the maximum depth of exploration.	3'/+1.5'
P-4	Lawn area approximately330 feet east of Mission Boulevard and 10 feet south of Bayside Lane.	63"	N/A	Three inches of topsoil underlain by old paralic deposits to the maximum depth of exploration.	Not encountered.

Prior to commencement of the field exploration activities, several site reconnaissance visits were performed to observe existing conditions and to select suitable locations for the soil borings and infiltration test holes. Subsequently, Underground Service Alert (USA) was contacted to coordinate clearance of the proposed boring and test hole locations with respect to existing buried utilities. The utility clearance effort revealed the presence of the following buried utilities: potable water and sanitary sewer pipelines; storm drains; natural gas and electrical transmission lines; and cable, telephone, and fiber optic lines.

Traffic control permits were obtained from the City of San Diego to perform the borings (B-2 and B-4) and pavement cores (C-2 and C-4) that are located within the public right-of-way. Borings B-1 and B-3, and percolation holes P-1 and P-4 which are located in Mission Bay Park were performed with prior verbal approval from the City of San Diego Parks & Recreation Department.

Due to the presence of shallow groundwater inside test holes P-1, P-2 and P-3, AGE was unable to perform infiltration testing inside these holes. The existing weep sump was installed on top of an existing City of San Diego sewer trench. Furthermore, when AGE attempted to perform the infiltration testing, AGE uncovered an 18-inch diameter green PVC pipe filled with 3/4-inch crushed rock (see photograph in Figure 6). Therefore, AGE was unable to perform infiltration testing inside the weep sump. Infiltration testing was only performed in test hole P-4.

3.3 Laboratory Testing

Selected soil samples obtained from the soil borings were tested in the laboratory to verify field classifications and evaluate certain engineering characteristics. The geotechnical laboratory tests were performed in general conformance with the American Society for Testing and Materials (ASTM) or other generally accepted testing procedures.

The laboratory tests included: in-place density and moisture content, maximum density and optimum moisture content, sieve (wash) analysis, consolidation, shear strength, and R-value. In addition, representative samples of the onsite soil materials were collected and delivered to Clarkson Laboratories and Supply, Inc. for chemical (analytical) testing to determine soil pH and resistivity, soluble sulfate and chloride concentrations, and bicarbonate content.

4.0 GEOLOGIC CONDITIONS

4.1 Geologic Setting and Site Physiography

The project alignments are located in Mission Beach, a narrow sandbar situated between the Pacific Ocean and Mission Bay. The sandbar is underlain by marine sediments which range from Pleistocene to Holocene in age. Hydraulically placed fill materials were added along the eastern and southern portions of the sandbar during development of Mission Bay from the 1940's into the 1950's. Shallow mechanically placed fill materials were also encountered in the study area.

4.2 Tectonic Setting

Tectonically, the San Diego region is situated in a broad zone of northwest-trending, predominantly right-slip faults that span the width of the Peninsular Ranges and extend offshore into the California Continental Borderland Province west of California and northern Baja California. At the latitude of San Diego, this zone extends from the San Clemente fault zone, located approximately 60 miles to the west, and the San Andreas fault located about 95 miles to the east.

Major active regional faults of tectonic significance include the Coronado Bank, San Diego Trough, San Clemente, and Newport Inglewood/Rose Canyon fault zones which are located offshore; the faults in Baja California, including the San Miguel-Vallecitos and Agua Blanca fault zones; and the faults located further to the east in Imperial Valley which include the Elsinore, San Jacinto and San Andreas fault zones.

4.3 Geologic Units

Based on their origin and compositional characteristics, the soil types encountered in the borings can be categorized into two geologic units which include (in order of increasing age) fill materials and old paralic deposits. A brief description of each unit is presented below.

4.3.1 Fill Materials

Hydraulically placed fill materials were encountered in borings B-1 and B-3 to depths of approximately 10 feet bgs. The hydraulic fill generally consists of fine to medium grained sand with silt and containing scattered sub-rounded gravel. During the field investigation we met refusal in boring B-3 on a large buried rock or concrete at a depth of 3 feet bgs. We moved approximately 10 feet to the east and re-drilled to the target depth.

Mechanically placed fill materials on the order of 12 inches or less in thickness were encountered in infiltration test holes P-1 thru P-4. These materials generally consist of silty sands and organic-rich topsoil for lawns and street trees. Documentation pertaining to the original placement of the fill materials is unavailable.

Fill materials were also encountered in SCS&T (2019) borings P-1 through P-3 which are located along Mission Boulevard to depth ranging between 2 to 5 feet bgs. Fill materials were encountered in SCS&T (2019) boring P-4 to the maximum depth of exploration of 21.5 feet bgs. The fill materials encountered in SCS&T (2019) borings posses the same consistency as those encountered in AGE's borings.

4.3.2 Old Paralic Deposits

Late to mid-Pleistocene age old paralic deposits (Kennedy and Tan, 2008) were encountered below fill materials in borings B-1 and B-3, and below paving in borings B-2 and B-4 to the maximum depth of exploration. These deposits are generally described as poorly sorted, moderately permeable, reddish brown interfingered strandline, beach, estuarine and colluvial deposits composed of siltstone, sandstone and conglomerate resting on a now emergent wave-cut platform preserved by regional uplift (Kennedy and Tan, 2008). The deposits can generally be excavated with conventional heavy duty construction equipment. Although not encountered during the field exploration, localized conglomerate layers may present difficult excavation conditions.

The old paralic deposits encountered in our test borings generally consisted of fine-to medium grained sands and silty sands with scattered to trace amounts of sub-rounded gravel and shell fragments. The soil deposits are generally uncemented, damp to wet, and in a medium dense to dense condition.

Old paralic deposits were encountered below the fill materials in SCS&T (2019) borings P-1 and P-3 to the maximum depth of exploration which ranges between 20.5 and 21 feet bgs. The old paralic deposits encountered in SCS&T (2019) borings posses the same consistency as those encountered in AGE's borings.

4.4 Groundwater

At the time of our field investigation, groundwater was measured in the soil borings and test holes at depths ranging from 3 feet to 11 feet bgs (approximate elevations -2 feet to +7 feet msl). Tidal coefficients in Mission Bay (Quivira Basin) on the days of the field exploration based on National Oceanic and Atmospheric Administration (NOAA) data are shown on the next page.

Date	Low Tide		High Tide	
	Time	Height (MLLW) Height (MSL)	Time	Height (MLLW) Height (MSL)
02/11/2019	7:34 am	-1.7 feet -4.5 feet	1:03 pm	+3.0 feet + 0.2 feet
02/12/2019	9:15 am	-1.5 feet -4.3 feet	3:07 pm	+2.6 feet +0.2 feet
02/13/2019	10:47 am	-0.9 feet -3.7 feet	5:18 pm	+2.7 feet -0.1 feet

No groundwater was encountered in infiltration hole P-4. Fill and formational materials encountered in the soil borings and infiltration test holes are generally considered to possess very high permeability characteristics. Based on the anticipated depth of excavations, it is anticipated that groundwater will be encountered along the project alignments during construction.

Groundwater was encountered in borings SCS&T (2019) borings between elevations + 1 and +2 feet msl. Monitoring with vibrating wire piezometer between March 5 and April 9l 2019 inside the borings indicate groundwater level fluctuation on the order of 0.5 to 0.75 foot.

5.0 DISCUSSIONS, OPINIONS AND RECOMMENDATIONS

5.1 Potential Geologic Hazards

The majority of the project study area is classified in the City of San Diego Seismic Safety Study (2008), as Hazard Category 52 - Other Terrain, defined as, "Other level areas, gently sloping to steep terrain, favorable geologic structure, Low Risk". The beach area in the eastern portion of the study area, as well as Mission Point Park in the southeast portion of the study area adjacent to the Mission Bay Channel is classified as Hazard Category 31 - Liquefaction, defined as, "High potential-Shallow groundwater, major drainages, hydraulic fills. Neither classifications are anticipated to affect the proposed project as described herein.

5.1.1 Faulting

The northwest trending Point Loma fault is mapped 2,000 feet east of the project study area (Kennedy, 1975; Kennedy and Tan, 2008), This fault is concealed below Mission Bay and Holocene age fill materials east of the project study area. To the southeast the mapped trace of the fault crosses the Point Loma peninsula, where it is concealed beneath Pleistocene age old paralic deposits. The Point Loma fault is classified in the City of San Diego Seismic Safety Study (2008) as "potentially active, inactive, presumed inactive, or activity unknown."

For the purpose of this project we consider the Rose Canyon fault zone (RCFZ) to represent the most significant seismic hazard. The RCFZ is a complex set of anastomosing and en-echelon, predominantly strike slip faults that extend from off the coast near Carlsbad to offshore south of downtown San Diego (Treiman, 1993). Previous geologic investigations on the RCFZ in the Rose

Creek area (Rockwell et. al., 1991) and in downtown San Diego (Patterson et. al., 1986) found evidence of multiple Holocene earthquakes. Based on these studies, several fault strands within the RCFZ have been classified as active faults, and are included in Alquist-Priolo Special Studies Zones. In San Diego Bay, this fault zone is believed to splay into multiple, subparallel strands; the most pronounced of which are the Silver Strand, Spanish Bight and Coronado Bank faults.

A study by Kleinfelder (2017) at the San Diego International Airport identified two zones of active faulting. One of these faults was named the East Bay fault and the second fault was determined to be a northward extension of the Spanish Bight fault. Recent study by Ninyo & Moore (2018) at Seaport Village found evidence of recent movement along a fault that was determined to be a northward extension of the active Coronado fault. The project alignments are not located within an Alquist-Priolo Earthquake Study Zone.

5.1.2 Fault Ground Rupture & Ground Lurching

There are no known (mapped) active or potentially active faults crossing the project alignments (Kennedy, 1975; Kennedy and Tan, 2008; City of San Diego, 2008). Therefore, the potential for fault ground rupture and ground lurching along the project alignments is considered insignificant.

5.1.3 Soil Liquefaction

Seismically-induced soil liquefaction is a phenomenon in which loose to medium dense, saturated granular materials undergo matrix rearrangement, develop high pore water pressure, and lose shear strength due to cyclic ground vibrations induced by earthquakes.

Hydraulically placed fill materials in the east and southerly portions of the project alignments are classified in the City of San Diego Seismic Safety Study (2008) as having a high liquefaction potential. The findings of our investigation determined that the hydraulic fill materials encountered in borings B-1 and B-3 are in a medium dense condition, and therefore are considered to have a low liquefaction potential. However, it is likely that liquefaction prone soil materials will be encountered during construction.

5.1.4 Landslides

A review of the published geologic maps indicates that there are no known (mapped) ancient landslides in the project study area (Kennedy, 1975; Kennedy and Tan, 2008; City of San Diego, 2008). Therefore, landsliding is not considered a significant risk.

5.1.5 Lateral Spread Displacement

The project alignments are located in an area that is flat, therefore, the risk of lateral spread displacement during a seismic event is considered remote.

5.1.6 Differential Seismic-Induced Settlement

Differential seismic settlement occurs when seismic shaking causes one type of soil to settle more than another type. It may also occur within a soil deposit with largely homogeneous properties if the seismic shaking is uneven due to variable geometry or thickness of the soil deposit. Based on the results of our investigation, it is our opinion that there is a slight potential of differential settlement in areas underlain by deep hydraulically placed man-made fills.

5.1.7 Secondary Hazards

The project alignments are located within the tsunami inundation zone (California Geological Survey, 2009). Therefore, there is a high potential of property damage from seismic-induced tsunamis. The project alignments are located within the Special Flood Hazard Areas, 100- and 500-year flood zone (FEMA Flood Insurance Rate Map, 2012). Therefore the potential for flooding along the project alignments is considered high to very high.

5.2 Soil Corrosivity

In accordance with the City of San Diego Water Facility Design Guidelines, Book 2, Chapter 7, soil is generally considered aggressive to concrete if its chloride concentration is greater than 300 parts per million (ppm) or sulfate concentration is greater than 1,000 ppm, or if the pH is 5.5 or less.

Analytical testing was performed on representative sample of the onsite soil materials to determine pH, resistivity, soluble sulfate, chlorides and bicarbonates content. The tests were performed in accordance with California Test Method Nos. 643, 417 and 422. A summary of the test results is presented in Table 2 below. Copies of the analytical laboratory test data reports are included in Appendix B.

Table 2

Summary of Corrosivity Test Results

	pH	Resistivity (ohm-cm)	Sulfate Conc. (ppm)	Chloride Conc. (ppm)	Bicarbonates Conc. (ppm)
B-1 Sample No. 4 @14'-15'	8.3	130	1,050	3,630	46
B-2 Sample No. 3 @8'-9'	9.3	3,200	70	50	66
B-3 Sample No.3 @9'-10'	9.3	7,700	30	30	66
B-4 Sample No. 4 @10'-11'	9.2	730	140	620	46

The test results indicate that some of the soils along the project alignments are considered aggressive to concrete. Therefore, Type 5 Portland Cement Concrete should be used for proposed facilities along the project alignments. It should be noted here that the most effective way to prevent sulfate attack is to keep the sulfate ions from entering the concrete in the first place. This can be done by using mix designs that give a low permeability (mainly by keeping the water/cement ratio low) and, if practical, by placing moisture barriers between the concrete and the soil.

AGE does not practice in the field of corrosion engineering. In the event that corrosion sensitive facilities are planned, we recommend that a corrosion engineer be retained to perform the necessary corrosion protection evaluation and design.

5.3 Expansive Soil

Based on visual observations and soil classifications, the soil materials encountered in the borings and test holes are considered to be non-expansive.

5.4 Fill Material

Fill material for trench backfill should be free of biodegradable material, hazardous substance contamination, other deleterious debris, and or rocks or hard lumps greater than 6 inches. If the fill material contains rocks or hard lumps, at least 70 percent (by weight) of its particles shall pass a U.S. Standard $^{3}/_{4}$-inch sieve. Fill material should consists of predominantly granular soil (less than 40 percent passing the U.S. Standard #200 sieve) with Expansion Index of less than 50.

The majority of the onsite soil materials are considered suitable for use as compacted backfill materials. It is noted that since the majority of the excavations will extend below the groundwater level, the majority of the soil materials generated from excavations along the project alignments will be wet, and will require drying prior to use as trench backfill materials.

5.5 Cut-and-Cover Construction

Since no changes to the existing ground surface along the cut-and-cover segment of the proposed storm drain pipeline alignment are planned, the net stress change in the underlying soils is considered negligible. Furthermore, the soils at the proposed invert level along the storm drain pipeline alignment are expected to provide a stable trench bottom. In the event that loose or disturbed soils are encountered at the trench bottom, it is recommended that they be over-excavated and replaced with pipe bedding or other approved materials. The depth of the overexcavation should be determined during construction by the City's Resident Engineer.

5.5.1 Soil and Excavation Characteristics

The materials within the anticipated depths of the storm drain pipe trench excavation will likely be comprised of materials which can be readily excavated with conventional heavy-duty construction equipment.

5.5.2 Pipe Loads and Settlement

Pipes should be designed for all loads applied by surrounding soils including dead load from soils, loads applied at the ground surface, uplift loads, and earthquake loads. Soil loading above and below the groundwater level may be estimated assuming a density of 100 pcf and 130 pcf, respectively, for properly compacted backfill materials.

Where a pipe changes direction abruptly, resistance to thrust forces can be provided by means of thrust blocks. For design purposes, for the passive resistance against thrust blocks embedded in dense formational material and/or properly compacted filled ground, an equivalent fluid density of 200 pcf may be used. Thrust blocks should be embedded a minimum of 3 feet beneath the ground surface.

Buried flexible pipes are generally designed to limit deflections caused by applied loads. The deflections can be estimated using the Modified Spangler equation. A modulus of soil reaction, E', equal to 1,000 and 2,000 psi may be used to represent a minimum of 6 inches of compacted pipe bedding materials of low plasticity (LL < 50) with less than 12 percent fines passing the #200 standard sieve and crushed rock materials, respectively.

5.5.3 Trench Backfill

Pipe Bedding Zone and Pipe Zone

"Pipe Bedding Zone" is defined as the area below the bottom of the pipe and extending over the full trench width, and should be at least 6 inches thick in order to provide a uniform firm foundation material directly beneath the pipe.

The "Pipe Zone" is defined as the full width of a trench from the bottom of the pipe to a horizontal level about 6 inches above the top (crown) of the pipe. In order to provide uniform support and to minimize external loads, trench widths should be selected such that a minimum clear space of 6 inches is provided on each side of the pipe. During backfilling, it is recommended that the backfill materials be placed on each side of the pipe simultaneously to avoid unbalanced loads on the pipe.

Backfill materials placed in the "Pipe Bedding Zone" and "Pipe Zone" should consist of clean, free draining sand or crushed rock. Sand should be free of clay, organic matter, and other deleterious materials and conform to the gradation shown in the following table.

Sieve Size	Percent Passing by Weight (percent)
½ inch	100
#4	75-100
#16	35-75
#50	10-40
#200	0-10

Crushed rock should conform to Section 200-1.2 and 200-1.3 of the Standard Specifications for Public Works Construction (SSPWC) for 3/4-inch crushed rock gradation. It must be noted that, since the native soil materials do not meet these specifications, import backfill materials will be required for the "Pipe Bedding Zone" and "Pipe Zone". If crushed rock is to be used for pipe zone and bedding backfill materials, we recommend that the rock materials be wrapped in geotextile filter fabric such as Mirafi 140N or equivalent. The purpose of the filter fabric is to prevent migration of fine grained materials from the backfill materials, and the sides and bottom of the trench into the rock bedding materials.

Above Pipe Zone

The "Above Pipe Zone" is defined as the full width of the trench from the top of the "Pipe Zone" to the finish grade or bottom of the pavement section. Backfill material placed in this zone should meet or exceed the criteria presented in Section 5.4. for either flowable fill or soil backfill.

5.5.4 Placement and Compaction of Backfill

Prior to placement, all soil backfill material should be moisture-conditioned, spread and placed in lifts (layers) not-to-exceed 6 inches in loose (uncompacted) thickness, and uniformly compacted to at least 90 percent relative compaction. During backfilling, the soil moisture content should be maintained at or within 2 to 3 percent above the optimum moisture content of the backfill materials. The maximum dry density and optimum moisture content of the backfill materials should be determined in the laboratory in accordance with the ASTM D1557 testing procedures. Field density testing shall be performed in accordance with either the Sand Cone Method (ASTM D1556) or the Nuclear Gauge Method (ASTM D2922 and D3017).

Small hand-operated compacting equipment should be used for compaction of the backfill materials to an elevation of at least 4 feet above the top (crown) of the pipes. Flooding or jetting should not be used to densify the backfill.

5.5.5 Groundwater Bouyant Uplift

AGE has performed a buoyancy analysis for 18-, 30-, 36- and 48-inch diameter RCP pipes which are proposed for the subject project. The analysis is included in Appendix C. The results indicate that the RCP pipes installed as recommended in this section with minimum 24 inches of cover are not expected to float. It is our opinion that no additional mitigation measures are required.

5.6 Buried Structures

It is recommended that any proposed buried structures be founded on firm native soils or approved compacted materials. In areas where loose or soft soils are encountered at the bottom of any manhole/box structure excavations, it is recommended that the loose/soft materials be removed and replaced with 3/4-inch crushed rock materials wrapped in geotextile fabric which meets or exceeds the specifications shown below.

Fabric Property	Min. Certified Values	Test Method
Grab Tensile Strength	300 lb	ASTM D 4632
Grab Tensile Elongation	35% (MAX)	ASTM D 4632
Burst Strength	600 psi	ASTM D 3786
Trapezoid Tear Strength	120 lb	ASTM D 4533
Puncture Strength	130 lb	ASTM D 4833

The actual extent of over-excavation of any loose/soft soil materials should be evaluated and determined in the field by the City's Resident Engineer.

5.6.1 Placement and Compaction of Backfill

Placement and compaction of backfill materials around the buried structures should be performed in accordance with the recommendations presented in Section 5.5.4 of this report.

5.6.2 Foundations

Bearing Capacity

For design of the buried structures which are founded on firm native soils an allowable soil bearing capacity of 2,000 psf may be used. In the event that loose and compressible soils are encountered at the bottom of the excavation for the proposed structures, we recommend that the structures be supported on a minimum of 24 inches of 3/4-inch crushed rock wrapped in geofabric. This allowable soil bearing value is for total dead and live loads, and may be increased by one third when considering seismic loads.

Anticipated Settlement

Under static condition, total settlement of the slab foundation is estimated to be less than 0.25 inch. Differential settlement between the center and the edge of the slab foundation is expected not to exceed 0.25 inch. No permanent deformation and/or post-construction settlement is anticipated, provided that backfill around the structures is properly compacted in accordance with the project specifications.

Resistance to Lateral Loads

Resistance to lateral loads may be developed by a combination of friction acting at the base of the slab foundation and passive earth pressure developed against the sides of the foundations below grade. Passive pressure and friction may be used in combination, without reduction, in determining the total resistance to lateral loads.

An allowable passive earth pressure of 200 psf per foot of foundation embedment below grade may be used for the sides of foundations placed against competent native soils. A coefficient of friction of 0.4 may be used for foundation cast directly on competent native soils or crushed rock wrapped in geofabric.

5.6.3 Walls Below Grade

Lateral earth pressures for walls below grade for structures less than 48 inches in horizontal dimensions may be treated as a shaft structure. Walls below grade for structures larger than 48 inches in horizontal dimensions should be designed to resist the lateral earth pressures presented in

Figures 7 and 8 provided that the wall backfill materials are properly placed and compacted in conformance with the recommendations presented in this report. Surcharge and foundation loads occurring within a horizontal distance equal to the wall height should be added to the lateral pressures as presented in Figures 9 and 10.

Buried structures located below the groundwater table will be subject to buoyant uplift forces. Geotechnical parameters for use in calculating uplift resistance of the surrounding backfill soil materials is presented in Figures 11 and 12.

5.7 Infiltration Testing

AGE attempted to perform infiltration testing in test hole P-4, but was unable to maintain a consistent free head inside the test hole during the 24-hour pre-soak period. During the test on February 14, 2019, AGE personnel had to add water into the test hole 24 times over a period of four (4) hours. The infiltration rate based on the last reading was calculated to be 90 inch per hour. It is our understanding that Rick Engineering is planning to install biofiltration basins with partial retention along the project alignments. It is our opinion that the soil underlying the project alignments are suitable for installation of partial retention biofiltration basins.

6.0 CONSTRUCTION-RELATED CONSIDERATIONS

6.1 Construction Dewatering

Groundwater and flowing sand conditions are anticipated to be encountered at or above the proposed pipe invert elevations along the project alignments. Because of the anticipated high rate of transmissivity of the underlying soils along the project alignments and the potential for encountering flowing sand condition, we recommend that groundwater be kept out of the trenched excavations using sheet piles in combination with sump pumps. Sheet piles should be extended to a depth of at least 10 feet below the bottom of the proposed trenched excavations.

The design, installation, and operation of any construction dewatering measures necessary for the project shall be the sole responsibility of the contractor.

6.2 Temporary Shoring

Since the anticipated pipe invert depths will be more than 4 feet below the ground surface, prevailing Federal and Cal OSHA safety regulations require that the trenched excavation be either sloped (if sufficient construction space or easement is available), shored, braced, or protected with approved sliding trench shield. Limited construction space, the presence of other buried utilities, and the need to avoid excessive community disruption dictate that a shored excavation will be needed along the entire pipeline alignment. Design and construction of temporary shoring should be the sole responsibility of the contractor.

Settlement

Settlement of existing street improvements and/or utilties adjacent to the shoring may occur in proportion to both the distance between shoring system and adjacent structures or utilities and the amount of horizontal deflection of the shoring system. Vertical settlement will be maximum directly adjacent to the shoring system, and decreases as the distance from the shoring increases. At a distance equal to the height of the shoring, settlement is expected to be negligible. Maximum vertical settlement is estimated to be on the order of 75 percent of the horizontal deflection of the shoring system. It is recommended that shoring be designed to limit the maximum horizontal deflection to 1-inch or less where structures or utilities are to be supported.

It is recommended that pre- and post-construction surveys be conducted to document existing site conditions. Documentation should include photographic and video surveys of the existing facilities and site improvements, as well as field surveys of building floors and pavement structures. We further recommend that a weekly survey of existing utilities be performed during the construction phase.

Lateral Earth Pressures

Temporary shoring should be designed to resist the pressure exerted by the retained soils and any additional lateral forces due to loads placed near the top of the excavation. For design of braced shorings supporting fill materials and old paralic deposits, the recommended lateral earth pressure should be 32H psf, where H is equal to the height of the retained earth in feet. Any surcharge loads would impose uniform lateral pressure of 0.3q, where "q" equals the uniform surcharge pressure. The surcharge pressure should be applied starting at a depth equal to the distance of the surcharge load from the top of the excavation. In the event that the bottom of the excavation is located below the groundwater level, hydrostatic pressure should be added to the lateral loads.

The recommended lateral earth pressures have been prepared based on the assumptions that the shored earth is level at the surface and that the shoring system is temporary in nature.

Lateral Bearing Capacity

Resistance to lateral loads will be provided by passive soil resistance. The allowable passive pressure for the fill materials and old paralic deposits may be assumed to be equivalent to a fluid weighing 200 pcf.

6.3 Environmental Considerations

The scope of AGE's investigation did not include the performance of a Phase I Environmental Site Assessment (Phase I ESA) to evaluate the possible presence of soil and/or groundwater contamination beneath the project alignments. During our subsurface investigation soil samples were field screened for the presence of volatile organics using a RAE Systems MiniRAE 3000 organic vapor meter (OVM). The field screening did not reveal elevated levels of volatile organics in the samples.

In the event that hazardous or toxic materials are encountered during the construction phase, the contractor should immediately notify the City and be prepared to handle and dispose of such materials in accordance with current industry practices and applicable Local, State and Federal regulations.

7.0 GENERAL CONDITIONS

7.1 Post-Investigation Services

Post-investigation geotechnical services are an important continuation of this investigation, and we recommend that the City's Construction Inspection Division performs the necessary geotechnical observation and testing services during construction. In the event that the City is unable to perform said services, it is recommended that our firm be retained to provide the services.

Sufficient and timely observation and testing should be performed during excavation, pipeline installation, backfilling and other related earthwork operations. The purpose of the geotechnical observation and testing is to correlate findings of this investigation with the actual subsurface conditions encountered during construction and to provide supplemental recommendations, if necessary.

7.2 Uncertainties and Limitations

The information presented in this report is intended for the sole use of Rick engineering and other members of the project design team and the City for project design purposes only and may not provide sufficient data to prepare an accurate bid. The contractor should be required to perform an independent evaluation of the subsurface conditions at the project site prior to submitting his/her bid.

AGE has observed and investigated the subsurface conditions only at selected locations along the project alignments. The findings and recommendations presented in this report are based on the assumption that the subsurface conditions beneath all project alignments do not deviate substantially from those encountered in the exploratory test pits. Consequently, modifications or changes to the recommendations presented herein may be necessary based on the actual subsurface conditions encountered during construction.

California, including San Diego County, is in an area of high seismic risk. It is generally considered economically unfeasible to build a totally earthquake-resistant project and it is, therefore, possible that a nearby large magnitude earthquake could cause damage at the project site.

Geotechnical engineering and geologic sciences are characterized by uncertainty. Professional judgments and opinions presented in this report are based partly on our evaluation and analysis of the technical data gathered during our present study, partly on our understanding of the scope of the proposed project, and partly on our general experience in geotechnical engineering.

In the performance of our professional services, we have complied with that level of care and skill ordinarily exercised by other members of the geotechnical engineering profession currently practicing under similar circumstances in southern California. Our services consist of professional consultation only, and no warranty of any kind whatsoever, expressed or implied, is made or intended in connection with the work performed. Furthermore, our firm does not guarantee the performance of the project in any respect.

AGE does not practice or consult in the field of safety engineering. The contractor will be responsible for the health and safety of his/her personnel and all subcontractors at the construction site. The contractor should notify the City if he or she considers any of the recommendations presented in this report to be unsafe.

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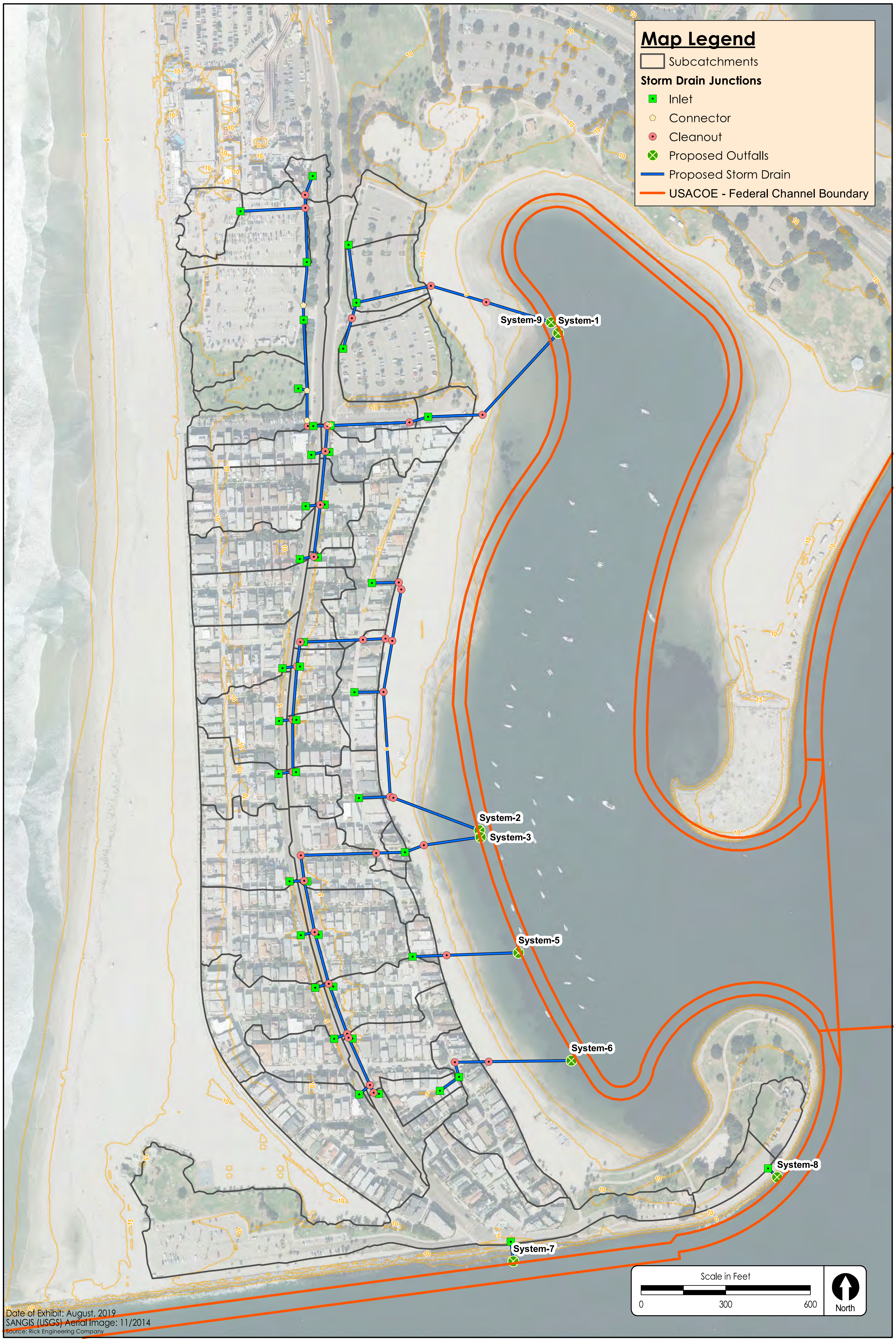
FIGURES



The City of SAN DIEGO | RICK ENGINEERING COMPANY

South Mission Beach - Storm Drain Improvements and GI

Storm Drain Alignment - System Outfalls

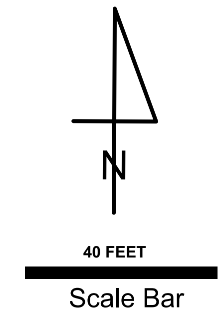
SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT ALIGNMENTS MAP

PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 1

LEGEND

P-1	Approximate Infiltration Test Hole Location
B-2	Approximate Boring Location
C-2	Approximate Pavement Core Location

SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT	LOCATION MAP	
PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 2



LEGEND

P-1	Approximate Infiltration Test Hole Location
B-2	Approximate Boring Location
C-2	Approximate Pavement Core Location

SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT	LOCATION MAP
PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 3

