REPORT OF GEOTECHNICAL INVESTIGATION
SEWER GROUP 806 BRIDGE CROSSING PROJECT
CITY OF SAN DIEGO

Submitted to:

MICHAEL BAKER INTERNATIONAL
5050 Avenida Encinas, Suite 260
Carlsbad, CA 92008

Prepared By:

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

AGE Project No. 185 GS-16-B

August 23, 2018
(Revised October 23, 2018)

AGE

Allied Geotechnical Engineers, Inc.

August 23, 2018
(Revised October 23, 2018)

Mr. Bo Burick, P.E.
Vice President
Michael Baker International
5050 Avenida Encinas, Suite 260
Carlsbad, CA 92008

Subject: **REPORT OF GEOTECHNICAL INVESTIGATION**
SEWER GROUP 806 BRIDGE CROSSING PROJECT
CITY OF SAN DIEGO
AGE Project No. 185 GS-16-B

Dear Mr. Burick:

Allied Geotechnical Engineers, Inc. is pleased to submit the accompanying report to present the findings, opinions, and recommendations of a geotechnical investigation that was performed to assist Michael Baker International 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.

Nichola Bar

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

No. 5363
Nicholas E. Barnes
No. 1893
Exp 05/31/2020

Sani Sutanto, P.E.
Senior Engineer

SANI SUTANTO
No. 51873
Exp 06/30/2020

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

REPORT OF GEOTECHNICAL INVESTIGATION
SEWER GROUP 806 BRIDGE CROSSING PROJECT
CITY OF SAN DIEGO

TABLE OF CONTENTS

TABLE OF CONTENTS
(Continued)

TABLE OF CONTENTS
(Continued)

Figures

TABLE OF CONTENTS
(Continued)

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 Michael Baker International (MBI) with their design of the Sewer Group 806 Bridge Crossing Project for the City of San Diego (City). The investigation was performed in conformance with AGE's proposal dated December 15, 2016 (Revised February 13, 2018), and the subconsultant agreement entered into by and between MBI and AGE on June 5, 2017.

This report has been prepared for the exclusive use of MBI 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 site is located in a northeast trending natural canyon in the gated Alvarado Estates development in the communities of Talmadge and College Heights in the City of San Diego. The approximate location of the project site is shown on the Location Map (Figure 1). Access to the project site is via an existing sewer access road. The entrance to the road is located approximately 1,000 feet southwest of the project site at the intersection of Yerba Santa Drive and Palo Verde Terrace. Nearby residential homesites are built along ridgelines overlooking the canyon, with the canyon bottom and side walls left largely undisturbed as open space. The canyon walls are moderately steep, with gradients of up to 3:1 (horizontal:vertical). The canyon is densely vegetated with mostly chaparral vegetation. Site elevations vary from approximately 188 feet above mean sea level (msl) to 194 feet msl. Nearby land uses include residential developments and open space.

A review of the project plans (MBI, undated) indicate that the Sewer Group 806 Bridge Crossing Project consists of the design and construction of a single span bridge at the location where the sewer access road crosses a small streambed between Manhole Nos. 156 and 219 as shown on the Site Plan (see Figure 2). The streambed is on the order of 10-feet in width and 4-feet in depth. We understand that the proposed bridge will have a minimum of 10 feet clear width and will be used to provide access for City maintenance crews and equipment. We further understand that the proposed bridge will be supported two 24-inch diameter Cast-In-Drilled-Hole (CIDH) piles at each abutment.

3.0 OBJECTIVE AND SCOPE OF INVESTIGATION

The objectives of this investigation were to characterize the subsurface conditions at the southwest and northeast sides of the stream crossing 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 proposed project, including the preliminary project plans, as-built utility maps, topographic maps, published geologic literature and maps, and AGE's in-house references.

3.2 Geotechnical Field Exploration

The field exploration program for this project was performed on July 31, 2018. One (1) soil boring and one (1) test pit were performed at the approximate locations shown on Figure 2. The soil boring was advanced with an all-terrain drill rig to a depth of 28 feet below the existing ground surface (bgs) and the test pit was excavated by manual labor to a depth of 5 feet bgs. A more detailed description of the excavation and sampling activities, and logs of the soil boring and test pit are presented in Appendix A.

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 boring and test pit. Subsequently, Underground Service Alert (USA) was contacted to coordinate clearance of the proposed boring and test pit locations with respect to existing buried utilities. Our utility clearance efforts revealed that the existing sewer pipeline is the only buried utility in the study area.

3.3 Laboratory Testing

Selected soil samples obtained from the soil boring and test pit 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, and shear strength. 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. A brief description of the tests that were performed and the final test results are presented in Appendix B.

4.0 GEOLOGIC CONDITIONS

4.1 Geologic Setting and Site Physiography

The project study area is located in a northeast trending canyon that drains to Alvarado Creek. Mapped geologic units in the study area consist of nearly flat-lying to gently southwest dipping, marine and non-marine sediments which range from Holocene to Eocene in age. Metavolcanic and metasedimentary basement rocks of Jurassic/Cretaceous age are mapped northeast of the project study area in Del Cerro and in Alvarado Creek near the SDSU campus (Kennedy and Tan, 2008). The basement rocks are non-conformably overlain by the sedimentary deposits. Shallow man-made fills are also present along the access road.

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 soil boring and test pit can be categorized into three geologic units which include (in order of increasing age) fill materials; young colluvial deposits; and Stadium Conglomerate. A brief description of each unit is presented below.

4.3.1 Fill Materials

Fill materials less than 1foot in thickness were encountered in the soil boring and test pit. The fill materials generally consist of silty sands containing scattered to abundant sub-rounded gravel and cobbles. The fill was in a dry condition, and is likely associated with construction of the sewer access roadway. Documentation pertaining to the original placement of the fill materials is unavailable.

4.3.2 Young Colluvial Deposits

Young colluvial deposits of Holocene to late Quaternary age were encountered to depths of up to 6-feet bgs in the soil boring and test pit. Although not mapped in the study area on the geologic map prepared by Kennedy and Tan (2008), similar colluvial deposits are described as poorly consolidated and poorly sorted sand and silt slopewash deposits. These deposits can generally be easily excavated with conventional heavy duty construction equipment.

The young colluvial deposits encountered in the test excavations generally consist of brown to dark brown silty sands and clayey sands with locally abundant sub-rounded gravel and cobbles up to 10" in maximum dimension. The deposits were in a loose to medium dense condition, and damp to wet.

4.3.3 Stadium Conglomerate

The Eocene age Stadium Conglomerate was encountered below young colluvial deposits at the soil boring and test pit, and extended to the maximum depths of exploration. The Stadium Conglomerate consists of a massive cobble-conglomerate with a yellowish brown silty sand matrix that is locally strongly cemented (Kennedy and Tan, 2008). The clasts are generally of rhyolite, dacite and quartzite composition, and are typically well rounded, elongated and flattened. The conglomerate is locally interbedded with lenses and layers of sandstone that is similar in composition to the matrix (Kennedy and Tan, 2008). The combination of strong cementation and locally abundant gravels and cobbles may pose difficult excavation conditions even for heavy duty construction equipment.

Stadium Conglomerate encountered in our soil boring and test pit consists of a dense to very dense cobble-conglomerate with a pale yellow, medium-grained sandstone matrix in a damp to wet condition.

4.4 Groundwater

At the time of our field investigation, groundwater was measured at a depth of 13 feet bgs in boring B-1 approximately 1 hour after the completion of drilling. Groundwater encountered in the boring appears to be a perched condition. Stadium Conglomerate encountered in the boring and test pit generally possesses very low permeability characteristics.

The database available at the Geotracker website (www.Geotracker.com) includes an environmental assessment report for Sullivan United, a storage and transfer company which formerly occupied a building located at 4660 Alvarado Canyon Road. This site is located approximately 1,300 feet northwest of the project study area north of the I-8 freeway and south of Alvarado Creek, at an elevation of 115 feet msl. In a Case Closure Summary prepared by the County of San Diego DEH (DEH, 2005) it was reported that the measured depth to groundwater varied from 27 feet to 63 feet bgs in 12 monitoring wells on the site.

Based on a review of the available data, the depth (elevation) of the regional groundwater table in the project study area is estimated to be well below the anticipated depths of excavation. However, it is considered probable that perched water conditions may be encountered during construction, especially during the rainy (wet) season.

5.0 DISCUSSIONS, OPINIONS AND RECOMMENDATIONS

5.1 Potential Geologic Hazards

The project study area is classified in the City of San Diego Seismic Safety Study (2008) as Hazard Category 53, “Level or sloping terrain, unfavorable geologic structure, low to moderate risk”. The classification is not expected to impact the proposed project. Based on the results of our study, several potential geologic hazards in the project study area are more fully described herein.

5.1.1 Faulting

There are no known (mapped) active faults in the project study zone. 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. The project study area is 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 study area (Kennedy, 1975; City of San Diego, 2008). Therefore, the potential for fault ground rupture and ground lurching 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.

The findings of our investigation indicate that the project study area is underlain with dense to very dense formational soils that are not considered to be liquefiable.

5.1.4 Landslides

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

5.1.5 Lateral Spread Displacement

The project study area is underlain by competent geologic units which are not considered susceptible to seismic-induced lateral spreading.

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 the young colluvial deposits.

5.1.7 Secondary Hazards

Given the elevation of the project study area and absence of large bodies of water, it is our opinion that the potential of property damage from seismic-induced tsunamis and/or seiches is considered remote. The project study area is not located within the 100- and 500-year flood zone (FEMA Flood Insurance Rate Map, 2012). There is a potential of property damage due to flooding during strong rainstorm events. A scour study was not performed for the proposed project.

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 samples 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 1 below. Copies of the analytical laboratory test data reports are included in Appendix B.

Table 1

Summary of Corrosivity Test Results

	pH	**Resistivity (ohm-cm)**	**Sulfate Conc. (ppm)**	**Chloride Conc. (ppm)**	**Bicarbonates Conc. (ppm)**
B-1 Sample No. 1 @3'-5'	7.8	570	160	85	120
TP-2 Sample No. 2 @4'-4.5'	7.9	340	170	830	40

The test results indicate that the soils at the project site are considered aggressive to concrete. Therefore, AGE recommends that Type V Portland Cement Concrete (high sulfate resistance) be used for proposed facilities at the project site. It should be noted here that the most effective way to prevent sulfate/chloride attack is to keep the sulfate/chloride 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 on-site materials are considered non-expansive or have a very low expansion potential.

