PTS# 652296

AGE

Allied Geotechnical Engineers, Inc.

April 10, 2020

Mr. Patrick Mulvey, P.E.
Project Manager
Infrastructure Engineering Corporation
14271 Danielson Street
Poway, CA 92064

Subject: **RESPONSE TO CITY OF SAN DIEGO**
DEVELOPMENT SERVICES DEPARTMENT
REVIEW COMMENTS PERTAINING TO
REPORT OF GEOTECHNICAL INVESTIGATION
SEWER AND STORM DRAIN GROUP JOB 828 REPLACEMENT PROJECT
CITY OF SAN DIEGO
AGE Project No. 179 GS-16-F

Dear Mr. Mulvey,

This letter provides our response to the City of San Diego Development Services Department review comments which we received on January 24, 2020. The general and specific comments that we received and our response are presented in the table below.

No.	Development Services Department Comment	AGE Response
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 is attached.
3	The geotechnical investigation report must contain a geologic/geotechnical map for the areas of slope reconstruction that shows the distribution of fill and geologic units, proposed construction, proposed key, and location of cross sections.	The scope of the current project does not include any slope reconstruction.

No.	Development Services Department Comment	AGE Response
4	Circumscribe the limits of anticipated remedial grading on the geologic/geotechnical map to delineate the proposed footprint of the project.	Earthwork operations for the proposed project are anticipated to be limited to conventional cut-and-cover trenched construction and trenchless construction. No remedial grading is anticipated for the proposed project.
5	The geotechnical investigation report must contain representative geologic/geotechnical cross-sections for the areas of the proposed slope reconstruction that show the existing and proposed grades, distribution of fill and geologic units, and approximate location of the proposed key and benches.	The scope of the current project does not include any slope reconstruction.
6	The project's geotechnical consultant should clarify if the proposed construction will destabilize or result in settlement of private structures or the public right of way.	Since the proposed project is limited to conventional cut-and-cover trenched construction and trenchless pipeline construction, and no grading is anticipated, it our opinion that the proposed project is not anticipated to destabilize or results in settlement of adjacent property of the right-of-way, nor will the proposed improvements add surcharge on existing improvements or structures. Refer to Section 5.7 - Summary and Conclusions of the report.
7	The project geotechnical consultant should provide a statement as to whether or not the site is suitable for the intended use and the proposed construction.	The project alignment is suitable for construction of sewer and storm drain pipelines as shown on the 100% Design Plans. Refer to Section 5.7 - Summary and Conclusions of the report.
8	The project geotechnical consultant must provide a professional opinion that the site of the proposed construction will be adequately stable following project completion.	The project alignment will be adequately stable following completion of the construction of sewer and storm drain pipelines as shown on the 100% Design Plans. Refer to Section 5.7 - Summary and Conclusions of the report.

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
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UPDATED REPORT OF GEOTECHNICAL INVESTIGATION SEWER AND STORM DRAIN GROUP JOB 828 REPLACEMENT PROJECT CITY OF SAN DIEGO

Submitted to:

INFRASTRUCTURE ENGINEERING CORPORATION
14271 Danielson Street
Poway, CA 92064

Prepared By:

ALLIED GEOTECHNICAL ENGINEERS, INC.
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Santee, California 92071-2685

AGE Project No. 179 GS-16-F

April 10, 2020

AGE
Allied Geotechnical Engineers, Inc.

May 17, 2018
(Updated April 10, 2020)

Mr. Patrick Mulvey, P.E.
Project Manager
Infrastructure Engineering Corporation
14271 Danielson Street
Poway, CA 92064

Subject: **UPDATED REPORT OF GEOTECHNICAL INVESTIGATION**
SEWER AND STORM DRAIN GROUP JOB 828 REPLACEMENT PROJECT
CITY OF SAN DIEGO
AGE Project No. 179 GS-16-F

Dear Mr. Mulvey:

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 Infrastructure Engineering Corporation with their design of the subject project. We have reviewed the 100% Design Plans prepared by Infrastructure Engineering Corporation, undated. It is our opinion that the 100% Design 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 transmitted through electronic mail on January 24, 2020.

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
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UPDATED REPORT OF GEOTECHNICAL INVESTIGATION
SEWER AND STORM DRAIN GROUP JOB 828 REPLACEMENT 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 Infrastructure Engineering Corporation (IEC) with their design of the Group Job 828 Sewer and Storm Drain Replacement Project for the City of San Diego (City). The investigation was performed in conformance with AGE's proposal dated August 17, 2016 (Revised May 12, 2017), and the subconsultant agreement entered into by and between IEC and AGE on March 29, 2017.

This report has been prepared for the exclusive use of IEC 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

Based on a review of the 100% Design Plans prepared by IEC, undated, it is our understanding that the scope of the Group Job 828 Sewer and Storm Drain Replacement Project includes the replacement of three (3) existing storm drain facilities, replacement of 9,100 feet of existing sewer pipe with new 12- to 15-inch diameter pipelines using trenchless construction methods, and construction of approximately 970 feet of 8-inch diameter pipeline in new trenches along Olive Street, Nutmeg Place, Palm Street, the alley between Quince Street and Palm Street, and the alley between Nutmeg Street and Olive Street. The proposed project alignment is shown on Figure 1 - Alignment Map. The three storm drain facilities are described below:

- Maple Street Storm Drain - from Maple Street (east of 28th Street) down into Switzer Canyon West. This segment is approximately 200 feet in length with a vertical drop of 60 feet;

- Olive Street Storm Drain - from Olive Street (between 30th Street and 31st Street) down into Switzer Canyon East. This segment is approximately 125 feet in length with a vertical drop of 70 feet; and

- Palm Street Storm Drain - from Palm Street (between 31st Street and 32nd Street) down into Switzer Canyon East. The segment is approximately 190 feet in length with a vertical drop of 50 feet.

The majority of the project alignment is located in Switzer Canyon Park and Balboa Park Municipal Golf Course. The northeast portion of the project alignment extends along several existing streets in the community of North Park (28th Street, Maple Street, 30th Street, Burlingame Street, Nutmeg Place, Alley Block II and Olive Street).