40 FEET

Scale Bar

N

LEGEND

P-1 Approximate Infiltration Test Hole Location

B-2 Approximate Boring Location

C-2 Approximate Pavement Core Location

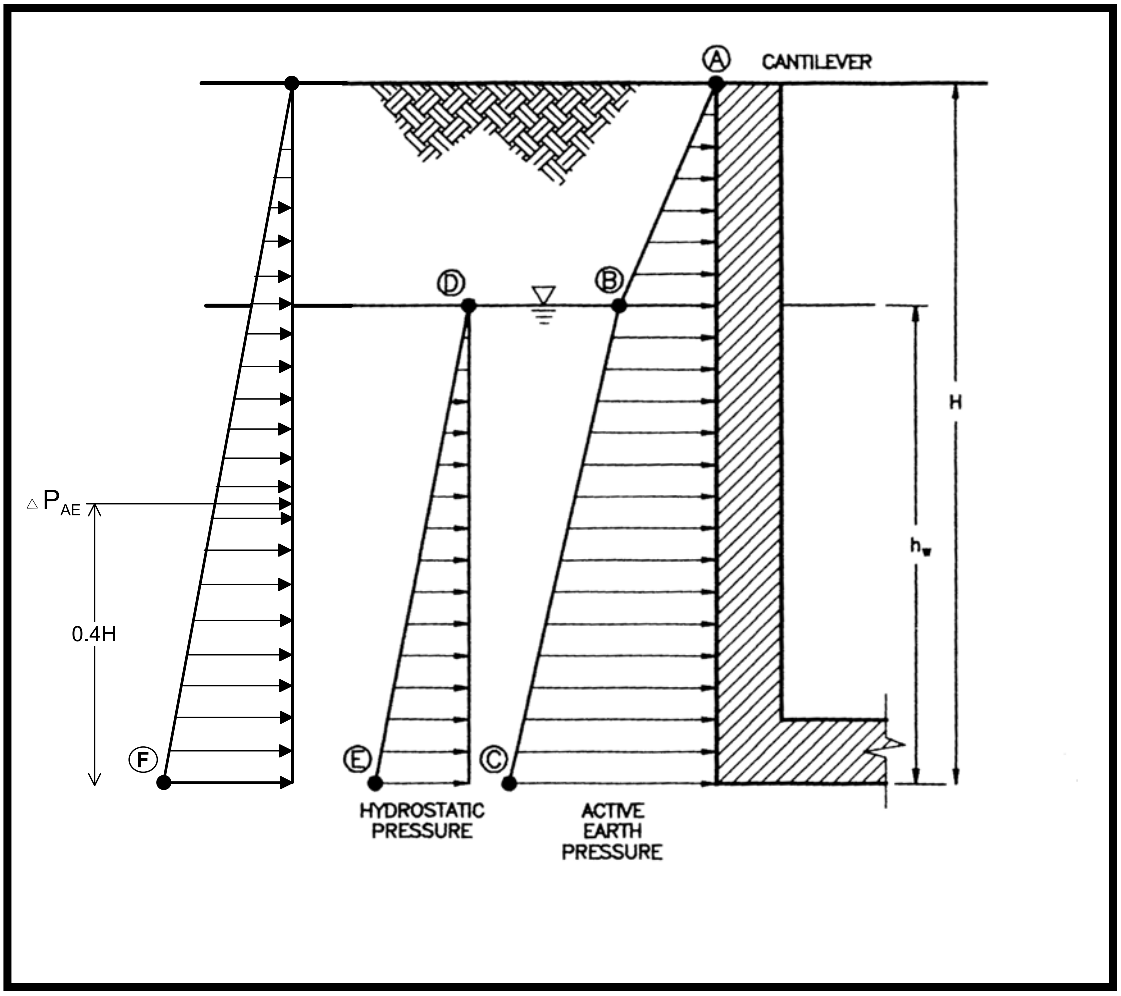
SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT		LOCATION MAP
PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 4

LEGEND

- P-1 Approximate Infiltration Test Hole Location
- B-2 Approximate Boring Location
- C-2 Approximate Pavement Core Location

SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT		LOCATION MAP
PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 5

SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT		WEEP SUMP PHOTOGRAPH
PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 6



NOTES

H = wall height in feet

h_w = water height above bottom of structure in feet

Lateral pressure values presented herein are based on the assumption that non-expansive backfill materials will be used to backfill behind walls

LATERAL PRESSURES

Earth Pressure

Ⓐ = 0
Ⓑ = $35\ (H-h_w)$, psf
Ⓒ = $35\ (H-h_w) + 20h_w$, psf

Hydrostatic Pressure

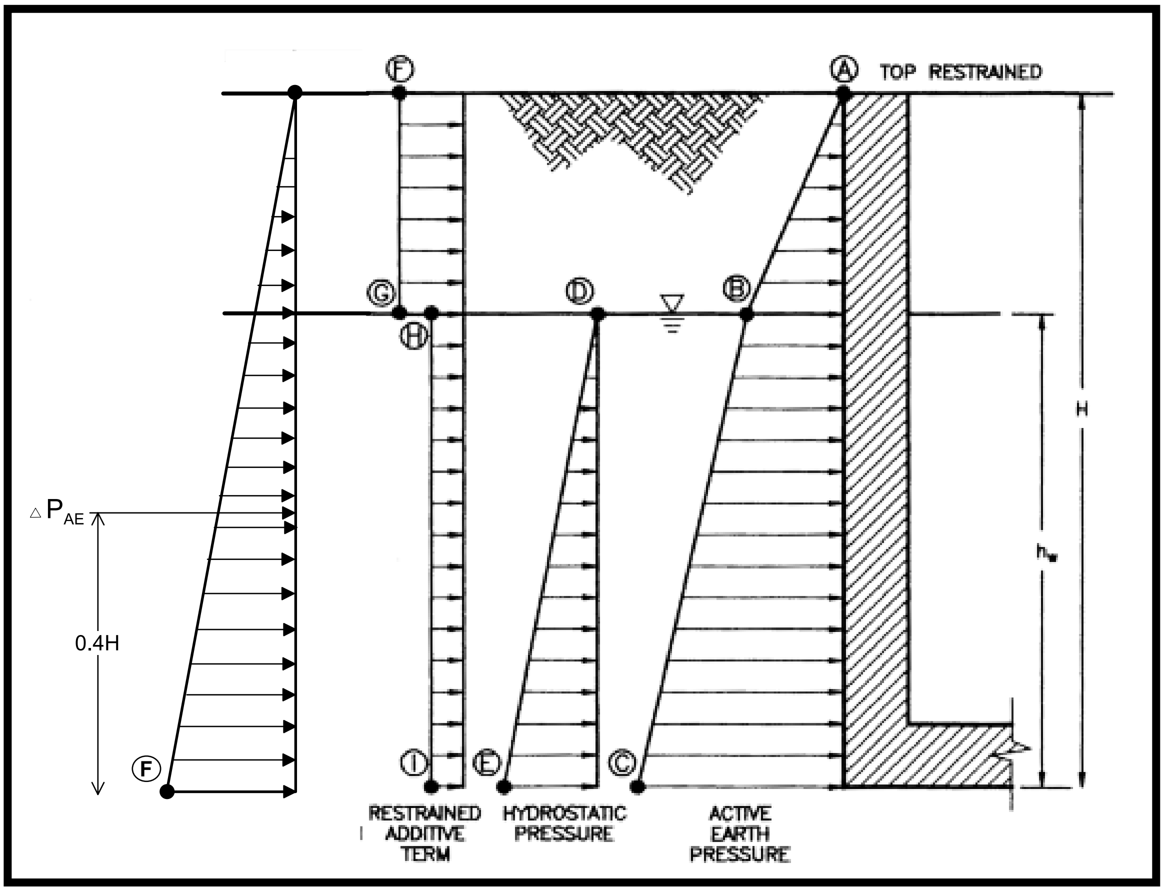
Ⓓ = 0
Ⓔ = $62.4h_w$

Dynamic Resultant Force

Ignored for shaft construction.

LATERAL PRESSURES FOR CANTILEVER WALLS
SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT

PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 7



NOTES

H = wall height in feet

h_w = water height above bottom of structure in feet

Lateral pressure values presented herein are based on the assumption that non-expansive backfill materials will be used to backfill behind walls

LATERAL PRESSURES

Earth Pressure
A = 0
B = 35 $(H-h_w)$, psf
C = 35 $(H-h_w) + 20h_w$, psf

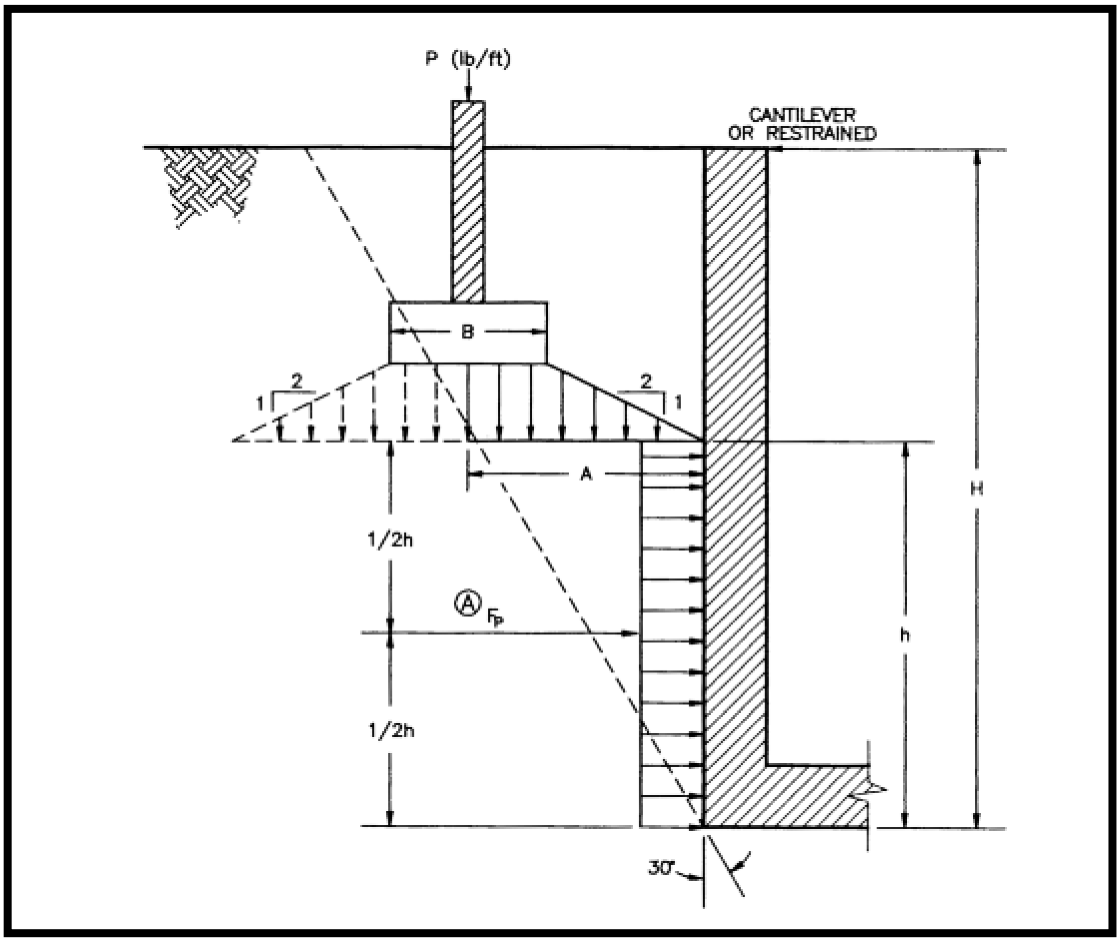
Hydrostatic Pressure
D = 0
E = $62.4h_w$

Restrained Additive Term
F = G = 10H, psf
H = I = 5H, psf

Dynamic Resultant Force
Ignored for shaft construction.

LATERAL PRESSURES FOR RESTRAINED WALLS
SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT

PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 8



NON-EXPANSIVE BACKFILL

F_p = M (A/B) P, lb/ft

A = h tan 30°, ft
M = 0.3 for cantilever wall
M = 0.4 for restrained wall

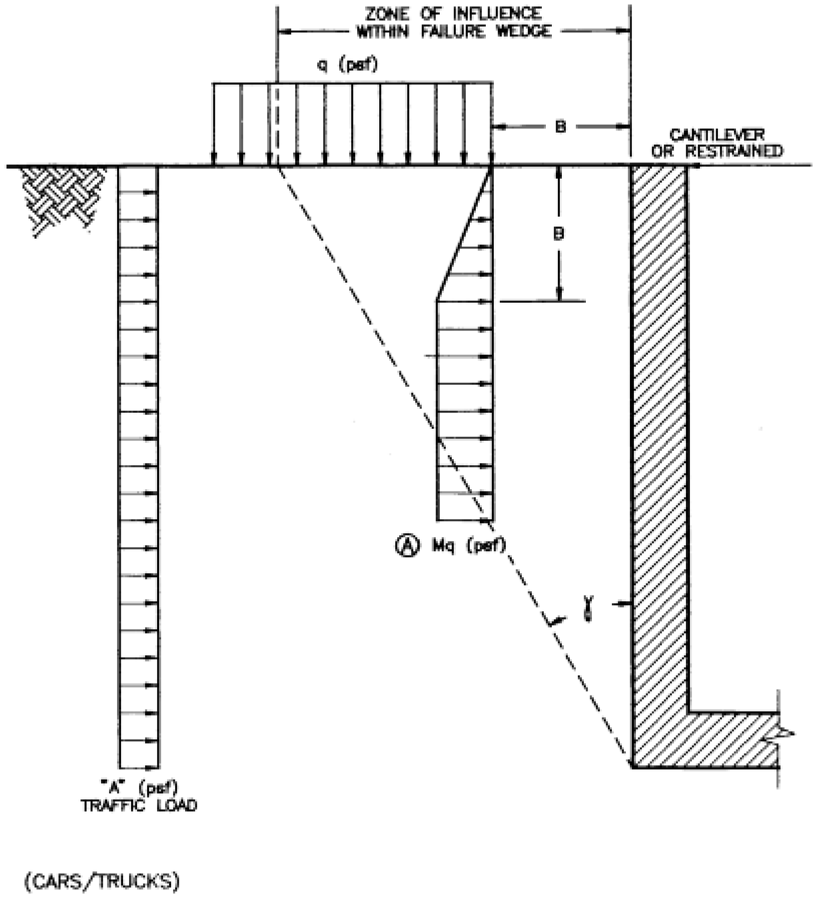
NOTES:

1. Surcharge pressure acting on wall is not affected by groundwater elevation.

2. Surcharge pressures shown are applicable for continuous footing only. Spread footings need to be evaluated individually.

FOUNDATION INDUCED WALL PRESSURE
SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT

PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 9



NON-EXPANSIVE BACKFILL

q = surcharge load (psf)
B = distance between wall and surcharge load, ft
M = 0.3 for cantilever wall
M = 0.4 for restrained wall
A = Mq, psf
"A" = 75 psf
$\gamma = 30^{o}$

NOTE: Surcharge pressure acting on wall is not affected by groundwater elevation.

TRAFFIC INDUCED WALL PRESSURES
SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT

PROJECT NO. 190 GS-18-D	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE 10**

APPENDIX A

FIELD EXPLORATION PROGRAM

APPENDIX A

FIELD EXPLORATION PROGRAM

The field exploration program for this project was performed on February 11 and 12, 2019. A total of four (4) soil borings, four (4) infiltration test holes, and two (2) pavement corings were performed at the approximate locations shown on Figures 2 through 5. In addition, AGE attempted to perform infiltration testing inside an existing weep sump located on the west side of Mission Boulevard, at the entrance of an alley located between Brighton Court and Capistrano Place. The soil borings were advanced to depths ranging from 15 feet to 16.5 feet below the existing ground surface (bgs). The infiltration test holes were hand-augured to depths ranging from 36 inches to 63 inches bgs. A brief description of the location and depth, pavement sections, groundwater level, and subsurface conditions encountered in the borings and infiltration test holes is presented in Table 1.

Borings B-2 and B-4 which were located in Mission Boulevard were performed with a CME-75 truck mounted drill rig. Borings B-1 and B-3 which were located on Mission Bay Park were performed with an all-terrain mounted drill rig. The soils encountered in the soil borings were visually classified and logged by an experienced engineering geologist from AGE. A Key to Logs is presented on Figures A-1 and A-2, and logs of the borings are presented on Figures A-3 thru A-6. The logs depict the various soil types encountered and indicate the depths at which samples were obtained for laboratory testing and analysis.

Prior to commencement of the field exploration activities, several site visits were performed to observe existing conditions and to select suitable locations for the soil borings and test holes. Subsequently, Underground Service Alert (USA) was contacted to coordinate clearance of the proposed boring and test hole locations with respect to existing buried utilities. The borings and test holes located in Mission Bay Park were performed in coordination with and with the approval from the City of San Diego Parks & Recreation Department.

During drilling, Standard Penetration Tests (SPT) were performed at selected depth intervals. The SPT tests involve the use of a specially manufactured “split spoon” sampler which is driven a distance of approximately 18 inches into the soils at the bottom of the borehole by dropping a 140-pound weight from a height of 30 inches. The number of blows required to penetrate each 6-inch increment was counted and recorded on the field logs, and have been used to evaluate the relative density and consistency of the materials. The blow counts were subsequently corrected for soil type, hammer model, groundwater and surcharge. The corrected blow counts are shown on the boring logs.

Relatively undisturbed samples were obtained by driving a 3-inch (OD) diameter standard California sampler with a special cutting tip and inside lining of thin brass rings into the soils at the bottom of the borehole. The sampler is driven a distance of approximately 18 inches into the soil at the bottom of the borehole by dropping a 140-pound weight from a height of 30 inches. A 6-inch long section of soil sample that was retained in the brass rings was extracted from the sampling tube and transported to our laboratory in close-fitting, waterproof containers. The samples were field screened for the presence of volatile organics using a RAE Systems MiniRAE 3000 organic vapor meter (OVM). The OVM readings are indicated on the logs. In addition, loose bulk samples were also collected.

Infiltration testing inside test hole P-4 was performed using Borehole Percolation Test Methods described in Appendix F - Storm Water Infiltration/Percolation BMPs of the City of San Diego Guidelines for Geotechnical Report (2011) and Appendix D - Approved Infiltration Rate Assessment Methods of the San Diego Region Model BMP Design Manual (2018).

Upon completion of the drilling, sampling and testing activities, the borings were backfilled using bentonite grout and/or bentonite chips to approximately 12 inches below the ground surface. Borings B-1 and B-3 which were located at the beach were capped with on-site beach sand. Borings B-2 and B-4 which were performed in Mission Boulevard were capped with rapid-set concrete to match the adjacent pavement surface. Pavement coreholes C-2 and C-4 were also capped with rapid-set concrete to match the adjacent pavement surface. The infiltration test holes were backfilled with soil cuttings generated during excavation.

KEY TO LOG OF BORING

DEPTH (FEET)	SAMPLES	BLOW COUNTS (BLOWS/FOOT)	OVM READING (PPM)	GRAPHIC LOG	SOIL DESCRIPTION	FIELD MOISTURE (% DRY WT.)	DRY DENSITY (PCF)	REMARKS
1								
2					Sample identification number			
3								
4	1				Approximate interval of bulk sample			
5								
6								
7								
8	2	28			Approximate interval of Standard California Sampler (SCS).			
9								
10					Number of blows required to advance sampler for the last 12-inch increment, or distance indicated.			
11								
12	3	33			Approximate interval of Standard Penetration Test (SPT).			
13								
14								
15								
16								
17					Approximate groundwater elevation			
18								
19								

(KEY TO LOG OF BORING CONTINUED ON FIGURE A-2)

KEY TO LOG OF BORING
(CONTINUED)

DEPTH (FEET)	SAMPLES	BLOW COUNTS (BLOWS/FOOT)	OVM READING (PPM)	GRAPHIC LOG	SOIL DESCRIPTION	FIELD MOISTURE (% DRY WT.)	DRY DENSITY (PCF)	REMARKS

—?— — —?— APPROXIMATE GEOLOGIC CONTACT

Strata symbols

Poorly graded sand with silt

1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19

GENERAL NOTES

1. Approximate elevations and locations of borings are based on the topographical maps provided by Rick Engineering Company, undated.
2. Soil descriptions are based on visual classification made during the field exploration and, where deemed appropriate, have been modified based on the results of laboratory tests.
3. Descriptions on the logs apply only at the specific locations and at the time the work was performed. They are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO. 190 GS-18-D

ALLIED GEOTECHNICAL ENGINEERS, INC.

FIGURE A-2

BORING NO. B-1

DATE OF DRILLING: February 11, 2019	TOTAL BORING DEPTH: 16.5'
GENERAL LOCATION: On the beach, 20' east of Bayside walk at San Fernando Place	
APPROXIMATE SURFACE ELEV.: + 7.3' msl	DRILLING CONTRACTOR: Tri-County Drilling
DRILLING METHOD: Hollow-Stem Auger	LOGGED BY: Nicholas Barnes

DEPTH (FEET)	SAMPLES	BLOW COUNTS BLOWS/FOOT	OVM READING (PPM)	GRAPHIC LOG	SOIL DESCRIPTION	FIELD MOISTURE % DRY WT.	DRY DENSITY LBS./CU. FT.	REMARKS
0–10					**HYDRAULIC FILL** Light gray, damp, medium grained poorly graded micaceous sand (SP-SM) with traces of broken shells.			
6	1	25	1.6			6.2	105.6	
10			?					?
11	2	17	0.1		**OLD PARALIC DEPOSITS** ▼ Dark greenish gray, wet, medium grained poorly graded micaceous sand (SP-SM)	22.2		
11.5	3							
14.5	4							
16	5	23				30.3	95.1	

NOTES:

Boring terminated at depth of 16.5' bgs. No refusal.
Water level measured at depth of 11' bgs 10 minutes after completion of the drilling operations.

PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE A-3

BORING NO. B-2

DATE OF DRILLING: February 12, 2019	TOTAL BORING DEPTH: 15’
GENERAL LOCATION: Southbound Mission Boulevard, approximately 40' south of San Fernando Place and 4' from median.	
APPROXIMATE SURFACE ELEV.: +6.3' msl	DRILLING CONTRACTOR: Tri-County Drilling
DRILLING METHOD: Hollow-Stem Auger	LOGGED BY: Nicholas Barnes

DEPTH (FEET)	SAMPLES	BLOW COUNTS BLOWS/FOOT	OVM READING (PPM)	GRAPHIC LOG	SOIL DESCRIPTION	FIELD MOISTURE % DRY WT.	DRY DENSITY LBS./CU. FT.	REMARKS
1					**PAVEMENT SECTION:** 4" A.C. over 8" P.C.C. underlain by 6" of miscellaneous base			
2					**OLD PARALIC DEPOSITS**			
3								
4					Greenish gray, wet, medium grained poorly-graded micaceous sand with silt (SP-SM) with traces of broken shells.			
5								
6	1	34				26.0		
	2							
7								
8	3							
9								
10	4	26				26.3		
11								
12								
13								
14	5					24.7		
15								

16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37

NOTES:

Boring terminated at depth of 15' bgs. No refusal.
Water level measured at depth of 4'-3" bgs at the completion of the drilling operation.

PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE A-4

BORING NO. B-3

DATE OF DRILLING: February 11, 2019	TOTAL BORING DEPTH: 16.5
GENERAL LOCATION: On the beach, 20' east of Bayside walk at Coronado Court	
APPROXIMATE SURFACE ELEV.: +5.2' msl	DRILLING CONTRACTOR: Tri-County Drilling
DRILLING METHOD: Hollow-Stem Auger	LOGGED BY: Nicholas Barnes

DEPTH (FEET)	SAMPLES	BLOW COUNTS BLOWS/FOOT	OVM READING (PPM)	GRAPHIC LOG	SOIL DESCRIPTION	FIELD MOISTURE % DRY WT.	DRY DENSITY LBS./CU. FT.	REMARKS
0–4					**HYDRAULIC FILL** Greenish gray, wet, fine- to medium-grained poorly graded micaceous silty sand (SP-SM) ▼			
6	1	27				25.5		
7–8	2							
9	3					21.4		
10			?					?
11	4	26			**OLD PARALIC DEPOSITS** Greenish gray, wet, fine- to medium-grained poorly graded micaceous silty sand (SP-SM)			Heaving sand. No sample recovery.
15	5					24.9		

NOTES:

First attempt encountered refusal at 3' bgs and the boring location was moved 10 feet to the east.
Boring terminated at depth of 16.5' bgs. No refusal.
Water level measured at depth of 4' bgs at the completion of the drilling operation.

PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE A-5

BORING NO. B-4

DATE OF DRILLING: February 12, 2019	TOTAL BORING DEPTH: 15
GENERAL LOCATION: Southbound Mission Boulevard, approximately 60' south of Brighton Court	
APPROXIMATE SURFACE ELEV.: +5' msl	DRILLING CONTRACTOR:
DRILLING METHOD:	LOGGED BY: Nicholas Barnes

DEPTH (FEET)	SAMPLES	BLOW COUNTS BLOWS/FOOT	OVM READING (PPM)	GRAPHIC LOG	SOIL DESCRIPTION	FIELD MOISTURE % DRY WT.	DRY DENSITY LBS./CU. FT.	REMARKS
1					**PAVEMENT SECTION:**			
2					4.5" A.C. over 7.5" P.C.C. underlain by 4" of miscellaneous base			
3					**OLD PARALIC DEPOSITS**			
4					Greenish gray, wet, medium grained poorly-graded micaceous sand with silt (SP-SM) with traces of broken shells and rounded gravels.			
5								
6	1 2	35						
7								
8	3		2.0			29.1		
9								
10	4		2.2			29.6		
11								
12								
13								
14	5		0.5			24.2		
15								

NOTES:

Boring terminated at depth of 15' bgs. No refusal. Water level measured at depth of 3'-3" bgs at the completion of the drilling operation.

PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE A-6

APPENDIX B

LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

Selected soil samples were tested in the laboratory to verify visual field classifications and to evaluate certain engineering characteristics. The testing was performed in accordance with the American Society for Testing and Materials (ASTM) or other generally accepted test methods, and included the following:

- Determination of in-place moisture content (ASTM D2216). The final test results are presented on the test pit logs;

- Determination of in-place dry density and moisture content (ASTM D2937) based on relatively undisturbed drive samples. The final test results are presented on the test pit logs;

- Maximum density and optimum moisture content (ASTM D1557). The final test results are presented on Figures B-1 thru B-3;

- Sieve analyses (ASTM D422), and the final test results are plotted as gradation curves on Figures B-4 and B-5;

- Direct shear test (ASTM D3080). The test results are presented on Figures B-6 and B-7; and

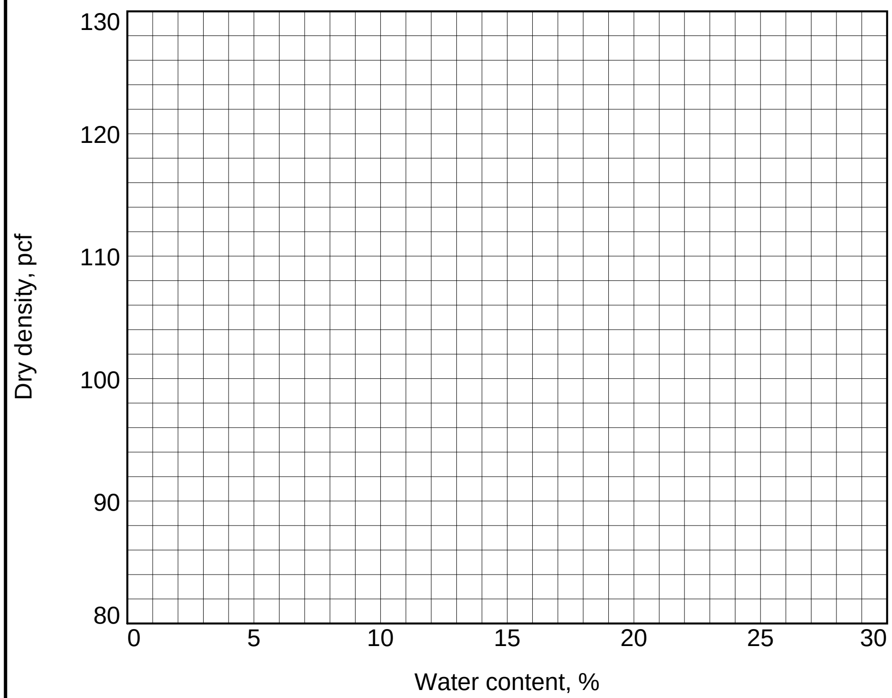
- Consolidation (ASTM D2435). The test results are presented on Figure B-8.

In addition, representative samples of the onsite soil materials were delivered to Clarkson Laboratory and Supply, Inc. for analytical (chemical) testing to determine soil pH and resistivity, soluble sulfate and chloride concentrations, and bicarbonate content. Copies of Clarkson's laboratory test data reports are included herein.

Representative samples of the soil materials underlying Mission Boulevard were delivered to Southern California Soil & Testing (SCS&T) for R-Value testing. Copies of SCS&T's laboratory test data reports are included herein.

COMPACTION TEST REPORT

Curve No.



Test Specification:
ASTM D 1557-91 Procedure A Modified

Preparation Method ______
Hammer Wt. 10 lb.
Hammer Drop 18 in.
Number of Layers five
Blows per Layer 25
Mold Size 0.03333 cu. ft.
Test Performed on Material Passing #4 **Sieve**

NM 6.2 **LL** ______ **PI** ______
Sp.G. (ASTM D 854) 2.6
%>#4 ______ **%<No.200** ______
USCS ______ **AASHTO** ______
Date Sampled ______
Date Tested ______
Tested By ______

TESTING DATA

	1	2	3	4	5	6
WM + WS						
WM						
WW + T #1						
WD + T #1						
TARE #1						
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE						
DRY DENSITY						

TEST RESULTS

Material Description
Dark greenish gray poorly-graded sand with silt (SP-SM)

Remarks:

Project No. 190 GS-18-D **Client:** Rick Engineering Company
Project: South Mission Beach Project

○ **Source of Sample:** B-1 **Depth:** 5 **Sample Number:** 1

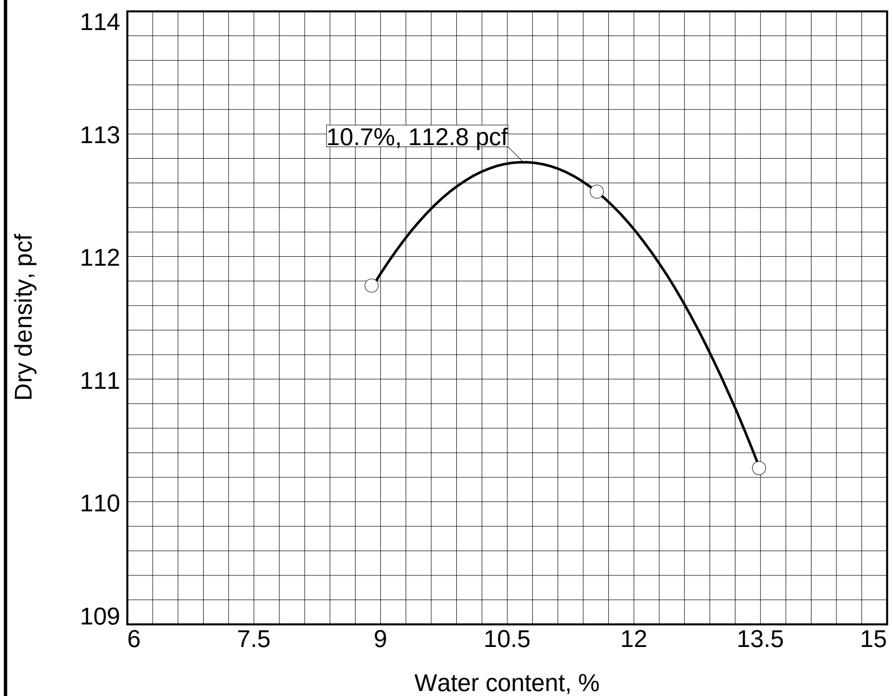
Allied Geotechnical Engineers, Inc.

Santee, CA

Checked by:
Title:

Figure B-1

COMPACTION TEST REPORT



Curve No.

Test Specification:
ASTM D 1557-91 Procedure A Modified

Preparation Method Wet
Hammer Wt. 10 lb.
Hammer Drop 18 in.
Number of Layers five
Blows per Layer 25
Mold Size 0.03333 cu. ft.
Test Performed on Material Passing #4 **Sieve**

NM 22.2 **LL** NV **PI**
Sp.G. (ASTM D 854)
%>#4 1.0 **%<No.200** 10.0
USCS SP-SM **AASHTO** A-3
Date Sampled 02/12/2019
Date Tested 02/21/2019
Tested By Nicholas Barnes

TESTING DATA

	1	2	3	4	5	6
WM + WS	5932.0	5926.0	5874.0			
WM	4034.0	4034.0	4034.0			
WW + T #1	531.6	520.0	493.9			
WD + T #1	482.6	466.7	459.5			
TARE #1	59.2	71.7	73.2			
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	11.6	13.5	8.9			
DRY DENSITY	112.5	110.3	111.8			

TEST RESULTS

Maximum dry density = 112.8 pcf

Optimum moisture = 10.7 %

Material Description

Dark greenish gray poorly-graded sand with silt (SP-SM)

Remarks:

Project No. 190 GS-18-D **Client:** Rick Engineering Company
Project: South Mission Beach Project

○ **Source of Sample:** B-1 **Depth:** 10 **Sample Number:** 2

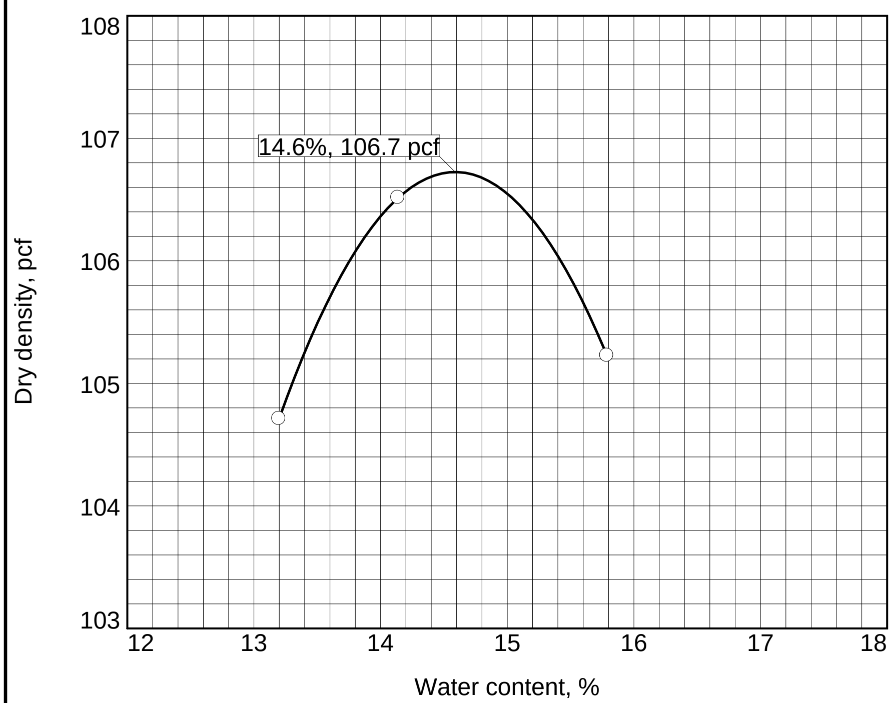
Allied Geotechnical Engineers, Inc.