5.4 Seismic Design Parameters

5.4.1 Caltrans Seismic Design Parameters

AGE has developed deterministic and probabilistic acceleration response spectra (ARS) curves for the bridge based on the methodology presented in Caltrans “Methodology for Developing Design

Response Spectrum for Use in Seismic Design Recommendations" (November 2012). The curves were developed using the current web-based Caltrans and United States Geological Survey (USGS) software.

A summary of the three closest fault zones to the project site is presented in Table 2 on the next page.

Table 2

Summary of Fault Parameters

	Rose Canyon Fault Zone (Silver Strand Section - Downtown Graben Fault)
Maximum Moment Magnitude	6.8
Fault Type	Strike-Slip (SS)
Fault Dip Angle	90 degree
Dip Direction	Vertical
Bottom of Rupture Plane	8 km
Top of Rupture Plane	0
Rrup*	8.471 km
Rjb*	8.471 km
Rx*	6.573 km
Fnorm*	0
Frev*	0

	Rose Canyon Fault Zone (San Diego Section)
Maximum Moment Magnitude	6.8
Fault Type	Strike-Slip (SS)
Fault Dip Angle	90 degree
Dip Direction	Vertical
Bottom of Rupture Plane	8 km
Top of Rupture Plane	0
Rrup*	8.988 km
Rjb*	8.988 km
Rx*	8.987 km
Fnorm*	0
Frev*	0

Table 2 (Continued)

Summary of Fault Parameters

	Rose Canyon fault zone (Silver Strand section-Silver Strand fault)
Maximum Moment Magnitude	6.8
Fault Type	Strike-Slip (SS)
Fault Dip Angle	90 degree
Dip Direction	Vertical
Bottom of Rupture Plane	8 km
Top of Rupture Plane	0
Rrup*	9.400 km
Rjb*	9.400 km
Rx*	7.065 km
Fnorm*	0
Frev*	0

* **Definition of Terms in Table 2**

Rrup -	Closest distance (km) to the fault rupture plane.
Rjb -	Joyner-Boore distance: The shortest horizontal distance to the surface projection of the rupture area. Rjb is zero if the site is located within that area.
Rx -	Horizontal distance to the fault trace or surface projection of the top of rupture plane. It is measured perpendicular to the fault (or the fictitious extension of the fault).
Fnorm -	Fault normal
Frev -	Fault reverse

Based on the fault parameters, Maximum Magnitude (Mmax) earthquake for the site is equal to 6.8. Based on the subsurface conditions observed in the boring and test pit, a V_{S30} of 600 m/s was used in the development of the ARS curves (Site Class C). The deterministic and probabilistic ARS curves and curve coordinates that have been developed are shown on Figures 3 and 4. The design response spectrum which is shown on Figure 5 is the upper envelope of the spectral values of the deterministic response spectrum, probabilistic response spectrum and the minimum response spectrum for the state of California. A maximum peak ground acceleration (PGA) of 0.289 g is estimated for the bridge.

5.4.2 California Building Code (CBC) 2016 Seismic Design Parameters

For structural design in accordance with the CBC 2016 (ASCE 7-10 procedures), the United States Geological Survey Design Maps (USGS, 2018) were used to calculate ground motion parameters for the project site. The Risk-Targeted Maximum Considered Earthquake (MCE_R) ground motion response acceleration is calculated based on the most severe earthquake effects considered by ASCE 7-10 determined for the orientation that resulted in the largest maximum response to the horizontal ground motions and with adjustment to the targeted risk. The Maximum Considered Earthquake Geometric Mean (MCE_G) is determined for the geometric peak ground acceleration and without adjustment for the targeted risk. The MCE_G Peak Ground Acceleration (PGA) adjusted for site effects (PGA_M) should be used for design and evaluation of liquefaction, lateral spreading, seismic settlements, and other soil related issues.

The calculated seismic design parameters are presented in Table 3 below for Site Class C. The design criteria are based on the soil profile type as determined by existing subsurface geologic conditions, on the proximity of the site to a nearby fault and on the maximum moment magnitude and slip rate of the nearby fault.

Table 3

Summary of CBC 2016 Seismic Design Parameters (Site Class C)

REFERENCE	PARAMETER
Table 20.3-1 Site Classification	Site Class = C
Figure 22-1	Ss = 0.953 g
Table 11.4-1 Site Coefficient Fa	Fa = 1.019
Figure 22-2	S_1 = 0.365 g
Table 11.4-2 Site Coefficient Fv	Fv = 1.435
Equation 11.4-1	S_{MS} = 0.971 g
Equation 11.4-2	S_{M1} = 0.524 g
Equation 11.4-3	S_{DS} = 0.647 g
Equation 11.4-5	S_{D1} = 0.349 g
Figure 22-12	T_L = 8 seconds
Figure 22-7	PGA = 0.387 g
Equation 11.8-1	PGA_M = 0.392 g
Figure 22-17	C_{RS} = 0.931
Figure 22-18	C_{R1} = 1.001

Figure 22-1	Ss Risk-Targeted Maximum Considered Earthquake (MCER) Ground Motion Parameter for the Conterminous United States for 0.2 s Spectral Response Acceleration (5% of Critical Damping), Site Class B.
Figure 22-2	S1Risk-Targeted Maximum Considered Earthquake (MCER) Ground Motion Parameter for the Conterminous United States for 1.0 s Spectral Response Acceleration (5% of Critical Damping), Site Class B.
Figure 22-12	Mapped Long-Period Transition Period, TL (s), for the Conterminous United States.
Figure 22-7	Maximum Considered Earthquake Geometric Mean (MCEG) PGA, %g, Site Class B for the Conterminous United States.
Figure 22-17	Mapped Risk Coefficient at 0.2 s Spectral Response Period, CRS.
Figure 22-18	Mapped Risk Coefficient at 1.0 s Spectral Response Period, CR1.

5.5 Earthwork Operations

Earthwork operations for the proposed project are anticipated to be limited to the foundation excavations for the proposed bridge and backfilling operations around the bridge foundation pile caps.

5.5.1 Soil and Excavation Characteristics

The majority of the on-site materials can be readily excavated with conventional heavy-duty construction equipment. Difficult excavation conditions within the highly cemented and/or highly conglomeratic Stadium Conglomerate may be anticipated, and may require the use of a rock breaker and/or jackhammer.

Fill material should be free of biodegradable material, hazardous substance contamination, other deleterious debris, and 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 30.

Soil materials generated from the young colluvial deposits and Stadium Conglomerate are likely to contain abundant gravel and cobbles, and may require selective screening of oversize materials if they are utilized as compacted fill. In lieu of screening, it may be more practical and economical for the Contractor to use select import fill materials for backfill.

5.5.2 Placement and Compaction of Backfill

Prior to placement, all backfill materials 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.

It is recommended that the upper 24 inches directly beneath slabs, pavement sections and the underlying base materials be compacted to at least 95 percent relative compaction. 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.

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 pipes. Flooding or jetting should not be used to densify the backfill. Appropriate compacting equipment should be used for compaction of backfill materials placed adjacent to underground facilities, structures and walls such as not to impose excessive loads on the facilities.

5.6 Bridge Foundation Recommendations

Based on the subsurface conditions encountered in the boring and test pit, CIDH piles at the abutments are considered suitable to provide support for the proposed bridge. The following foundation recommendations were designed in accordance with the 2014 AASHTO LRFD Bridge Design Specification (6th Edition) with CA Amendments.

Preliminary general foundation information and design loads provided by MBI are shown in Tables 4 and 5, respectively.

Table 4

General Foundation Information

Support No.	Pile Type	Finished Grade Elevation (ft)	Cut-off Elevation (ft)	Pile Cap Size (ft)		Permissible Settlement Under Service Load (in)	Number of Piles Per Support
				B	L		
Abut 1	24-inch CIDH	188.08	182.25	3.5	13.5	2	2
Abut 2	24-inch CIDH	186.37	182.25	3.5	13.5	2	2

Table 5

Preliminary Design Load

Support No.	Service - I Limit State (kips)		Strength/Construction Limit State (kips)				Extreme Limit State (kips)			
			Compression		Tension		Compression		Tension	
	Total Load per Support	Permanent Loads per Support	Per Support	Max per Pile	Per Support	Max per Pile	Per Support	Max per Pile	Per Support	Max per Pile
Abut 1	140	80	190	100	0	0	110	60	0	0
Abut 2	140	80	190	100	0	0	110	60	0	0

It is anticipated that concrete placement for the CIDH concrete piles can be performed using conventional methods, and casing and slurry displacement methods will not be required. The calculated “Nominal Axial Resistance” of the CIDH concrete piles was based on skin friction and tip resistance.

The zones used to calculate skin friction of the CIDH concrete piles are shown in Table 6. The soil parameters used for the vertical load resistance analyses are also presented in Table 6 below. The skin friction for the rock socket is calculated using Rowe R.K. and Armitage H.H. (1984) based on rock unconfined compressive strength of 800 psi (weak rock). The axial resistance curves, including Extreme Event I (seismic case), showing the axial resistance versus depth are presented on Figure 6.

The effect of scour was not considered since the fill, alluvial and colluvial deposits are not considered in the design of the pile. A resistance factor of 1.0 per the Caltrans Amendments (Caltrans, 2014b) was used for both Extreme Event Limit States.

Table 6

CIDH Concrete Pile

Support Location	**Top of Rock Elevation (ft. msl)**	**Top of Skin Friction Zone Elevation (ft. msl)**	**Bottom of Skin Friction Zone Elevation (ft. msl)**	**Specified Tip Elevation (ft. msl)**	**Nominal Skin Friction (ksf)**
Abut 1	+185	+182.25	+172.25	+172.25	10
Abut 2	+185	+182.25	+172.25	+172.25	10

NOTE: * Recommended minimum embedment of 10 feet into the Stadium Conglomerate control.