It is our understanding that the replacement sewer pipelines range between 8-inch to 15-inch in diameter, and that the majority of the sewer will be installed using trenchless construction methods. Conventional cut-and-cover construction methods will be employed where the sewer follows the existing streets in the North Park area. The sewer will be installed in a steel casing where the pipe crosses below 30th Street. Soil cover above the sewer will typically vary from 3-feet to 15-feet, with approximately 65 feet of soil cover where the pipe crosses below 30th Street.

It is anticipated that the storm drain replacement will be performed using conventional cut-and-cover construction methods. All three storm drain segment will be replaced with 18-inch diameter reinforced concrete pipe (RCP) with 3 feet to 4 feet soil cover above the pipe crest.

Existing improvements along and adjacent to the project alignment include Balboa Park Municipal Golf Course, Switzer Canyon, residential neighborhoods, and open space. The topography along the project alignment varies from gentle to moderate sloping. Open space within Switzer Canyon is vegetated with a variety of native and non-native shrubs and trees.

3.0 OBJECTIVE AND SCOPE OF INVESTIGATION

The objectives of this investigation were to characterize the subsurface conditions along the project alignment 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 alignment, including the preliminary project plans, as-built utility maps, topographic maps, published geologic literature and maps, and AGE's in-house references. A listing of references that were reviewed is presented in Section 8.0.

3.2 Geotechnical Field Exploration

The initial field exploration program for this project was performed during the period between February 21 and April 17, 2018. A total of six (6) test pits and two (2) soil borings were performed at the approximate locations shown on Figure 1. A third boring was performed on July 30, 2018 and the approximate boring location is also shown on Figure 1. The pits were excavated using manual labor to depths ranging from 3.5 feet to 8 feet below the existing ground surface (bgs). The soil borings were advanced to depths ranging from 3.5 feet to 58 feet bgs. A more detailed description of the excavation and sampling activities, and logs of the test pits and borings are presented in Appendix A.

Drilling refusal on boulders was encountered in boring B-1 which is located on the east side of 30th Street in the vicinity of Burlingame Drive. The boring was terminated at a depth of 58 feet below the existing ground surface (approximate elevation +215 feet msl), approximately 8 feet above the proposed 15-inch diameter sewer pipe invert elevation which is planned to be installed using trenchless construction methods. Subsequently, boring B-3 was performed at the bottom of the canyon on the east side of 30th Street to a depth of 31 feet bgs (approximate elevation +187 feet msl).

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 test pits. Subsequently, Underground Service Alert (USA) was contacted to coordinate clearance of the proposed pit locations with respect to existing buried utilities. The utility clearance efforts 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.

3.3 Laboratory Testing

Selected soil samples obtained from the test pits and 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, 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 Switzer Canyon, a southeast trending canyon incised into a mesa top in North Park and Balboa Park. 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 Pliocene in age. Man-made fills and Holocene age alluvial deposits were also encountered at various locations throughout 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 soil borings and test pits can be categorized into five geologic units which include (in order of increasing age) fill materials; wash deposits; young colluvial deposits; very old paralic deposits; and the San Diego Formation. A Generalized Geologic Map of the project alignment is shown on Figure 2. A brief description of each unit is presented below.

4.3.1 Fill Materials

Fill materials were encountered in all of the test pits and soil borings with the exception of test pit TP-3 and boring B-2. The fill materials extended to the maximum depth of exploration of 8 feet bgs in test pits TP-1 and TP-6, to the maximum depth of exploration of 58 feet bgs in boring B-1, and to the depth of 8 feet bgs in boring B-3. Fill materials encountered in the test pits generally consist of silty sands and sandy silts containing scattered to locally abundant sub-rounded to sub-angular gravel and cobbles, and possibly boulders in boring B-1. Traces of glass and metal were locally encountered in the fill. Although the test pits are located in open areas, the fill materials encountered in the test pits may be associated with nearby residential developments along the side walls of Switzer Canyon. Documentation pertaining to the original placement of the fill materials is unavailable.

Boring B-1 was performed on a causeway which crosses Switzer Canyon. The causeway was built in 1957 to replace the aging 30th Street trolley bridge. Soil materials utilized in the construction were reportedly imported from Nile Street in North Park (www.infogalactic.com). The fill materials encountered in boring B-1 generally consist of silty sands with trace to locally abundant gravel and cobbles. Blow counts performed during sampling indicate that the fill materials are moderately well compacted, but comparison of in-situ dry densities of the fill with the laboratory determined maximum dry density performed in accordance with ASTM test method D-1557 indicates that the majority of the fill is below the generally accepted current compaction standard of 90 percent. Documentation pertaining to the original placement of the fill materials is unavailable.

The fill materials encountered in boring B-3 generally consists of silty sands and sandy silts containing scattered sub-rounded gravel. Abundant sub-rounded cobbles up to 6-inches in maximum dimension were also observed on the ground surface adjacent to the boring. The fill materials may be associated with prior installation of the causeway which crosses Switzer Canyon. Documentation pertaining to the original placement of the fill materials is unavailable.

4.3.2 Wash Deposits

Wash deposits of late Holocene age (Kennedy & Tan, 2008) were encountered at surface grade in test pit TP-3, which was excavated in the active creek channel. The wash deposits are underlain by the San Diego Formation. The wash deposits are generally described as unconsolidated bouldery to sandy alluvium of active to recently active stream channels (Kennedy and Tan, 2008). The presence of abundant boulders can pose difficult excavation conditions for conventional heavy duty construction equipment and trenchless construction.

Wash deposits encountered in test pit TP-3 generally consist of unconsolidated silty sand containing abundant gravel and cobbles up to 10" in maximum dimension. Clasts are typically sub-rounded and locally fractured.

4.3.3 Young Colluvial Deposits

Although not shown on the published geologic map, young colluvial deposits were encountered below the fill materials in all of the test pits with the exception of test pit TP-3. Test pits TP-1 and TP-5 were each terminated in the colluvial deposits, and in test pits TP-2 and TP-4 the young colluvial deposits were underlain by the San Diego Formation. The colluvial deposits are described as poorly consolidated and poorly sorted sand and silt slopewash deposits (Kennedy and Tan, 2008). These deposits can generally be easily excavated with conventional heavy duty construction equipment.