Santee, CA

Checked by: Sani Sutanto
Title: Project Manager

Figure B-2

COMPACTION TEST REPORT



Curve No.

Test Specification:
ASTM D 1557-91 Procedure A Modified

Preparation Method Wet
Hammer Wt. 10 lb.
Hammer Drop 18 in.
Number of Layers five
Blows per Layer 25
Mold Size 0.03333 cu. ft.
Test Performed on Material Passing #4 **Sieve**

NM **LL** NV **PI**
Sp.G. (ASTM D 854)
%>#4 0.1 **%<No.200** 5.1
USCS SP-SM **AASHTO** A-3
Date Sampled 02/12/2019
Date Tested 02/21/2019
Tested By Nicholas Barnes

TESTING DATA

	1	2	3	4	5	6
WM + WS	5826.0	5872.0	5876.0			
WM	4034.0	4034.0	4034.0			
WW + T #1	491.0	475.3	486.0			
WD + T #1	440.0	424.2	428.2			
TARE #1	53.6	62.8	62.1			
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	13.2	14.1	15.8			
DRY DENSITY	104.7	106.5	105.2			

TEST RESULTS

Maximum dry density = 106.7 pcf

Optimum moisture = 14.6 %

Material Description

Greenish gray poorly graded sand with silt (SP-SM)

Remarks:

Project No. 190 GS-18-D **Client:** Rick Engineering Company
Project: South Mission Beach Project

○ **Source of Sample:** B-3 **Depth:** 6

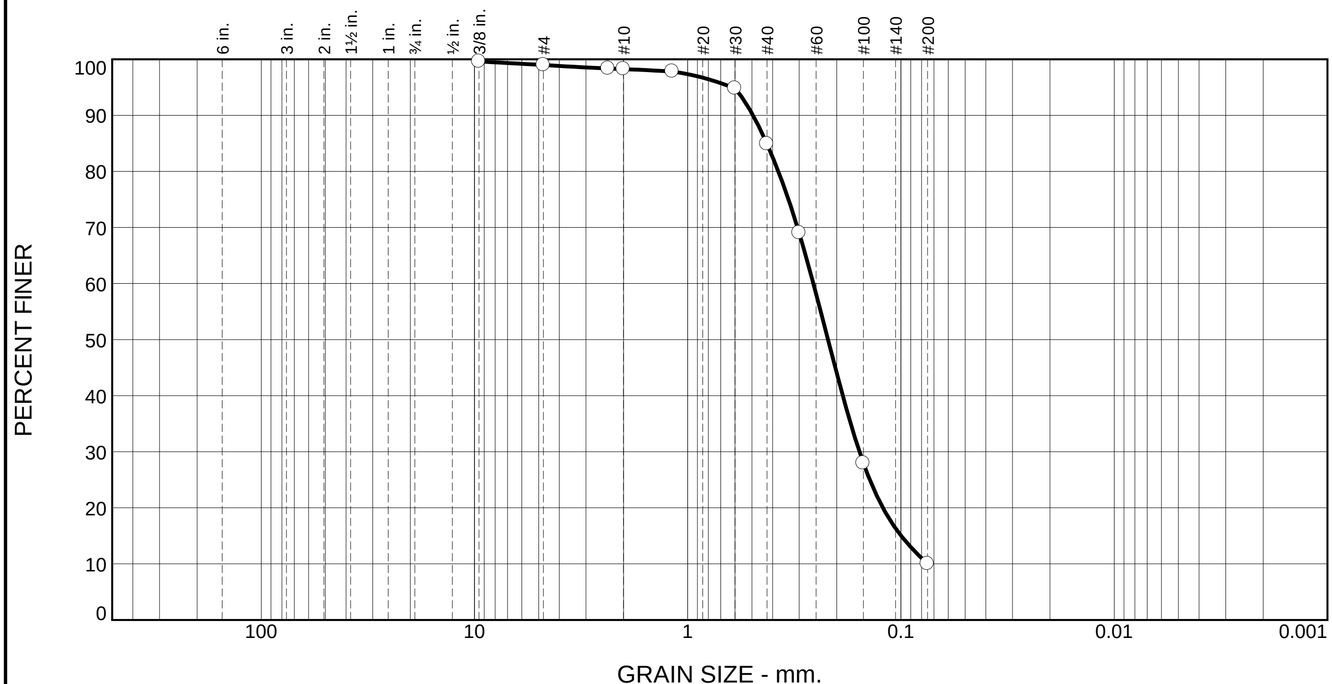
Allied Geotechnical Engineers, Inc.

Santee, CA

Checked by: Sani Sutanto
Title: Project Manager

Figure B-3

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
			0.8	13.3	74.9	10.0	

Test Results (ASTM D 422 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.375	99.6		
#4	99.0		
#8	98.4		
#10	98.2		
#16	97.8		
#30	94.8		
#40	84.9		
#50	69.0		
#100	27.9		
#200	10.0		

* (no specification provided)

Material Description

Dark greenish gray poorly-graded sand with silt (SP-SM)

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SP-SM **AASHTO (M 145)=** A-3

Coefficients

D_{90}= 0.4940	D_{85}= 0.4263	D_{60}= 0.2575
D_{50}= 0.2199	D_{30}= 0.1566	D_{15}= 0.0998
D_{10}=	C_u=	C_c=

Remarks

Date Received: **Date Tested:** 02/21/2019

Tested By: Nicholas Barnes

Checked By: Sani Sutanto

Title: Project Manager

Source of Sample: B-1 **Depth:** 10 **Date Sampled:** 02/12/2019

Sample Number: 2

Allied Geotechnical Engineers, Inc.

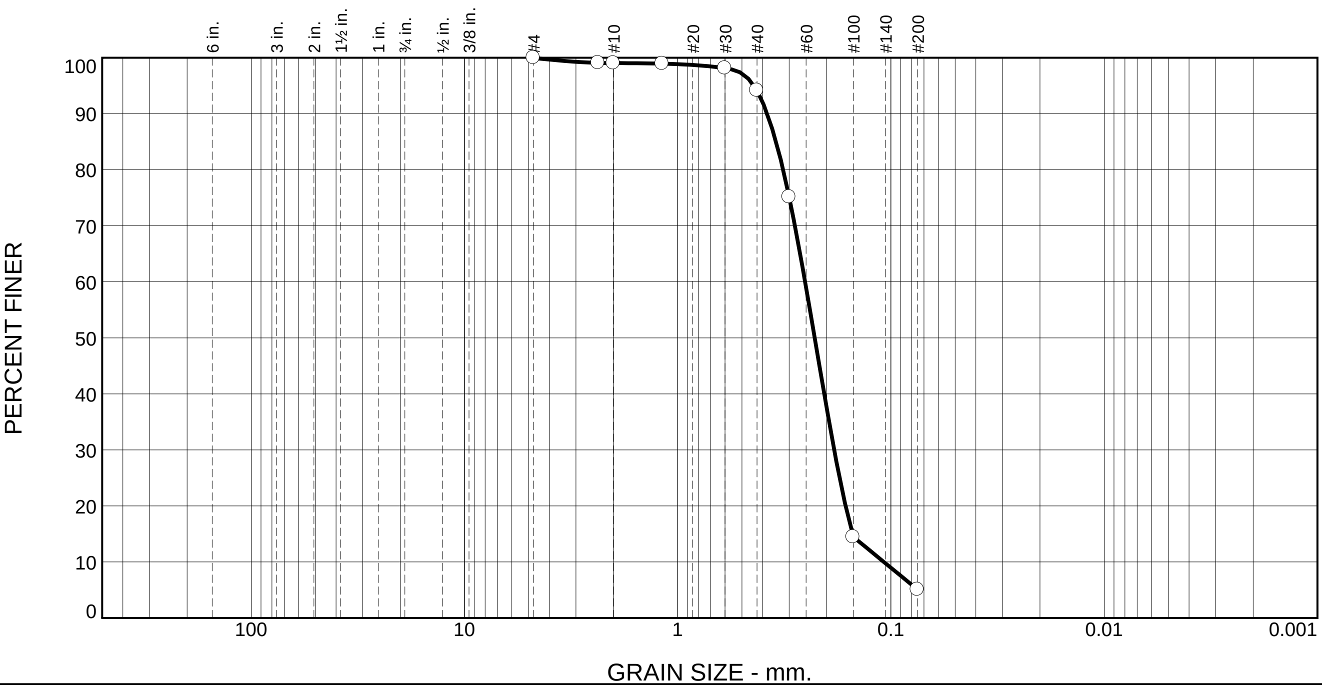
Santee, CA

Client: Rick Engineering Company

Project: South Mission Beach Project

Project No: 190 GS-18-D **Figure** B-4

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
			0.9	4.9	89.0	5.1	

Test Results (ASTM D 422 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	99.9		
#8	99.1		
#10	99.0		
#16	98.9		
#30	98.1		
#40	94.1		
#50	75.1		
#100	14.5		
#200	5.1		

* (no specification provided)

Material Description

Greenish gray poorly graded sand with silt (SP-SM)

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=**

Classification

USCS (D 2487)= SP-SM **AASHTO (M 145)=** A-3

Coefficients

D_{90}= 0.3806 D_{85}= 0.3460 D_{60}= 0.2527
D_{50}= 0.2279 D_{30}= 0.1846 D_{15}= 0.1514
D_{10}= 0.1079 C_u= 2.34 C_c= 1.25

Remarks

Date Received: 02/12/2019 **Date Tested:** 02/21/2019

Tested By: Nicholas Barnes

Checked By: Sani Sutanto

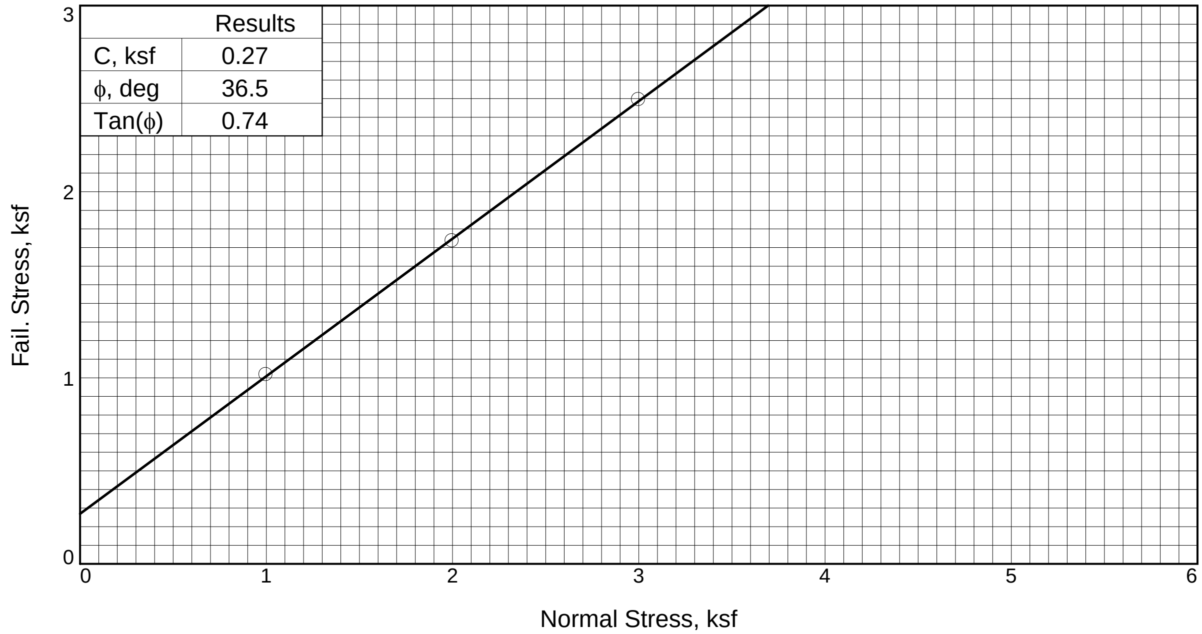
Title: Project Manager

Source of Sample: B-3 **Depth:** 6 **Date Sampled:** 02/12/2019

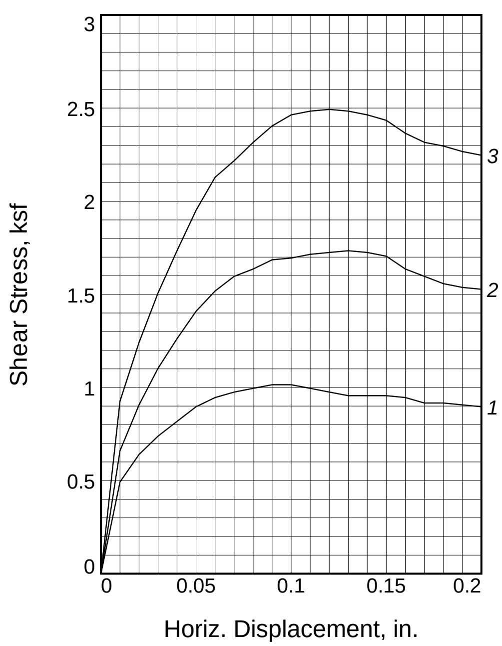
Allied Geotechnical Engineers, Inc.

Santee, CA

Client: Rick Engineering Company
Project: South Mission Beach Project
Project No: 190 GS-18-D **Figure** B-5



	Results
C, ksf	0.27
ϕ, deg	36.5
Tan(ϕ)	0.74



	Sample No.	1	2	3
Initial	Water Content, %	31.5	31.0	30.7
	Dry Density, pcf	94.1	95.8	94.6
	Saturation, %	113.0	116.0	111.6
	Void Ratio	0.7253	0.6944	0.7159
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	0.0	0.0	0.0
	Dry Density, pcf	95.4	96.8	95.3
	Saturation, %	0.0	0.0	0.0
	Void Ratio	0.7012	0.6774	0.7039
	Diameter, in.	2.38	2.38	2.38
	Height, in.	0.99	0.99	0.99
	Normal Stress, ksf	1.00	2.00	3.00
	Fail. Stress, ksf	1.02	1.73	2.49
	Displacement, in.	0.09	0.13	0.12
	Ult. Stress, ksf			
	Displacement, in.			
	Strain rate, in./min.	0.008	0.008	0.008

Sample Type: Ring
Description:

Assumed Specific Gravity= 2.6
Remarks:

Figure B-6

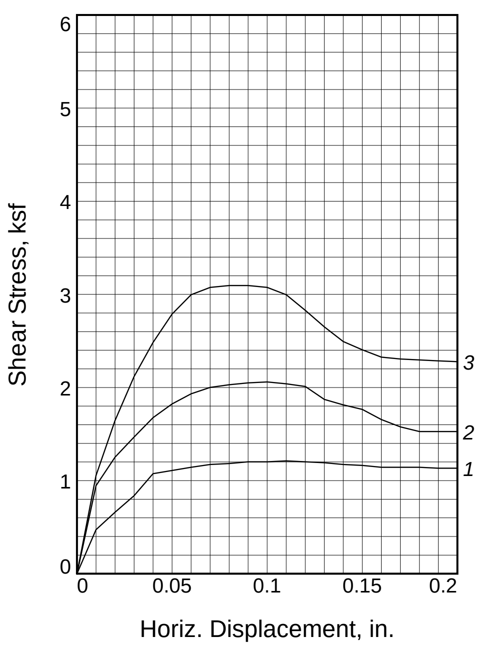
Client: Rick Engineering Company

Project: South Mission Beach Project

Source of Sample: B-1 **Depth:** 15
Sample Number: 5
Proj. No.: 190 GS-18-D **Date Sampled:** 02/12/2019

DIRECT SHEAR TEST REPORT
Allied Geotechnical Engineers, Inc.
Santee, CA

Tested By: Nicholas Barnes



Sample No.		1	2	3
Initial	Water Content, %	22.0	22.1	23.1
	Dry Density, pcf	103.1	102.8	101.4
	Saturation, %	99.7	99.1	99.8
	Void Ratio	0.5742	0.5795	0.6010
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	22.5	22.8	23.2
	Dry Density, pcf	105.7	105.0	102.8
	Saturation, %	109.5	108.3	104.4
	Void Ratio	0.5349	0.5463	0.5785
	Diameter, in.	2.38	2.38	2.38
	Height, in.	0.97	0.98	0.99
Normal Stress, ksf		1.00	2.00	3.00
Fail. Stress, ksf		1.21	2.06	3.09
Displacement, in.		0.11	0.10	0.08
Ult. Stress, ksf				
Displacement, in.				
Strain rate, in./min.		0.008	0.008	0.008

Sample Type: Ring
Description:

Assumed Specific Gravity= 2.6
Remarks:

Figure B-7

Client: Rick Engineering Company

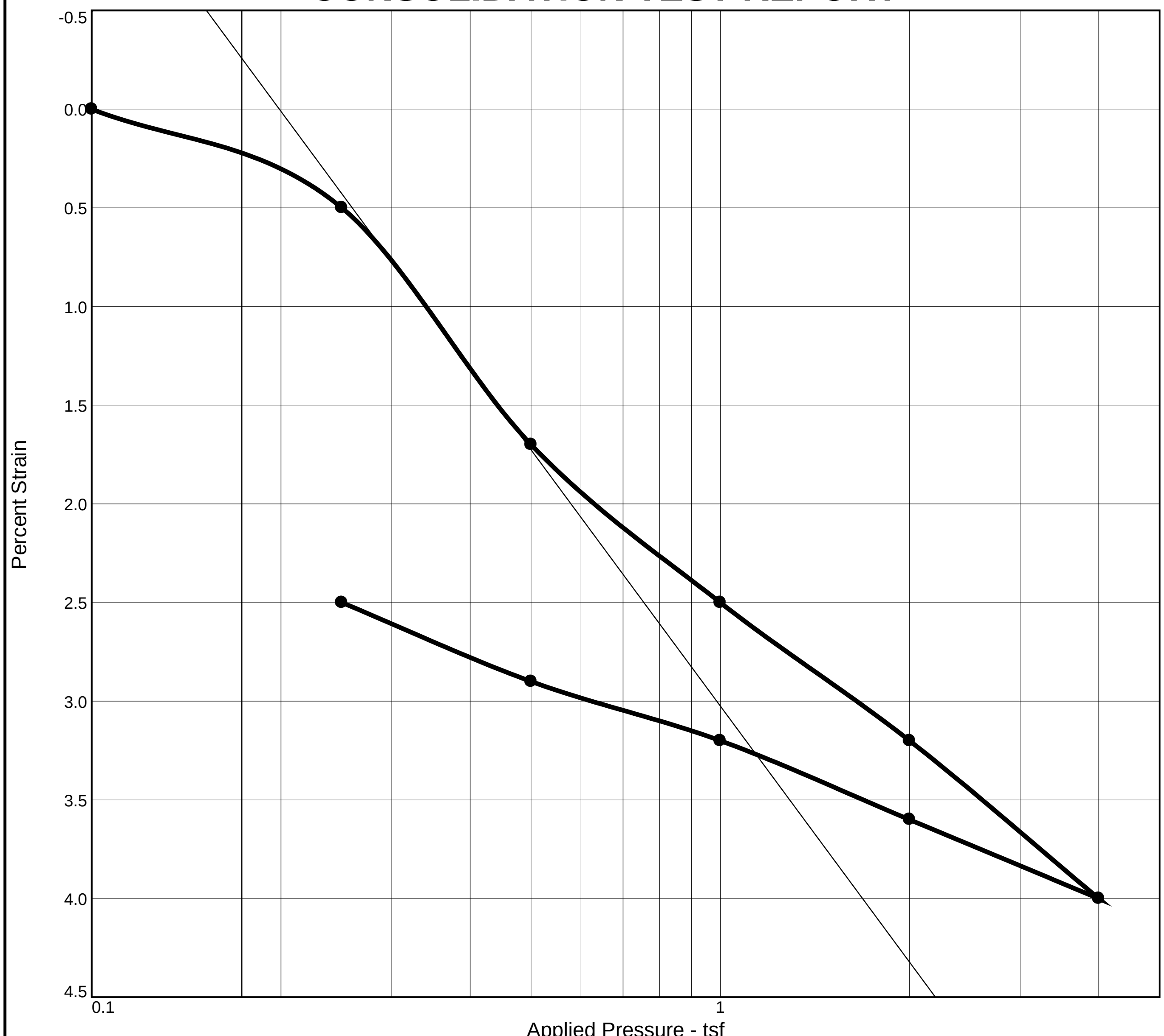
Project: South Mission Beach Project

Source of Sample: B-4 **Depth:** 5
Sample Number: 1
Proj. No.: 190 GS-18-D **Date Sampled:** 02/11/2019

DIRECT SHEAR TEST REPORT
Allied Geotechnical Engineers, Inc.
Santee, CA

Tested By: Nicholas Barnes

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
34.0 %	7.1 %	98.2			2.6	0.1	0.2	0.07		0.543

MATERIAL DESCRIPTION	USCS	AASHTO
Dark greenish gray poorly-graded sand with silt (SP-SM)		

Project No. 190 GS-18-D **Client:** Rick Engineering Company

Project: South Mission Beach Project

Source of Sample: B-1 **Depth:** 5 **Sample Number:** 1

Remarks:

Allied Geotechnical Engineers, Inc.

Santee, CA

Figure B-8

Tested By: Nicholas Barnes **Checked By:** Sani Sutanto

L A B O R A T O R Y   R E P O R T

Telephone (619) 425-1993   Fax 425-7917   Established 1928

C L A R K S O N   L A B O R A T O R Y   A N D   S U P P L Y   I N C.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
A N A L Y T I C A L   A N D   C O N S U L T I N G   C H E M I S T S

Date: February 19, 2019
Purchase Order Number: 190GS18-D
Sales Order Number: 43345
Account Number: ALLG
To:
--
Allied Geotechnical Engineers
1810 Gillespie Way Ste 104
El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO7200-4   Customers Phone: 449-5900
Fax: 449-5902

Sample Designation:
--
One soil sample received on 02/15/19 at 9:00am,
from South Mission Beach Green Infrastructure Project marked as B-4#4@10'-11'

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 9.2

Water Added (ml)	Resistivity (ohm-cm)
10	2200
5	1500
5	1100
5	930
5	880
5	750
5	730
5	830
5	840

27 years to perforation for a 16 gauge metal culvert.
35 years to perforation for a 14 gauge metal culvert.
48 years to perforation for a 12 gauge metal culvert.
62 years to perforation for a 10 gauge metal culvert.
75 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417   0.014% (140 ppm)

Water Soluble Chloride Calif. Test 422   0.062% (620 ppm)

Bicarbonate (as $CaCO_3$)   46 ppm
(on a saturated soil paste extract)

Laura Torres

Laura Torres
LT/dbb

L A B O R A T O R Y   R E P O R T

Telephone (619) 425-1993   Fax 425-7917   Established 1928

C L A R K S O N   L A B O R A T O R Y   A N D   S U P P L Y   I N C.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
A N A L Y T I C A L   A N D   C O N S U L T I N G   C H E M I S T S

Date: February 19, 2019
Purchase Order Number: 190GS18-D
Sales Order Number: 43345
Account Number: ALLG
To:

Allied Geotechnical Engineers
1810 Gillespie Way Ste 104
El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO7200-3   Customers Phone: 449-5900
Fax: 449-5902
Sample Designation:

One soil sample received on 02/15/19 at 9:00am,
from South Mission Beach Green Infrastructure Project marked as B-3#3@9'-10'

Analysis By California Test 643, 1999, Department of Transportation
Division of Construction, Method for Estimating the Service Life of
Steel Culverts.

pH 9.3

Water Added (ml)	Resistivity (ohm-cm)
10	39000
5	29000
5	19000
5	14000
5	10000
5	8800
5	7700
5	8300
5	9300

71 years to perforation for a 16 gauge metal culvert.
92 years to perforation for a 14 gauge metal culvert.
127 years to perforation for a 12 gauge metal culvert.
162 years to perforation for a 10 gauge metal culvert.
198 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate	Calif. Test 417	0.003% (30 ppm)
Water Soluble Chloride	Calif. Test 422	0.003% (30 ppm)

Bicarbonate (as $CaCO_3$)   66 ppm
(on a saturated soil paste extract)

Laura Torres

Laura Torres
LT/dbb

L A B O R A T O R Y   R E P O R T

Telephone (619) 425-1993   Fax 425-7917   Established 1928

C L A R K S O N   L A B O R A T O R Y   A N D   S U P P L Y   I N C.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
A N A L Y T I C A L   A N D   C O N S U L T I N G   C H E M I S T S

Date: February 19, 2019
Purchase Order Number: 190GS18-D
Sales Order Number: 43345
Account Number: ALLG
To:
--
Allied Geotechnical Engineers
1810 Gillespie Way Ste 104
El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO7200-2   Customers Phone: 449-5900
Fax: 449-5902
Sample Designation:
--
One soil sample received on 02/15/19 at 9:00am,
from South Mission Beach Green Infrastructure Project marked as B-2#3@8'-9'

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 9.3

Water Added (ml)	Resistivity (ohm-cm)
10	13000
5	9500
5	6900
5	5100
5	4000
5	3500
5	3200
5	3500
5	3600

49 years to perforation for a 16 gauge metal culvert.
64 years to perforation for a 14 gauge metal culvert.
89 years to perforation for a 12 gauge metal culvert.
113 years to perforation for a 10 gauge metal culvert.
138 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417   0.007% (70 ppm)

Water Soluble Chloride Calif. Test 422   0.005% (50 ppm)

Bicarbonate (as $CaCO_3$)   66 ppm
(on a saturated soil paste extract)

Laura Torres

Laura Torres
LT/dbb

L A B O R A T O R Y   R E P O R T

Telephone (619) 425-1993   Fax 425-7917   Established 1928

C L A R K S O N   L A B O R A T O R Y   A N D   S U P P L Y   I N C.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
A N A L Y T I C A L   A N D   C O N S U L T I N G   C H E M I S T S

Date: February 19, 2019
Purchase Order Number: 190GS18-D
Sales Order Number: 43345
Account Number: ALLG
To:
--
Allied Geotechnical Engineers
1810 Gillespie Way Ste 104
El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO7200-1   Customers Phone: 449-5900
Fax: 449-5902
Sample Designation:
--
One soil sample received on 02/15/19 at 9:00am,
from South Mission Beach Green Infrastructure Project
marked as B-1#4@14'-15'.

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 8.3

Water Added (ml)	Resistivity (ohm-cm)
20	270
5	220
5	140
5	130
5	130
5	130
5	130
5	140
5	160

13 years to perforation for a 16 gauge metal culvert.
17 years to perforation for a 14 gauge metal culvert.
24 years to perforation for a 12 gauge metal culvert.
30 years to perforation for a 10 gauge metal culvert.
37 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417   0.105% (1050 ppm)

Water Soluble Chloride Calif. Test 422   0.363% (3630 ppm)

Bicarbonate (as $CaCO_3$)   46 ppm
(on a saturated soil paste extract)

Laura Torres

Laura Torres
LT/dbb

SCST, LLC - San Diego
LEA: 47, Exp: 04/25/2021
6280 Riverdale Street
San Diego, CA 92120
Phone: (619) 280-4321
Fax: (619) 280-4717

R-Value

Cal 301, ASTM D2844

Report Date: 3/11/2019

Client:	**Project:**
Allied Geotechnical Engineering 9500 Cuyamaca Street #102 Santee, CA 92071-2685	180035L Allied Geotechnical 2018 Lab Testing 9500 Cuyamaca Street Suite 102 Santee CA 9207...

In accordance with your request, SCST has performed the subject laboratory testing. Test results are presented in the attached report.

If you have any additional questions or concerns, please contact us at 619.280.4321

Respectfully Submitted,
SCST, Inc.

In accordance with your request, SCST has performed the subject laboratory testing. Test results are presented in the attached report.

If you have any additional questions or concerns, please contact us at 619.280.4321

Respectfully Submitted,
SCST, Inc.

In accordance with your request, SCST has performed the subject laboratory testing. Test results are presented in the attached report. See R-Value 37891.pdf in the documents section at the end of this report.

If you have any additional questions or concerns, please contact us at 619.280.4321

Respectfully Submitted,
SCST, Inc.