Pile spacing at Abutments 1 and 2 is anticipated to be in excess of three times the pile diameter of 24 feet. Therefore, a pile group reduction factor does not need to be applied to the piles.

Design soil input parameters for lateral pile capacity analyses using the software LPILE are presented in Tables 7 and 8 for use by the project structural engineer. The final design tip elevations should be determined based on the controlling value of axial and lateral demand. AGE recommends that the CIDH piles be embedded a minimum of 10 feet into the Stadium Conglomerate.

Table 7

LPILE Soil Parameters

Soil ID	**Soil Type**	**Description**	**Soil Model**	**Effective Unit Weight (pcf)**	**Est. Undrained Shear Strength (C_u) (psf)**	**Angle of Internal Friction (degrees)**	**Subgrade Modulus for Laterally Loaded Soil (K) (lb/cu. in)**	**Unfconfined Compressive Strength (psi)**	**Strain (E50) (%)**
1	Fill & Young Colluvial Deposits	Medium dense silty sand and clayey sand	Sand	90	0	35	90	N.A.	N.A.
2	Stadium Conglomerate	Rock	Weak Rock	120	0	N.A.	N.A.	800	N.A.

Table 8

LPILE Soil Profile at Abutments 1 and 2

Soil ID	**Depth (feet)**	**Elevation (feet msl)**
1	0-7	+185 to +192
2	7 & deeper	+ 185 and deeper

Resistance to lateral loads may be developed by a combination of friction acting at the base of the pile caps and passive earth pressure developed against the sides of the pile caps 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 resistance of 350 psf per foot of pile cap embedment below grade may be used for the sides of foundations placed against the Stadium Conglomerate. The maximum recommended allowable passive pressure is 3,500 psf. An allowable passive earth pressure of 250 psf per foot of pile cap embedment below grade may be used for the sides of foundations placed against properly compacted filled ground and young colluvial deposits. The maximum recommended allowable passive pressure is 2,500 psf. A coefficient of friction of 0.40 may be used for foundations cast directly on the Stadium Conglomerate.

5.7 Bridge Abutment Walls

We recommend that bridge abutment walls be backfilled with soil materials which have less than 40 percent passing the standard #200 sieve and not less than 70 percent passing the U.S. standard 3/4-inch sieve, expansion index of less than 30 and minimum internal friction angle of 35^{o}. In addition, the backfill materials should not contain any organic debris, rocks or hard lumps greater than 6 inches, or other deleterious materials. All backfill soils should be compacted to at least 90 percent of maximum dry density as determined in the laboratory by the ASTM D1557 testing procedure.

For design of properly backfilled abutment walls, an active soil pressure equivalent to that generated by a fluid weighing 35 and 61 pounds per cubic foot, for level and 2:1 (horizontal : vertical) sloped backfill, respectively, may be used for design of the wall assuming that they are free to rotate at the top at least 0.001H (where H is the height of the wall). An at-rest soil pressure equivalent to that generated by a fluid weighing 60 pounds per cubic foot may be used for design of wall restrained at the top. Traffic surcharge occurring within a horizontal distance equal to the wall height should be added as lateral pressure equal to a uniformly distributed load of 75 psf along the entire face of the wall.

Calculation for the Seismic Active Earth Pressure was performed in accordance with the procedure outlined in Section 11.6.5.3 of the AASHTO LRFD Bridge Design Specifications 6th Edition (2012) using the Mononobe-Okabe (M-O) Method. The Horizontal Acceleration Coefficient (K_h) is estimated to be 1/2 of PGA_M and equal to 0.196 (Site Class C), Vertical Acceleration Coefficient (K_v) is assumed to be zero. The backfill material is assumed to have a unit weight of 110 pcf, friction angle of 35^o and cohesion value of 600 psf. The calculated Seismic Active Earth Pressure Coefficient (K_{AE}) is equal to 0.10 for retaining structures up to 25 feet in height.

Based on the conditions described above, a triangular pressure distribution of 11 pcf (equivalent fluid pressure) may be used for the Seismic Active Earth Pressure. This seismic earth pressures may be assumed to act at 0.4H from the bottom of the wall and are applicable for both cantilever and braced conditions. Forces resulting from wall inertia effects are expected to be relatively minor for non-gravity walls and/or walls retaining less than 5 feet of backfill materials, and may be ignored in estimating the seismic lateral earth pressure.

6.0 CONSTRUCTION-RELATED CONSIDERATIONS

6.1 Construction Dewatering

The depth of the local groundwater table is expected to be below the anticipated depth of the proposed excavations for this project. We therefore do not anticipate the need for dewatering of excavations made during construction. The contractor should, however, anticipate the possible need for sump pumps in the event that localized perched water conditions are encountered during construction. 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 Excavations

Excavation and safety during construction are the sole responsibility of the contractor. Excavations should be performed in accordance with applicable Local, State, and prevailing Federal and Cal OSHA safety regulations to prevent excessive ground movement and failure. Unsupported temporary excavations in the fill materials similar to those encountered in the boring and test pit may be constructed at an inclination no steeper than 1.5 : 1 (horizontal to vertical), or flatter, up to a maximum height of 15 feet. Unsupported temporary excavations in the formational materials similar to those encountered in the boring and test pit may be constructed at an inclination no steeper than 3/4 : 1 (horizontal to vertical), or flatter, up to a maximum height of 15 feet. Temporary construction slopes are considered to have a factor of safety against deep-seated failure in excess of 1.2 under static conditions.

Observations will need to be performed during site grading to check that no adverse conditions, geologic features or discontinuities are exposed in the excavation which may necessitate shoring or tie-backs. The contractor should exercise caution and provide adequate safety measures during excavations to protect equipment and/or personnel working directly below any excavation. Adequate safety measures include, but are not limited to, providing proper drainage control above and below the excavation, and elimination of any surcharge within a lateral distance equal to the height of the excavations.

6.3 Temporary Shoring

The contractor shall be responsible for the design and installation of temporary shoring for all vertical excavations in excess of 4 feet in height. Design and installation of shoring should be in accordance with the requirements specified by the State of California, Division of Occupational Safety and Health, Department of Industrial Relations (CAL OSHA). Furthermore, it should be the contractor's responsibility to provide adequate and safe support for all excavations and nearby located improvements which could be damaged by earth movement.

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 1/2-inch or less where existing structures or utilities are to be protected.

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, the recommended lateral earth pressure should be 32H psf, where H is equal to the height of the retained earth in feet. For braced shoring supporting formational materials, the recommended lateral earth pressures may be reduced to 20H psf. 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.

Resistance to lateral loads will be provided by passive soil resistance. The allowable passive pressure for the fill materials and colluvial deposits may be assumed to be equivalent to a fluid weighing 250 pcf. Allowable lateral bearing pressure in fill material should not exceed 2,500 psf. Allowable passive pressure for undisturbed formational materials may be assumed to be equivalent to a fluid weighing 350 pcf, with maximum allowable lateral bearing pressure of 3,500 psf.

6.4 CIDH Piles Construction Considerations

It is anticipated that standard continuous flight rotary augers may be used for construction of the proposed CIDH piles. The need for the use of drilling fluids and or slurry displacement method for the installation of the proposed CIDH piles are not anticipated.

Zones with abundant gravels and cobbles were encountered during the subsurface investigation. The contractor should be prepared to use drilling buckets in the event that the augers are unable to extract the cuttings from the drilled shafts. No boulders and/or hard rocks were encountered during the subsurface investigation. Therefore, the need for rock coring is not anticipated on this project. Since the gravels and cobbles are he use of temporary steel casings is not anticipated. In the event that steel casings are required, the casings may be extracted from the shafts as the concrete is placed.

It is recommended that prior to placement of reinforcing steel, all footing shafts be downhole inspected to confirm and verify the soil type at the bottom of the shafts and minimum depth embedment into the Scripps Formation. Furthermore, prior to placement of concrete, it is recommended that the shafts be cleaned of all loose materials with a cleanout bucket. Concrete should be placed by using a tremie or pump pipe which can be adjusted to permit free discharge of concrete and lowered rapidly, if needed, without excessive contact with the sides of the shaft.

6.5 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 site. 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 MBI 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 at the project site. The findings and recommendations presented in this report are based on the assumption that the subsurface conditions beneath the remainder of the site do not deviate substantially from those encountered in the exploratory soil boring and test pit. 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

City of San Diego, Public Works Department, “Standard Drawings for Public Work Construction”, 2012 Edition.

City of San Diego Seismic Safety Study, Geologic Hazards and Faults, Sheet No. 22, 2008 edition.

City of San Diego, “ Sewer Group 806 Pre Design Map”, dated August 11, 2015.

County of San Diego DEH, “Case Closure Summary, Unauthorized Release #H05236-001, Sullivan United, 4660 Alvarado Canyon Road, San Diego, California, 92120", report dated December 7, 2005.

Department of Conservation, California Geological Survey Regulatory Hazard Zones Maps for Earthquake Faults, Liquefaction and Landslide Zones, 2009.

Geotracker Data Base - (http://geotracker.waterboards.ca.gov).

International Conference of Building Officials, 1997, Maps of Known Active Fault Near Source Zones in California and Adjacent Portions of Nevada.

Kennedy, M.P., 1975, Geology of the San Diego Metropolitan Area, California: California Division of Mines & Geology, Bulletin 200.

Kennedy, M.P., et.al., 1975, Character and Recency of Faulting, San Diego Metropolitan Area, California: California Division of Mines and Geology, Special Report 123.

Kennedy, M.P, and Tan, S.S, 2008, “Geologic Map of the San Diego 30' x 60' Quadrangle, California”, Digital Preparation by U.S. Geological Survey.

Lindvall, S.C., and Rockwell, T.K., 1995, “Holocene Activity of the Rose Canyon fault zone in San Diego, California” in Journal of Geophysical Research, v. 100, No. B12, pp. 24,121 - 124,132.

Marshall, M., 1989, "Detailed Gravity Studies and the Tectonics of the Rose Canyon--Point Loma--La Nacion Fault System, San Diego, California" in Proceedings of Workshop on "The Seismic Risk in the San Diego Region: Special Focus on the Rose Canyon Fault System" (Glenn Roquemore, et.al, Editors).

Michael Baker Internation, “Plans for the Construction of San Diego Sewer Maintenance Bridge”, plans, undated.