The young colluvial deposits encountered in the test pits generally consist of fine to medium grained silty sands and local clayey sand with scattered to abundant sub-rounded and sub-angular gravel and cobbles. Trace roots and rootlets were also encountered in the colluvial deposits. The deposits were typically in a medium dense to dense condition, and damp to wet.

4.3.4 Very Old Paralic Deposits

Pleistocene age very old paralic deposits were encountered below paving at boring B-2, extending to the maximum depth of excavation. 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). Locally strong cementation and conglomerate layers can present difficult excavation conditions even with conventional heavy duty construction equipment.

The very old paralic deposits encountered in our test boring consist of silty sand containing abundant sub-rounded gravel and cobbles in a dense to very dense condition. We encountered refusal on cemented cobble-conglomerate at a depth of 3.5 feet bgs.

4.3.5 San Diego Formation

The San Diego Formation was encountered below the wash deposits and/or young colluvial deposits in all of the test pits with the exception of TP-1 and TP-5, and below the fill materials in boring B-3. Where encountered, the San Diego Formation extended to the maximum depths of exploration.

The San Diego Formation predominantly consists of a yellow brown and gray, fine to medium grained, poorly indurated marine sandstone and a reddish brown, transitional marine and non-marine pebble and cobble-conglomerate (Kennedy and Tan, 2008). Thin beds of bentonite, marl, and brown mudstone may also be encountered in the unit. Based on fossil assemblages, the San Diego Formation has been assigned an early Pleistocene and late Pliocene age. The San Diego Formation can generally be easily excavated with conventional heavy duty construction equipment.

San Diego Formation encountered in our soil test pits and boring B-1 consists of yellow brown to olive, dense to very dense, fine-grained silty sandstone in a damp condition. The sandstone is moderately to strongly cemented.

4.4 Groundwater

At the time of our field investigation, no groundwater and/or seepage was encountered in the test pits and soil borings. Formational materials encountered in the pits and soil boring generally possess low to moderate permeability characteristics.

Review of the Geotracker website (www.Geotracker.com) did not reveal any nearby groundwater elevation data or wells. Based on a review of the available data, the depth (elevation) of the regional groundwater table beneath the project alignment is estimated to be well below the anticipated depths of excavation. It must be noted, however, that localized perched water conditions may be encountered along the project alignment, especially during the rainy (wet) season. Flowing water may also be encountered in Switzer Canyon following strong rainstorm events.

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 52, "Other level areas, gently sloping to steep terrain, favorable geologic structure, Low Risk". The classification is not expected to impact the proposed project. Based on the results of our study, several potential geologic hazards are identified along the project corridor which are more fully described herein.

5.1.1 Faulting

The Texas Street fault crosses the project alignment in a north-south direction near 28th Street (City of San Diego Seismic Safety Study, 2008). The fault is classified in the study as "potentially active, inactive, presumed inactive, or activity unknown". However, the Texas Street fault is not mapped by Kennedy and Tan (2008) and is generally considered by most local experts to be inactive. 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 alignment 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 alignment (Kennedy, 1975; Kennedy and Tan, 2008). Therefore, the potential for fault ground rupture and ground lurching along the alignment 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 determined that the project alignment is underlain with dense/stiff formational soils that are considered to have a very low to negligible liquefaction potential.

5.1.4 Landslides

A review of the published geologic maps indicate that the project alignment does not cross any known (mapped) ancient landslides(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 alignment 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 areas underlain by deep mechanically placed man-made fills.

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 alignment is not located within the 100- and 500-year flood zone (FEMA Flood Insurance Rate Map, 2012). However, seasonal flooding along the bottom of Switzer Canyon during a heavy precipitation event should be anticipated.

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 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)
TP-1 Sample No. 2 @4'-5'	7.1	2,500	72	11	12
TP-2 Sample No. 3 @3.5''-4'	7.8	750	110	150	42
TP-3 Sample No. 2 @5'-5.5'	6.5	920	110	75	8
TP-4 Sample No. 4 @5'-5.5'	7.0	820	150	11	N/A
TP-5 Sample No. 2 @ 3'-3.5'	5.9	4,200	78	21	10
TP-6 Sample No. 3 @ 4'-4.5'	7.9	1,600	200	11	42
B-3 Sample No. 7 @23'-24'	8.9	2,500	100	21	66

The test results indicate that soils along the project alignments are not considered aggressive to concrete. Therefore, Type I and Type 2 Portland Cement Concrete may 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 on-site materials are considered non-expansive or have a low expansion potential.

5.4 Fill Material

5.4.1 Flowable Fill

Flowable fill refers to a cementitious slurry consisting of a mixture of fine aggregate or filler, water, and cementitious material(s), which is used as a fill or backfill in lieu of compacted earth. The mixture is capable of filling all voids in irregular excavations and hard to reach places, self-leveling, and hardens in a matter of a few hours without the need for compaction. Flowable fill may be used for trench backfill and slope reconstruction

Flowable fill for the subject project should be designed with a compressive strength that will allow excavation with heavy machinery at a maximum compressive strength of 200 psi at 1 year and maximum unit weight not to exceed 115 pounds per cubic foot (pcf). The coefficient of permeability of the flowable fill should be equal or greater than that of the surrounding soil.

Flowable fill should have minimal subsidence and bleed water shrinkage. Evaporation of bleed water should not result in shrinkage of more than 1/8 inch per foot of flowable fill depth (for mixes containing high fly ash content) when measured in accordance with ASTM C 940 test method "Standard Test Method for Expansion and Bleeding of Freshly Mixed Grouts for Preplaced-Aggregate Concrete in the Laboratory".

Flowable fill should be sampled and tested in the field in conformance with either ASTM C 94 or C 685. Samples for tests should be taken for every 150 cubic yards of material, or fraction thereof, for each day's placement. Tests should include temperature reading and four compressive strength cylinders. Compressive strength sampling and testing should conform to ASTM D 4832 with one specimen tested at 7 days, two at 28 days, and one held for each batch of four specimens.

Perform installation of flowable fill only when approved by the Resident Engineer, and when existing and forecasted weather conditions are within the limits established by the manufacturer of the materials and products used. The mix design should produce a consistency that will result in a flowable product at the time of placement which does not require manual means to move it into place. Placement of the flowable fill should be performed in accordance with the manufacturer's mix design specifications. Flowable fill materials are considered suitable for use in slope reconstruction and as trenched excavation backfill.