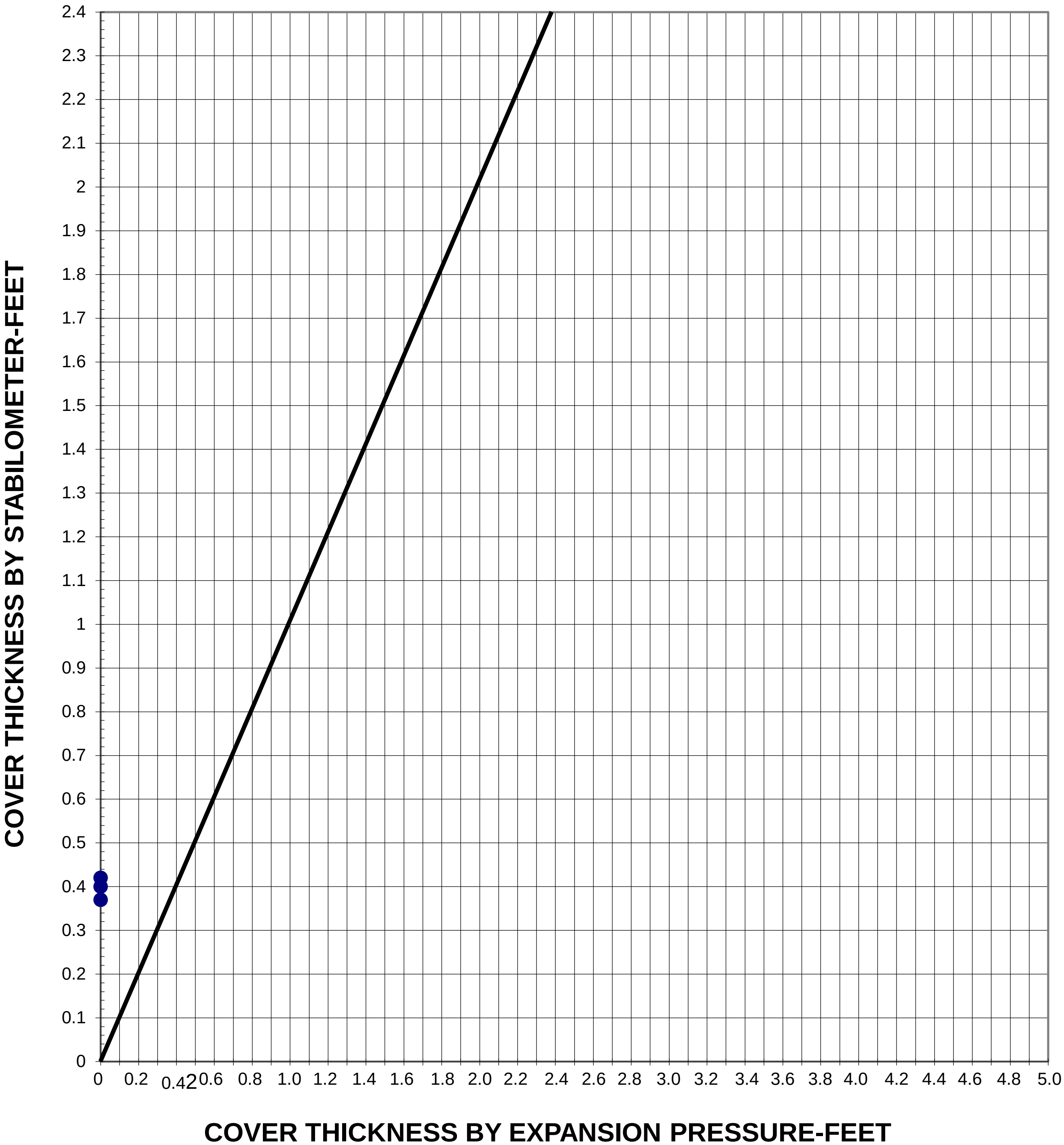
SCST, LLC
Corporate Headquarters
6280 Riverdale Street
San Diego, CA 92120
T 877.215.4321
P 619.280.4321
F 619.280.4717
W www.scst.com

Job Name:	Allied Geotechnical 2018 Lab Testing	**Job Number:**	180035L
Client:	Allied Geotechnical Engineering	**Sample No.:**	37891
Date:	3/5/2019	**By:**	DRB
Location:	B-4-2 @ 5'-8'		
Description:	Light Tan Sand		

CTM 301 Resistance Value of Treated and Untreated Bases, Subbases and Basement Soils

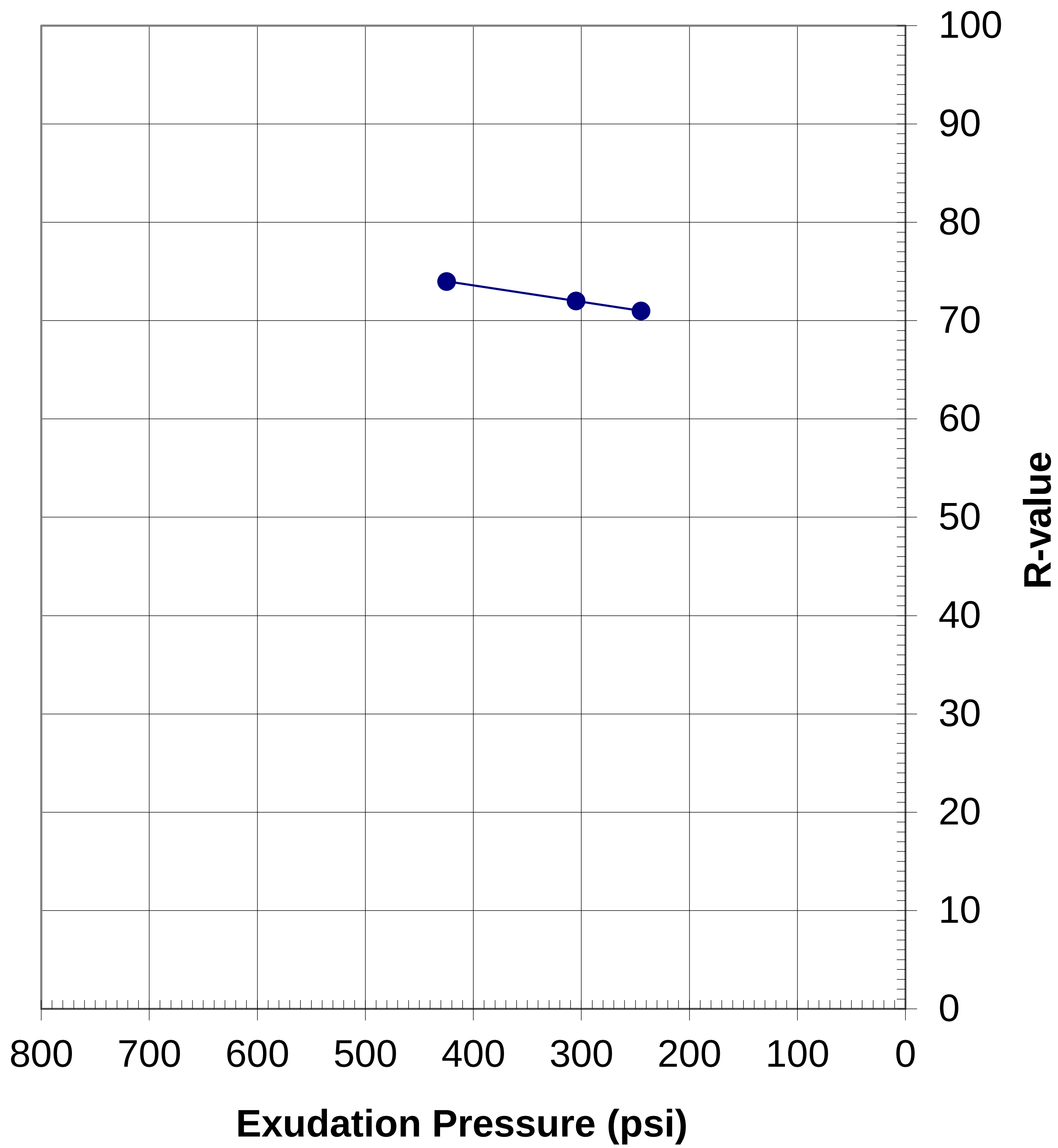
Test Specimen		A	B	C	D
Date Tested		3/5/2019	3/5/2019	3/5/2019	
Compactor Air Pressure	PSI	350	350	350	
Initial Moisture	%	0.4	0.4	0.4	
Soil Wt. Added	GRAMS	850	850	850	
Water Added	ML	90	103	84	
Water Added	%	10.6	12.2	9.9	
Moisture At Compaction	%	11	12.6	10.3	
Weight of Briquette & Tare	GRAMS	2983	2985	2979	
Net Weight of Briquette	GRAMS	929	944	924	
Briquette Height	IN	2.49	2.52	2.52	
Density	PCF	101.8	100.8	100.7	
Exudation Pressure	PSI	305	245	425	
Expansion Pressure	PSF	0	0	0	
PH at 1000 Pounds	PSI	13	13	12	
PH at 2000 Pounds	PSI	23	24	22	
Displacement	Turns	5.65	5.70	5.60	
R' Value		72	71	74	
Stabilometer Thickness	FT	0.4	0.42	0.37	
Expansion Thickness	FT	0	0	0	
Expansion Dial Reading		0000	0000	0000	
R' Value Modifier		0	0	0	
Corrected R-Value		72	71	74	
R-Value by Exudation Pressure			72		
Gravel Equivalent		0	0	0	
Traffic Index		4.5	4.5	4.5	
R-Value by Expansion Pressure			N/A		
R-Value at Equivalent			**72**		

EXPANSION PRESSURE CHART



SC ST	Job Name:	Allied Geotechnical 2018 Lab Testing		
	By:	DRB	Date:	3/5/2019
	Job No.:	180035L	Sample No.:	B-4-2 @ 5'-8'
	Gravel Equ:	0	Plate No.:	

R-value By Exudation Pressure



Job Name:	Allied Geotechnical 2018 Lab Testing		
By:	DRB	Date:	3/5/2019
Job No.:	180035L	Sample No.:	B-4-2 @ 5'-8'
R-Value by Ex.:	72	Plate No.:	

SCST, LLC - San Diego
LEA: 47, Exp: 04/25/2021
6280 Riverdale Street
San Diego, CA 92120
Phone: (619) 280-4321
Fax: (619) 280-4717

R-Value

Cal 301, ASTM D2844

Report Date: 3/11/2019

Client:	**Project:**
Allied Geotechnical Engineering 9500 Cuyamaca Street #102 Santee, CA 92071-2685	180035L Allied Geotechnical 2018 Lab Testing 9500 Cuyamaca Street Suite 102 Santee CA 9207...

In accordance with your request, SCST has performed the subject laboratory testing. Test results are presented in the attached report.

If you have any additional questions or concerns, please contact us at 619.280.4321

Respectfully Submitted,
SCST, Inc.

In accordance with your request, SCST has performed the subject laboratory testing. Test results are presented in the attached report.

If you have any additional questions or concerns, please contact us at 619.280.4321

Respectfully Submitted,
SCST, Inc.

In accordance with your request, SCST has performed the subject laboratory testing. Test results are presented in the attached report. See R-Value 37892.pdf in the documents section at the end of this report.

If you have any additional questions or concerns, please contact us at 619.280.4321

Respectfully Submitted,
SCST, Inc.

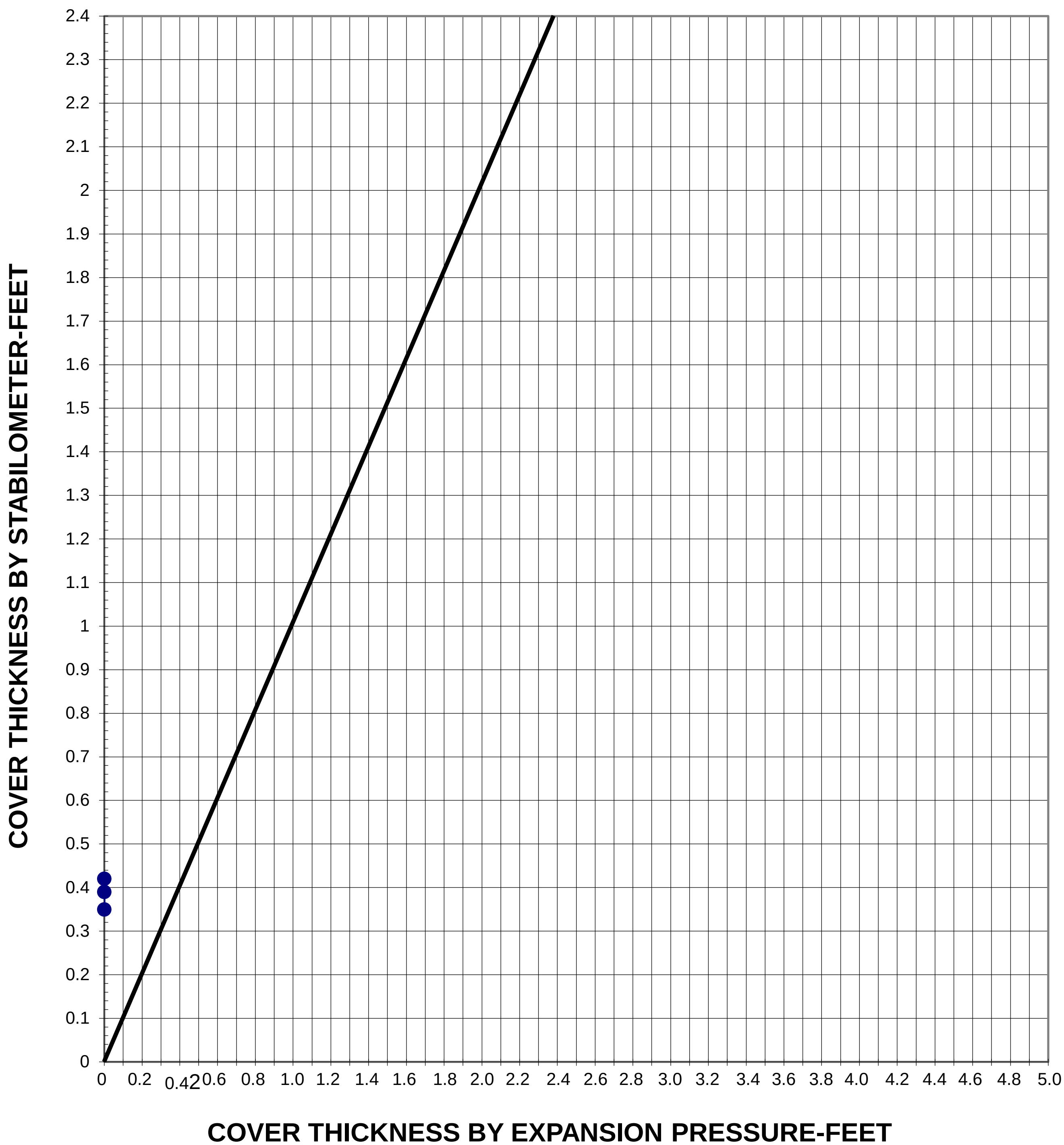
SCST, LLC
Corporate Headquarters
6280 Riverdale Street
San Diego, CA 92120
T 877.215.4321
P 619.280.4321
F 619.280.4717
W www.scst.com

Job Name:	Allied Geotechnical 2018 Lab Testing	**Job Number:**	180035L
Client:	Allied Geotechnical Engineering	**Sample No.:**	37892
Date:	3/5/2019	**By:**	DRB
Location:	B-2-2 @ 5'-8'		
Description:	Light Grey Brown Silty Sand		

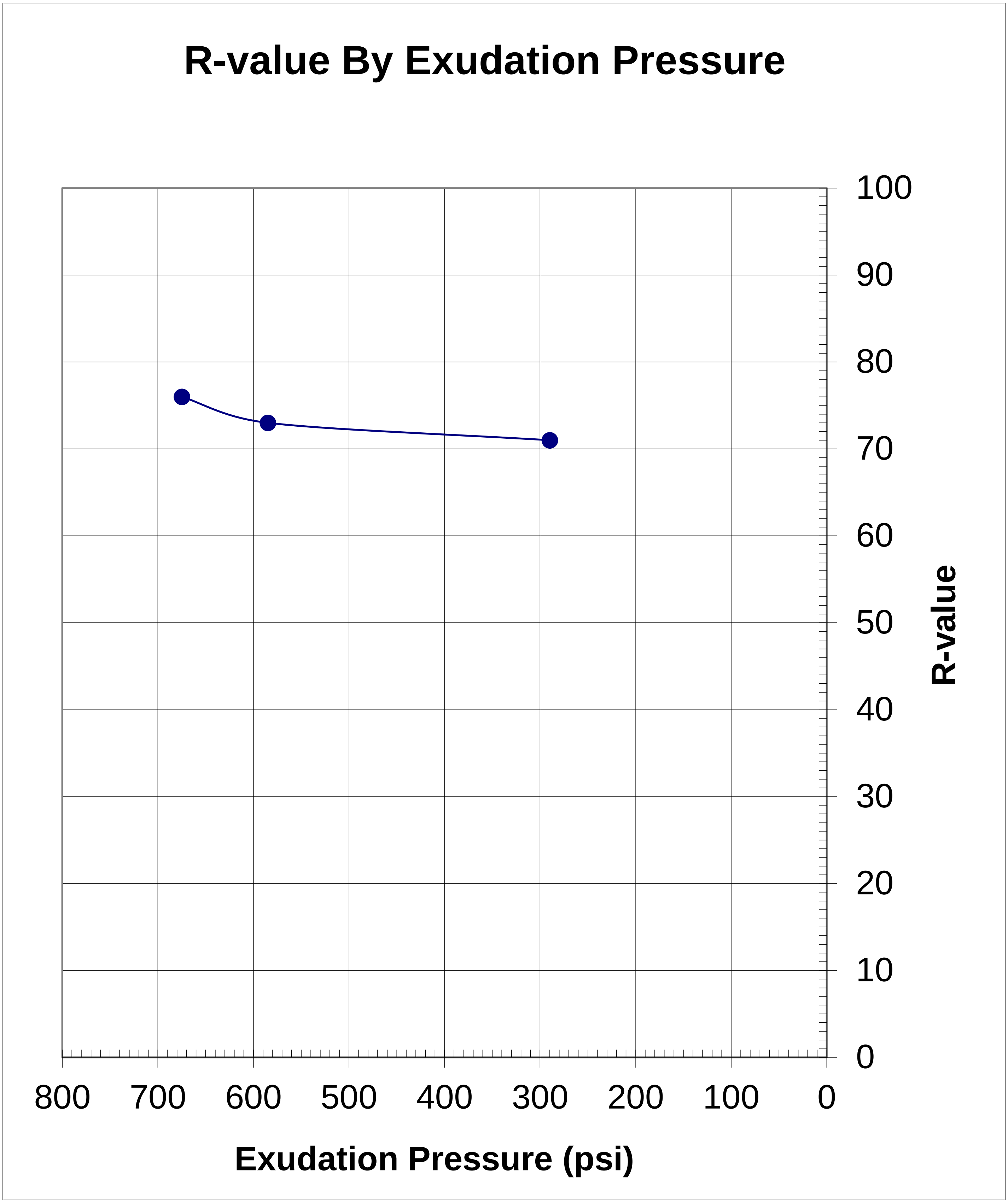
CTM 301 Resistance Value of Treated and Untreated Bases, Subbases and Basement Soils

Test Specimen		A	B	C	D
Date Tested		3/5/2019	3/5/2019	3/5/2019	
Compactor Air Pressure	PSI	350	350	350	
Initial Moisture	%	0.7	0.7	0.7	
Soil Wt. Added	GRAMS	910	900	890	
Water Added	ML	85	95	108	
Water Added	%	9.4	10.6	12.2	
Moisture At Compaction	%	10.1	11.3	12.9	
Weight of Briquette & Tare	GRAMS	3100	3097	3101	
Net Weight of Briquette	GRAMS	986	985	989	
Briquette Height	IN	2.56	2.53	2.49	
Density	PCF	106.0	106.0	106.6	
Exudation Pressure	PSI	675	585	290	
Expansion Pressure	PSF	0	0	0	
PH at 1000 Pounds	PSI	14	14	15	
PH at 2000 Pounds	PSI	24	25	26	
Displacement	Turns	5.00	5.10	5.20	
R' Value		74	73	71	
Stabilometer Thickness	FT	0.35	0.39	0.42	
Expansion Thickness	FT	0	0	0	
Expansion Dial Reading		0000	0000	0000	
R' Value Modifier		2	0	0	
Corrected R-Value		76	73	71	
R-Value by Exudation Pressure			71		
Gravel Equivalent		0	0	0	
Traffic Index		4.5	4.5	4.5	
R-Value by Expansion Pressure			N/A		
R-Value at Equivalent			**71**		

EXPANSION PRESSURE CHART



Job Name:	Allied Geotechnical 2018 Lab Testing		
By:	DRB	Date:	3/5/2019
Job No.:	180035L	Sample No.:	B-2-2 @ 5'-8'
Gravel Equ:	0	Plate No.:	



Job Name:	Allied Geotechnical 2018 Lab Testing		
By:	DRB	Date:	3/5/2019
Job No.:	180035L	Sample No.:	B-2-2 @ 5'-8'
R-Value by Ex.:	71	Plate No.:	

APPENDIX C

RCP PIPES BUOYANCY CALCULATIONS

Project No. 190 GS-18-D
Appendix C, Sheet 1

South Mission Beach Green Infrastructure RCP Pipes Buoyancy Calculations

Pipe Material: Reinforced Concrete
Pipe Dimensions:

OD	= 54 inches = 4.5 feet
ID	= 48 inches = 4.0 feet
Conc. Unit Weight	= 150 pcf
Compacted Fill Unit Weight Above Water	= 130 pcf
Compacted Fill Unit Weight Below Water	= 125 pcf (include 62.4 pcf for unit weight of water)
Water Unit Weight	= 62.4 pcf

Note: Assume pipe in empty for calculation purposes & GW level at ground surface elevation.

Pipe Weight per Unit Length =
$((3.14 \times 2.25^2) - (3.14 \times 2.0^2))$ cf/ft x 150 pcf =
(15.9 - 12.6) cf/ft x 150 pcf =
3.3 cf/ft x 150 pcf = 495.0 pounds/ft

Bouyant Uplift per Unit Length of Pipe =
(3.14×2.25^2) cf/ft x 62.4 pcf = 992.2 pounds/ft

Excess Bouyant Uplift =
992.2 pounds/ft - 495.0 pounds/ft = 497.2 pounds/ft

Minimum Thickness of Compacted Backfill Cover = (497.2 pounds/ft / (4.5 feet x 125 pcf)) x 1.5 (F.S.) =
1.325 foot rounded to 1.5 foot.

South Mission Beach Green Infrastructure RCP Pipes Buoyancy Calculations

Pipe Material: Reinforced Concrete
Pipe Dimensions:

OD	= 42 inches = 3.5 feet
ID	= 36 inches = 3.0 feet
Conc. Unit Weight	= 150 pcf
Compacted Fill Unit Weight Above Water	= 130 pcf
Compacted Fill Unit Weight Below Water	= 125 pcf (include 62.4 pcf for unit weight of water)
Water Unit Weight	= 62.4 pcf

Note: Assume pipe in empty for calculation purposes & GW level at ground surface elevation.

Pipe Weight per Unit Length =
$((3.14 \times 1.75^2) - (3.14 \times 1.5^2))$ cf/ft x 150 pcf =
(9.6 - 7.1) cf/ft x 150 pcf =
2.5 cf/ft x 150 pcf = 375 pounds/ft

Bouyant Uplift per Unit Length of Pipe =
(3.14×1.75^2) cf/ft x 62.4 pcf = 600.0 pounds/ft

Excess Bouyant Uplift =
600.0 pounds/ft - 375.0 pounds/ft = 225.0 pounds/ft

Minimum Thickness of Compacted Backfill Cover = (225.0 pounds/ft / (3.5 feet x 125 pcf)) x 1.5 (F.S.) =
0.77 foot rounded to 1.0 foot.

Project No. 190 GS-18-D
Appendix C, Sheet 3

South Mission Beach Green Infrastructure RCP Pipes Buoyancy Calculations

Pipe Material: Reinforced Concrete
Pipe Dimensions:
OD = 36 inches = 3.0 feet
ID = 30 inches = 2.5 feet
Conc. Unit Weight = 150 pcf
Compacted Fill Unit Weight Above Water = 130 pcf
Compacted Fill Unit Weight Below Water = 125 pcf (include 62.4 pcf for unit weight of water)
Water Unit Weight = 62.4 pcf

Note: Assume pipe in empty for calculation purposes & GW level at ground surface elevation.

Pipe Weight per Unit Length =
((3.14×1.5^2) - (3.14×1.25^2)) cf/ft x 150 pcf =
(7.1 - 4.9) cf/ft x 150 pcf =
2.2 cf/ft x 150 pcf = 330.0 pounds/ft

Bouyant Uplift per Unit Length of Pipe =
(3.14×1.5^2) cf/ft x 62.4 pcf = 443.0 pounds/ft

Excess Bouyant Uplift =
443.0 pounds/ft - 330.0 pounds/ft = 113.0 pounds/ft

Minimum Thickness of Compacted Backfill Cover = (113.0 pounds/ft / (3 feet x 125 pcf)) x 1.5 (F.S.) =
0.452 foot rounded to 0.5 foot.

Project No. 190 GS-18-D
Appendix C, Sheet 4

South Mission Beach Green Infrastructure RCP Pipes Buoyancy Calculations

Pipe Material: Reinforced Concrete
Pipe Dimensions:
OD = 22 inches = 1.83 feet
ID = 18 inches = 1.5 feet
Conc. Unit Weight = 150 pcf
Compacted Fill Unit Weight Above Water = 130 pcf
Compacted Fill Unit Weight Below Water = 125 pcf (include 62.4 pcf for unit weight of water)
Water Unit Weight = 62.4 pcf

Note: Assume pipe in empty for calculation purposes & GW level at ground surface elevation.

Pipe Weight per Unit Length =
$((3.14 \times 0.915^2) - (3.14 \times 0.75^2))$ cf/ft x 150 pcf =
(2.64 - 1.77) cf/ft x 150 pcf =
0.87 cf/ft x 150 pcf = 130.5 pounds/ft

Bouyant Uplift per Unit Length of Pipe =
(3.14×0.915^2) cf/ft x 62.4 pcf = 164.7 pounds/ft

Excess Bouyant Uplift =
164.7 pounds/ft - 130.5 pounds/ft = 34.2 pounds/ft

Minimum Thickness of Compacted Backfill Cover = (34.2 pounds/ft / (1.83 feet x 125 pcf)) x 1.5 (F.S.) =
0.22 foot rounded to 0.5 foot.

November 27, 2019

Mr. Kevin Gibson, P.E.
Project Manager
Rick Engineering Company
5620 Friars Road
San Diego, CA

Subject: **RESPONSE TO CITY OF SAN DIEGO DEVELOPMENT SERVICES DEPARTMENT REVIEW COMMENTS PERTAINING TO REPORT OF GEOTECHNICAL INVESTIGATION SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT CITY OF SAN DIEGO AGE Project No. 190 GS-18-D**

Dear Kevin,

This letter provides our response to Mr. Kreg Mills of the City of San Diego Development Services Department review comments dated October 7, 2019. The general and specific comments that we received and our response are presented in the table below.

No.	Development Services Department Comment	AGE Response
1	Updated Report of Geotechnical Investigation, South Mission Beach Green Infrastructure Project, City of San Diego, prepared by Allied Geotechnical Engineers, Inc. dated March 15, 2019 (their project no. 170446P4.1) Development Plans for South Mission Beach Storm Drain and Green Infrastructure, Drawing no. 41306-D, prepared by Rick Engineering Company for the City of San Diego Transportation and Storm Water, dated August 2, 2019	Reference only.
2	The project's geotechnical consultant must submit an addendum geotechnical report or update letter for the purpose of an environmental review that specifically addresses the proposed development plans and the following:	An updated report which specifically addresses the 100% Submittal project plans prepared by Rick Engineering Company is attached.

No.	Development Services Department Comment	AGE Response
3	To delineate the proposed footprint of the project, the project's geotechnical consultant should clarify if remedial grading will be anticipated outside the limits of grading or limits of work currently shown on the referenced development plans.	Remedial grading outside the limit of proposed excavations for the subject project is not anticipated (first paragraph of Section 5.5 - Cut-and-Cover Construction page 19 of the Updated Geotechnical Repor).
4	The development plans indicate Infiltration is proposed for the bioretention basins. The geotechnical consultant must address the feasibility of on-site storm water disposal/ infiltration systems and potential impacts regarding fill settlement, piping of soil, and premature failure of pavement. The geotechnical consultant should indicate whether or not the proposed on-site storm water disposal/ infiltration systems will have adverse impacts on adjacent properties.	The basins are not anticipated to cause settlement, soil piping, premature pavement failure and/or adverse impacts on adjacent properties. A discussion is presented in Section 5.7 - Infiltration Testing and Basins on page 26 of the Updated Geotechnical Report.
5	The project's geotechnical consultant should provide a conclusion regarding if the proposed development will destabilize or result in settlement of adjacent property or the right of way.	Based on a review of the 100% Submittal project plans prepared by Rick Engineering, the proposed project is not anticipated to destabilize or results in settlement of adjacent property or the right-of-way (first paragraph of Section 5.1 - Potential Geologic Hazards page 13 of the Updated Geotechnical Report).
6	Submit a digital copy (on CD or USB data storage device) of all geotechnical reports submitted for review with the next re-submittal.	Reference only.

If you have any questions regarding the contents of this letter or if we may be of further assistance, please feel free to give us a call.

Very truly yours,

ALLIED GEOTECHNICAL ENGINEERS, INC.

Sani Sutanto, P.E.
Project Manager

REGISTERED PROFESSIONAL ENGINEER
SANI SUTANTO
No. 51873
Exp. 06/30/2020
CIVIL
STATE OF CALIFORNIA

SS/TJL:cal

Review Information

Cycle Type:	2 Submitted (Multi-Discipline)	**Submitted:**	08/30/2019	Deemed Complete on 08/30/2019
Reviewing Discipline:	LDR-Geology	**Cycle Distributed:**	08/30/2019	
Reviewer:	**Mills, Kreg**	**Assigned:**	09/03/2019	
	(619) 446-5295	**Started:**	09/18/2019	
	Kmills@sandiego.gov	**Review Due:**	09/30/2019	
Hours of Review:	4.00	**Completed:**	09/24/2019	**COMPLETED ON TIME**
Next Review Method:	Submitted (Multi-Discipline)	**Closed:**	10/07/2019	

. The review due date was changed to 10/03/2019 from 10/03/2019 per agreement with customer.
. The reviewer has indicated they want to review this project again. Reason chosen by the reviewer: First Review Issues.
. We request a 2nd complete submittal for LDR-Geology on this project as: Submitted (Multi-Discipline).
. The reviewer has requested more documents be submitted.
. Your project still has 5 outstanding review issues with LDR-Geology (all of which are new).
. Last month LDR-Geology performed 66 reviews, 92.4% were on-time, and 76.8% were on projects at less than < 3 complete submittals.

646245-2 (9/24/2019)

REFERENCES REVIEWED:

☒ 1 Updated Report of Geotechnical Investigation, South Mission Beach Green Infrastructure Project, City of San Diego, prepared by Allied Geotechnical Engineers, Inc. dated March 15, 2019 (their project no. 170446P4.1) Development Plans for South Mission Beach Storm Drain and Green Infrastructure, Drawing no. 41306-D, prepared by Rick Engineering Company for the City of San Diego Transportation and Storm Water, dated August 2, 2019

REVIEW COMMENTS:

☐ 2 The project's geotechnical consultant must submit an addendum geotechnical report or update letter for the purpose of an environmental review that specifically addresses the proposed development plans and the following: (New Issue)

☐ 3 To delineate the proposed footprint of the project, the project's geotechnical consultant should clarify if remedial grading will be anticipated outside the limits of grading or limits of work currently shown on the referenced development plans. (New Issue)

☐ 4 The development plans indicate Infiltration is proposed for the bioretention basins. The geotechnical consultant must address the feasibility of on-site storm water disposal/ infiltration systems and potential impacts regarding fill settlement, piping of soil, and premature failure of pavement. The geotechnical consultant should indicate whether or not the proposed on-site storm water disposal/ infiltration systems will have adverse impacts on adjacent properties. (New Issue)

☐ 5 The project's geotechnical consultant should provide a conclusion regarding if the proposed development will destabilize or result in settlement of adjacent property or the right of way. (New Issue)

☐ 6 Submit a digital copy (on CD or USB data storage device) of all geotechnical reports submitted for review with the next re-submittal. (New Issue)

UPDATED REPORT OF GEOTECHNICAL INVESTIGATION SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT CITY OF SAN DIEGO

Submitted to:

RICK ENGINEERING COMPANY
5620 Friars Road
San Diego, CA

Prepared By:

ALLIED GEOTECHNICAL ENGINEERS, INC.
9500 Cuyamaca Street, Suite 102
Santee, California 92071-2685

AGE Project No. 190 GS-18-D

March 8, 2019
(Updated November 27, 2019)

March 8, 2019
(Updated November 27, 2019)

Mr. Kevin Gibson, P.E.
Project Manager
Rick Engineering Company
5620 Friars Road
San Diego, CA

Subject: **UPDATED REPORT OF GEOTECHNICAL INVESTIGATION**
SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT
CITY OF SAN DIEGO
AGE Project No. 190 GS-18-D

Dear Mr. Gibson:

Allied Geotechnical Engineers, Inc. is pleased to submit the accompanying updated report to present the findings, opinions, and recommendations of a geotechnical investigation that was performed to assist Rick Engineering Company with their design of the subject project. We have reviewed the 100% Submittal plans for the subject project, prepared by Rick Engineering Company, undated. It is our opinion that the 100% Submittal project plans were prepared in conformance with the design recommendations provided herein. This report incorporates our response to the review comments that we received from the City of San Diego Development Services Department dated October 7, 2019.

We appreciate the opportunity to be of service on this project. If you have any questions regarding the contents of this report or need further assistance, please feel free to contact our office.

Sincerely,

ALLIED GEOTECHNICAL ENGINEERS, INC.

Nicholas E. Barnes, P.G., C.E.G.
Senior Geologist

Sani Sutanto, P.E.
Project Manager

PROFESSIONAL GEOLOGIST No. 5383
Nicholas E. Barnes
No. 1893
CERTIFIED ENGINEERING GEOLOGIST
Exp. 05/31/2020
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
SANI SUTANTO
No. 51873
Exp. 06/30/2020
CIVIL
STATE OF CALIFORNIA

NEB/SS/TJL:cal
Distr. (1 electronic) Addressee

9500 Cuyamaca Street, Suite 102 ■ Santee, California 92071-2685 Phone 619.449.5900 Fax 619.449.5902

UPDATED REPORT OF GEOTECHNICAL INVESTIGATION
SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT
CITY OF SAN DIEGO

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Figures

Appendices

1.0 INTRODUCTION

Allied Geotechnical Engineers, Inc. (AGE) is pleased to submit this report to present the findings, opinions, and recommendations of a geotechnical investigation conducted to assist Rick Engineering Company (Rick Engineering) with their design of the South Mission Beach Green Infrastructure Project for the City of San Diego (City). The investigation was performed in conformance with AGE's proposal dated July 11, 2018 (revised July 24, 2018), and the subconsultant agreement entered into by and between Rick Engineering and AGE on November 1, 2018. This report was previously updated on September 10, 2019 to incorporate the results of a subsurface geotechnical investigation performed by Southern California Soil and Testing, Inc. (SCS&T), dated April 16, 2019 and to provide additional recommendations to mitigate the groundwater bouyant uplift forces on the proposed storm drain pipelines. The current updated report incorporates the results of our review of Rick Engineering's 100% Submittal project plans and response to City of San Diego Development Services Department review comments dated October 7, 2019.

This report has been prepared for the exclusive use of Rick Engineering and its design team and the City in their design of the project as described herein. The information presented in this report is not sufficient for any other uses or the purposes of other parties.

2.0 SITE AND PROJECT DESCRIPTION

The project alignments Map is shown on Figure 1. Based on a review of the 100% Submittal plans prepared by Rick Engineering Company, undated, it is our understanding that the scope of the proposed project will include the following:

- design and construction of approximately 88 feet of concrete lined channel;
- design and construction of approximately 6,253 feet of storm drain pipelines;
- design and construction of approximately 142 feet of encased storm drain;
- design and construction of 16 feet of culvert;
- design and construction of five (5) biofiltration basins and three (3) bioretention basins identified as Basins 1 through 8 on the 100% Submittal project plans; and
- design and construction of associated headwalls, inlets, connectors, cleanouts, outlets, tidegates and weep sumps.

The proposed project alignments extend along public right-of-ways in the South Mission Beach area of San Diego. The proposed pipelines will consist of 18-, 30-, 36- and 48-inch diameter reinforced concrete pipes (RCP) and 6-inch diameter PVC pipes. It is anticipated that the proposed pipelines will be installed using conventional cut-and-cover construction method with cover thickness on the order of 2 to 13 feet above the pipe crown.

Existing improvements along the project alignments include a mix of residential and commercial developments as well as Mission Beach and Mission Bay Park. The topography along the project alignments varies from level to very gently sloping with elevations which vary from sea level to approximately 13 feet above mean sea level (msl).