Patterson, R.H., D.L. Schug, and B.E. Ehleringer, 1986, "Evidence of Recent Faulting in Downtown San Diego, California" in Geological Society of America, Abstracts With Programs, v. 18, No. 2, p. 169.

Rockwell, T.K., et.al., 1991, "Minimum Holocene Slip Rate for the Rose Canyon Fault in SanDiego, California" in Environmental Perils in the San Diego Region (P.L. Abbott and W.J. Elliott, editors): San Diego Association of Geologists, pp. 37-46.

Treiman, J.A., 1993, “The Rose Canyon Fault Zone, Southern California”, California Division of Mines and Geology Open File Report No. 93-02.

Standard Specifications for Public Works Construction ("Green Book"), including the Regional Standards, 2010 Edition.

Aerial Photographs

U.S. Department of Agriculture black and white aerial photograph Nos. AXN-3M- 98 and 99 (dated 1953)

FIGURES

Approximate Sewer Group 806 Bridge Crossing Location

SEWER GROUP 806 BRIDGE CROSSING PROJECT

LOCATION MAP

PROJECT NO.
185 GS-16-B

ALLIED GEOTECHNICAL ENGINEERS, INC.

FIGURE 1

SEWER GROUP 806 BRIDGE CROSSING PROJECT

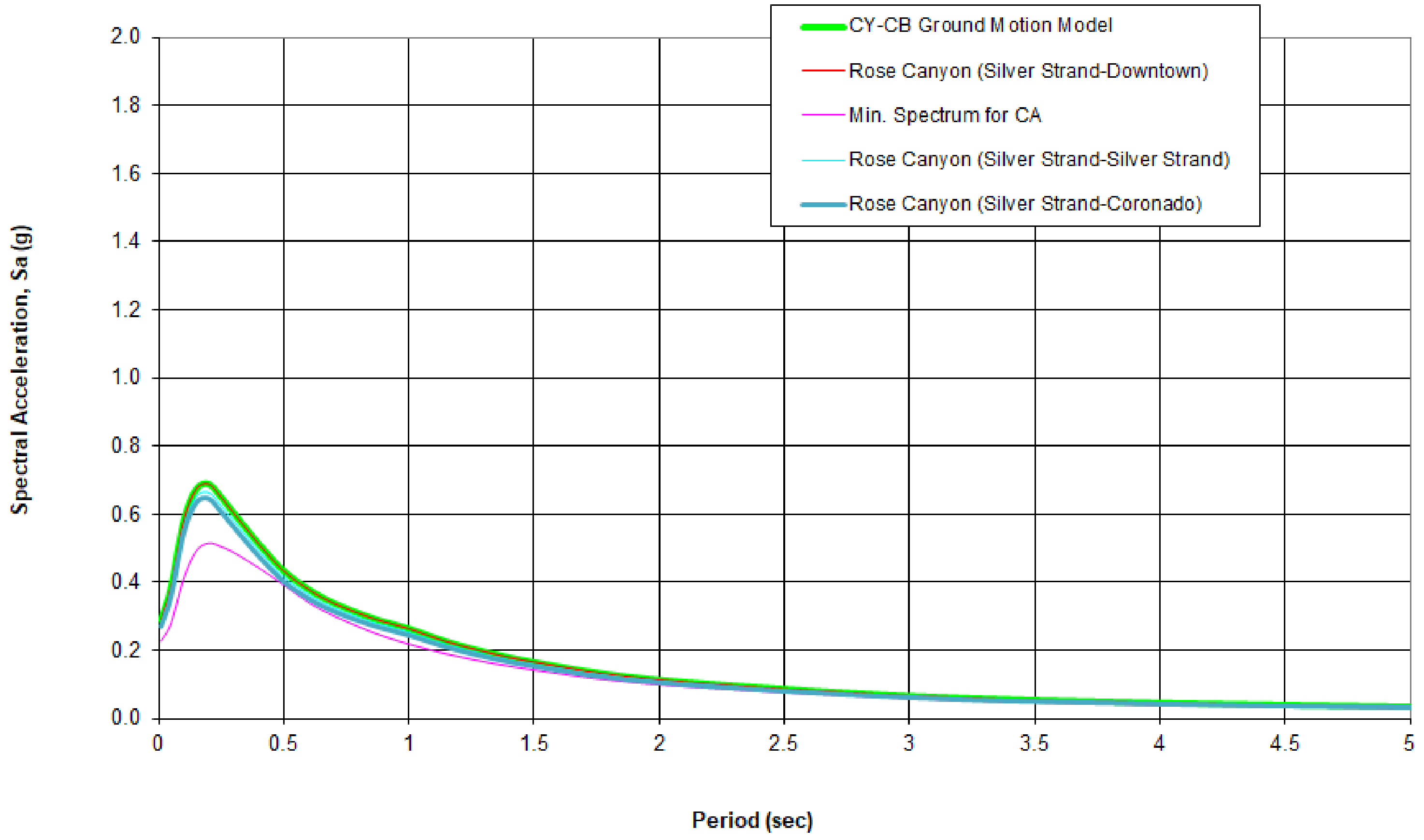
SITE PLAN

PROJECT NO.
185 GS-16-B

ALLIED GEOTECHNICAL ENGINEERS, INC.

FIGURE 2

Deterministic ARS (5% Damping)
Comparison of Spreadsheet vs ARS Online Results

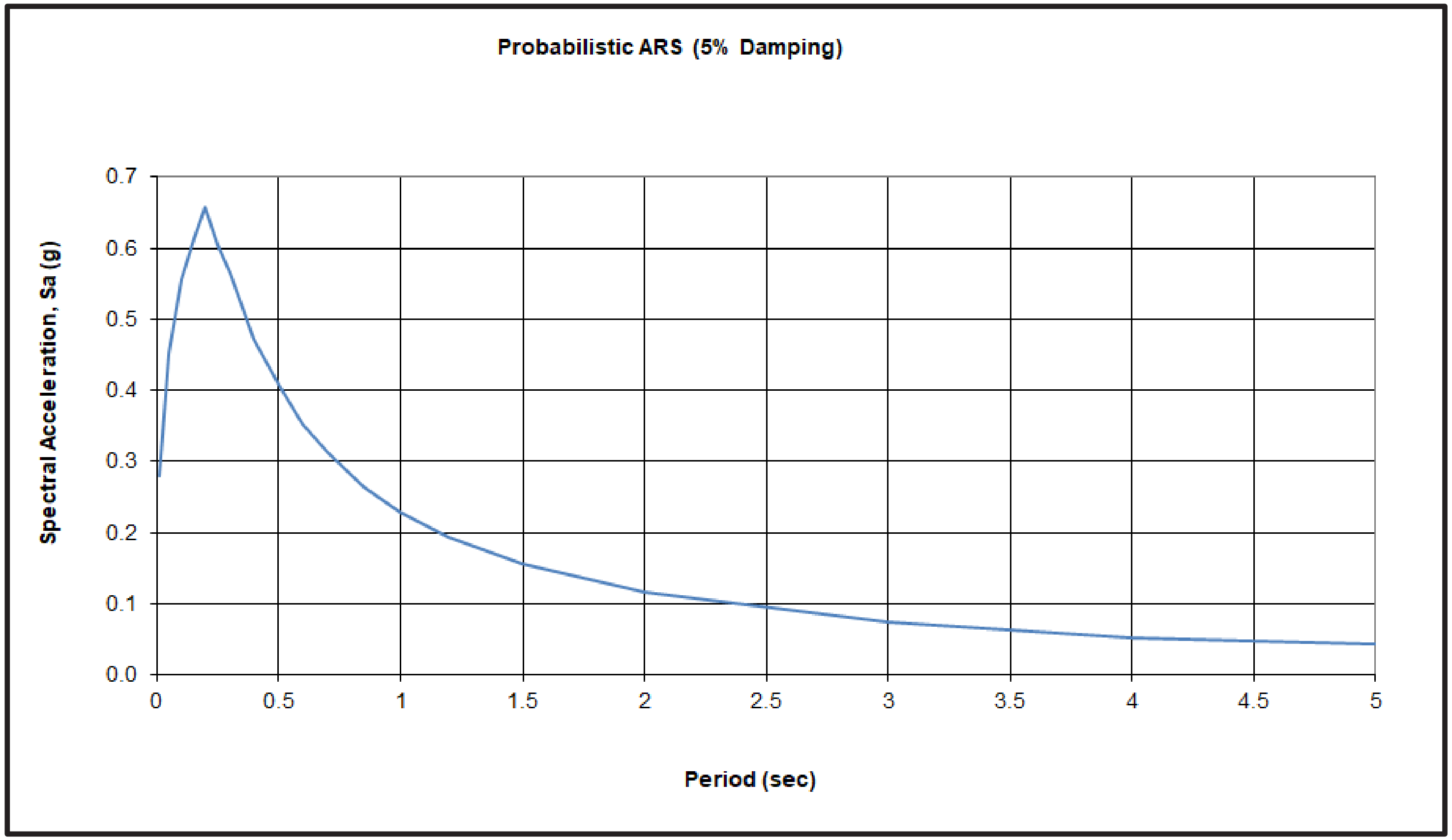


DETERMINISTIC ARS
SEWER GROUP 806 BRIDGE CROSSING PROJECT

FIGURE 3

ALLIED GEOTECHNICAL ENGINEERS, INC.