5.4.2 Soil Backfill

Fill material for trench backfill and slope reconstruction 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 onsite soil materials generated from excavations within the San Diego Formation are considered suitable for use as compacted backfill materials. Materials generated from excavations in the young colluvial deposits, the wash deposits and Very Old Paralic Deposits may require selective screening to remove large (in excess of 6 inches in maximum dimension) rock clasts prior to placement as compacted fill. The contractor may find it more cost efficient to use import fill materials in lieu of employing selective screening methods to remove large rock clasts.

5.5 Cut-and-Cover Construction

Since no significant changes to the existing ground surface along the cut-and-cover segment of the proposed storm drain and sewer pipeline alignments 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 majority of the materials within the anticipated depths of the storm drain and sewer pipe trench excavations will likely be comprised of materials which can be readily excavated with conventional heavy-duty construction equipment. Excavation within the Very Old Paralic Deposits which is anticipated to be encountered in the vicinity of Olive Street, Nutmeg Place, Palm Street, alley between Quince Street and Palm Street, and alley between Nutmeg Street and Olive Street, may require jackhammering operations. Materials generated from excavations within the colluvial deposits, the wash deposits and Very Old Paralic Deposits are generally not considered suitable for use as backfill materials due to their high cobble content and may require selective screening to remove large (in excess of 6 inches in maximum dimension) rock clasts prior to placement as compacted fill. However, due to space limitation, using import fill to backfill the trenched excavations may be more cost effective.

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 the groundwater level may be estimated assuming a density of 100 pcf for the 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 350 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 below.

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. Compaction is not required in the event that flowable fill is used to backfill the trenched excavations.

5.5.5 Concrete Anchor/Cutoff Wall

We recommend that for segments of the proposed storm drain and sewer pipelines that are installed at a slope of 3 : 1 (horizontal : vertical), or steeper, concrete anchors and/or cutoff walls be used to provide support for both the storm drain pipe and the trench backfill. Concrete anchor and/or cutoff wall may be designed in accordance with Drawing Numbers SDS-114 or SDS-115 of the City of San Diego - Standard Drawings for Public Works Construction. Based on the slope gradient, subsurface conditions, and depth of excavation, when and if it is necessary, we recommend that the anchor and/or cutoff wall be installed at approximate 25-foot intervals.

5.6 Trenchless Construction

It is anticipated that the majority of the proposed sewer pipelines which will be constructed using trenchless construction method will extend through the San Diego Formation which is considered favorable for both Microtunnel Boring Machine (MTM) and Horizontal Directional Drilling (HDD). Trenchless construction across 30th Street causeway may require lowering the pipe invert elevation on the east side of 30th Street to elevation +206 feet MSL for the pipeline to stay out of the fill materials and within the San Diego Formation. Trenchless construction in the vicinity of Olive Street, Nutmeg Place, Palm Street, alley between Quince Street and Palm Street, and alley between Nutmeg Street and Olive Street will likely encounter Very Old Paralic Deposits. Trenchless construction in this area may require the use of Jack & Bore construction method using oversized steel casing.

For trenchless construction, the formational unit can be classified as being firm as described by the Tunnelman's Ground Classification System (Bickel & Kuesel, 1995). For assessing the stability of the San Diego Formation and Very Old Paralic Deposits, the formations may be modeled as having an undrained shear strength of 2,500 psf and 3,500 psf, respectively.

The following formula may be used to estimate ground deformation due to the trenchless construction operations.

$$d_{\max} = (2.5i/V_s)$$

$d_{\max}$ is maximum ground settlement;
i is equal to K times the depth to the center of the pipe; and
V_s is the volume loss due to the excavation per foot of pipe.

For the formational units at the project site, we recommend using a K of 0.25 and a V_s equal to 1 percent of the excavated face. Ground settlement adjacent to the trenchless alignment may be estimated using the following equation.

$$d = d_{\max} \exp(-x^2/2i^2)$$

x is the distance from the centerline of the pipe (feet);
i is defined as Kz where z is the depth to the center of the pipe (feet); and
d is the ground displacement at x.

We recommend using a coefficient of 0.55 for steel casing against soil and 0.88 for concrete against soil. We further recommend using a unit weight of 130 pcf for calculating the normal pressure acting on the casing.

5.7 Summary and Conclusions

We have reviewed the "100% Design Plans” prepared by IEC, undated. It is our opinion that the project plans were prepared in conformance with the design recommendations provided herein. Since the proposed project is limited to conventional cut-and-cover trenched construction and trenchless pipeline construction, and no grading is anticipated, it our opinion that the proposed project will not destabilize or results in settlement of adjacent property of the right-of-way, nor will the proposed improvements add surcharge on existing improvements or structures. The project alignment is suitable for construction of sewer and storm drain pipelines as shown on the 100% Design Plans. The project alignment will be adequately stable following completion of the construction of sewer and storm drain pipelines as shown on the 100% Design Plans.

6.0 CONSTRUCTION-RELATED CONSIDERATIONS

6.1 Construction Dewatering

The depth of the local groundwater table is expected to be well below the anticipated depth of the proposed excavations for this project. No groundwater or seepage was encountered in the borings and test pits. 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. Localized perched water conditions would most likely occur at the interface between fill materials and formational materials. 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, wash deposits and young colluvial deposits, 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 the San Diego Formation and Very Old Paralic Deposits, 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. 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 alluvial deposits may be assumed to be equivalent to a fluid weighing 250 pcf. Allowable lateral bearing pressure in fill material, wash deposits and young colluvial deposits should not exceed 2,500 psf. Allowable passive pressure for Very Old Paralic Deposits and San Diego Formation may be assumed to be equivalent to a fluid weighing 350 pcf, with maximum allowable lateral bearing pressure of 3,500 psf.