3.0 OBJECTIVE AND SCOPE OF INVESTIGATION

The objectives of this investigation were to characterize the subsurface conditions along the project alignments and to develop geotechnical recommendations for use in the design of the currently proposed project. The scope of our investigation included several tasks which are described in more detail in the following sections.

3.1 Information Review

This task involved a review of readily available information pertaining to the project study area, including the preliminary project plans, as-built utility maps, topographic maps, published geologic literature and maps, and AGE's in-house references. In addition AGE also perform a review of a report of Geotechnical Investigation prepared by Souther California Soil & Testing for the Mission Beach Water and Sewer Replacement City of San Diego Task #17CD03, dated April 16, 2019.

3.2 Geotechnical Field Exploration

The field exploration program for this project was performed on February 11 and 12, 2019. A total of four (4) soil borings, four (4) infiltration test holes, and two (2) pavement corings were performed at the approximate locations shown on Figures 2 through 5. In addition, AGE attempted to perform infiltration testing inside an existing weep sump located on the west side of Mission Boulevard, at the entrance of an alley located between Brighton Court and Capistrano Place. The soil borings were advanced to depths ranging from 15 feet to 16.5 feet below the existing ground surface (bgs). The infiltration test holes were hand-augured to depths ranging from 36 inches to 63 inches bgs. A brief description of the location and depth, pavement sections, groundwater level, and subsurface conditions encountered in the borings and infiltration test holes is presented in Table 1 on the next page. A more detailed description of the excavation and sampling activities, and logs of the soil borings are presented in Appendix A.

Table 1

Summary of Subsurface Conditions

Boring & Test Hole ID	Location	Depth (Feet)	Existing Pavement Section	Subsurface Conditions	Estimated Groundwater Depth/ Elevation (Feet bgs/feet msl)
B-1	Mission Bay beach, approximately 10 feet east of Bayside Walk at intersection with San Fernando Place.	16.5	N/A	Hydraulic fill to 10 feet and old paralic deposits to the maximum depth of exploration.	11/-3.7
B-2	Southbound Mission Boulevard, approximately 40 feet south of San Fernando Place and 4 feet west of the center median.	15	4" A.C. over 8" P.C.C. underlain by 6" miscellaneous base.	Old paralic deposits to the maximum depth of exploration.	4.25/+2.0
C-2	Southbound Mission Boulevard, approximately 40 feet south of San Fernando Place and 12 feet east of the curb.	N/A	4.5" A.C. over 9.5" P.C.C. Unable to differentiate base materials.	N/A	N/A
B-3	Mission Bay beach, approximately 20 feet east of Bayside Walk at intersection with Coronado Court.	16.5	N/A	Hydraulic fill to 10 feet and old paralic deposits to the maximum depth of exploration.	4/+1.2
B-4	Southbound Mission Boulevard, approximately 60 feet south of Brighton Court and 4 feet west of the center median.	15	4.5" A.C. over 7.5" P.C.C. underlain by 4" miscellaneous base.	Old paralic deposits to the maximum depth of exploration.	3.25/+1.75

Table 1 (continued)

Summary of Subsurface Conditions

Boring & Test Hole ID	Location	Depth (Feet)	Existing Pavement Section	Subsurface Conditions	Estimated Groundwater Depth/ Elevation (Feet bgs/feet msl)
C-4	Northbound Mission Boulevard, approximately 60 feet south of Brighton Court and 12 feet west of the curb.	N/A	6" A.C., 6" P.C.C., 2" miscellaneous base.	N/A	N/A
P-1	Lawn area approximately 30 feet east of Mission Boulevard and 240 feet north of San Fernando Place.	62"	N/A	Four inches of topsoil underlain by old paralic deposits to the maximum depth of exploration.	3'/+2.7'
P-2	Tree planter on east side of Mission Boulevard approximately 20 feet north of Deal Court.	36"	N/A	Twelve inches of topsoil underlain by old paralic deposits to the maximum depth of exploration.	3'/+1.9'
P-3	Tree planter on west side of Mission Boulevard approximately 15 feet north of Balboa Court.	48"	N/A	Twelve inches of topsoil underlain by old paralic deposits to the maximum depth of exploration.	3'/+1.5'
P-4	Lawn area approximately330 feet east of Mission Boulevard and 10 feet south of Bayside Lane.	63"	N/A	Three inches of topsoil underlain by old paralic deposits to the maximum depth of exploration.	Not encountered.

Prior to commencement of the field exploration activities, several site reconnaissance visits were performed to observe existing conditions and to select suitable locations for the soil borings and infiltration test holes. Subsequently, Underground Service Alert (USA) was contacted to coordinate clearance of the proposed boring and test hole locations with respect to existing buried utilities. The utility clearance effort revealed the presence of the following buried utilities: potable water and sanitary sewer pipelines; storm drains; natural gas and electrical transmission lines; and cable, telephone, and fiber optic lines.

Traffic control permits were obtained from the City of San Diego to perform the borings (B-2 and B-4) and pavement cores (C-2 and C-4) that are located within the public right-of-way. Borings B-1 and B-3, and percolation holes P-1 and P-4 which are located in Mission Bay Park were performed with prior verbal approval from the City of San Diego Parks & Recreation Department.

Due to the presence of shallow groundwater inside test holes P-1, P-2 and P-3, AGE was unable to perform infiltration testing inside these holes. The existing weep sump was installed on top of an existing City of San Diego sewer trench. Furthermore, when AGE attempted to perform the infiltration testing, AGE uncovered an 18-inch diameter green PVC pipe filled with 3/4-inch crushed rock (see photograph in Figure 6). Therefore, AGE was unable to perform infiltration testing inside the weep sump. Infiltration testing was only performed in test hole P-4.

3.3 Laboratory Testing

Selected soil samples obtained from the soil borings were tested in the laboratory to verify field classifications and evaluate certain engineering characteristics. The geotechnical laboratory tests were performed in general conformance with the American Society for Testing and Materials (ASTM) or other generally accepted testing procedures.

The laboratory tests included: in-place density and moisture content, maximum density and optimum moisture content, sieve (wash) analysis, consolidation, shear strength, and R-value. In addition, representative samples of the onsite soil materials were collected and delivered to Clarkson Laboratories and Supply, Inc. for chemical (analytical) testing to determine soil pH and resistivity, soluble sulfate and chloride concentrations, and bicarbonate content.

4.0 GEOLOGIC CONDITIONS

4.1 Geologic Setting and Site Physiography

The project alignments are located in Mission Beach, a narrow sandbar situated between the Pacific Ocean and Mission Bay. The sandbar is underlain by marine sediments which range from Pleistocene to Holocene in age. Hydraulically placed fill materials were added along the eastern and southern portions of the sandbar during development of Mission Bay from the 1940's into the 1950's. Shallow mechanically placed fill materials were also encountered in the study area.

4.2 Tectonic Setting

Tectonically, the San Diego region is situated in a broad zone of northwest-trending, predominantly right-slip faults that span the width of the Peninsular Ranges and extend offshore into the California Continental Borderland Province west of California and northern Baja California. At the latitude of San Diego, this zone extends from the San Clemente fault zone, located approximately 60 miles to the west, and the San Andreas fault located about 95 miles to the east.

Major active regional faults of tectonic significance include the Coronado Bank, San Diego Trough, San Clemente, and Newport Inglewood/Rose Canyon fault zones which are located offshore; the faults in Baja California, including the San Miguel-Vallecitos and Agua Blanca fault zones; and the faults located further to the east in Imperial Valley which include the Elsinore, San Jacinto and San Andreas fault zones.

4.3 Geologic Units

Based on their origin and compositional characteristics, the soil types encountered in the borings can be categorized into two geologic units which include (in order of increasing age) fill materials and old paralic deposits. A brief description of each unit is presented below.

4.3.1 Fill Materials

Hydraulically placed fill materials were encountered in borings B-1 and B-3 to depths of approximately 10 feet bgs. The hydraulic fill generally consists of fine to medium grained sand with silt and containing scattered sub-rounded gravel. During the field investigation we met refusal in boring B-3 on a large buried rock or concrete at a depth of 3 feet bgs. We moved approximately 10 feet to the east and re-drilled to the target depth.

Mechanically placed fill materials on the order of 12 inches or less in thickness were encountered in infiltration test holes P-1 thru P-4. These materials generally consist of silty sands and organic-rich topsoil for lawns and street trees. Documentation pertaining to the original placement of the fill materials is unavailable.

Fill materials were also encountered in SCS&T (2019) borings P-1 through P-3 which are located along Mission Boulevard to depth ranging between 2 to 5 feet bgs. Fill materials were encountered in SCS&T (2019) boring P-4 to the maximum depth of exploration of 21.5 feet bgs. The fill materials encountered in SCS&T (2019) borings posses the same consistency as those encountered in AGE's borings.

4.3.2 Old Paralic Deposits

Late to mid-Pleistocene age old paralic deposits (Kennedy and Tan, 2008) were encountered below fill materials in borings B-1 and B-3, and below paving in borings B-2 and B-4 to the maximum depth of exploration. These deposits are generally described as poorly sorted, moderately permeable, reddish brown interfingered strandline, beach, estuarine and colluvial deposits composed of siltstone, sandstone and conglomerate resting on a now emergent wave-cut platform preserved by regional uplift (Kennedy and Tan, 2008). The deposits can generally be excavated with conventional heavy duty construction equipment. Although not encountered during the field exploration, localized conglomerate layers may present difficult excavation conditions.

The old paralic deposits encountered in our test borings generally consisted of fine-to medium grained sands and silty sands with scattered to trace amounts of sub-rounded gravel and shell fragments. The soil deposits are generally uncemented, damp to wet, and in a medium dense to dense condition.

Old paralic deposits were encountered below the fill materials in SCS&T (2019) borings P-1 and P-3 to the maximum depth of exploration which ranges between 20.5 and 21 feet bgs. The old paralic deposits encountered in SCS&T (2019) borings posses the same consistency as those encountered in AGE's borings.

4.4 Groundwater

At the time of our field investigation, groundwater was measured in the soil borings and test holes at depths ranging from 3 feet to 11 feet bgs (approximate elevations -2 feet to +7 feet msl). Tidal coefficients in Mission Bay (Quivira Basin) on the days of the field exploration based on National Oceanic and Atmospheric Administration (NOAA) data are shown on the next page.

Date	Low Tide		High Tide	
	Time	Height (MLLW) Height (MSL)	Time	Height (MLLW) Height (MSL)
02/11/2019	7:34 am	-1.7 feet -4.5 feet	1:03 pm	+3.0 feet + 0.2 feet
02/12/2019	9:15 am	-1.5 feet -4.3 feet	3:07 pm	+2.6 feet +0.2 feet
02/13/2019	10:47 am	-0.9 feet -3.7 feet	5:18 pm	+2.7 feet -0.1 feet

No groundwater was encountered in infiltration hole P-4. Fill and formational materials encountered in the soil borings and infiltration test holes are generally considered to possess very high permeability characteristics. Based on the anticipated depth of excavations, it is anticipated that groundwater will be encountered along the project alignments during construction.

Groundwater was encountered in borings SCS&T (2019) borings between elevations + 1 and +2 feet msl. Monitoring with vibrating wire piezometer between March 5 and April 9l 2019 inside the borings indicate groundwater level fluctuation on the order of 0.5 to 0.75 foot.

Groundwater flow rate is anticipated to be low due to the flat topography. Depending on the locations within the project study area, groundwater flow is anticipated to be toward the Pacific Ocean or San Diego Bay. Based on the results of our study, shallow groundwater and highly permeable soil materials are present beneath the project alignment. Given these site conditions, significant groundwater inflows can be expected in anticipated deep excavations required for the construction project unless adequate engineering measures are taken to mitigate the groundwater inflow. It must be noted that large variations in the elevation of the groundwater table should be expected in response to seasonal and tidal fluctuations in San Diego Bay and the Pacific Ocean.

5.0 DISCUSSIONS, OPINIONS AND RECOMMENDATIONS

5.1 Potential Geologic Hazards

The majority of the project study area is classified in the City of San Diego Seismic Safety Study (2008), as Hazard Category 52 - Other Terrain, defined as, "Other level areas, gently sloping to steep terrain, favorable geologic structure, Low Risk". The beach area in the eastern portion of the study area, as well as Mission Point Park in the southeast portion of the study area adjacent to the Mission Bay Channel is classified as Hazard Category 31 - Liquefaction, defined as, "High potential-Shallow groundwater, major drainages, hydraulic fills. Neither classifications are anticipated to affect the proposed project as described herein. Based on a review of the 100% Submittal project plans prepared by Rick Engineering, the proposed project is not anticipated to destabilize or results in settlement of adjacent property or the right-of-way.

5.1.1 Faulting

The northwest trending Point Loma fault is mapped 2,000 feet east of the project study area (Kennedy, 1975; Kennedy and Tan, 2008), This fault is concealed below Mission Bay and Holocene age fill materials east of the project study area. To the southeast the mapped trace of the fault crosses the Point Loma peninsula, where it is concealed beneath Pleistocene age old paralic deposits. The Point Loma fault is classified in the City of San Diego Seismic Safety Study (2008) as "potentially active, inactive, presumed inactive, or activity unknown."

For the purpose of this project we consider the Rose Canyon fault zone (RCFZ) to represent the most significant seismic hazard. The RCFZ is a complex set of anastomosing and en-echelon, predominantly strike slip faults that extend from off the coast near Carlsbad to offshore south of downtown San Diego (Treiman, 1993). Previous geologic investigations on the RCFZ in the Rose

Creek area (Rockwell et. al., 1991) and in downtown San Diego (Patterson et. al., 1986) found evidence of multiple Holocene earthquakes. Based on these studies, several fault strands within the RCFZ have been classified as active faults, and are included in Alquist-Priolo Special Studies Zones. In San Diego Bay, this fault zone is believed to splay into multiple, subparallel strands; the most pronounced of which are the Silver Strand, Spanish Bight and Coronado Bank faults.

A study by Kleinfelder (2017) at the San Diego International Airport identified two zones of active faulting. One of these faults was named the East Bay fault and the second fault was determined to be a northward extension of the Spanish Bight fault. Recent study by Ninyo & Moore (2018) at Seaport Village found evidence of recent movement along a fault that was determined to be a northward extension of the active Coronado fault. The project alignments are not located within an Alquist-Priolo Earthquake Study Zone.

5.1.2 Fault Ground Rupture & Ground Lurching

There are no known (mapped) active or potentially active faults crossing the project alignments (Kennedy, 1975; Kennedy and Tan, 2008; City of San Diego, 2008). Therefore, the potential for fault ground rupture and ground lurching along the project alignments is considered insignificant.

5.1.3 Soil Liquefaction

Seismically-induced soil liquefaction is a phenomenon in which loose to medium dense, saturated granular materials undergo matrix rearrangement, develop high pore water pressure, and lose shear strength due to cyclic ground vibrations induced by earthquakes.

Hydraulically placed fill materials in the east and southerly portions of the project alignments are classified in the City of San Diego Seismic Safety Study (2008) as having a high liquefaction potential. The findings of our investigation determined that the hydraulic fill materials encountered in borings B-1 and B-3 are in a medium dense condition, and therefore are considered to have a low liquefaction potential. However, it is likely that liquefaction prone soil materials will be encountered during construction.

5.1.4 Hydroconsolidation

At the time of our field investigation, groundwater was measured in the soil borings and test holes at depths ranging from 3 feet to 11 feet bgs (approximate elevations -2 feet to +7 feet msl). Based on the subsurface conditions encountered during drilling, the results of the consolidation test and the elevation of the groundwater beneath the project study area, hydroconsolidation is not anticipated to pose a hazard within the project study area.

5.1.5 Landslides

A review of the published geologic maps indicates that there are no known (mapped) ancient landslides in the project study area (Kennedy, 1975; Kennedy and Tan, 2008; City of San Diego, 2008). Therefore, landsliding is not considered a significant risk.

5.1.6 Lateral Spread Displacement

The project alignments are located in an area that is flat, therefore, the risk of lateral spread displacement during a seismic event is considered remote.

5.1.7 Differential Seismic-Induced Settlement

Differential seismic settlement occurs when seismic shaking causes one type of soil to settle more than another type. It may also occur within a soil deposit with largely homogeneous properties if the seismic shaking is uneven due to variable geometry or thickness of the soil deposit. Based on the results of our investigation, it is our opinion that there is a slight potential of differential settlement in areas underlain by deep hydraulically placed man-made fills.

5.1.8 Secondary Hazards

The project alignments are located within the tsunami inundation zone (California Geological Survey, 2009). Therefore, there is a high potential of property damage from seismic-induced tsunamis. The project alignments are located within the Special Flood Hazard Areas, 100- and 500-year flood zone (FEMA Flood Insurance Rate Map, 2012). Therefore the potential for flooding along the project alignments is considered high to very high.

5.2 Soil Corrosivity

In accordance with the City of San Diego Water Facility Design Guidelines, Book 2, Chapter 7, soil is generally considered aggressive to concrete if its chloride concentration is greater than 300 parts per million (ppm) or sulfate concentration is greater than 1,000 ppm, or if the pH is 5.5 or less.

Analytical testing was performed on representative sample of the onsite soil materials to determine pH, resistivity, soluble sulfate, chlorides and bicarbonates content. The tests were performed in accordance with California Test Method Nos. 643, 417 and 422. A summary of the test results is presented in Table 2 below. Copies of the analytical laboratory test data reports are included in Appendix B.

Table 2

Summary of Corrosivity Test Results

	pH	Resistivity (ohm-cm)	Sulfate Conc. (ppm)	Chloride Conc. (ppm)	Bicarbonates Conc. (ppm)
B-1 Sample No. 4 @14'-15'	8.3	130	1,050	3,630	46
B-2 Sample No. 3 @8'-9'	9.3	3,200	70	50	66
B-3 Sample No.3 @9'-10'	9.3	7,700	30	30	66
B-4 Sample No. 4 @10'-11'	9.2	730	140	620	46

The test results indicate that some of the soils along the project alignments are considered aggressive to concrete. Therefore, Type 5 Portland Cement Concrete should be used for proposed facilities along the project alignments. It should be noted here that the most effective way to prevent sulfate attack is to keep the sulfate ions from entering the concrete in the first place. This can be done by using mix designs that give a low permeability (mainly by keeping the water/cement ratio low) and, if practical, by placing moisture barriers between the concrete and the soil.

AGE does not practice in the field of corrosion engineering. In the event that corrosion sensitive facilities are planned, we recommend that a corrosion engineer be retained to perform the necessary corrosion protection evaluation and design.

5.3 Expansive Soil

Based on visual observations and soil classifications, the soil materials encountered in the borings and test holes are considered to be non-expansive.

5.4 Fill Material

Fill material for trench backfill should be free of biodegradable material, hazardous substance contamination, other deleterious debris, and or rocks or hard lumps greater than 6 inches. If the fill material contains rocks or hard lumps, at least 70 percent (by weight) of its particles shall pass a U.S. Standard $^3/_4$-inch sieve. Fill material should consists of predominantly granular soil (less than 40 percent passing the U.S. Standard #200 sieve) with Expansion Index of less than 50.

The majority of the onsite soil materials are considered suitable for use as compacted backfill materials. It is noted that since the majority of the excavations will extend below the groundwater level, the majority of the soil materials generated from excavations along the project alignments will be wet, and will require drying prior to use as trench backfill materials.

5.5 Cut-and-Cover Construction

Since no changes to the existing ground surface along the cut-and-cover segment of the proposed storm drain pipeline alignment are planned, the net stress change in the underlying soils is considered negligible. Furthermore, the soils at the proposed invert level along the storm drain pipeline alignment are expected to provide a stable trench bottom. In the event that loose or disturbed soils are encountered at the trench bottom, it is recommended that they be over-excavated and replaced with pipe bedding or other approved materials. The depth of the overexcavation should be determined during construction by the City's Resident Engineer. Remedial grading outside the limit of proposed excavations for the subject project is not anticipated.

5.5.1 Soil and Excavation Characteristics

The materials within the anticipated depths of the storm drain pipe trench excavation will likely be comprised of materials which can be readily excavated with conventional heavy-duty construction equipment.

5.5.2 Pipe Loads and Settlement

Pipes should be designed for all loads applied by surrounding soils including dead load from soils, loads applied at the ground surface, uplift loads, and earthquake loads. Soil loading above and below the groundwater level may be estimated assuming a density of 100 pcf and 130 pcf, respectively, for properly compacted backfill materials.

Where a pipe changes direction abruptly, resistance to thrust forces can be provided by means of thrust blocks. For design purposes, for the passive resistance against thrust blocks embedded in dense formational material and/or properly compacted filled ground, an equivalent fluid density of 200 pcf may be used. Thrust blocks should be embedded a minimum of 3 feet beneath the ground surface.

Buried flexible pipes are generally designed to limit deflections caused by applied loads. The deflections can be estimated using the Modified Spangler equation. A modulus of soil reaction, E', equal to 1,000 and 2,000 psi may be used to represent a minimum of 6 inches of compacted pipe bedding materials of low plasticity (LL < 50) with less than 12 percent fines passing the #200 standard sieve and crushed rock materials, respectively.

5.5.3 Trench Backfill

Pipe Bedding Zone and Pipe Zone

"Pipe Bedding Zone" is defined as the area below the bottom of the pipe and extending over the full trench width, and should be at least 6 inches thick in order to provide a uniform firm foundation material directly beneath the pipe.

The "Pipe Zone" is defined as the full width of a trench from the bottom of the pipe to a horizontal level about 6 inches above the top (crown) of the pipe. In order to provide uniform support and to minimize external loads, trench widths should be selected such that a minimum clear space of 6 inches is provided on each side of the pipe. During backfilling, it is recommended that the backfill materials be placed on each side of the pipe simultaneously to avoid unbalanced loads on the pipe.

Backfill materials placed in the "Pipe Bedding Zone" and "Pipe Zone" should consist of clean, free draining sand or crushed rock. Sand should be free of clay, organic matter, and other deleterious materials and conform to the gradation shown in the following table.

Sieve Size	Percent Passing by Weight (percent)
½ inch	100
#4	75-100
#16	35-75
#50	10-40
#200	0-10

Crushed rock should conform to Section 200-1.2 and 200-1.3 of the Standard Specifications for Public Works Construction (SSPWC) for 3/4-inch crushed rock gradation. It must be noted that, since the native soil materials do not meet these specifications, import backfill materials will be required for the "Pipe Bedding Zone" and "Pipe Zone". If crushed rock is to be used for pipe zone and bedding backfill materials, we recommend that the rock materials be wrapped in geotextile filter fabric such as Mirafi 140N or equivalent. The purpose of the filter fabric is to prevent migration of fine grained materials from the backfill materials, and the sides and bottom of the trench into the rock bedding materials.

Above Pipe Zone

The "Above Pipe Zone" is defined as the full width of the trench from the top of the "Pipe Zone" to the finish grade or bottom of the pavement section. Backfill material placed in this zone should meet or exceed the criteria presented in Section 5.4. for either flowable fill or soil backfill.

5.5.4 Placement and Compaction of Backfill

Prior to placement, all soil backfill material should be moisture-conditioned, spread and placed in lifts (layers) not-to-exceed 6 inches in loose (uncompacted) thickness, and uniformly compacted to at least 90 percent relative compaction. During backfilling, the soil moisture content should be maintained at or within 2 to 3 percent above the optimum moisture content of the backfill materials. The maximum dry density and optimum moisture content of the backfill materials should be determined in the laboratory in accordance with the ASTM D1557 testing procedures. Field density testing shall be performed in accordance with either the Sand Cone Method (ASTM D1556) or the Nuclear Gauge Method (ASTM D2922 and D3017).

Small hand-operated compacting equipment should be used for compaction of the backfill materials to an elevation of at least 4 feet above the top (crown) of the pipes. Flooding or jetting should not be used to densify the backfill.

5.5.5 Groundwater Bouyant Uplift

AGE has performed a buoyancy analysis for 18-, 30-, 36- and 48-inch diameter RCP pipes and 6-inch diameter PVC pipes which are proposed for the subject project. The analysis is included in Appendix C. The results indicate that the RCP and PVC pipes installed as recommended in this section with crushed rock backfill, within the Pipe Bedding Zone and Pipe Zone, wrapped in geofabric as described in Section 5.5.3 and minimum 24 inches of cover are not expected to float. It is our opinion that no additional mitigation measures are required.

5.6 Buried Structures

It is recommended that any proposed buried structures be founded on firm native soils or approved compacted materials. In areas where loose or soft soils are encountered at the bottom of any manhole/box structure excavations, it is recommended that the loose/soft materials be removed and replaced with 3/4-inch crushed rock materials wrapped in geotextile fabric which meets or exceeds the specifications shown below.

Fabric Property	Min. Certified Values	Test Method
Grab Tensile Strength	300 lb	ASTM D 4632
Grab Tensile Elongation	35% (MAX)	ASTM D 4632
Burst Strength	600 psi	ASTM D 3786
Trapezoid Tear Strength	120 lb	ASTM D 4533
Puncture Strength	130 lb	ASTM D 4833

The actual extent of over-excavation of any loose/soft soil materials should be evaluated and determined in the field by the City's Resident Engineer.

5.6.1 Placement and Compaction of Backfill

Placement and compaction of backfill materials around the buried structures should be performed in accordance with the recommendations presented in Section 5.5.4 of this report.

5.6.2 Foundations

Bearing Capacity

For design of the buried structures which are founded on firm native soils an allowable soil bearing capacity of 2,000 psf may be used. In the event that loose and compressible soils are encountered at the bottom of the excavation for the proposed structures, we recommend that the structures be supported on a minimum of 24 inches of 3/4-inch crushed rock wrapped in geofabric. This allowable soil bearing value is for total dead and live loads, and may be increased by one third when considering seismic loads.

Anticipated Settlement

Under static condition, total settlement of the slab foundation is estimated to be less than 0.25 inch. Differential settlement between the center and the edge of the slab foundation is expected not to exceed 0.25 inch. No permanent deformation and/or post-construction settlement is anticipated, provided that backfill around the structures is properly compacted in accordance with the project specifications.

Resistance to Lateral Loads

Resistance to lateral loads may be developed by a combination of friction acting at the base of the slab foundation and passive earth pressure developed against the sides of the foundations below grade. Passive pressure and friction may be used in combination, without reduction, in determining the total resistance to lateral loads.

An allowable passive earth pressure of 200 psf per foot of foundation embedment below grade may be used for the sides of foundations placed against competent native soils. A coefficient of friction of 0.4 may be used for foundation cast directly on competent native soils or crushed rock wrapped in geofabric.

5.6.3 Walls Below Grade

Lateral earth pressures for walls below grade for structures less than 48 inches in horizontal dimensions may be treated as a shaft structure. Walls below grade for structures larger than 48 inches in horizontal dimensions should be designed to resist the lateral earth pressures presented in

Figures 7 and 8 provided that the wall backfill materials are properly placed and compacted in conformance with the recommendations presented in this report. Surcharge and foundation loads occurring within a horizontal distance equal to the wall height should be added to the lateral pressures as presented in Figures 9 and 10.

Buried structures located below the groundwater table will be subject to buoyant uplift forces. Geotechnical parameters for use in calculating uplift resistance of the surrounding backfill soil materials is presented in Figures 11 and 12.

5.7 Infiltration Testing and Basins

AGE attempted to perform infiltration testing in test hole P-4, but was unable to maintain a consistent free head inside the test hole during the 24-hour pre-soak period. During the test on February 14, 2019, AGE personnel had to add water into the test hole 24 times over a period of four (4) hours. The infiltration rate based on the last reading was calculated to be 90 inch per hour.

The 100% Submittal project plans indicate that Basins 1 through 5 will be located in landscaped areas adjacent to beach access parking facilities on the east and west side of Mission Boulevard to the north of San Fernando Place. Basins 6 through 8 will be located along the southern edge of the South Mission Beach Park at the southern end of the project study area. None of the basins are located in close proximity to existing structures.

Infiltrated water flow is anticipated to be toward Mariner Basin for basins 1, 3 and 5, Pacific Ocean for Basins 2 and 4, and toward Mission Bay for Basins 6, 7 and 8. As discussed under Sections 5.1 through 5.3 of this report, faulting, hydroconsolidation, landslide and expansive soil are not considered to pose as hazards within the project study area. The project study area covers relatively level area with no measurable slopes and the basins are not located adjacent to any filled or cut slopes. Therefore, the potential for slope instability within the project area is considered very low.

Hydraulically placed fill materials in the east and southerly portions of the project study area are classified in the City of San Diego Seismic Safety Study (2008) as having a high liquefaction potential. The findings of our investigation determined that the hydraulic fill materials encountered in borings B-1 and B-3 are in a medium dense condition, and therefore are considered to have a low liquefaction potential. Furthermore, the groundwater elevation in the area of the proposed basins at the time of the geotechnical investigation is shallow (3' to 4' below the ground surface). The groundwater level is already at an elevation of maximum liquefaction potential. Therefore, additional water infiltration is not anticipated to increase the hazard level.

Basins 6 and 7 are not located in the vicinity of the infiltration tests. However, the borings indicate the entire project study area is underlain by uniform poorly graded silty sand (SP-SM). Due to the presence of shallow groundwater, AGE was unable to perform infiltration tests in P-1 through P-3. However, the rate in P-4 which is located the in the vicinity of Basins 6 and 7 were very high. AGE was unable to maintain a consistent free head inside the test hole during the 24-hour pre-soak period. During the test on February 14, 2019, AGE personnel had to add water into the test hole 24 times over a period of four (4) hours. The infiltration rate based on the last reading was calculated to be 90 inch per hour. The test results indicate that the underlying on-site soil possess very high infiltration rate. Due to the very high infiltration rate, lateral moisture infiltration beneath existing pavement is not anticipated. Therefore, soil piping beneath pavement sections and premature pavement sections failure are not anticipated.

The proposed basins are not anticipated to adversely impact the groundwater quality within the project study area**.** Vertical distance to the regional groundwater table is anticipated to be less than 10 feet. A search of the Geotracker data base does not reveal the presence of any water supply wells within 100 feet of the project study area.

6.0 CONSTRUCTION-RELATED CONSIDERATIONS

6.1 Construction Dewatering

Groundwater and flowing sand conditions are anticipated to be encountered at or above the proposed pipe invert elevations along the project alignments. Because of the anticipated high rate of transmissivity of the underlying soils along the project alignments and the potential for encountering flowing sand condition, we recommend that groundwater be kept out of the trenched excavations using sheet piles in combination with sump pumps. Sheet piles should be extended to a depth of at least 10 feet below the bottom of the proposed trenched excavations.

The design, installation, and operation of any construction dewatering measures necessary for the project shall be the sole responsibility of the contractor.

6.2 Temporary Shoring

Since the anticipated pipe invert depths will be more than 4 feet below the ground surface, prevailing Federal and Cal OSHA safety regulations require that the trenched excavation be either sloped (if sufficient construction space or easement is available), shored, braced, or protected with approved sliding trench shield. Limited construction space, the presence of other buried utilities, and the need to avoid excessive community disruption dictate that a shored excavation will be needed along the entire pipeline alignment. Design and construction of temporary shoring should be the sole responsibility of the contractor.

Settlement

Settlement of existing street improvements and/or utilties adjacent to the shoring may occur in proportion to both the distance between shoring system and adjacent structures or utilities and the amount of horizontal deflection of the shoring system. Vertical settlement will be maximum directly adjacent to the shoring system, and decreases as the distance from the shoring increases. At a distance equal to the height of the shoring, settlement is expected to be negligible. Maximum vertical settlement is estimated to be on the order of 75 percent of the horizontal deflection of the shoring system. It is recommended that shoring be designed to limit the maximum horizontal deflection to 1-inch or less where structures or utilities are to be supported.

It is recommended that pre- and post-construction surveys be conducted to document existing site conditions. Documentation should include photographic and video surveys of the existing facilities and site improvements, as well as field surveys of building floors and pavement structures. We further recommend that a weekly survey of existing utilities be performed during the construction phase.

Lateral Earth Pressures

Temporary shoring should be designed to resist the pressure exerted by the retained soils and any additional lateral forces due to loads placed near the top of the excavation. For design of braced shorings supporting fill materials and old paralic deposits, the recommended lateral earth pressure should be 32H psf, where H is equal to the height of the retained earth in feet. Any surcharge loads would impose uniform lateral pressure of 0.3q, where "q" equals the uniform surcharge pressure. The surcharge pressure should be applied starting at a depth equal to the distance of the surcharge load from the top of the excavation. In the event that the bottom of the excavation is located below the groundwater level, hydrostatic pressure should be added to the lateral loads.

The recommended lateral earth pressures have been prepared based on the assumptions that the shored earth is level at the surface and that the shoring system is temporary in nature.

Lateral Bearing Capacity

Resistance to lateral loads will be provided by passive soil resistance. The allowable passive pressure for the fill materials and old paralic deposits may be assumed to be equivalent to a fluid weighing 200 pcf.

6.3 Environmental Considerations

The scope of AGE's investigation did not include the performance of a Phase I Environmental Site Assessment (Phase I ESA) to evaluate the possible presence of soil and/or groundwater contamination beneath the project alignments. During our subsurface investigation soil samples were field screened for the presence of volatile organics using a RAE Systems MiniRAE 3000 organic vapor meter (OVM). The field screening did not reveal elevated levels of volatile organics in the samples.

In the event that hazardous or toxic materials are encountered during the construction phase, the contractor should immediately notify the City and be prepared to handle and dispose of such materials in accordance with current industry practices and applicable Local, State and Federal regulations.