PROJECT NO.
185 GS-16-B

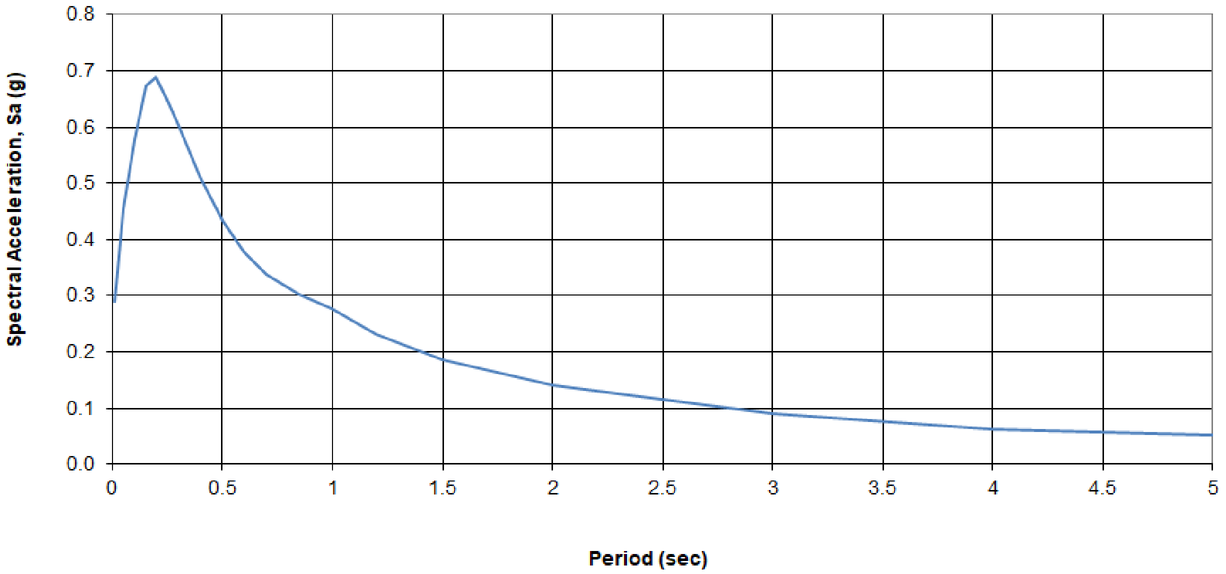


PROBABILISTIC ARS/USGS DEAGGREGATION HAZARD DATA
(5% EXCEEDANCE PROBABILITY IN 50 YEAR)
SEWER GROUP 806 BRIDGE CROSSING PROJECT

FIGURE 4

ALLIED GEOTECHNICAL ENGINEERS, INC.

PROJECT NO.
185 GS-16-B



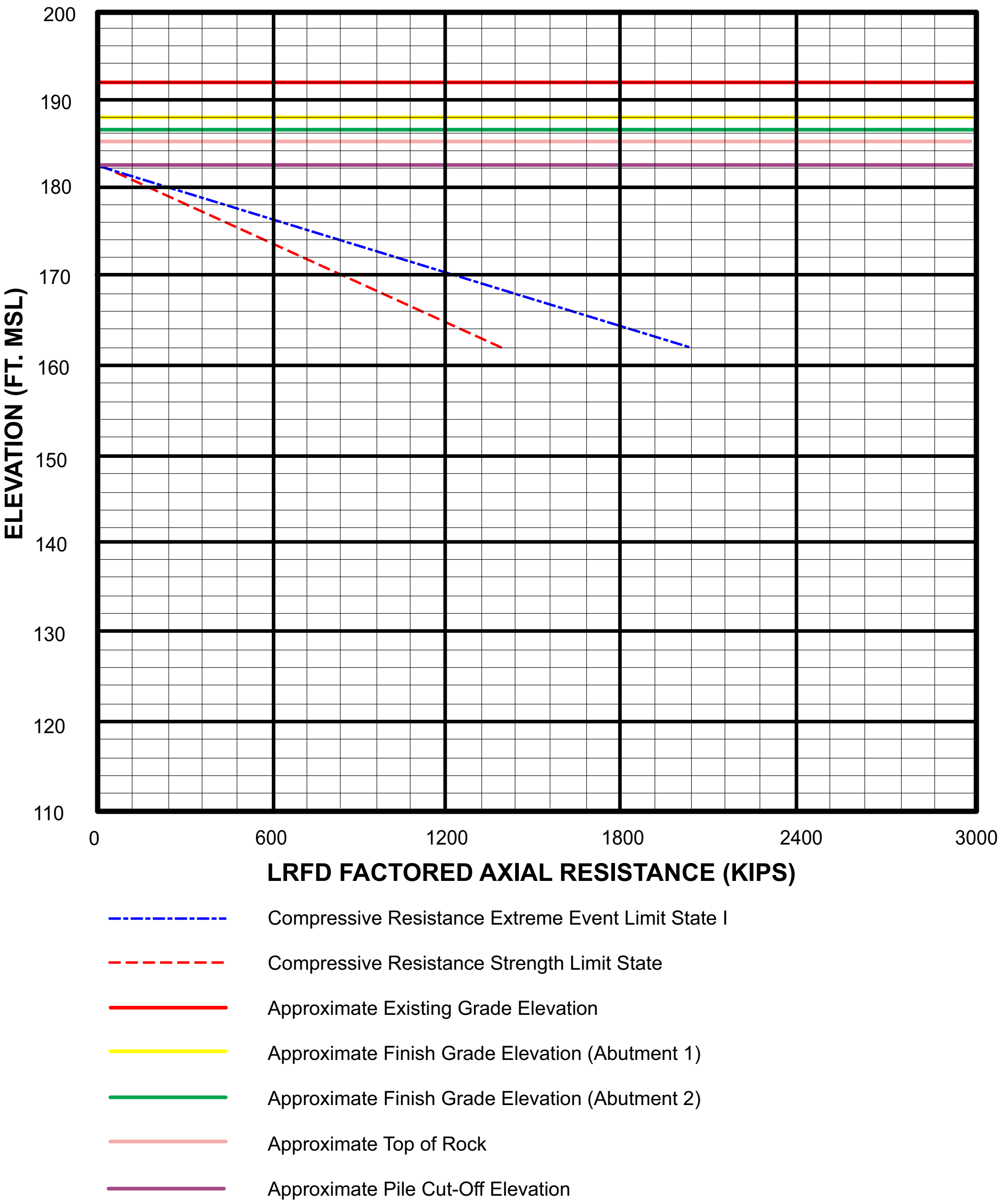
T (sec)	Base Spectrum S(a)
0.01	0.289
0.05	0.453
0.1	0.578
0.15	0.673
0.2	0.688
0.25	0.649
0.3	0.603
0.4	0.512
0.5	0.433
0.6	0.378
0.7	0.338
0.85	0.302
1	0.276
1.2	0.231
1.5	0.186
2	0.141
3	0.089
4	0.063
5	0.052

ARS ENVELOPE DESIGN SPECTRAL ACCELERATION
SEWER GROUP 806 BRIDGE CROSSING PROJECT

**PROJECT NO.
185 GS-16-B**

ALLIED GEOTECHNICAL ENGINEERS, INC.

FIGURE 5



NOTES:

1. Resistance is based on combination of skin friction and end bearing.
2. Applied resistance factor is 0.7 for Strength Limit State and 1.0 for Extreme Event Limit State I.
3. Uplift is not provided.
4. No liquefaction downdrag anticipated.

LRFD FACTORED AXIAL RESISTANCE
24-INCH DIAMETER CIDH PILES (ABUTMENTS 1 AND 2)
SEWER GROUP 806 BRIDGE CROSSING PROJECT

PROJECT NO.
185 GS-16-B

ALLIED GEOTECHNICAL ENGINEERS, INC.

FIGURE 6

APPENDIX A

FIELD EXPLORATION PROGRAM

APPENDIX A

FIELD EXPLORATION PROGRAM

The field exploration program for this project was performed on July 31, 2018, and included the advancement of one (1) soil boring and one (1) test pit at the approximate locations shown on the Site Plan (Figure 2). The soil boring was performed by Tri-County Drilling with a CME-75 all-terrain drill rig to a depth of 28 feet below the existing ground surface (bgs), and the test pit was performed by Mansolf Excavation using manual labor to a depth of 5 feet bgs. The soils encountered in the soil boring and test pit 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 boring and test pit are presented on Figures A-3 and A-4. 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 boring and test pit. Subsequently, Underground Service Alert (USA) was contacted to coordinate clearance of the proposed test pit locations with respect to existing buried utilities.

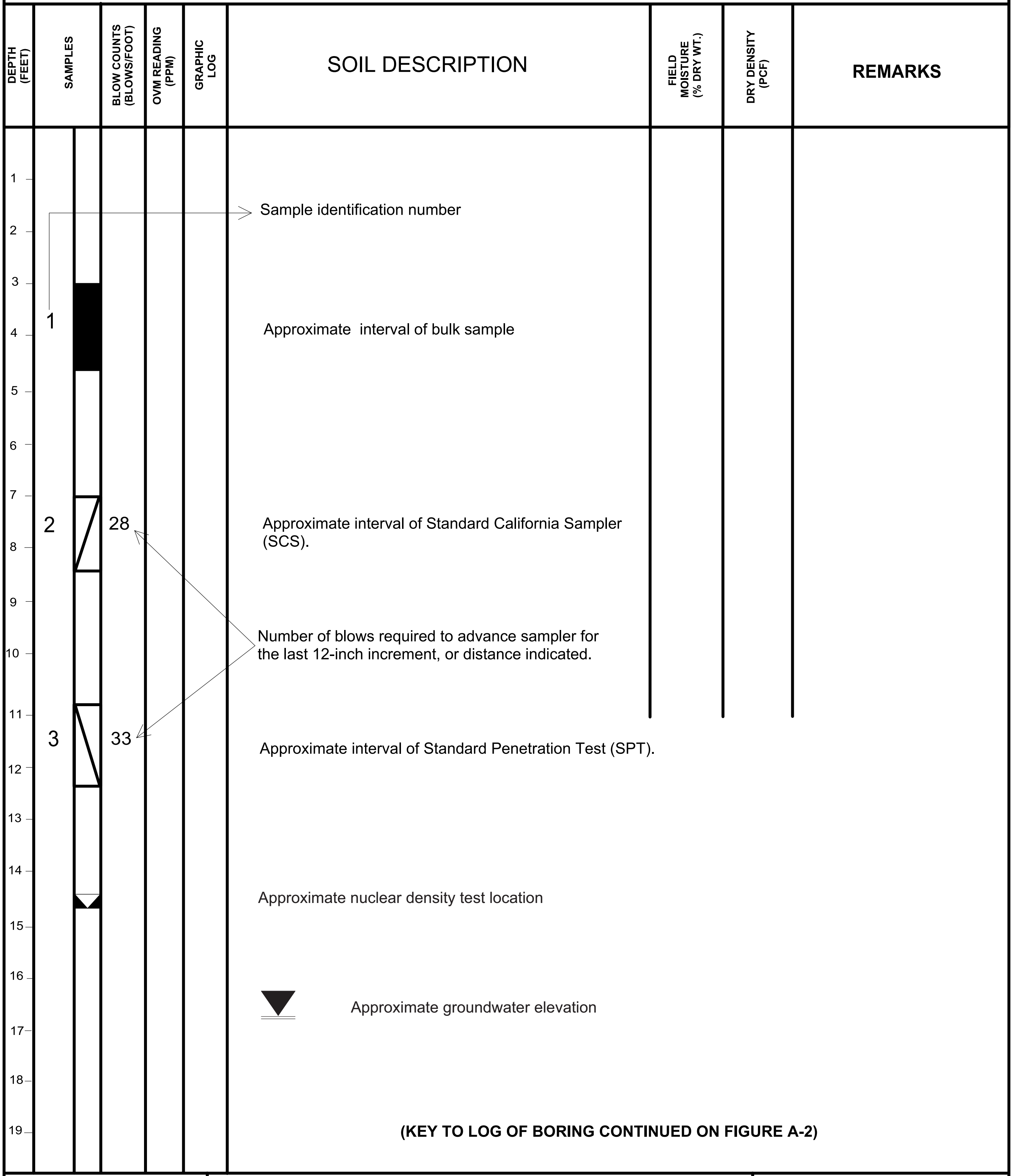
During the excavation, moisture and density test readings were taken in the test pit using a nuclear soil gauge (ASTM D6938-10). In addition, loose bulk samples were also collected. The samples were field screened for the presence of volatile organics using a RAE Systems MiniRAE 3000 organic vapor meter (OVM). The OVM readings are indicated on the logs.