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 alignment. 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 IEC 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 alignment. 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 borings and 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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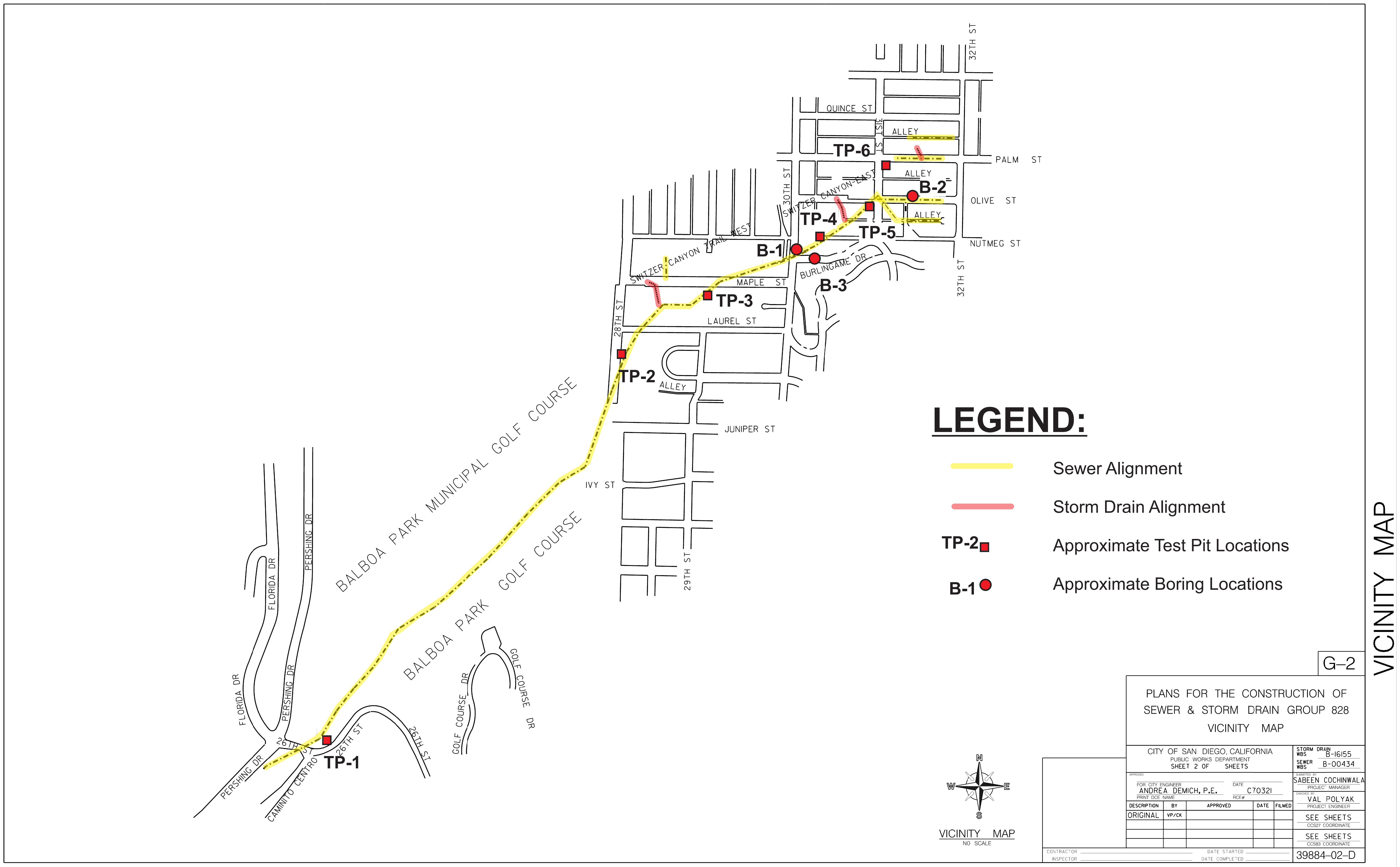
Aerial Photographs

U.S. Department of Agriculture black and white aerial photograph Nos. AXN-3M- 216 and 217 (dated 1953)