7.0 GENERAL CONDITIONS

7.1 Post-Investigation Services

Post-investigation geotechnical services are an important continuation of this investigation, and we recommend that the City's Construction Inspection Division performs the necessary geotechnical observation and testing services during construction. In the event that the City is unable to perform said services, it is recommended that our firm be retained to provide the services.

Sufficient and timely observation and testing should be performed during excavation, pipeline installation, backfilling and other related earthwork operations. The purpose of the geotechnical observation and testing is to correlate findings of this investigation with the actual subsurface conditions encountered during construction and to provide supplemental recommendations, if necessary.

7.2 Uncertainties and Limitations

The information presented in this report is intended for the sole use of Rick engineering and other members of the project design team and the City for project design purposes only and may not provide sufficient data to prepare an accurate bid. The contractor should be required to perform an independent evaluation of the subsurface conditions at the project site prior to submitting his/her bid.

AGE has observed and investigated the subsurface conditions only at selected locations along the project alignments. The findings and recommendations presented in this report are based on the assumption that the subsurface conditions beneath all project alignments do not deviate substantially from those encountered in the exploratory test pits. Consequently, modifications or changes to the recommendations presented herein may be necessary based on the actual subsurface conditions encountered during construction.

California, including San Diego County, is in an area of high seismic risk. It is generally considered economically unfeasible to build a totally earthquake-resistant project and it is, therefore, possible that a nearby large magnitude earthquake could cause damage at the project site.

Geotechnical engineering and geologic sciences are characterized by uncertainty. Professional judgments and opinions presented in this report are based partly on our evaluation and analysis of the technical data gathered during our present study, partly on our understanding of the scope of the proposed project, and partly on our general experience in geotechnical engineering.

In the performance of our professional services, we have complied with that level of care and skill ordinarily exercised by other members of the geotechnical engineering profession currently practicing under similar circumstances in southern California. Our services consist of professional consultation only, and no warranty of any kind whatsoever, expressed or implied, is made or intended in connection with the work performed. Furthermore, our firm does not guarantee the performance of the project in any respect.

AGE does not practice or consult in the field of safety engineering. The contractor will be responsible for the health and safety of his/her personnel and all subcontractors at the construction site. The contractor should notify the City if he or she considers any of the recommendations presented in this report to be unsafe.

8.0 REFERENCES

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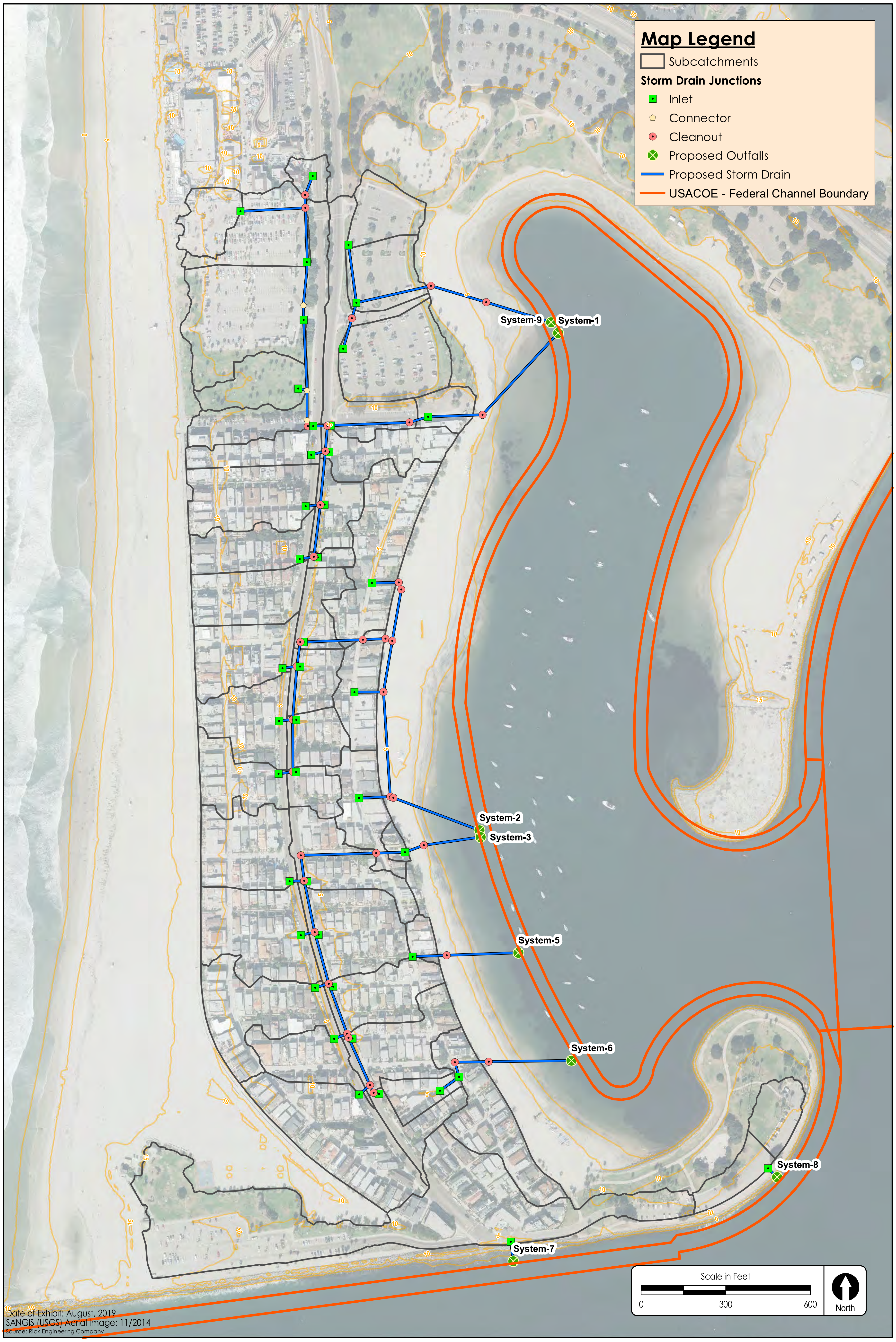
FIGURES

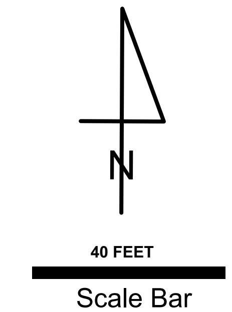


The City of SAN DIEGO | RICK ENGINEERING COMPANY

DRAFT South Mission Beach - Storm Drain Improvements and GI
Storm Drain Alignment - System Outfalls

SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT
ALIGNMENTS MAP

PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 1

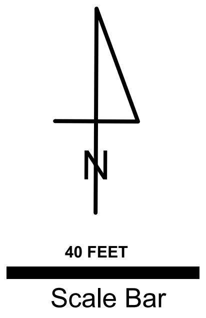


LEGEND

P-1	Approximate Infiltration Test Hole Location
B-2	Approximate Boring Location
C-2	Approximate Pavement Core Location

SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT	**LOCATION MAP**

PROJECT NO. 190 GS-18-D	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE 2**



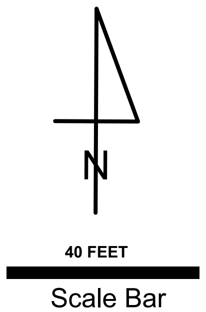
LEGEND

P-1 Approximate Infiltration Test Hole Location

B-2 Approximate Boring Location

C-2 Approximate Pavement Core Location

SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT		LOCATION MAP
PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 3



LEGEND

P-1	Approximate Infiltration Test Hole Location
B-2	Approximate Boring Location
C-2	Approximate Pavement Core Location

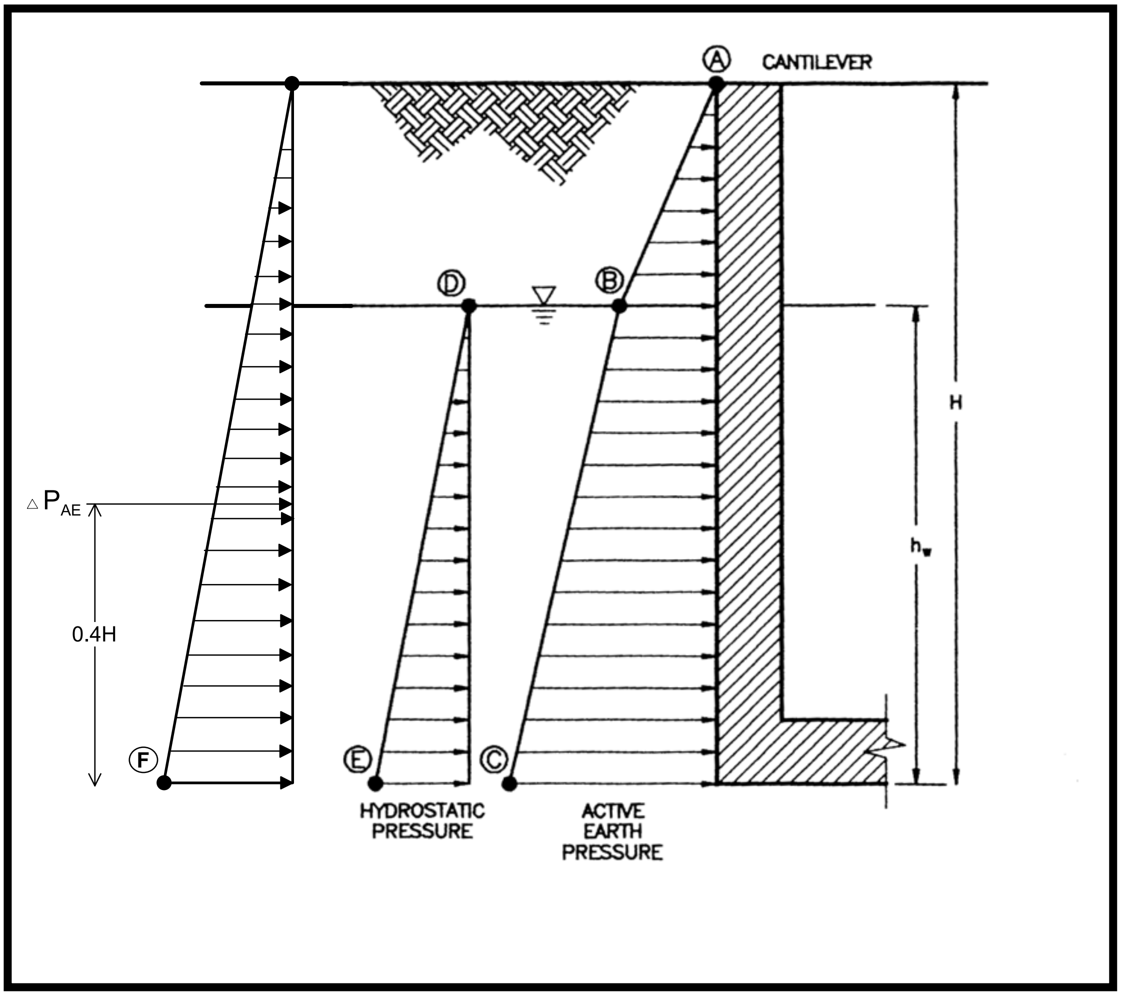
SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT	LOCATION MAP	
PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 4

LEGEND

P-1	Approximate Infiltration Test Hole Location
B-2	Approximate Boring Location
C-2	Approximate Pavement Core Location

SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT		LOCATION MAP
PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 5

SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT		WEEP SUMP PHOTOGRAPH
PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 6



NOTES

H = wall height in feet

h_w = water height above bottom of structure in feet

Lateral pressure values presented herein are based on the assumption that non-expansive backfill materials will be used to backfill behind walls

LATERAL PRESSURES

Earth Pressure

A = 0

B = 35 ($H-h_w$), psf

C = 35 ($H-h_w$) + $20h_w$, psf

Hydrostatic Pressure

D = 0

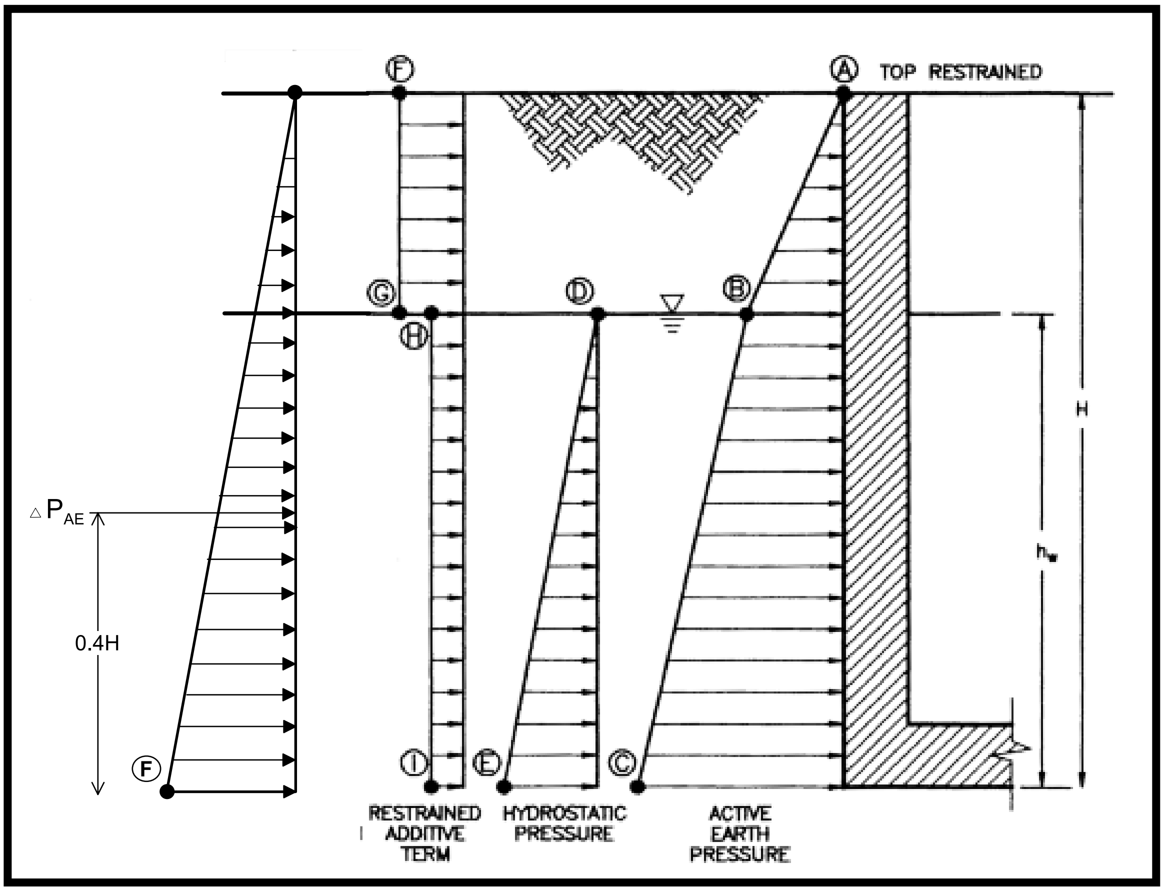
E = $62.4h_w$

Dynamic Resultant Force

Ignored for shaft construction.

LATERAL PRESSURES FOR CANTILEVER WALLS
SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT

PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 7



NOTES

H = wall height in feet

h_w = water height above bottom of structure in feet

Lateral pressure values presented herein are based on the assumption that non-expansive backfill materials will be used to backfill behind walls

LATERAL PRESSURES

Earth Pressure
A = 0
B = 35 $(H-h_w)$, psf
C = 35 $(H-h_w) + 20h_w$, psf

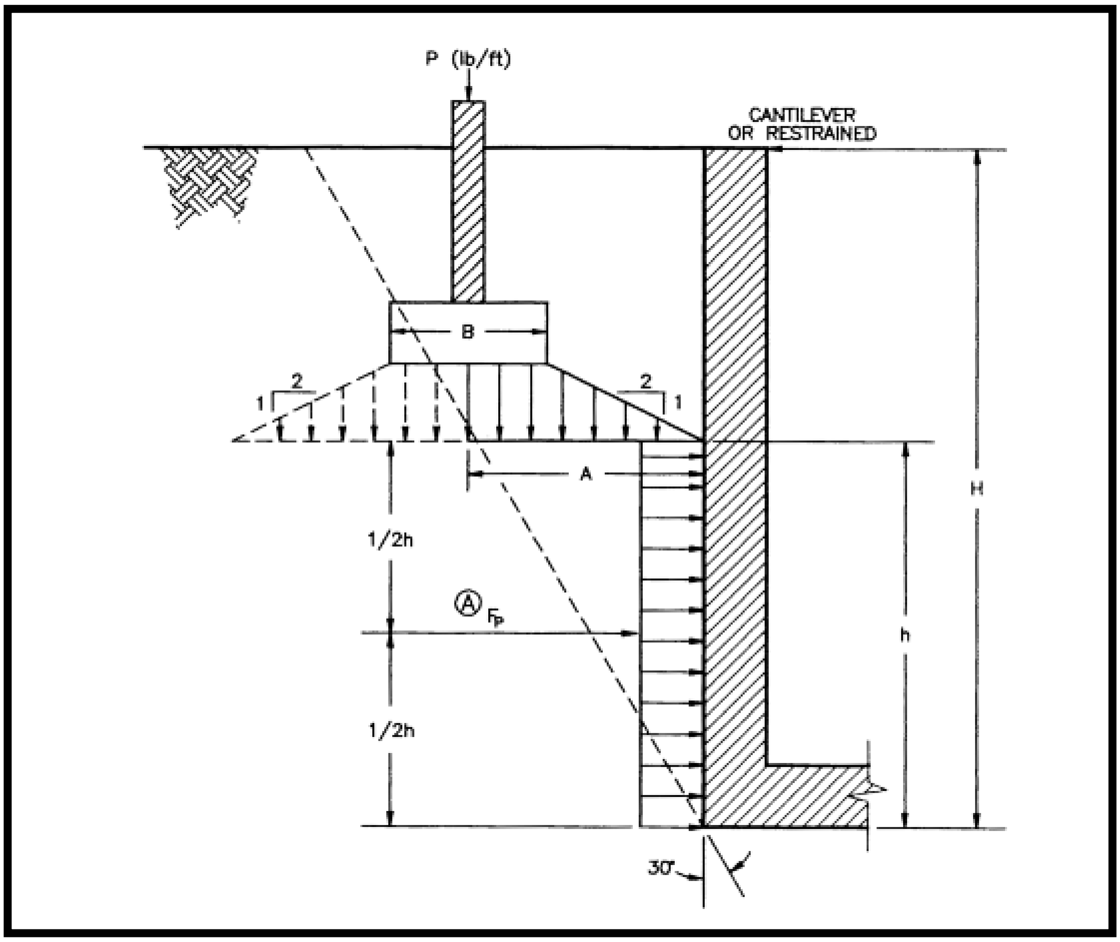
Hydrostatic Pressure
D = 0
E = $62.4h_w$

Restrained Additive Term
F = G = 10H, psf
H = I = 5H, psf

Dynamic Resultant Force
Ignored for shaft construction.

LATERAL PRESSURES FOR RESTRAINED WALLS
SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT

PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 8



NON-EXPANSIVE BACKFILL

F_p = M (A/B) P, lb/ft

A = h tan 30°, ft
M = 0.3 for cantilever wall
M = 0.4 for restrained wall

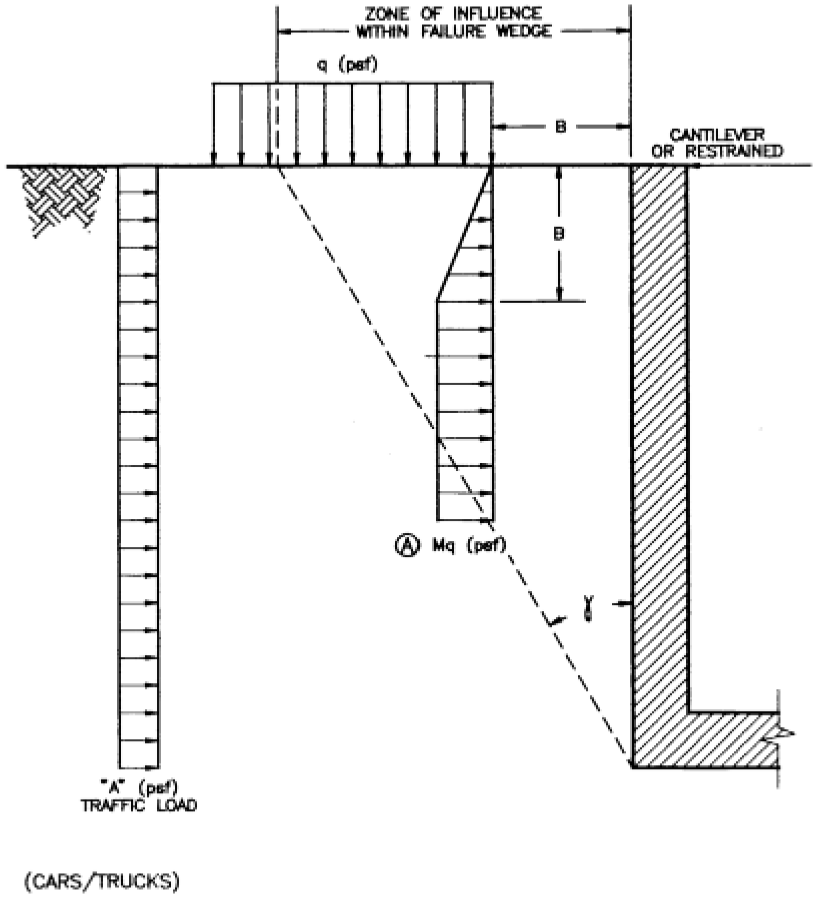
NOTES:

1. Surcharge pressure acting on wall is not affected by groundwater elevation.

2. Surcharge pressures shown are applicable for continuous footing only. Spread footings need to be evaluated individually.

FOUNDATION INDUCED WALL PRESSURE
SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT

PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 9



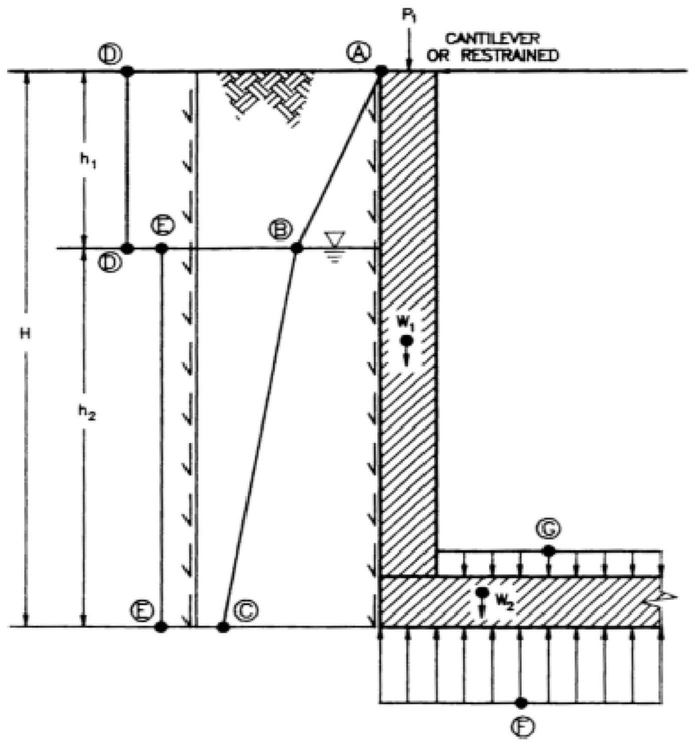
NON-EXPANSIVE BACKFILL

q = surcharge load (psf)
B = distance between wall and surcharge load, ft
M = 0.3 for cantilever wall
M = 0.4 for restrained wall
A = Mq, psf
"A" = 75 psf
$\gamma = 30^{o}$

NOTE: Surcharge pressure acting on wall is not affected by groundwater elevation.

TRAFFIC INDUCED WALL PRESSURES
SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT

PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 10



PROPERLY COMPACTED BACKFILL

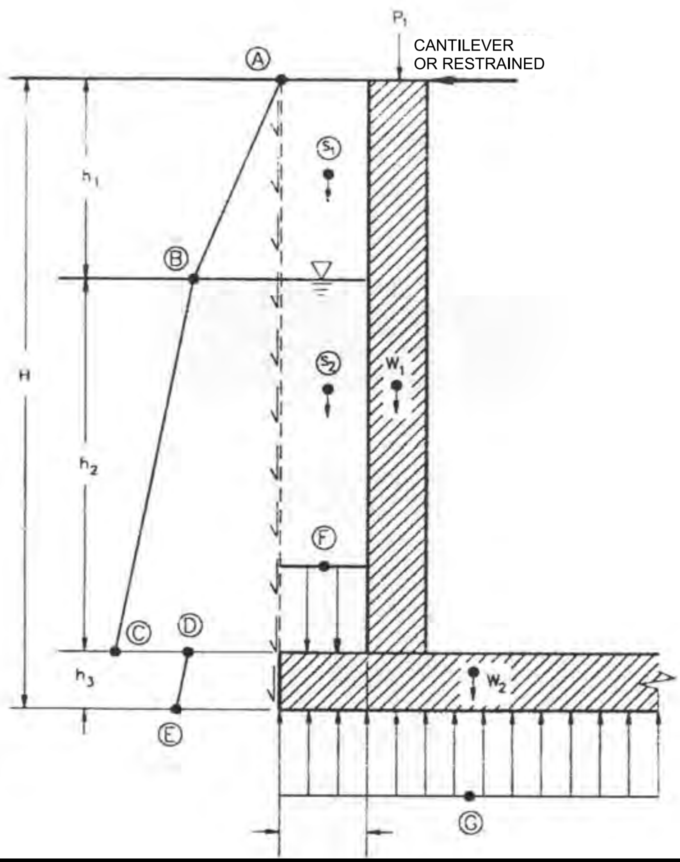
Soil Friction, psf
A = 0
B = $22h_1$
C = $22h_1 + 11\ h_2$
D = 7H*
E = 4H*

Hydrostatic Pressure, psf
F = $62.4\ h_2$

NOTE: * D and E are only applicable for restrained walls and should be ignored if walls are to be designed as simple cantilever

UPLIFT RESISTANCE FOR WALLS WITHOUT EXTENSION
SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT

PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 11



PROPERLY COMPACTED BACKFILL

Soil Friction, psf

- A = 0
- B = $40h_1$
- C = $40h_1 + 20\ h_2$
- D = $24h_1 + 12\ h_2$
- E = $24h_1 + 12\ h_2 + 12\ h_3$

Soil Weights - Within Vertical Prism, pcf

- S_1 = 130 (above groundwater)
- S_2 = 62 (below groundwater)

Hydrostatic Pressure, psf

- F = $62.4\ h_2$
- G = $62.4\ (h_2 + h_3)$

UPLIFT RESISTANCE FOR WALLS WITH EXTENSION
SOUTH MISSION BEACH GREEN INFRASTRUCTURE PROJECT

PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 12

APPENDIX A

FIELD EXPLORATION PROGRAM

APPENDIX A

FIELD EXPLORATION PROGRAM

The field exploration program for this project was performed on February 11 and 12, 2019. A total of four (4) soil borings, four (4) infiltration test holes, and two (2) pavement corings were performed at the approximate locations shown on Figures 2 through 5. In addition, AGE attempted to perform infiltration testing inside an existing weep sump located on the west side of Mission Boulevard, at the entrance of an alley located between Brighton Court and Capistrano Place. The soil borings were advanced to depths ranging from 15 feet to 16.5 feet below the existing ground surface (bgs). The infiltration test holes were hand-augured to depths ranging from 36 inches to 63 inches bgs. A brief description of the location and depth, pavement sections, groundwater level, and subsurface conditions encountered in the borings and infiltration test holes is presented in Table 1.

Borings B-2 and B-4 which were located in Mission Boulevard were performed with a CME-75 truck mounted drill rig. Borings B-1 and B-3 which were located on Mission Bay Park were performed with an all-terrain mounted drill rig. The soils encountered in the soil borings were visually classified and logged by an experienced engineering geologist from AGE. A Key to Logs is presented on Figures A-1 and A-2, and logs of the borings are presented on Figures A-3 thru A-6. The logs depict the various soil types encountered and indicate the depths at which samples were obtained for laboratory testing and analysis.

Prior to commencement of the field exploration activities, several site visits were performed to observe existing conditions and to select suitable locations for the soil borings and test holes. Subsequently, Underground Service Alert (USA) was contacted to coordinate clearance of the proposed boring and test hole locations with respect to existing buried utilities. The borings and test holes located in Mission Bay Park were performed in coordination with and with the approval from the City of San Diego Parks & Recreation Department.

During drilling, Standard Penetration Tests (SPT) were performed at selected depth intervals. The SPT tests involve the use of a specially manufactured “split spoon” sampler which is driven a distance of approximately 18 inches into the soils at the bottom of the borehole by dropping a 140-pound weight from a height of 30 inches. The number of blows required to penetrate each 6-inch increment was counted and recorded on the field logs, and have been used to evaluate the relative density and consistency of the materials. The blow counts were subsequently corrected for soil type, hammer model, groundwater and surcharge. The corrected blow counts are shown on the boring logs.

Relatively undisturbed samples were obtained by driving a 3-inch (OD) diameter standard California sampler with a special cutting tip and inside lining of thin brass rings into the soils at the bottom of the borehole. The sampler is driven a distance of approximately 18 inches into the soil at the bottom of the borehole by dropping a 140-pound weight from a height of 30 inches. A 6-inch long section of soil sample that was retained in the brass rings was extracted from the sampling tube and transported to our laboratory in close-fitting, waterproof containers. The samples were field screened for the presence of volatile organics using a RAE Systems MiniRAE 3000 organic vapor meter (OVM). The OVM readings are indicated on the logs. In addition, loose bulk samples were also collected.

Infiltration testing inside test hole P-4 was performed using Borehole Percolation Test Methods described in Appendix F - Storm Water Infiltration/Percolation BMPs of the City of San Diego Guidelines for Geotechnical Report (2011) and Appendix D - Approved Infiltration Rate Assessment Methods of the San Diego Region Model BMP Design Manual (2018).

Upon completion of the drilling, sampling and testing activities, the borings were backfilled using bentonite grout and/or bentonite chips to approximately 12 inches below the ground surface. Borings B-1 and B-3 which were located at the beach were capped with on-site beach sand. Borings B-2 and B-4 which were performed in Mission Boulevard were capped with rapid-set concrete to match the adjacent pavement surface. Pavement coreholes C-2 and C-4 were also capped with rapid-set concrete to match the adjacent pavement surface. The infiltration test holes were backfilled with soil cuttings generated during excavation.

KEY TO LOG OF BORING

DEPTH (FEET)	SAMPLES	BLOW COUNTS (BLOWS/FOOT)	OVM READING (PPM)	GRAPHIC LOG	SOIL DESCRIPTION	FIELD MOISTURE (% DRY WT.)	DRY DENSITY (PCF)	REMARKS
1								
2					Sample identification number			
3								
4	1				Approximate interval of bulk sample			
5								
6								
7								
8	2	28			Approximate interval of Standard California Sampler (SCS).			
9								
10					Number of blows required to advance sampler for the last 12-inch increment, or distance indicated.			
11								
12	3	33			Approximate interval of Standard Penetration Test (SPT).			
13								
14								
15								
16								
17					Approximate groundwater elevation			
18								
19								

(KEY TO LOG OF BORING CONTINUED ON FIGURE A-2)

PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE A-1

KEY TO LOG OF BORING
(CONTINUED)

DEPTH (FEET)	SAMPLES	BLOW COUNTS (BLOWS/FOOT)	OVM READING (PPM)	GRAPHIC LOG	SOIL DESCRIPTION	FIELD MOISTURE (% DRY WT.)	DRY DENSITY (PCF)	REMARKS

1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19

—?— —— —?— APPROXIMATE GEOLOGIC CONTACT

Strata symbols

Poorly graded sand with silt

GENERAL NOTES

1. Approximate elevations and locations of borings are based on the topographical maps provided by Rick Engineering Company, undated.

2. Soil descriptions are based on visual classification made during the field exploration and, where deemed appropriate, have been modified based on the results of laboratory tests.

3. Descriptions on the logs apply only at the specific locations and at the time the work was performed. They are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE A-2

BORING NO. B-1

DATE OF DRILLING: February 11, 2019	TOTAL BORING DEPTH: 16.5’
GENERAL LOCATION: On the beach, 20' east of Bayside walk at San Fernando Place	
APPROXIMATE SURFACE ELEV.: + 7.3' msl	DRILLING CONTRACTOR: Tri-County Drilling
DRILLING METHOD: Hollow-Stem Auger	LOGGED BY: Nicholas Barnes

DEPTH (FEET)	SAMPLES	BLOW COUNTS BLOWS/FOOT	OVM READING (PPM)	GRAPHIC LOG	SOIL DESCRIPTION	FIELD MOISTURE % DRY WT.	DRY DENSITY LBS./CU. FT.	REMARKS
0–10					**HYDRAULIC FILL** Light gray, damp, medium grained poorly graded micaceous sand (SP-SM) with traces of broken shells.			
6	1	25	1.6			6.2	105.6	
10			?					?
11	2	17	0.1		**OLD PARALIC DEPOSITS** Dark greenish gray, wet, medium grained poorly graded micaceous sand (SP-SM)	22.2		
11.5	3							
14.5	4							
16	5	23				30.3	95.1	

NOTES:

Boring terminated at depth of 16.5' bgs. No refusal.
Water level measured at depth of 11' bgs 10 minutes after completion of the drilling operations.

PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE A-3

BORING NO. B-2

DATE OF DRILLING: February 12, 2019	TOTAL BORING DEPTH: 15’
GENERAL LOCATION: Southbound Mission Boulevard, approximately 40' south of San Fernando Place and 4' from median.	
APPROXIMATE SURFACE ELEV.: +6.3' msl	DRILLING CONTRACTOR: Tri-County Drilling
DRILLING METHOD: Hollow-Stem Auger	LOGGED BY: Nicholas Barnes

DEPTH (FEET)	SAMPLES	BLOW COUNTS BLOWS/FOOT	OVM READING (PPM)	GRAPHIC LOG	SOIL DESCRIPTION	FIELD MOISTURE % DRY WT.	DRY DENSITY LBS./CU. FT.	REMARKS
1					**PAVEMENT SECTION:** 4" A.C. over 8" P.C.C. underlain by 6" of miscellaneous base			
2					**OLD PARALIC DEPOSITS**			
3–5					▼ Greenish gray, wet, medium grained poorly-graded micaceous sand with silt (SP-SM) with traces of broken shells.			
6	1	34				26.0		
6	2							
8	3							
10	4	26				26.3		
14	5					24.7		
15								

NOTES:

Boring terminated at depth of 15' bgs. No refusal.
Water level measured at depth of 4'-3" bgs at the completion of the drilling operation.

PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE A-4

BORING NO. B-3

DATE OF DRILLING: February 11, 2019	TOTAL BORING DEPTH: 16.5
GENERAL LOCATION: On the beach, 20' east of Bayside walk at Coronado Court	
APPROXIMATE SURFACE ELEV.: +5.2' msl	DRILLING CONTRACTOR: Tri-County Drilling
DRILLING METHOD: Hollow-Stem Auger	LOGGED BY: Nicholas Barnes

DEPTH (FEET)	SAMPLES	BLOW COUNTS BLOWS/FOOT	OVM READING (PPM)	GRAPHIC LOG	SOIL DESCRIPTION	FIELD MOISTURE % DRY WT.	DRY DENSITY LBS./CU. FT.	REMARKS
0–4					**HYDRAULIC FILL** Greenish gray, wet, fine- to medium-grained poorly graded micaceous silty sand (SP-SM) ▼			
6	1	27				25.5		
7	2							
9	3					21.4		
10			?					?
10.5	4	26			**OLD PARALIC DEPOSITS** Greenish gray, wet, fine- to medium-grained poorly graded micaceous silty sand (SP-SM)			Heaving sand. No sample recovery.
15	5					24.9		

NOTES:

First attempt encountered refusal at 3' bgs and the boring location was moved 10 feet to the east.
Boring terminated at depth of 16.5' bgs. No refusal.
Water level measured at depth of 4' bgs at the completion of the drilling operation.

PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE A-5

BORING NO. B-4

DATE OF DRILLING: February 12, 2019	TOTAL BORING DEPTH: 15
GENERAL LOCATION: Southbound Mission Boulevard, approximately 60' south of Brighton Court	
APPROXIMATE SURFACE ELEV.: +5' msl	DRILLING CONTRACTOR:
DRILLING METHOD:	LOGGED BY: Nicholas Barnes

DEPTH (FEET)	SAMPLES	BLOW COUNTS BLOWS/FOOT	OVM READING (PPM)	GRAPHIC LOG	SOIL DESCRIPTION	FIELD MOISTURE % DRY WT.	DRY DENSITY LBS./CU. FT.	REMARKS
1					**PAVEMENT SECTION:**			
2					4.5" A.C. over 7.5" P.C.C. underlain by 4" of miscellaneous base			
3					**OLD PARALIC DEPOSITS**			
4					Greenish gray, wet, medium grained poorly-graded micaceous sand with silt (SP-SM) with traces of broken shells and rounded gravels.			
6	1	35						
6	2							
8	3		2.0			29.1		
10	4		2.2			29.6		
14	5		0.5			24.2		
15								

NOTES:

Boring terminated at depth of 15' bgs. No refusal. Water level measured at depth of 3'-3" bgs at the completion of the drilling operation.

PROJECT NO. 190 GS-18-D	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE A-6

APPENDIX B

LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

Selected soil samples were tested in the laboratory to verify visual field classifications and to evaluate certain engineering characteristics. The testing was performed in accordance with the American Society for Testing and Materials (ASTM) or other generally accepted test methods, and included the following:

- Determination of in-place moisture content (ASTM D2216). The final test results are presented on the test pit logs;

- Determination of in-place dry density and moisture content (ASTM D2937) based on relatively undisturbed drive samples. The final test results are presented on the test pit logs;

- Maximum density and optimum moisture content (ASTM D1557). The final test results are presented on Figures B-1 and B-2;

- Sieve analyses (ASTM D422), and the final test results are plotted as gradation curves on Figures B-3 and B-4;

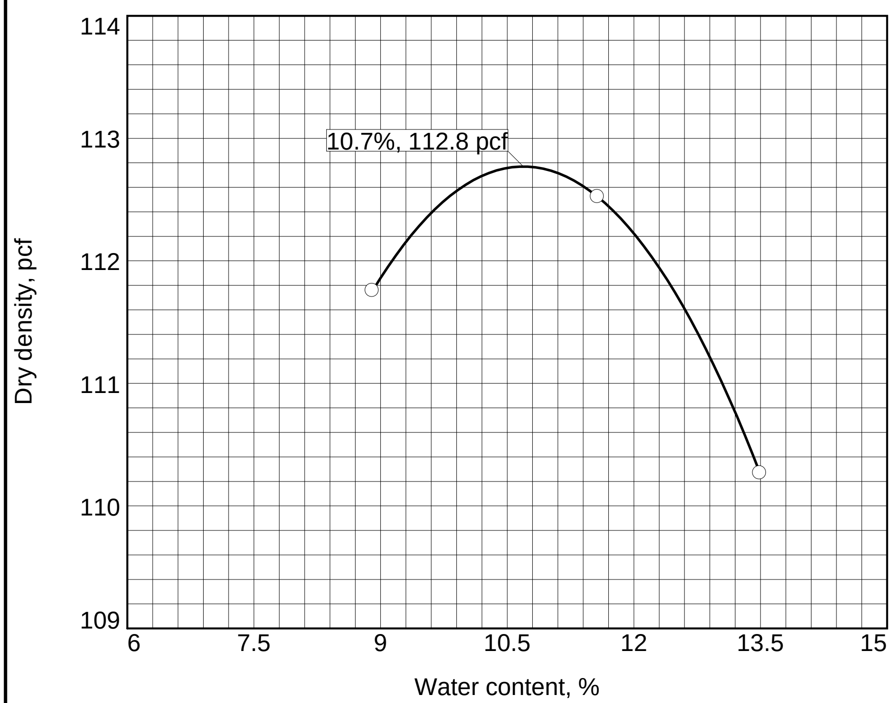
- Direct shear test (ASTM D3080). The test results are presented on Figures B-5 and B-6; and

- Consolidation (ASTM D2435). The test results are presented on Figure B-7.

In addition, representative samples of the onsite soil materials were delivered to Clarkson Laboratory and Supply, Inc. for analytical (chemical) testing to determine soil pH and resistivity, soluble sulfate and chloride concentrations, and bicarbonate content. Copies of Clarkson's laboratory test data reports are included herein.

Representative samples of the soil materials underlying Mission Boulevard were delivered to Southern California Soil & Testing (SCS&T) for R-Value testing. Copies of SCS&T's laboratory test data reports are included herein.

COMPACTION TEST REPORT



Curve No.

Test Specification:
ASTM D 1557-91 Procedure A Modified

Preparation Method Wet
Hammer Wt. 10 lb.
Hammer Drop 18 in.
Number of Layers five
Blows per Layer 25
Mold Size 0.03333 cu. ft.
Test Performed on Material Passing #4 **Sieve**

NM 22.2 **LL** NV **PI**
Sp.G. (ASTM D 854)
%>#4 1.0 **%<No.200** 10.0
USCS SP-SM **AASHTO** A-3
Date Sampled 02/12/2019
Date Tested 02/21/2019
Tested By Nicholas Barnes

TESTING DATA

	1	2	3	4	5	6
WM + WS	5932.0	5926.0	5874.0			
WM	4034.0	4034.0	4034.0			
WW + T #1	531.6	520.0	493.9			
WD + T #1	482.6	466.7	459.5			
TARE #1	59.2	71.7	73.2			
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	11.6	13.5	8.9			
DRY DENSITY	112.5	110.3	111.8			

TEST RESULTS

Maximum dry density = 112.8 pcf

Optimum moisture = 10.7 %

Material Description

Dark greenish gray poorly-graded sand with silt (SP-SM)

Remarks:

Project No. 190 GS-18-D **Client:** Rick Engineering Company
Project: South Mission Beach Project

○ **Source of Sample:** B-1 **Depth:** 10 **Sample Number:** 2

Checked by: Sani Sutanto
Title: Project Manager

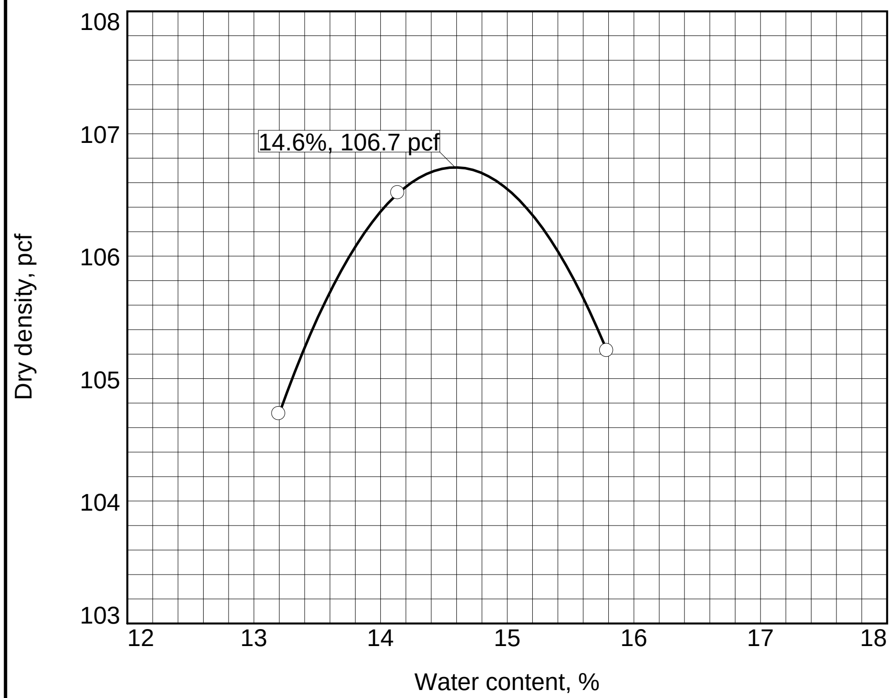
Allied Geotechnical Engineers, Inc.

Santee, CA

Figure B-1

COMPACTION TEST REPORT

Curve No.



Test Specification:
ASTM D 1557-91 Procedure A Modified

Preparation Method Wet
Hammer Wt. 10 lb.
Hammer Drop 18 in.
Number of Layers five
Blows per Layer 25
Mold Size 0.03333 cu. ft.
Test Performed on Material Passing #4 **Sieve**

NM **LL** NV **PI**
Sp.G. (ASTM D 854)
%>#4 0.1 **%<No.200** 5.1
USCS SP-SM **AASHTO** A-3
Date Sampled 02/12/2019
Date Tested 02/21/2019
Tested By Nicholas Barnes

TESTING DATA

	1	2	3	4	5	6
WM + WS	5826.0	5872.0	5876.0			
WM	4034.0	4034.0	4034.0			
WW + T #1	491.0	475.3	486.0			
WD + T #1	440.0	424.2	428.2			
TARE #1	53.6	62.8	62.1			
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	13.2	14.1	15.8			
DRY DENSITY	104.7	106.5	105.2			

TEST RESULTS

Maximum dry density = 106.7 pcf

Optimum moisture = 14.6 %

Material Description

Greenish gray poorly graded sand with silt (SP-SM)

Remarks:

Project No. 190 GS-18-D **Client:** Rick Engineering Company
Project: South Mission Beach Project

○ **Source of Sample:** B-3 **Depth:** 6

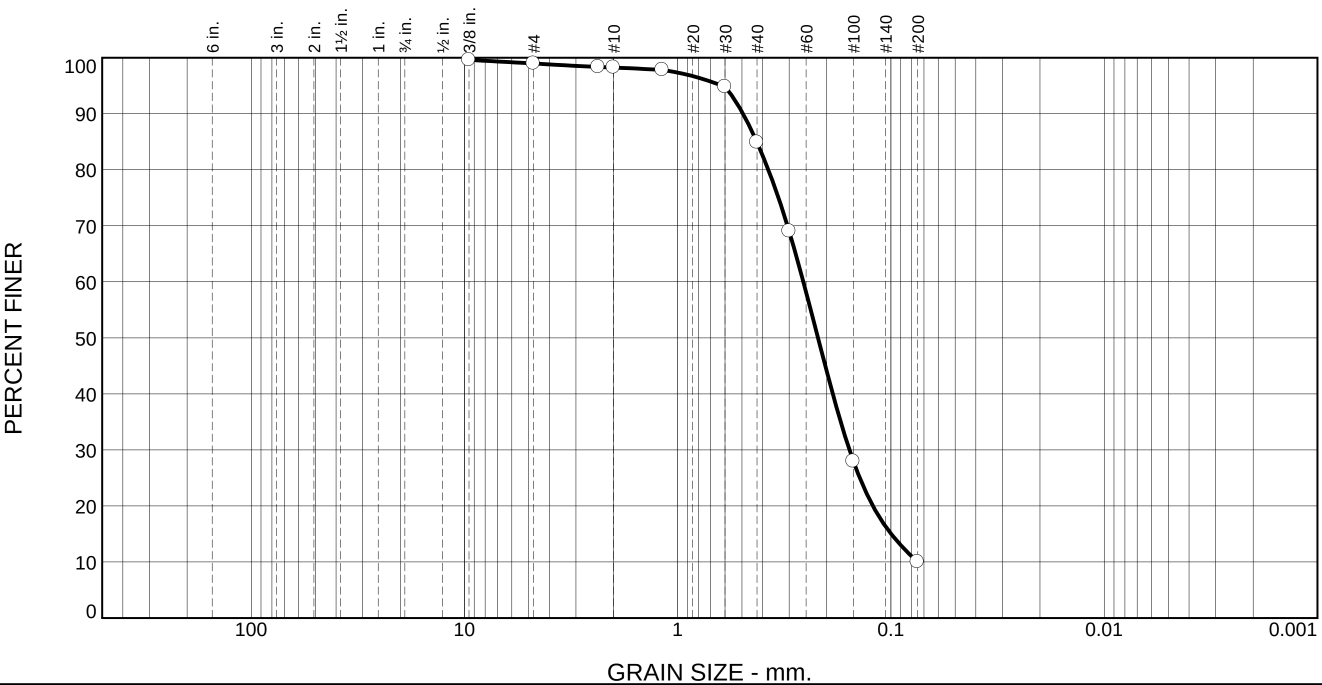
Checked by: Sani Sutanto
Title: Project Manager

Allied Geotechnical Engineers, Inc.

Santee, CA

Figure B-2

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
			0.8	13.3	74.9	10.0	

Test Results (ASTM D 422 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.375	99.6		
#4	99.0		
#8	98.4		
#10	98.2		
#16	97.8		
#30	94.8		
#40	84.9		
#50	69.0		
#100	27.9		
#200	10.0		

* (no specification provided)

Material Description

Dark greenish gray poorly-graded sand with silt (SP-SM)

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=** NP

Classification

USCS (D 2487)= SP-SM **AASHTO (M 145)=** A-3

Coefficients

D_{90}= 0.4940 D_{85}= 0.4263 D_{60}= 0.2575
D_{50}= 0.2199 D_{30}= 0.1566 D_{15}= 0.0998
D_{10}= C_u= C_c=

Remarks

Date Received: **Date Tested:** 02/21/2019

Tested By: Nicholas Barnes

Checked By: Sani Sutanto

Title: Project Manager

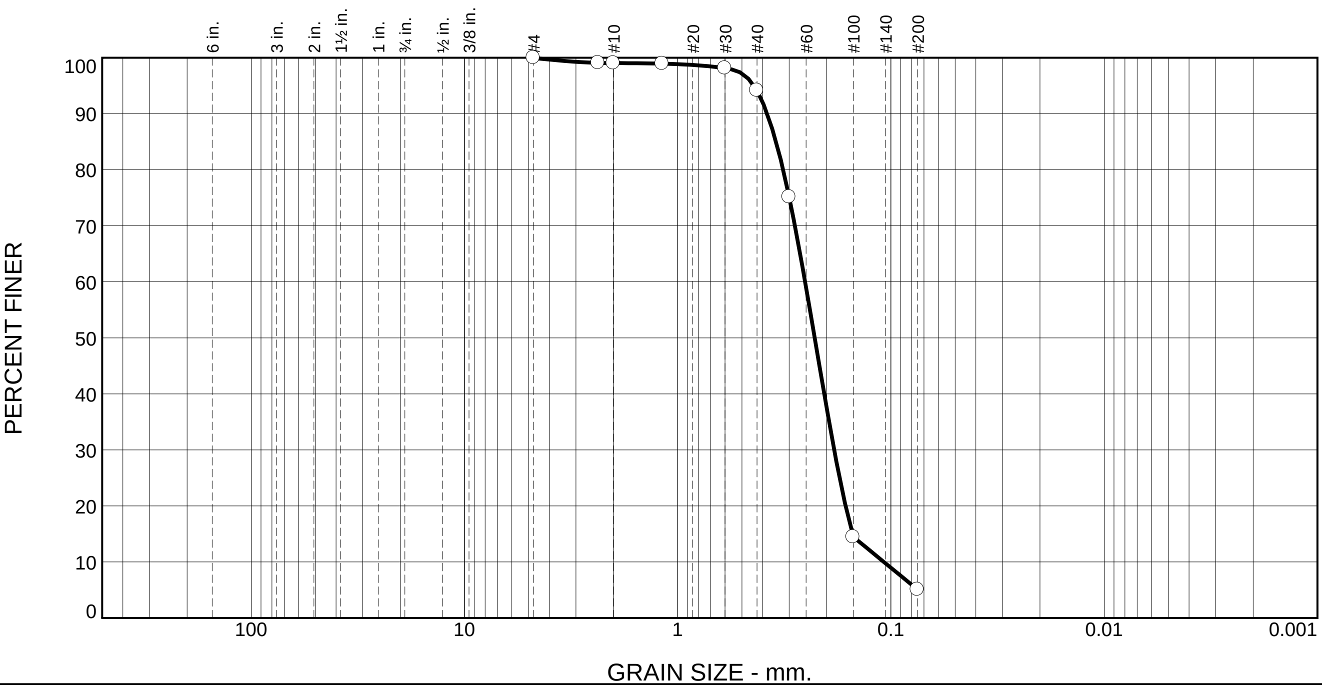
Source of Sample: B-1 **Depth:** 10
Sample Number: 2
Date Sampled: 02/12/2019

Allied Geotechnical Engineers, Inc.

Santee, CA

Client: Rick Engineering Company
Project: South Mission Beach Project
Project No: 190 GS-18-D **Figure** B-3

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
			0.9	4.9	89.0	5.1	

Test Results (ASTM D 422 & ASTM D 1140)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#4	99.9		
#8	99.1		
#10	99.0		
#16	98.9		
#30	98.1		
#40	94.1		
#50	75.1		
#100	14.5		
#200	5.1		

* (no specification provided)

Material Description

Greenish gray poorly graded sand with silt (SP-SM)

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=**

Classification

USCS (D 2487)= SP-SM **AASHTO (M 145)=** A-3

Coefficients

D_{90}= 0.3806 D_{85}= 0.3460 D_{60}= 0.2527
D_{50}= 0.2279 D_{30}= 0.1846 D_{15}= 0.1514
D_{10}= 0.1079 C_u= 2.34 C_c= 1.25

Remarks

Date Received: 02/12/2019 **Date Tested:** 02/21/2019

Tested By: Nicholas Barnes

Checked By: Sani Sutanto

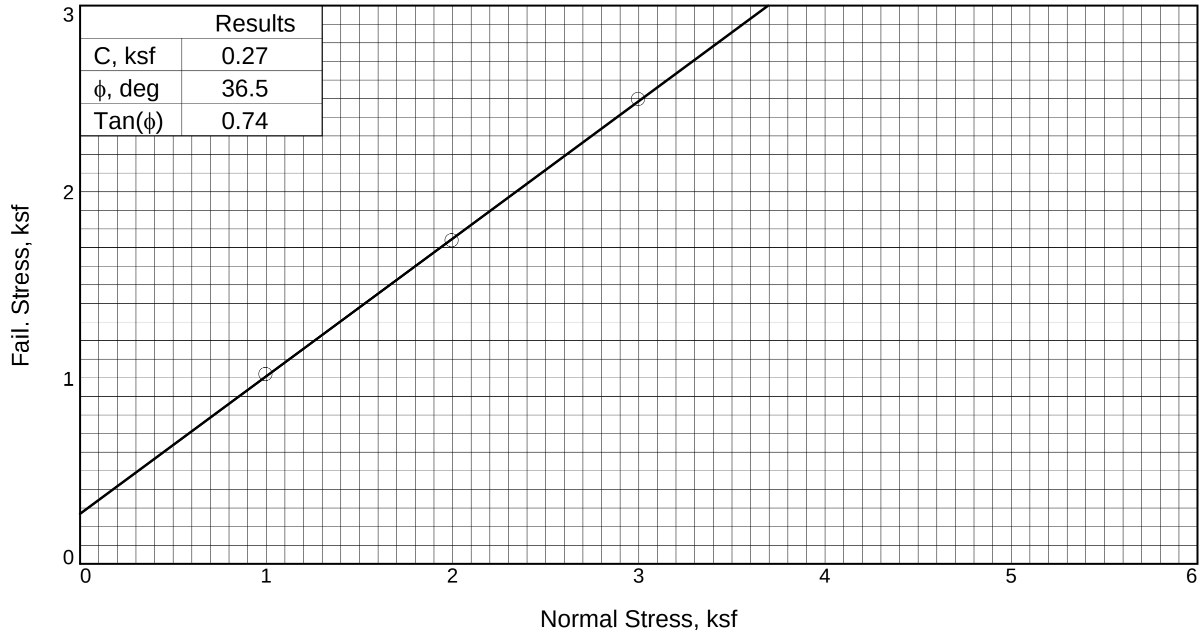
Title: Project Manager

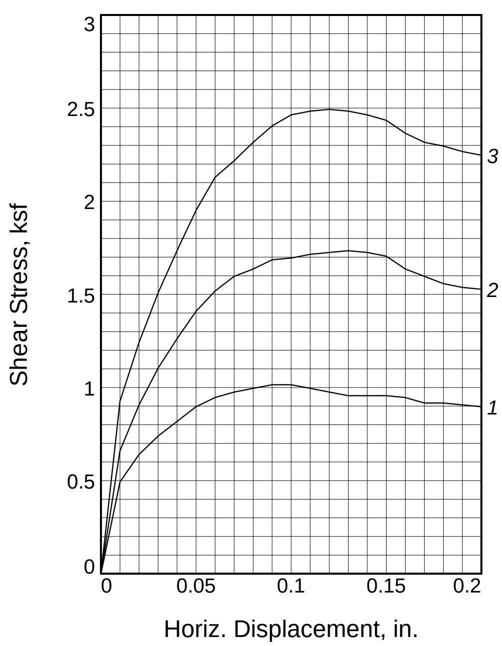
Source of Sample: B-3 **Depth:** 6 **Date Sampled:** 02/12/2019

Allied Geotechnical Engineers, Inc.

Santee, CA

Client: Rick Engineering Company
Project: South Mission Beach Project
Project No: 190 GS-18-D **Figure** B-4





Sample No.		1	2	3
Initial	Water Content, %	31.5	31.0	30.7
	Dry Density, pcf	94.1	95.8	94.6
	Saturation, %	113.0	116.0	111.6
	Void Ratio	0.7253	0.6944	0.7159
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	0.0	0.0	0.0
	Dry Density, pcf	95.4	96.8	95.3
	Saturation, %	0.0	0.0	0.0
	Void Ratio	0.7012	0.6774	0.7039
	Diameter, in.	2.38	2.38	2.38
	Height, in.	0.99	0.99	0.99
Normal Stress, ksf		1.00	2.00	3.00
Fail. Stress, ksf		1.02	1.73	2.49
Displacement, in.		0.09	0.13	0.12
Ult. Stress, ksf				
Displacement, in.				
Strain rate, in./min.		0.008	0.008	0.008

Sample Type: Ring
Description:

Assumed Specific Gravity= 2.6
Remarks:

Figure B-5

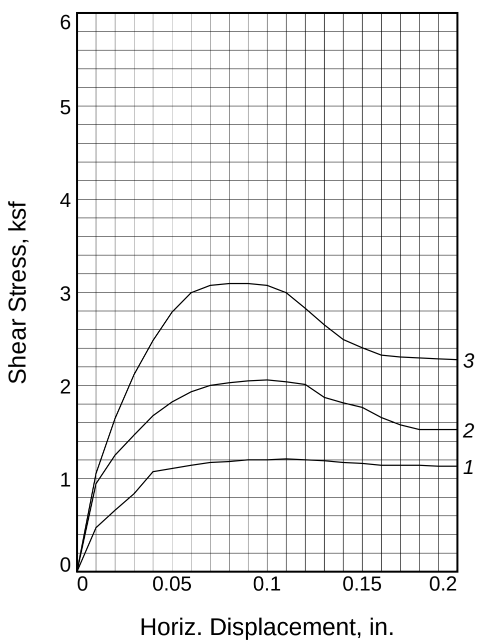
Client: Rick Engineering Company

Project: South Mission Beach Project

Source of Sample: B-1 **Depth:** 15
Sample Number: 5
Proj. No.: 190 GS-18-D **Date Sampled:** 02/12/2019

DIRECT SHEAR TEST REPORT
Allied Geotechnical Engineers, Inc.
Santee, CA

Tested By: Nicholas Barnes



Sample No.		1	2	3
Initial	Water Content, %	22.0	22.1	23.1
	Dry Density, pcf	103.1	102.8	101.4
	Saturation, %	99.7	99.1	99.8
	Void Ratio	0.5742	0.5795	0.6010
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	22.5	22.8	23.2
	Dry Density, pcf	105.7	105.0	102.8
	Saturation, %	109.5	108.3	104.4
	Void Ratio	0.5349	0.5463	0.5785
	Diameter, in.	2.38	2.38	2.38
	Height, in.	0.97	0.98	0.99
Normal Stress, ksf		1.00	2.00	3.00
Fail. Stress, ksf		1.21	2.06	3.09
Displacement, in.		0.11	0.10	0.08
Ult. Stress, ksf				
Displacement, in.				
Strain rate, in./min.		0.008	0.008	0.008

Sample Type: Ring
Description:

Assumed Specific Gravity= 2.6
Remarks:

Figure B-6

Client: Rick Engineering Company

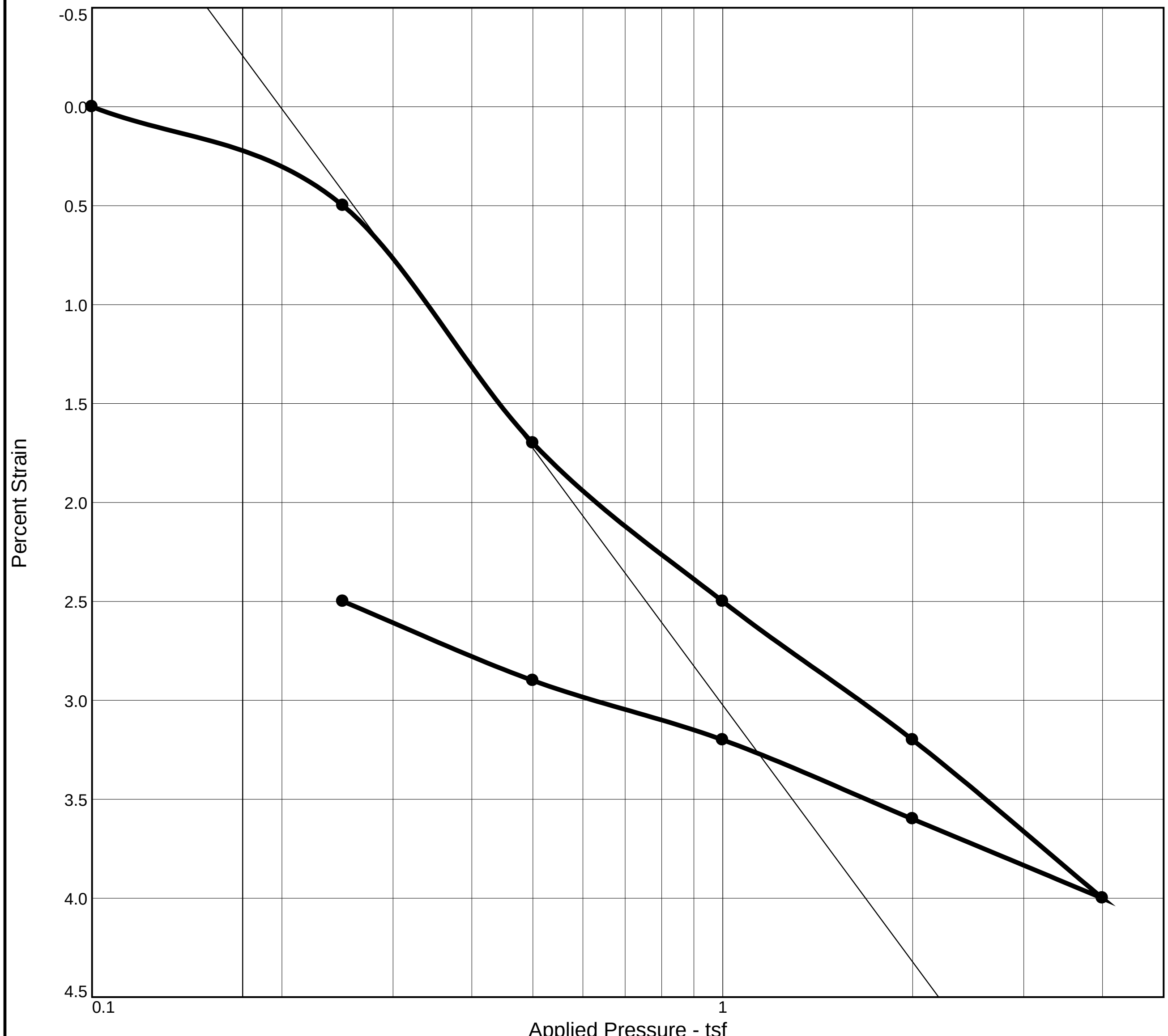
Project: South Mission Beach Project

Source of Sample: B-4 **Depth:** 5
Sample Number: 1
Proj. No.: 190 GS-18-D **Date Sampled:** 02/11/2019

DIRECT SHEAR TEST REPORT
Allied Geotechnical Engineers, Inc.
Santee, CA

Tested By: Nicholas Barnes

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
34.0 %	7.1 %	98.2			2.6	0.1	0.2	0.07		0.543

MATERIAL DESCRIPTION	USCS	AASHTO
Dark greenish gray poorly-graded sand with silt (SP-SM)		

Project No. 190 GS-18-D **Client:** Rick Engineering Company

Project: South Mission Beach Project

Source of Sample: B-1 **Depth:** 5 **Sample Number:** 1

Allied Geotechnical Engineers, Inc.