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 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 12 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.

Upon completion of the drilling and sampling activities, boring B-1 was backfilled using bentonite grout and bentonite chips to approximately 12 inches below the ground surface and capped with excess soil cuttings. The test pit was backfilled with excess soil cuttings and compacted.

KEY TO LOG OF BORING



Sample identification number

Approximate interval of bulk sample

Approximate interval of Standard California Sampler (SCS).

Number of blows required to advance sampler for the last 12-inch increment, or distance indicated.

Approximate interval of Standard Penetration Test (SPT).

Approximate nuclear density test location

Approximate groundwater elevation

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

PROJECT NO.
185 GS-16-B

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 clayey silty sand**
▩	**Silty gravel**

GENERAL NOTES

1. Approximate elevations and locations of borings are based on the topographical maps provided by Michael Baker International, 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 boring logs apply only at the specific boring locations and at the time the borings were performed. They are not warranted to be representative of subsurface conditions at other locations or times.

PROJECT NO. 185 GS-16-B	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE A-2

BORING NO. B-1

DATE OF DRILLING: July 31, 2018
TOTAL BORING DEPTH: 28 feet
GENERAL LOCATION: Southwest side of proposed bridge
APPROXIMATE SURFACE ELEV.: 192 feet msl
DRILLING CONTRACTOR: Tri-County Drilling
DRILLING METHOD: 8" HSA/4" Air Rotary
LOGGED BY: Nicholas Barnes

DEPTH (FEET)	SAMPLES	BLOW COUNTS BLOWS/FOOT	OVM READING (PPM)	SOIL DESCRIPTION	FIELD MOISTURE % DRY WT.	DRY DENSITY LBS./CU. FT.	REMARKS
				FILL: brown, dry, silty sand (SM) with subrounded and fractured gravels and cobbles up to 4" in maximum dimension.			Started drilling using 8-inch hollow-stem auger.
1							
2							
3				**YOUNG COLLUVIAL DEPOSITS**			
4	1			Brown to dark brown, dry to damp, medium dense, silty sand (SM) and clayey sand (SC) with scattered sub-rounded gravel and cobbles.			
5	2	100+	0.0		3.8	120.9	
6				? ?			?
7				**STADIUM CONGLOMERATE**			
8				Massive gravel-cobble conglomerate with pale yellow, medium-grained, silty sand matrix.			
9							
10	3	100+	0.0		4.1		At 10 feet converted to air rotary due to cobbles.
11							
12							
13							
14							
15	4						
16							
17							
18							
19							
20							
21							
22							
23							
24							
25							
26							
27	5	100+		▼			No sample recovery.
28							

NOTES:

Boring terminated at depth of 28 feet bgs due to water flowing into the boring. Further drilling would cause water to flow into the adjacent creek bed.

29
30
31
32
33
34
35
36
37

PROJECT NO.
185 GS-16-B

ALLIED GEOTECHNICAL ENGINEERS, INC.

FIGURE A-3

TEST PIT NO. TP-2

DATE OF DRILLING: July 31, 2018	TOTAL BORING DEPTH: 5 feet
GENERAL LOCATION: Northeast side of proposed bridge	
APPROXIMATE SURFACE ELEV.: 190 feet msl	DRILLING CONTRACTOR: Mansolf Excavation
DRILLING METHOD: Manually excavated test pit	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
					Fill Materials			
1	1				Brown, dry, silty sand (SM) with scattered sub-rounded and fractured gravel and cobbles up to 4" in maximum dimension	11.9	91.8	Trace amounts of roots and rootlets up to 0.5” in maximum dimension.
2	2 3					19.4	82.0	
3	4				**YOUNG COLLUVIAL DEPOSITS**	18.9	89.3	
4	5		0.0					
5				▨	Brown to dark brown, damp, silty sand (SM) and clayey sand (SC) with trace to scattered sub-rounded gravel and cobbles up to 10" in maximum dimension.			
6								
7					**STADIUM CONGLOMERATE**			
8					Massive gravel-cobble conglomerate with a pale yellow, damp, medium-grained silty sand matrix.			
9								
10								
11								
12								
13								
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								
26								
27								
28								
29								
30								
31								
32								
33								
34								
35								
36								
37								

NOTES

Test pit terminated at a depth of 5 feet bgs.
No seepage or groundwater were encountered at the time of the excavation.

PROJECT NO. 185 GS-16-B	ALLIED GEOTECHNICAL ENGINEERS, INC.	**FIGURE A-4**

APPENDIX B

LABORATORY TESTING

APPENDIX B

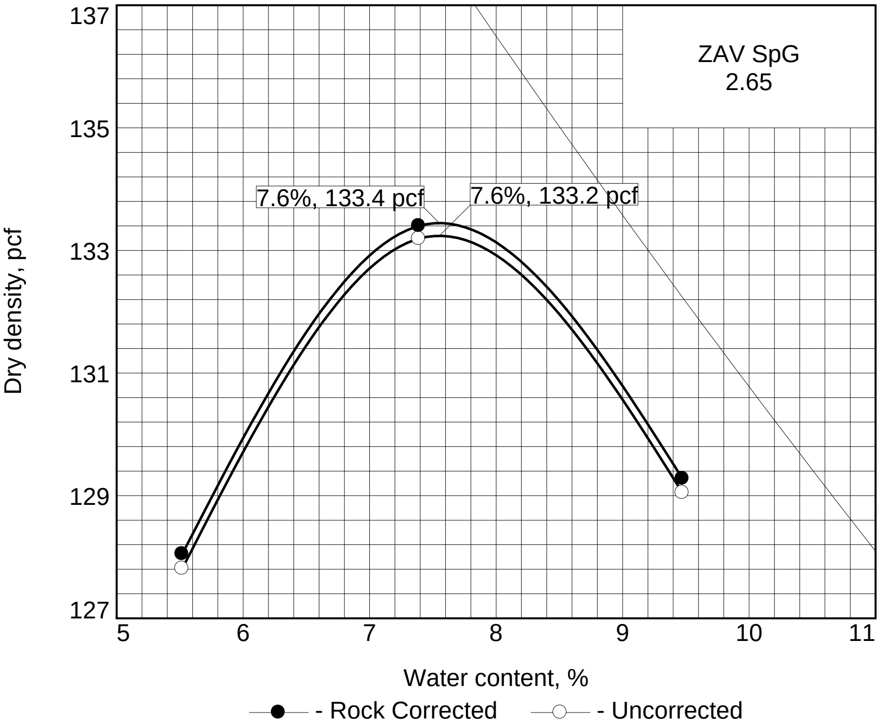
LABORATORY TESTING

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 boring and 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 boring and 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.

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.

COMPACTION TEST REPORT



Curve No.

Test Specification:
ASTM D 1557-91 Procedure C Modified
ASTM D4718-15 Oversize Corr. Applied to Each Test Point

Preparation Method
Hammer Wt. 10 lb.
Hammer Drop 18 in.
Number of Layers five
Blows per Layer 56
Mold Size 0.075 cu. ft.
Test Performed on Material Passing 3/4 in. **Sieve**

NM **LL** NV **PI** NP
Sp.G. (ASTM D 854) 2.65
%>3/4 in. 0.9 **%<No.200** 28.8
USCS SM **AASHTO** A-2-4(0)
Date Sampled 7-31-2018
Date Tested 08/08/2018
Tested By Nicholas Barnes

TESTING DATA

	1	2	3	4	5	6
WM + WS	11186.0	11464.0	11404.0			
WM	6598.0	6598.0	6598.0			
WW + T #1	1143.4	1200.7	1042.1			
WD + T #1	1091.8	1126.6	961.8			
TARE #1	156.8	123.9	114.2			
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	5.5	7.4	9.5			
DRY DENSITY	128.0	133.4	129.3			

ROCK CORRECTED TEST RESULTS	UNCORRECTED
Maximum dry density = 133.4 pcf	133.2 pcf
Optimum moisture = 7.6 %	7.6 %

Material Description

Pale yellow, medium-grained, silty sand matrix.

Remarks:

Project No. 185 GS-16-B **Client:**
Project: SEWER GROUP 806 BRIDGE CROSSING PROJECT

○ **Source of Sample:** B-1 **Depth:** 14

Allied Geotechnical Engineers, Inc.