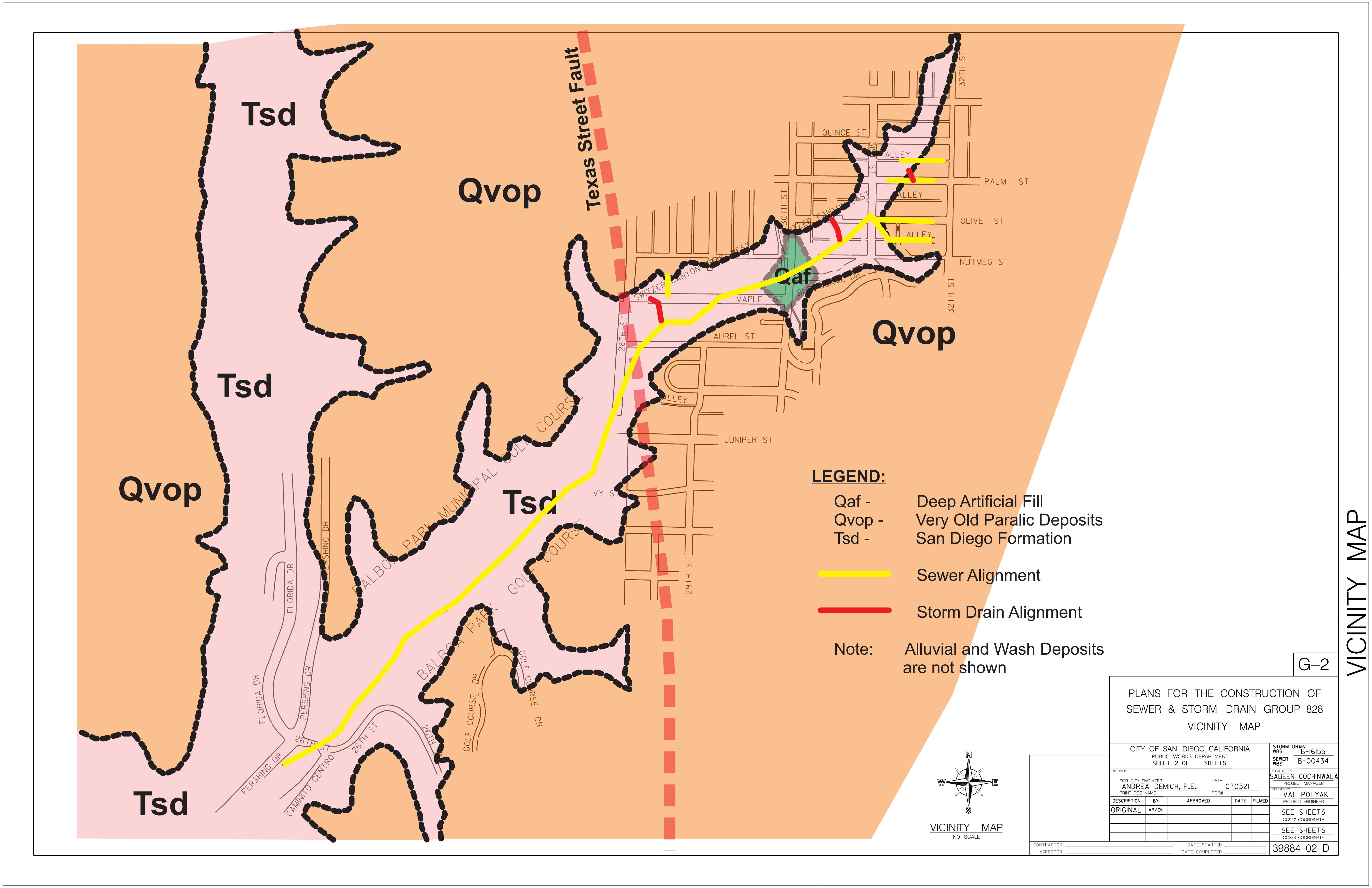
SEWER AND STORM DRAIN GROUP JOB 828 REPLACEMENT PROJECT

ALIGNMENT MAP

PROJECT NO.
179 GS-16-F

ALLIED GEOTECHNICAL ENGINEERS, INC.

FIGURE 1



LEGEND:

Qaf - Deep Artificial Fill
Qvop - Very Old Paralic Deposits
Tsd - San Diego Formation

🟨 Sewer Alignment

🟥 Storm Drain Alignment

Note: Alluvial and Wash Deposits are not shown

SEWER AND STORM DRAIN GROUP JOB 828 REPLACEMENT PROJECT

GENERALIZED GEOLOGIC MAP

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ALLIED GEOTECHNICAL ENGINEERS, INC.

FIGURE 2

APPENDIX A

FIELD EXPLORATION PROGRAM

APPENDIX A

FIELD EXPLORATION PROGRAM

The initial field exploration program for this project was performed during the period between February 21 and April 17, 2018. A total of six (6) test pits and two (2) soil borings were performed at the approximate locations shown on Figure 1. A third boring was performed on July 30, 2018 and the approximate boring location is also shown on Figure 1. The test pits were performed using manual labor to depths ranging from 4 feet to 8 feet below the existing ground surface (bgs), and the borings were performed with a CME-75 or equivalent truck mounted drill rig to depths ranging from 3.5 feet to 58 feet bgs. The soils encountered in the test pits and 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 and test pits are presented on Figures A-3 thru A-12. 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 test pits and soil borings. 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 pits using a nuclear soil gauge (ASTM D6938-10). In addition, 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 test pits. The sampler was driven a distance of approximately 12 inches into the soil at the bottom of the test pit with a drop weight. 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.

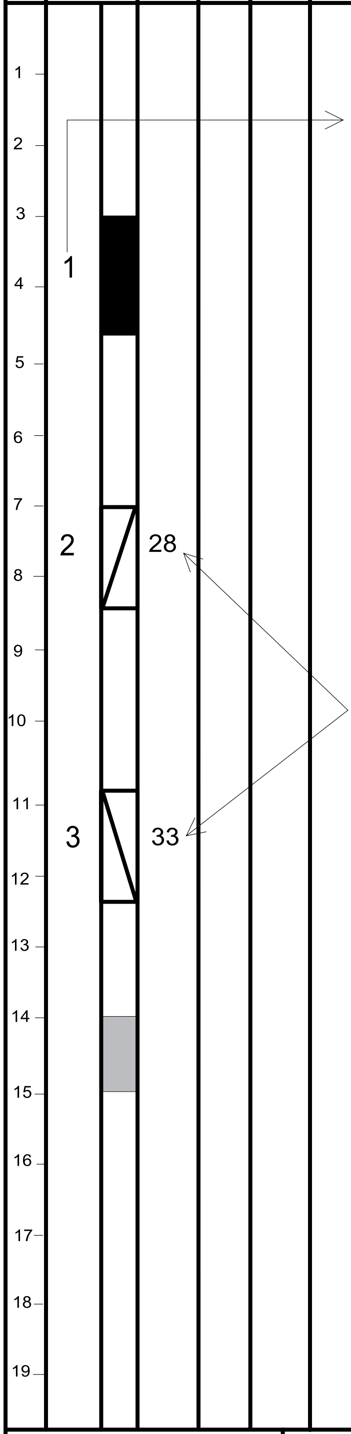
During drilling, Standard Penetration Tests (SPT) were performed at selected depth intervals. The SPT tests involve the use of a specially manufactured “split spoon” sampler which is driven into the soils at the bottom of the borehole by dropping a 140-pound weight from a height of 30 inches. The number 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 collected from the the test pits and borings 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 boring and test pit logs. In addition, loose bulk samples were also collected.

Upon completion of the field exploration activities, all of the test pits were backfilled with excess soil cuttings and compacted. Upon completion of the drilling and sampling activities, boring B-1 was backfilled using bentonite grout to approximately 12 inches below the ground surface. The boring was performed in a dirt area, and was capped with excess soil cuttings. Borings B-2 and B-3 were backfilled using bentonite chips to approximately 12 inches below the ground surface and capped with rapid-set concrete to match the adjacent pavement surface.

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



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

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

PROJECT NO. 179 GS-16-F	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
					—?— —?— APPROXIMATE GEOLOGIC CONTACT			
1								
2								
3								
4								
5								
6								
7								
8								
9								
10								
11								
12								
13								
14								
15								
16								
17								
18								
19								

Strata symbols

▥	**Silty sand**
▩	**Silty gravel and sand**
▨	**Clayey sand**
▩	**Silty gravel**

GENERAL NOTES

1. Approximate elevations and locations of borings are based on the topographical maps provided by Infrastructure Engineering Corporation, 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.