Santee, CA

Remarks:

Figure B-7

Tested By: Nicholas Barnes **Checked By:** Sani Sutanto

L A B O R A T O R Y   R E P O R T

Telephone (619) 425-1993   Fax 425-7917   Established 1928

C L A R K S O N L A B O R A T O R Y A N D S U P P L Y I N C.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
A N A L Y T I C A L A N D C O N S U L T I N G C H E M I S T S

Date: February 19, 2019
Purchase Order Number: 190GS18-D
Sales Order Number: 43345
Account Number: ALLG
To:
--
Allied Geotechnical Engineers
1810 Gillespie Way Ste 104
El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO7200-4   Customers Phone: 449-5900
Fax: 449-5902
Sample Designation:
--
One soil sample received on 02/15/19 at 9:00am,
from South Mission Beach Green Infrastructure Project marked as B-4#4@10'-11'

Analysis By California Test 643, 1999, Department of Transportation
Division of Construction, Method for Estimating the Service Life of
Steel Culverts.

pH 9.2

Water Added (ml)	Resistivity (ohm-cm)
10	2200
5	1500
5	1100
5	930
5	880
5	750
5	730
5	830
5	840

27 years to perforation for a 16 gauge metal culvert.
35 years to perforation for a 14 gauge metal culvert.
48 years to perforation for a 12 gauge metal culvert.
62 years to perforation for a 10 gauge metal culvert.
75 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417   0.014% (140 ppm)

Water Soluble Chloride Calif. Test 422   0.062% (620 ppm)

Bicarbonate (as $CaCO_3$)   46 ppm
(on a saturated soil paste extract)

Laura Torres

Laura Torres
LT/dbb

L A B O R A T O R Y   R E P O R T

Telephone (619) 425-1993   Fax 425-7917   Established 1928

C L A R K S O N   L A B O R A T O R Y   A N D   S U P P L Y   I N C.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
A N A L Y T I C A L   A N D   C O N S U L T I N G   C H E M I S T S

Date: February 19, 2019
Purchase Order Number: 190GS18-D
Sales Order Number: 43345
Account Number: ALLG
To:
--
Allied Geotechnical Engineers
1810 Gillespie Way Ste 104
El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO7200-3   Customers Phone: 449-5900
Fax: 449-5902
Sample Designation:
--
One soil sample received on 02/15/19 at 9:00am,
from South Mission Beach Green Infrastructure Project marked as B-3#3@9'-10'

Analysis By California Test 643, 1999, Department of Transportation
Division of Construction, Method for Estimating the Service Life of
Steel Culverts.

pH 9.3

Water Added (ml)	Resistivity (ohm-cm)
10	39000
5	29000
5	19000
5	14000
5	10000
5	8800
5	7700
5	8300
5	9300

71 years to perforation for a 16 gauge metal culvert.
92 years to perforation for a 14 gauge metal culvert.
127 years to perforation for a 12 gauge metal culvert.
162 years to perforation for a 10 gauge metal culvert.
198 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417   0.003% (30 ppm)

Water Soluble Chloride Calif. Test 422   0.003% (30 ppm)

Bicarbonate (as $CaCO_3$)   66 ppm
(on a saturated soil paste extract)

Laura Torres
____
Laura Torres
LT/dbb

LABORATORY REPORT

Telephone (619) 425-1993 Fax 425-7917 Established 1928

CLARKSON LABORATORY AND SUPPLY INC.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
ANALYTICAL AND CONSULTING CHEMISTS

Date: February 19, 2019
Purchase Order Number: 190GS18-D
Sales Order Number: 43345
Account Number: ALLG
To:

Allied Geotechnical Engineers
1810 Gillespie Way Ste 104
El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO7200-2 Customers Phone: 449-5900
Fax: 449-5902
Sample Designation:

One soil sample received on 02/15/19 at 9:00am,
from South Mission Beach Green Infrastructure Project marked as B-2#3@8'-9'

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 9.3

Water Added (ml)	Resistivity (ohm-cm)
10	13000
5	9500
5	6900
5	5100
5	4000
5	3500
5	3200
5	3500
5	3600

49 years to perforation for a 16 gauge metal culvert.
64 years to perforation for a 14 gauge metal culvert.
89 years to perforation for a 12 gauge metal culvert.
113 years to perforation for a 10 gauge metal culvert.
138 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417 0.007% (70 ppm)

Water Soluble Chloride Calif. Test 422 0.005% (50 ppm)

Bicarbonate (as $CaCO_3$) 66 ppm
(on a saturated soil paste extract)

Laura Torres

Laura Torres
LT/dbb

L A B O R A T O R Y   R E P O R T

Telephone (619) 425-1993   Fax 425-7917   Established 1928

C L A R K S O N L A B O R A T O R Y A N D S U P P L Y I N C.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
A N A L Y T I C A L A N D C O N S U L T I N G C H E M I S T S

Date: February 19, 2019
Purchase Order Number: 190GS18-D
Sales Order Number: 43345
Account Number: ALLG
To:

Allied Geotechnical Engineers
1810 Gillespie Way Ste 104
El Cajon, CA 92020
Attention: Sani Sutanto

Laboratory Number: SO7200-1   Customers Phone: 449-5900
Fax: 449-5902

Sample Designation:

One soil sample received on 02/15/19 at 9:00am,
from South Mission Beach Green Infrastructure Project
marked as B-1#4@14'-15'.

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 8.3

Water Added (ml)	Resistivity (ohm-cm)
20	270
5	220
5	140
5	130
5	130
5	130
5	130
5	140
5	160

13 years to perforation for a 16 gauge metal culvert.
17 years to perforation for a 14 gauge metal culvert.
24 years to perforation for a 12 gauge metal culvert.
30 years to perforation for a 10 gauge metal culvert.
37 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417 — 0.105% (1050 ppm)

Water Soluble Chloride Calif. Test 422 — 0.363% (3630 ppm)

Bicarbonate (as $CaCO_3$) — 46 ppm
(on a saturated soil paste extract)

Laura Torres

Laura Torres
LT/dbb

SCST, LLC - San Diego
LEA: 47, Exp: 04/25/2021
6280 Riverdale Street
San Diego, CA 92120
Phone: (619) 280-4321
Fax: (619) 280-4717

R-Value

Cal 301, ASTM D2844

Report Date: 3/11/2019

Client:	**Project:**
Allied Geotechnical Engineering 9500 Cuyamaca Street #102 Santee, CA 92071-2685	180035L Allied Geotechnical 2018 Lab Testing 9500 Cuyamaca Street Suite 102 Santee CA 9207...

In accordance with your request, SCST has performed the subject laboratory testing. Test results are presented in the attached report.

If you have any additional questions or concerns, please contact us at 619.280.4321

Respectfully Submitted,
SCST, Inc.

In accordance with your request, SCST has performed the subject laboratory testing. Test results are presented in the attached report.

If you have any additional questions or concerns, please contact us at 619.280.4321

Respectfully Submitted,
SCST, Inc.

In accordance with your request, SCST has performed the subject laboratory testing. Test results are presented in the attached report. See R-Value 37891.pdf in the documents section at the end of this report.

If you have any additional questions or concerns, please contact us at 619.280.4321

Respectfully Submitted,
SCST, Inc.

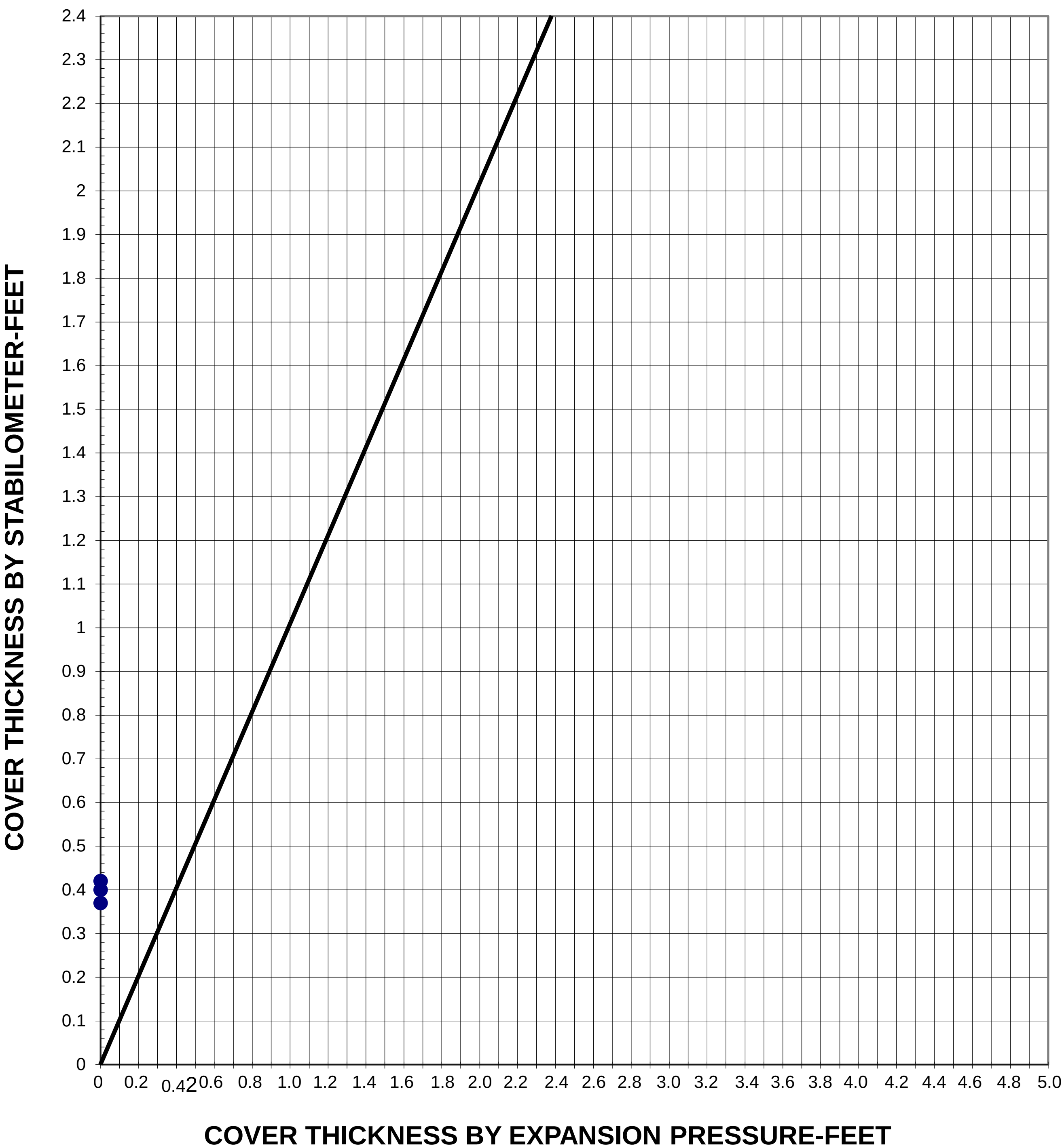
SCST, LLC
Corporate Headquarters
6280 Riverdale Street
San Diego, CA 92120
T 877.215.4321
P 619.280.4321
F 619.280.4717
W www.scst.com

Job Name:	Allied Geotechnical 2018 Lab Testing	**Job Number:**	180035L
Client:	Allied Geotechnical Engineering	**Sample No.:**	37891
Date:	3/5/2019	**By:**	DRB
Location:	B-4-2 @ 5'-8'		
Description:	Light Tan Sand		

CTM 301 Resistance Value of Treated and Untreated Bases, Subbases and Basement Soils

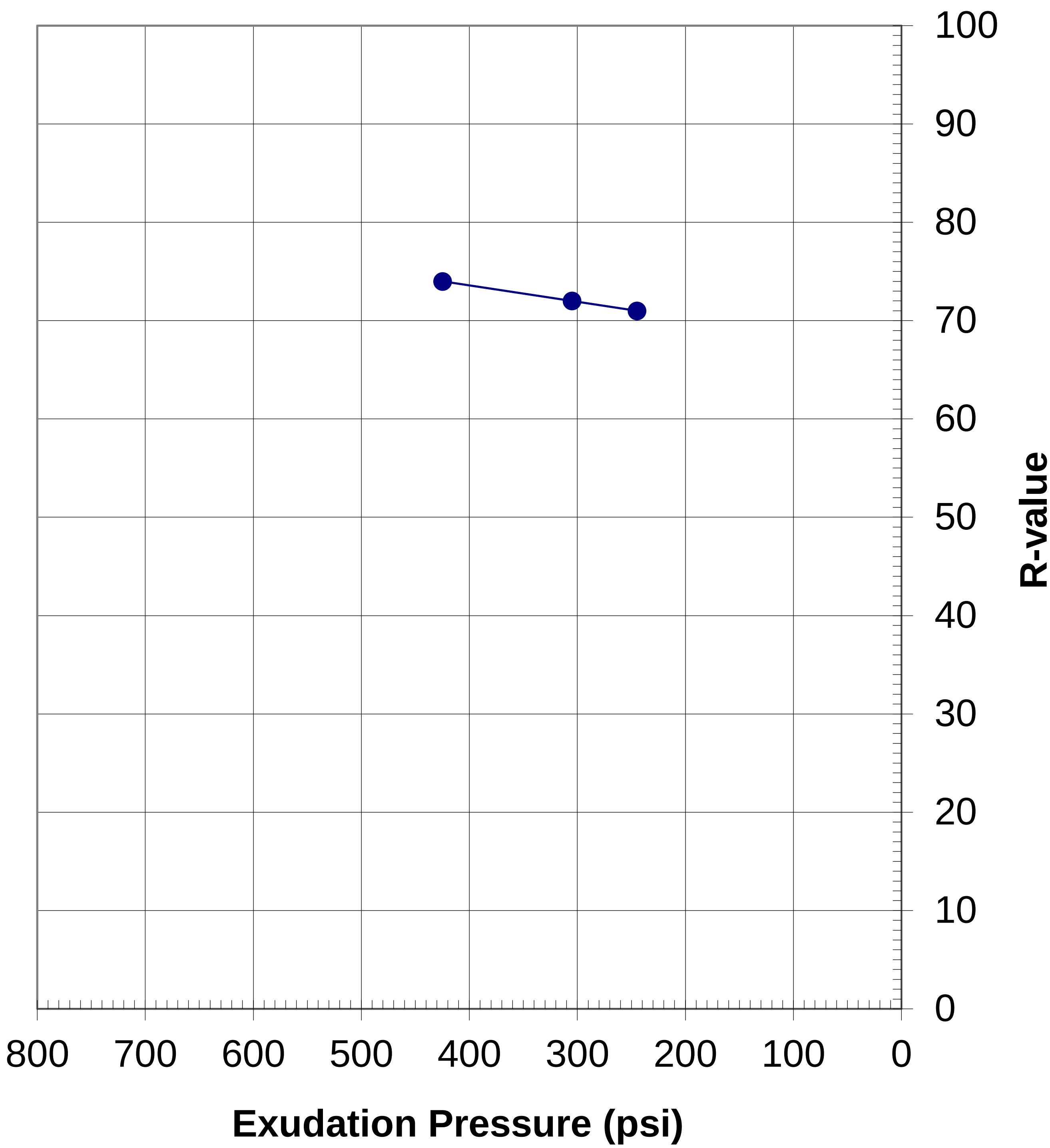
Test Specimen		**A**	**B**	**C**	**D**
Date Tested		3/5/2019	3/5/2019	3/5/2019	
Compactor Air Pressure	**PSI**	350	350	350	
Initial Moisture	**%**	0.4	0.4	0.4	
Soil Wt. Added	**GRAMS**	850	850	850	
Water Added	**ML**	90	103	84	
Water Added	**%**	10.6	12.2	9.9	
Moisture At Compaction	**%**	11	12.6	10.3	
Weight of Briquette & Tare	**GRAMS**	2983	2985	2979	
Net Weight of Briquette	**GRAMS**	929	944	924	
Briquette Height	**IN**	2.49	2.52	2.52	
Density	**PCF**	101.8	100.8	100.7	
Exudation Pressure	**PSI**	305	245	425	
Expansion Pressure	**PSF**	0	0	0	
PH at 1000 Pounds	**PSI**	13	13	12	
PH at 2000 Pounds	**PSI**	23	24	22	
Displacement	**Turns**	5.65	5.70	5.60	
R' Value		72	71	74	
Stabilometer Thickness	**FT**	0.4	0.42	0.37	
Expansion Thickness	**FT**	0	0	0	
Expansion Dial Reading		0000	0000	0000	
R' Value Modifier		0	0	0	
Corrected R-Value		72	71	74	
R-Value by Exudation Pressure			72		
Gravel Equivalent		0	0	0	
Traffic Index		4.5	4.5	4.5	
R-Value by Expansion Pressure			N/A		
R-Value at Equivalent			**72**		

EXPANSION PRESSURE CHART



SCST	Job Name:	Allied Geotechnical 2018 Lab Testing		
	By:	DRB	Date:	3/5/2019
	Job No.:	180035L	Sample No.:	B-4-2 @ 5'-8'
	Gravel Equ:	0	Plate No.:	

R-value By Exudation Pressure



Job Name:	Allied Geotechnical 2018 Lab Testing		
By:	DRB	Date:	3/5/2019
Job No.:	180035L	Sample No.:	B-4-2 @ 5'-8'
R-Value by Ex.:	72	Plate No.:	

R-Value

Cal 301, ASTM D2844

Report Date: 3/11/2019

SCST, LLC - San Diego
LEA: 47, Exp: 04/25/2021
6280 Riverdale Street
San Diego, CA 92120
Phone: (619) 280-4321
Fax: (619) 280-4717

Client:	**Project:**
Allied Geotechnical Engineering 9500 Cuyamaca Street #102 Santee, CA 92071-2685	180035L Allied Geotechnical 2018 Lab Testing 9500 Cuyamaca Street Suite 102 Santee CA 9207...

In accordance with your request, SCST has performed the subject laboratory testing. Test results are presented in the attached report.

If you have any additional questions or concerns, please contact us at 619.280.4321

Respectfully Submitted,
SCST, Inc.

In accordance with your request, SCST has performed the subject laboratory testing. Test results are presented in the attached report.

If you have any additional questions or concerns, please contact us at 619.280.4321

Respectfully Submitted,
SCST, Inc.

In accordance with your request, SCST has performed the subject laboratory testing. Test results are presented in the attached report. See R-Value 37892.pdf in the documents section at the end of this report.

If you have any additional questions or concerns, please contact us at 619.280.4321

Respectfully Submitted,
SCST, Inc.

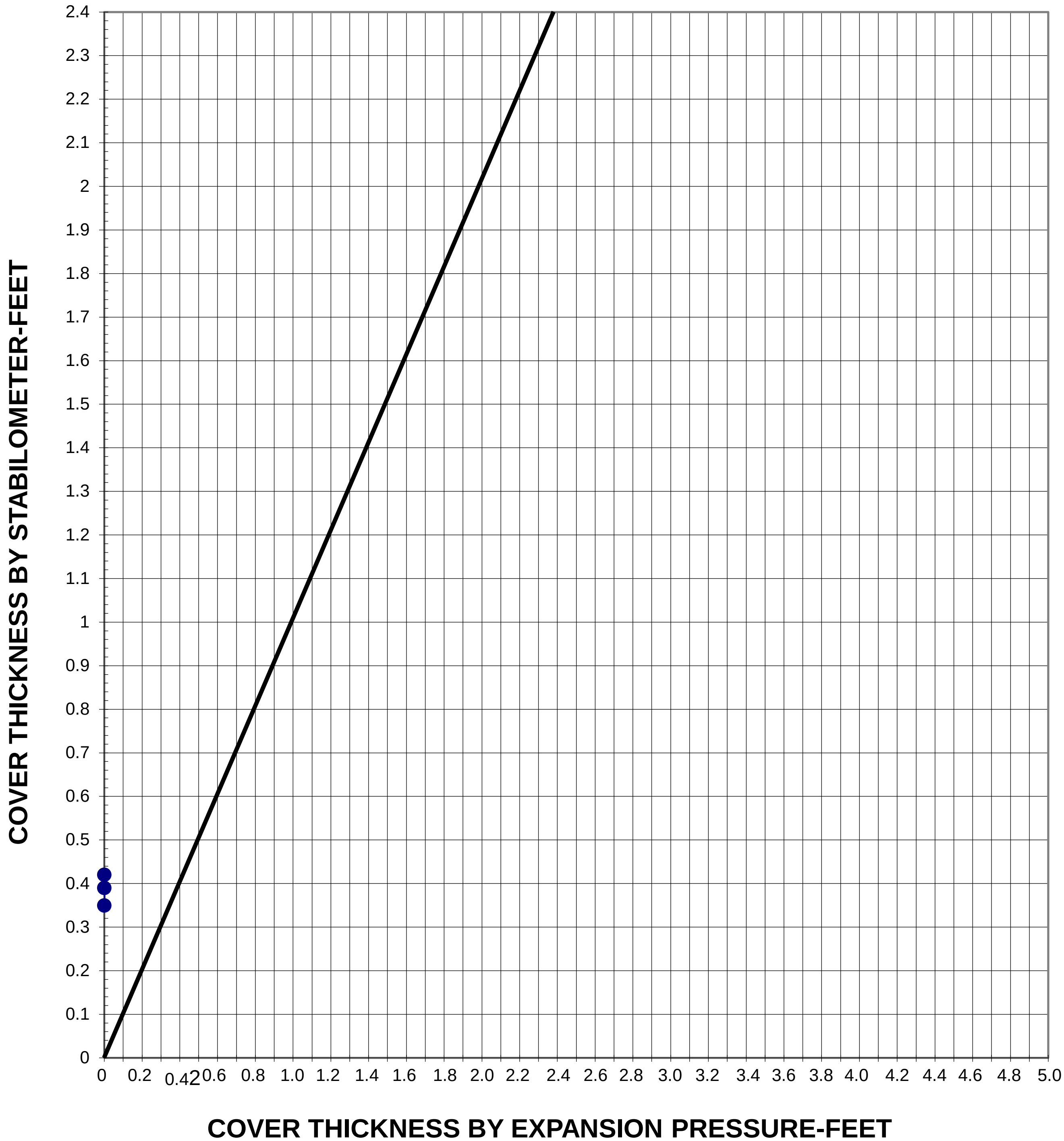
SCST, LLC
Corporate Headquarters
6280 Riverdale Street
San Diego, CA 92120
T 877.215.4321
P 619.280.4321
F 619.280.4717
W www.scst.com

Job Name:	Allied Geotechnical 2018 Lab Testing	**Job Number:**	180035L
Client:	Allied Geotechnical Engineering	**Sample No.:**	37892
Date:	3/5/2019	**By:**	DRB
Location:	B-2-2 @ 5'-8'		
Description:	Light Grey Brown Silty Sand		

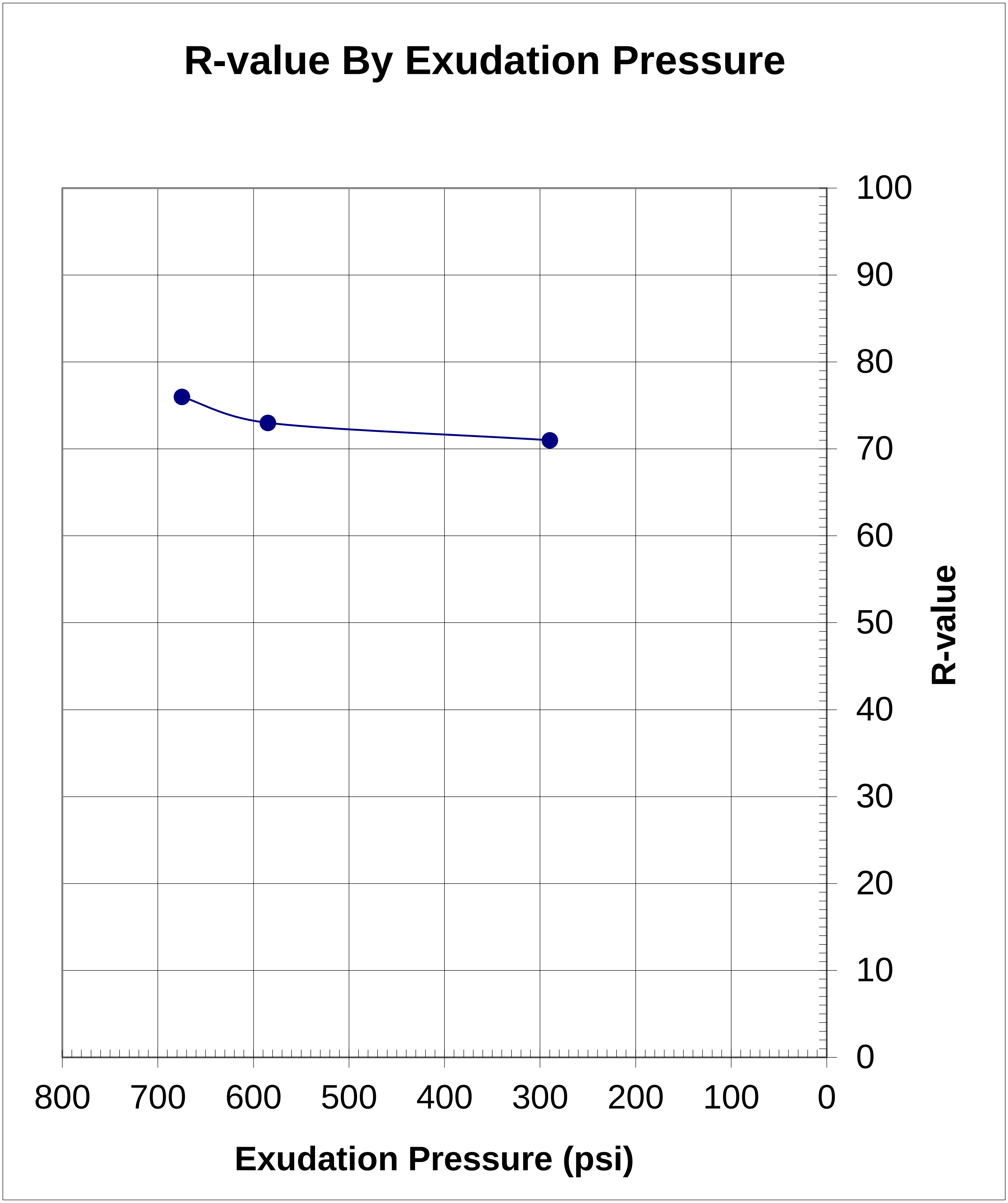
CTM 301 Resistance Value of Treated and Untreated Bases, Subbases and Basement Soils

Test Specimen		A	B	C	D
Date Tested		3/5/2019	3/5/2019	3/5/2019	
Compactor Air Pressure	PSI	350	350	350	
Initial Moisture	%	0.7	0.7	0.7	
Soil Wt. Added	GRAMS	910	900	890	
Water Added	ML	85	95	108	
Water Added	%	9.4	10.6	12.2	
Moisture At Compaction	%	10.1	11.3	12.9	
Weight of Briquette & Tare	GRAMS	3100	3097	3101	
Net Weight of Briquette	GRAMS	986	985	989	
Briquette Height	IN	2.56	2.53	2.49	
Density	PCF	106.0	106.0	106.6	
Exudation Pressure	PSI	675	585	290	
Expansion Pressure	PSF	0	0	0	
PH at 1000 Pounds	PSI	14	14	15	
PH at 2000 Pounds	PSI	24	25	26	
Displacement	Turns	5.00	5.10	5.20	
R' Value		74	73	71	
Stabilometer Thickness	FT	0.35	0.39	0.42	
Expansion Thickness	FT	0	0	0	
Expansion Dial Reading		0000	0000	0000	
R' Value Modifier		2	0	0	
Corrected R-Value		76	73	71	
R-Value by Exudation Pressure			71		
Gravel Equivalent		0	0	0	
Traffic Index		4.5	4.5	4.5	
R-Value by Expansion Pressure			N/A		
R-Value at Equivalent			**71**		

EXPANSION PRESSURE CHART



Job Name:	Allied Geotechnical 2018 Lab Testing		
By:	DRB	Date:	3/5/2019
Job No.:	180035L	Sample No.:	B-2-2 @ 5'-8'
Gravel Equ:	0	Plate No.:	



Job Name:	Allied Geotechnical 2018 Lab Testing		
By:	DRB	Date:	3/5/2019
Job No.:	180035L	Sample No.:	B-2-2 @ 5'-8'
R-Value by Ex.:	71	Plate No.:	

APPENDIX C

RCP PIPES BUOYANCY CALCULATIONS

Project No. 190 GS-18-D
Appendix C, Sheet 1

South Mission Beach Green Infrastructure RCP Pipes Buoyancy Calculations

Pipe Material: Reinforced Concrete
Pipe Dimensions:

OD	= 54 inches = 4.5 feet
ID	= 48 inches = 4.0 feet
Conc. Unit Weight	= 150 pcf
Compacted Fill Unit Weight Above Water	= 130 pcf
Compacted Fill Unit Weight Below Water	= 125 pcf (include 62.4 pcf for unit weight of water)
Water Unit Weight	= 62.4 pcf

Note: Assume pipe in empty for calculation purposes & GW level at ground surface elevation.

Pipe Weight per Unit Length =
$((3.14 \times 2.25^2) - (3.14 \times 2.0^2))$ cf/ft x 150 pcf =
(15.9 - 12.6) cf/ft x 150 pcf =
3.3 cf/ft x 150 pcf = 495.0 pounds/ft

Bouyant Uplift per Unit Length of Pipe =
(3.14×2.25^2) cf/ft x 62.4 pcf = 992.2 pounds/ft

Excess Bouyant Uplift =
992.2 pounds/ft - 495.0 pounds/ft = 497.2 pounds/ft

Minimum Thickness of Compacted Backfill Cover = (497.2 pounds/ft / (4.5 feet x 125 pcf)) x 1.5 (F.S.) =
1.325 foot rounded to 1.5 foot.

South Mission Beach Green Infrastructure RCP Pipes Buoyancy Calculations

Pipe Material: Reinforced Concrete
Pipe Dimensions:

OD	= 42 inches = 3.5 feet
ID	= 36 inches = 3.0 feet
Conc. Unit Weight	= 150 pcf
Compacted Fill Unit Weight Above Water	= 130 pcf
Compacted Fill Unit Weight Below Water	= 125 pcf (include 62.4 pcf for unit weight of water)
Water Unit Weight	= 62.4 pcf

Note: Assume pipe in empty for calculation purposes & GW level at ground surface elevation.

Pipe Weight per Unit Length =
$((3.14 \times 1.75^2) - (3.14 \times 1.5^2))$ cf/ft x 150 pcf =
$(9.6 - 7.1)$ cf/ft x 150 pcf =
2.5 cf/ft x 150 pcf = 375 pounds/ft

Bouyant Uplift per Unit Length of Pipe =
(3.14×1.75^2) cf/ft x 62.4 pcf = 600.0 pounds/ft

Excess Bouyant Uplift =
600.0 pounds/ft - 375.0 pounds/ft = 225.0 pounds/ft

Minimum Thickness of Compacted Backfill Cover = (225.0 pounds/ft / (3.5 feet x 125 pcf)) x 1.5 (F.S.) =
0.77 foot rounded to 1.0 foot.

Project No. 190 GS-18-D
Appendix C, Sheet 3

South Mission Beach Green Infrastructure RCP Pipes Buoyancy Calculations

Pipe Material: Reinforced Concrete
Pipe Dimensions:
OD = 36 inches = 3.0 feet
ID = 30 inches = 2.5 feet
Conc. Unit Weight = 150 pcf
Compacted Fill Unit Weight Above Water = 130 pcf
Compacted Fill Unit Weight Below Water = 125 pcf (include 62.4 pcf for unit weight of water)
Water Unit Weight = 62.4 pcf

Note: Assume pipe in empty for calculation purposes & GW level at ground surface elevation.

Pipe Weight per Unit Length =
$((3.14 \times 1.5^2) - (3.14 \times 1.25^2))$ cf/ft x 150 pcf =
(7.1 - 4.9) cf/ft x 150 pcf =
2.2 cf/ft x 150 pcf = 330.0 pounds/ft

Bouyant Uplift per Unit Length of Pipe =
(3.14×1.5^2) cf/ft x 62.4 pcf = 443.0 pounds/ft

Excess Bouyant Uplift =
443.0 pounds/ft - 330.0 pounds/ft = 113.0 pounds/ft

Minimum Thickness of Compacted Backfill Cover = (113.0 pounds/ft / (3 feet x 125 pcf)) x 1.5 (F.S.) =
0.452 foot rounded to 0.5 foot.

Project No. 190 GS-18-D
Appendix C, Sheet 4

South Mission Beach Green Infrastructure RCP Pipes Buoyancy Calculations

Pipe Material: Reinforced Concrete
Pipe Dimensions:
OD = 22 inches = 1.83 feet
ID = 18 inches = 1.5 feet
Conc. Unit Weight = 150 pcf
Compacted Fill Unit Weight Above Water = 130 pcf
Compacted Fill Unit Weight Below Water = 125 pcf (include 62.4 pcf for unit weight of water)
Water Unit Weight = 62.4 pcf

Note: Assume pipe in empty for calculation purposes & GW level at ground surface elevation.

Pipe Weight per Unit Length =
$((3.14 \times 0.915^2) - (3.14 \times 0.75^2))$ cf/ft x 150 pcf =
(2.64 - 1.77) cf/ft x 150 pcf =
0.87 cf/ft x 150 pcf = 130.5 pounds/ft

Bouyant Uplift per Unit Length of Pipe =
(3.14×0.915^2) cf/ft x 62.4 pcf = 164.7 pounds/ft

Excess Bouyant Uplift =
164.7 pounds/ft - 130.5 pounds/ft = 34.2 pounds/ft

Minimum Thickness of Compacted Backfill Cover = (34.2 pounds/ft / (1.83 feet x 125 pcf)) x 1.5 (F.S.) =
0.22 foot rounded to 0.5 foot.

Project No. 190 GS-18-D
Appendix C, Sheet 5

South Mission Beach Green Infrastructure PVC Pipes Buoyancy Calculations

Pipe Material: PVC Schedule 40
Pipe Dimensions:

OD	= 6.625 inches = 0.552 feet
ID	= 6.065 inches = 0.505 feet
PVC pipe weight per foot of length	= 3 lbs
Compacted Fill Unit Weight Above Water	= 130 pcf
Compacted Fill Unit Weight Below Water	= 125 pcf (include 62.4 pcf for unit weight of water)
Water Unit Weight	= 62.4 pcf

Note: Assume pipe in empty for calculation purposes & GW level at ground surface elevation.

Pipe Weight per Unit Length = 3 pounds/ft

Bouyant Uplift per Unit Length of Pipe =
(3.14×0.276^2) cf/ft x 62.4 pcf = 14.9 pounds/ft - rounded to 15 pounds/ft

Excess Bouyant Uplift =
14.9 pounds/ft - 3.0 pounds/ft = 11.9 pounds/ft

Minimum Thickness of Compacted Backfill Cover = (11.9 pounds/ft / (0.552 feet x 125 pcf)) x 1.5 (F.S.) =
0.25 foot rounded to 0.5 foot.