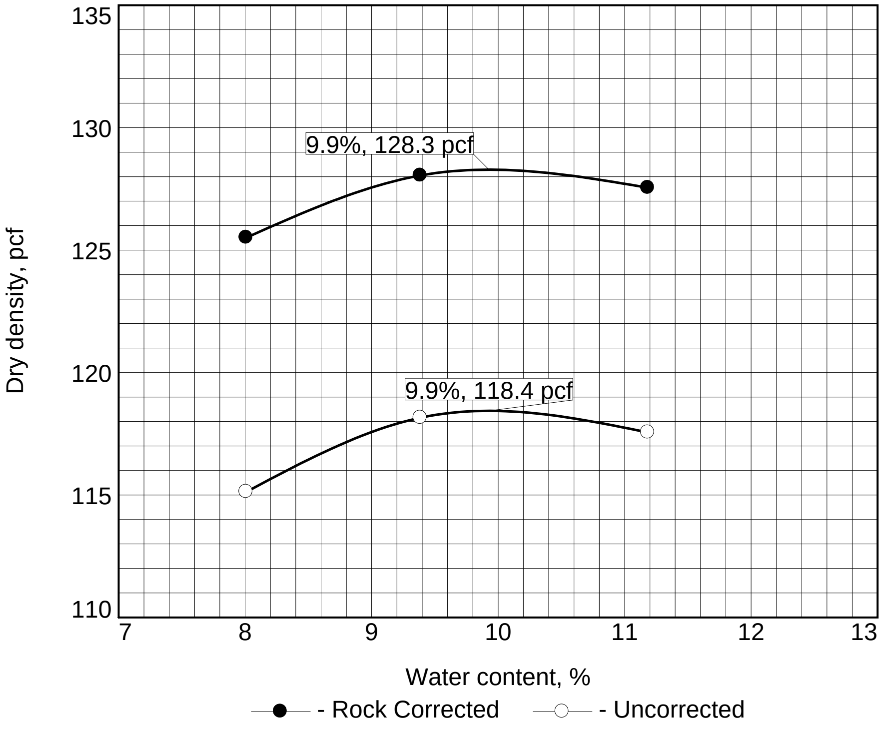
Santee, CA

Checked by: Sani Sutanto
Title: Project Manager

Figure B-1

COMPACTION TEST REPORT

Curve No.



Test Specification:
ASTM D 1557-91 Procedure A Modified
ASTM D4718-15 Oversize Corr. Applied to Each Test Point

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 11.9 **LL** NV **PI** NP

Sp.G. (ASTM D 854)

%>#4 29.0 **%<No.200** 25.2

USCS SM **AASHTO** A-2-4(0)

Date Sampled 7-31-2018

Date Tested 08/08/2018

Tested By Nicholas Barnes

TESTING DATA

	1	2	3	4	5	6
WM + WS	5916.0	5990.0	6012.0			
WM	4036.0	4036.0	4036.0			
WW + T #1	378.4	375.0	400.4			
WD + T #1	355.5	349.0	366.5			
TARE #1	69.6	72.0	63.4			
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	8.0	9.4	11.2			
DRY DENSITY	125.5	128.0	127.5			

ROCK CORRECTED TEST RESULTS	UNCORRECTED
Maximum dry density = 128.3 pcf	118.4 pcf
Optimum moisture = 9.9 %	9.9 %

Material Description

Brown to dark brown, damp, silty sand (SM) with gravels.

Remarks:

Project No. 185 GS-16-B **Client:**

Project: SEWER GROUP 806 BRIDGE CROSSING PROJECT

○ **Source of Sample:** TP-2 **Depth:** 1 **Sample Number:** 1

Allied Geotechnical Engineers, Inc.

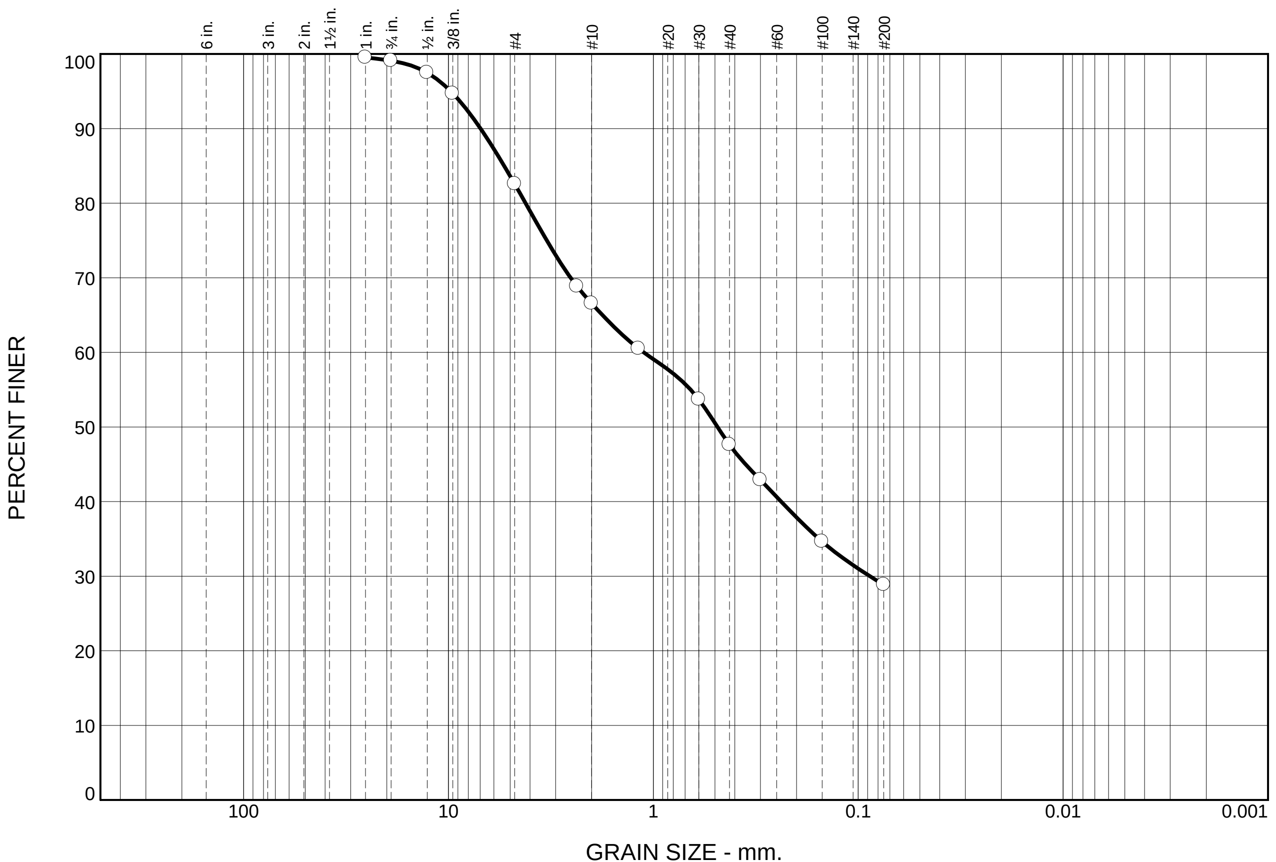
Santee, CA

Checked by: Sani Sutanto

Title: Project Manager

Figure B-2

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○			16.5	16.0	19.0	18.8	28.8	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-1		14	Pale yellow, medium-grained, silty sand matrix.	SM

Allied Geotechnical Engineers, Inc.

Santee, CA

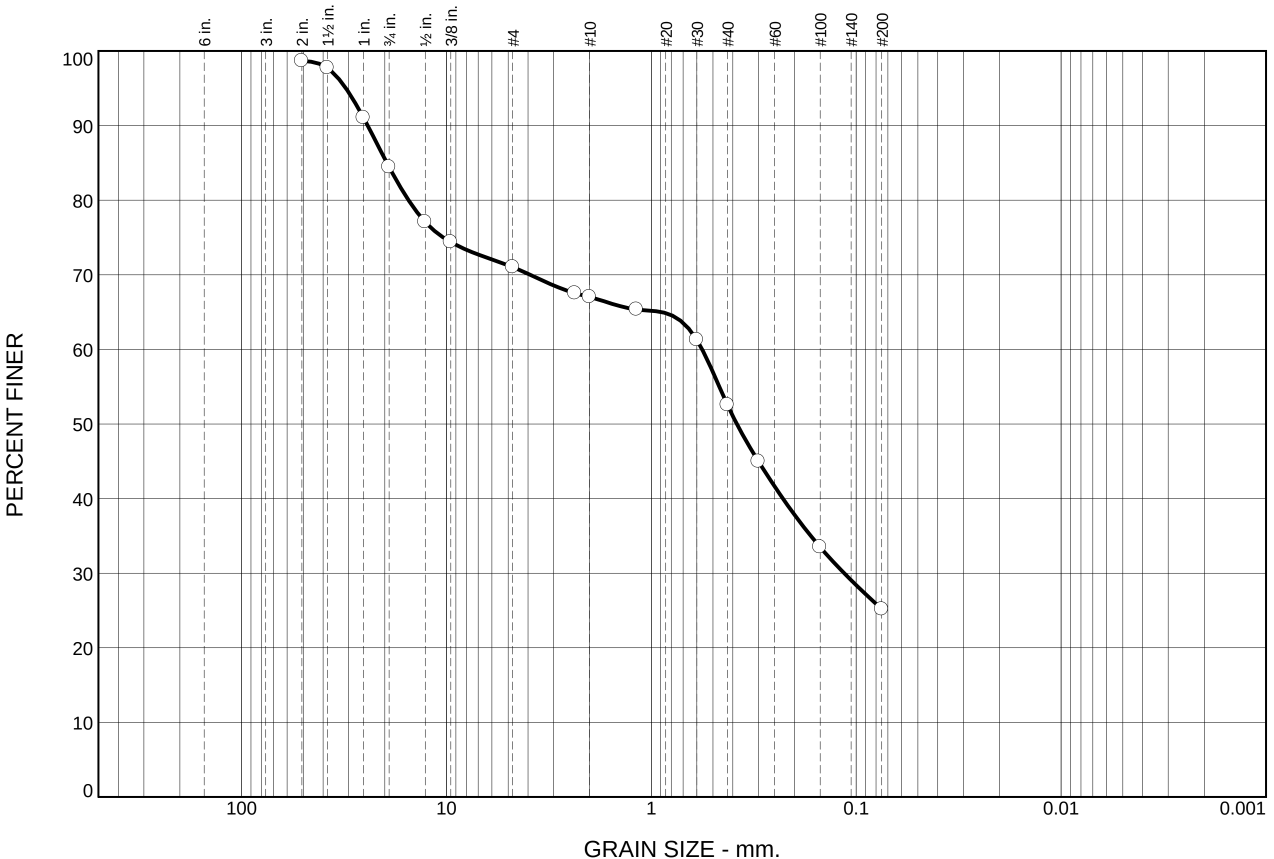
Client:

Project: SEWER GROUP 806 BRIDGE CROSSING PROJECT

Project No.: 185 GS-16-B **Figure** B-3

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

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○			13.4	4.0	14.5	27.3	25.2	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	TP-2	1	1	Brown to dark brown, damp, silty sand (SM) with gravels.	SM

Allied Geotechnical Engineers, Inc.