BORING NO. B-1

DATE OF DRILLING: 04/17/2018
TOTAL BORING DEPTH: 58 FEET
GENERAL LOCATION: EAST SHOULDER OF 30TH STREET, 40' NORTH OF STORMDRAIN INLET (STATION 62+75)
APPROXIMATE SURFACE ELEV.: 276 FEET MSL
DRILLING CONTRACTOR: TRI-COUNTY DRILLING
DRILLING METHOD: 8-INCH HSA
LOGGED BY: NICK 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					**FILL**			
2					Pale yellow, dry to damp silty sand (SM) with abundant subrounded gravel and cobble up to 8" in maximum dimension.			
3								
4								
5								
6	1	36	0		Light yellow brown to pale olive, damp, micaceous silty fine-grained sand (SM) with subrounded gravel up to 3" in maximum dimension.	14.1	102.5	
7	2							
8								
9								
10								
11	3	19	0.2		Light yellow brown to brownish yellow, damp, micaceous silty fine-grained sand (SM) and Sandy Silt (ML) with trace of subrounded gravel up to 3/4" in maximum dimension.	15.6		
12								
13								
14								
15								
16	4	43	0.1		Light yellow brown to olive yellow, damp, micaceous silty fine-grained sand with trace to scattered subrounded gravel up to 2" in maximum dimension.	13.2	96.4	
17								
18								
19								
20								
21	5	38	0.3		Light yellow brown to pale olive, damp, micaceous silty fine-grained sand (SM) with fractured gravel.	11.4		
22								
23								
24								
25								
26	6	94	0.6			8.7	112.8	
27	7							
28								
29								
30								
31	8	61	0.3			9.8		
32								
33								
34								
35	9	100+	0.4			10.3	102.2	
36								
37								

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ALLIED GEOTECHNICAL ENGINEERS, INC.

FIGURE A-3

BORING NO. B-1

DATE OF DRILLING: 04/17/2018
TOTAL BORING DEPTH: 58 FEET
GENERAL LOCATION: EAST SHOULDER OF 30TH STREET, 40' NORTH OF STORMDRAIN INLET (STATION 62+75)
APPROXIMATE SURFACE ELEV.: 276 FEET MSL
DRILLING CONTRACTOR: TRI-COUNTY DRILLING
DRILLING METHOD: 8-INCH HSA
LOGGED BY: NICK 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
38								
39								
40	10	100+			Scattered to locally abundant gravels and cobbles below a depth of 40 feet			No recovery
41								
42								
43								
44								
45	11	100+						No recovery
46								
47								
48								
49								
50	12	100+						No recovery
51								
52								
53								
54								
55	13	100+	0.2			10.7	99.9	
56								
57								
58								
59								
60								
61								
62								
63								
64								
65								
66								
67								
68								
69								
70								
71								
72								
73								
74								

NOTES:

Drilling encountered refusal on large cobbles at 58' bgs
No groundwater and or seepage encountered during the drilling operations

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FIGURE A-4

BORING NO. B-2

DATE OF DRILLING: 04/17/18
TOTAL BORING DEPTH: 2 FEET
GENERAL LOCATION: NORTH SIDE OF OLIVE STREET APPROXIMATELY 30' EAST OF NUTMEG PLACE (STATION 4+40)
APPROXIMATE SURFACE ELEV.: +297 FEET MSL
DRILLING CONTRACTOR: TRI-COUNTY DRILLING
DRILLING METHOD: 8-INCH HSA
LOGGED BY: NICK 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** 6" A.C., no base			
2	1				**VERY OLD PARALIC DEPOSITS** Reddish yellow, damp, silty sand (SM) with scattered to abundant sub-rounded gravel and cobbles			
3								
4								
5								
6					**NOTES:**			
7								
8					Drilling encountered refusal on cobbles at 3.5' bgs No groundwater and or seepage encountered during the drilling operations			
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								

BORING NO. B-3

DATE OF DRILLING: 7-30-18
TOTAL BORING DEPTH: 31
GENERAL LOCATION: City access road on extension of Burlingame Drive, approximately 145' east of 30th Street
APPROXIMATE SURFACE ELEV.: 218' MSL
DRILLING CONTRACTOR: Tri-County Drilling
DRILLING METHOD: 8" HSA
LOGGED BY: N. 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
					PAVEMENT SECTION			
1								
2					2" A.C., no base			
3					**FILL**			
4					Dark brown to yellow brown, damp, gravelly silty sand (SM).			
5								
6	1	31	0.4			2.4		
7	2							
8			?		? ?			?
9					**SAN DIEGO FORMATION**			
10					Light gray to brownish yellow, damp, dense, fine to medium-grained, micaceous, silty sandstone (SM)			
11	3	38				20.5	102.9	
12								
13								
14					Light olive gray to olive, damp, stiff, sandy siltstone (ML).			
15								
16	4	29	0.2			14.5		
17								
18					Light gray with pale reddish yellow oxide staining, damp, dense to very dense, fine-grained, micaceous, silty sandstone (SM).			
19								
20								
21	5	100+	0.3			11.4	96.0	
22	6							
23	7							
24								
25								
26	8	50				15.6		
27								
28								
29								
30	9	100+				12.7	90.9	
31								
32								
33								
34								
35								
36								
37								

NOTES:

Boring terminated at depth of 31' bgs
No groundwater and/or seepage encountered during the drilling operations.

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FIGURE A-6

TEST PIT: TP-1

DATE OF EXCAVATION: 02/23/2018
TOTAL DEPTH: 7 FEET
GENERAL LOCATION: 26TH STREET EAST OF PERSHING DRIVE (STATION 6+00)
APPROXIMATE SURFACE ELEV.: +98 FEET MSL
EXCAVATION CONTRACTOR: MANSOLF EXCAVATION
EXCAVATION METHOD: MANUAL EXCAVATION
LOGGED BY: NICK 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					**FILL**	9.8	119.6	
2	1				Yellow brown, moist, loose to medium dense silty sand (SM) with occasional cobbles up to 4" in maximum dimension	14.3	89.5	
3						13.6	90.1	
4	2		?	▨				?
5				▨				
6	3			▨	?			
7				▨	**YOUNG COLLUVIAL DEPOSITS**			
8					Medium brown, moist, loose silty sand (SM) with approximately 30% to 40% gravels and cobbles up to 6" in maximum dimension			
9								
10								
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NOTES:

Bottom of test pit at 7' bgs
No groundwater and/or seepage observed in the test pit at the time of excavation