Santee, CA

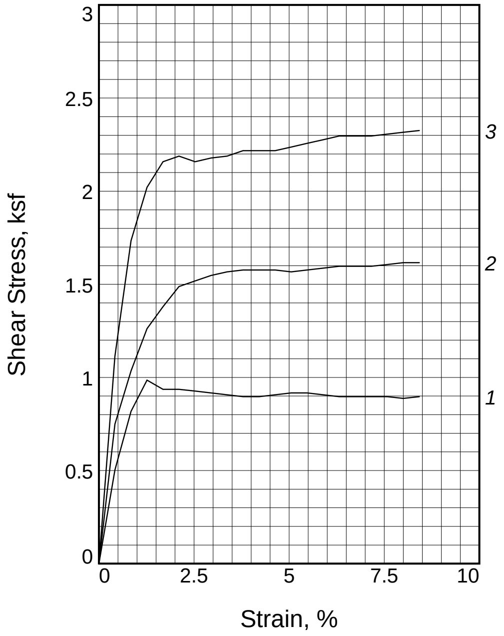
Client:

Project: SEWER GROUP 806 BRIDGE CROSSING PROJECT

Project No.: 185 GS-16-B **Figure** B-4

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

	Results
C, ksf	0.30
ϕ, deg	33.8
Tan(ϕ)	0.67



Sample No.		1	2	3
Initial	Water Content, %	10.3	9.7	9.7
	Dry Density, pcf	114.9	115.0	115.7
	Saturation, %	62.2	58.7	60.1
	Void Ratio	0.4400	0.4389	0.4293
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	16.4	15.9	15.4
	Dry Density, pcf	115.2	114.4	115.1
	Saturation, %	99.7	94.2	93.1
	Void Ratio	0.4357	0.4461	0.4379
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.01
Normal Stress, ksf		1.00	2.00	3.00
Fail. Stress, ksf		0.99	1.62	2.33
Strain, %		1.3	8.0	8.4
Ult. Stress, ksf				
Strain, %				
Strain rate, in./min.		0.007	0.007	0.007

Sample Type: remold

Description: Pale yellow, medium-grained, silty sand matrix.

LL= NV **PI=** NP

Specific Gravity= 2.65

Remarks:

Figure B-5

Client:

Project: SEWER GROUP 806 BRIDGE CROSSING PROJECT

Source of Sample: B-1 **Depth:** 14

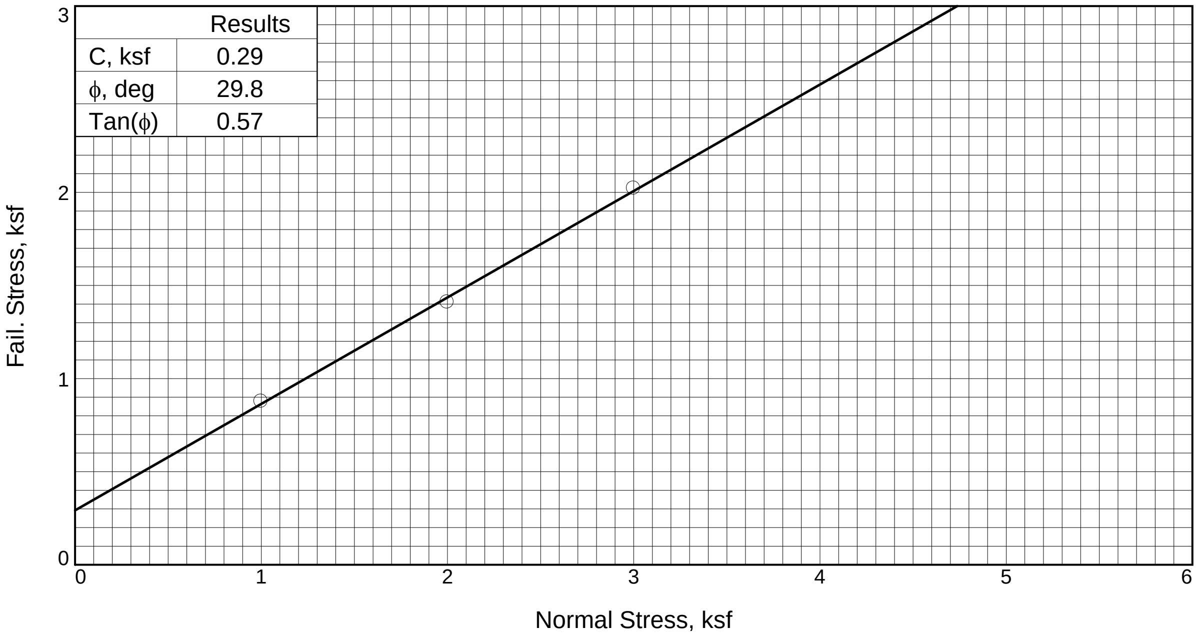
Proj. No.: 185 GS-16-B **Date Sampled:** 7-31-2018

DIRECT SHEAR TEST REPORT

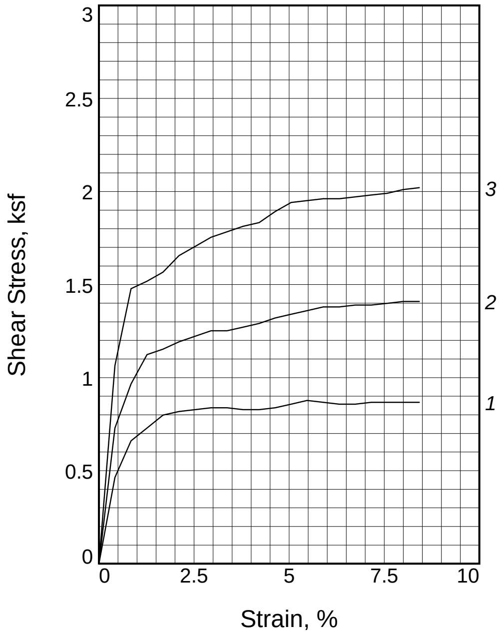
Allied Geotechnical Engineers, Inc.

Santee, CA

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



	Results
C, ksf	0.29
ϕ, deg	29.8
Tan(ϕ)	0.57



Sample No.		1	2	3
Initial	Water Content, %	10.5	11.4	10.8
	Dry Density, pcf	107.0	107.3	107.8
	Saturation, %	51.1	55.7	53.4
	Void Ratio	0.5465	0.5415	0.5341
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	23.0	23.2	21.7
	Dry Density, pcf	106.3	106.2	106.2
	Saturation, %	109.6	109.9	103.2
	Void Ratio	0.5558	0.5585	0.5572
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.01	1.01	1.01
Normal Stress, ksf		1.00	2.00	3.00
Fail. Stress, ksf		0.88	1.41	2.02
Strain, %		5.5	8.0	8.4
Ult. Stress, ksf				
Strain, %				
Strain rate, in./min.		0.007	0.007	0.007

Sample Type: remold
Description: Brown to dark brown, damp, silty sand (SM) with gravels.
LL= NV **PI=** NP
Specific Gravity= 2.65
Remarks:

Figure B-6

Client:

Project: SEWER GROUP 806 BRIDGE CROSSING PROJECT

Source of Sample: TP-2 **Depth:** 1
Sample Number: 1
Proj. No.: 185 GS-16-B **Date Sampled:** 7-31-2018

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

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

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: August 8, 2018
Purchase Order Number: 185 GS 16-B
Sales Order Number: 41100
Account Number: ALLG

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

Laboratory Number: SO6960-1 Customers Phone: 449-5900
Fax: 449-5902
Sample Designation:
--
One soil sample received on 08/02/18 at 9:00am, taken from Sewer Group 806 Project# 185 GS 16-B marked as B-1 #1 @ 3'-5'.

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

pH 7.8

Water Added (ml)	Resistivity (ohm-cm)
10	3000
5	1500
5	800
5	570
5	580
5	610

24 years to perforation for a 16 gauge metal culvert.
32 years to perforation for a 14 gauge metal culvert.
44 years to perforation for a 12 gauge metal culvert.
56 years to perforation for a 10 gauge metal culvert.
68 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417	0.016% (160ppm)
Water Soluble Chloride Calif. Test 422	0.009% (85ppm)
Bicarbonate (as $CaCO_3$) (on a 1:3 water extraction)	120ppm

Laura Torres

Laura Torres
LT/ilv

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: August 8, 2018
Purchase Order Number: 185 GS 16-B
Sales Order Number: 41100
Account Number: ALLG

To:

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

Laboratory Number: SO6960-2 Customers Phone: 449-5900
Fax: 449-5902

Sample Designation:

One soil sample received on 08/02/18 at 9:00am, taken from Sewer Group 806 Project# 185 GS 16-B marked as TP-2 #2 @ 4'-4.5'.

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

pH 7.9

Water Added (ml)	Resistivity (ohm-cm)
10	1100
5	670
5	440
5	350
5	340
5	340
5	350
5	360

20 years to perforation for a 16 gauge metal culvert.
26 years to perforation for a 14 gauge metal culvert.
35 years to perforation for a 12 gauge metal culvert.
45 years to perforation for a 10 gauge metal culvert.
55 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate	Calif. Test 417	0.017% (170ppm)
Water Soluble Chloride	Calif. Test 422	0.083% (830ppm)
Bicarbonate (as $CaCO_3$) (on a 1:3 water extraction)		40ppm

Laura Torres

Laura Torres
LT/ilv