TEST PIT: TP-2

DATE OF EXCAVATION: 02/21/2018
TOTAL DEPTH: 4 FEET
GENERAL LOCATION: EAST OF 28TH STREET SOUTH OF LAUREL STREET (STATION 43+00)
APPROXIMATE SURFACE ELEV.: +172 FEET MSL
EXCAVATION CONTRACTOR: MANSOLF EXCAVATION
EXCAVATION METHOD: MANUAL EXCAVATION
LOGGED BY: NICK 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			
1					Yellow brown to reddish brown, dry to moist silty	10.3	89.8	
2	1		?		sand with cobbles up to 6" in maximum dimension	10.7	109.9	?
3	2				**YOUNG COLLUVIAL DEPOSITS**	10.7	119.4	
4	3		?		Grades into reddish brown to brownish gray, wet clayey			?
5					sand (SC) with cobbles up to 6" in maximum dimension **SAN DIEGO FORMATION**			
6								
7					Yellow brown, dense, moist silty sand (SM)			
8								
9					**NOTES:**			
10					Bottom of test pit at 4' bgs			
11					No groundwater and/or seepage observed in the test pit at the time of excavation			
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PROJECT NO.
179 GS-16-F

ALLIED GEOTECHNICAL ENGINEERS, INC.

FIGURE A-8

TEST PIT: TP-4

DATE OF EXCAVATION: 02/22/2018
TOTAL DEPTH: 5.5 FEET
GENERAL LOCATION: WEST OF NUTMEG STREET CUL-DE-SAC, WEST OF 31ST STREET (STATION 64+46)
APPROXIMATE SURFACE ELEV.: +219 FEET MSL
EXCAVATION CONTRACTOR: MANSOLF EXCAVATION
EXCAVATION METHOD: MANUAL EXCAVATION
LOGGED BY: NICK 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					**FILL**			
1	1				Medium to dark brown, moist silty sand (SM) with cobbles up to 8" in maximum dimension	7.5	103.4	
2			?		? ?	12.2	93.5	?
3	2				**YOUNG COLLUVIAL DEPOSITS**	5.0		
4					Dark brown, moist, dense silty fine to medium grained sand (SM) with scattered gravels and cobbles up to 6" in maximum dimension			
5	3		?			7.4	103.6	?
6	4				**SAN DIEGO FORMATION**			
7					Yellow brown, moist, dense to very dense silty sand (SM)			

NOTES:

Bottom of test pit at 5.5' bgs
No groundwater and/or seepage observed in the test pit at the time of excavation

PROJECT NO.
179 GS-16-F

ALLIED GEOTECHNICAL ENGINEERS, INC.

FIGURE A-9

TEST PIT: TP-3

DATE OF EXCAVATION: 02/21/2018	TOTAL DEPTH: 6 FEET
GENERAL LOCATION: SOUTH OF MAPLE STREET (STATION 55+00)	
APPROXIMATE SURFACE ELEV.: +196 FEET MSL	EXCAVATION CONTRACTOR: MANSOLF EXCAVATION
EXCAVATION METHOD: MANUAL EXCAVATION	LOGGED BY: NICK 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					**YOUNG ALLUVIAL DEPOSITS**			
2								
3	1				Gray brown to light yellow brown, damp, silty gravel with sand (GM) with subrounded and fractured clasts up to 10" in maximum dimension			
4								
5			?		?			?
6	2 3				**SAN DIEGO FORMATION** Olive green, damp, dense, silty fine grained sandstone (SM)	12.4	94.9	
7								
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NOTES:

Bottom of test pit at 6' bgs
No groundwater and/or seepage observed in the test pit at the time of excavation

PROJECT NO. 179 GS-16-F	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE A-10

TEST PIT: TP-5

DATE OF EXCAVATION: 02/22/2018
TOTAL DEPTH: 7 FEET
GENERAL LOCATION: SOUTH OF OLIVE STREET AND WEST OF 31ST STREET (STATION 69+80)
APPROXIMATE SURFACE ELEV.: +230 FEET MSL
EXCAVATION CONTRACTOR: MANSOLF EXCAVATION
EXCAVATION METHOD: MANUAL EXCAVATION
LOGGED BY: NICK 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 2	1				**FILL** Brown, damp, silty fine sand (SM) with abundant subrounded and feactured gravels and cobbles up to 6" in maximum dimension and traces of broken glass	9.1 9.4	118.4 118.47	
3 4 5 6 7	2		?		**YOUNG COLLUVIAL DEPOSITS** Dark gray, damp to wet, silty fine to medium grained sand (SM) with abundant gravels and cobbles up to 8" in maximum dimension			?

NOTES:

Bottom of test pit at 7' bgs
Seepage observed at a depth of 6.5' in the test pit at the time of excavation

8 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

PROJECT NO.
179 GS-16-F

ALLIED GEOTECHNICAL ENGINEERS, INC.

FIGURE A-11

TEST PIT: TP-6

DATE OF EXCAVATION: 02/22/2018
TOTAL DEPTH: 5.5 FEET
GENERAL LOCATION: SOUTH OF PALM STREET AND EAST OF 31ST STREET (STATION 1+00)
APPROXIMATE SURFACE ELEV.: +250 FEET MSL
EXCAVATION CONTRACTOR: MANSOLF EXCAVATION
EXCAVATION METHOD: MANUAL EXCAVATION
LOGGED BY: NICK 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				**FILL**	7.9		
1					Medium brown, loose to medium dense silty medium fine sand with broken glass and rusted metals	12.5	82.2	
2	2		?		**YOUNG COLLUVIAL DEPOSITS**	13.3	106.1	?
3						9.1	110.5	
4	3		?		Yellow brown, damp, silty sand (SM) with abundant gravels and cobbles up to 6" in maximum dimension			?
5	4				**SAN DIEGO FORMATION**	7.7	108.9	
6					Yellow brown, moist to wet, medium dense silty sand (SM)			
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NOTES:

Bottom of test pit at 5.5' bgs
No groundwater and/or seepage observed in the test pit at the time of excavation

PROJECT NO.
179 GS-16-F

ALLIED GEOTECHNICAL ENGINEERS, INC.

FIGURE A-12

APPENDIX B

LABORATORY TESTING

APPENDIX B

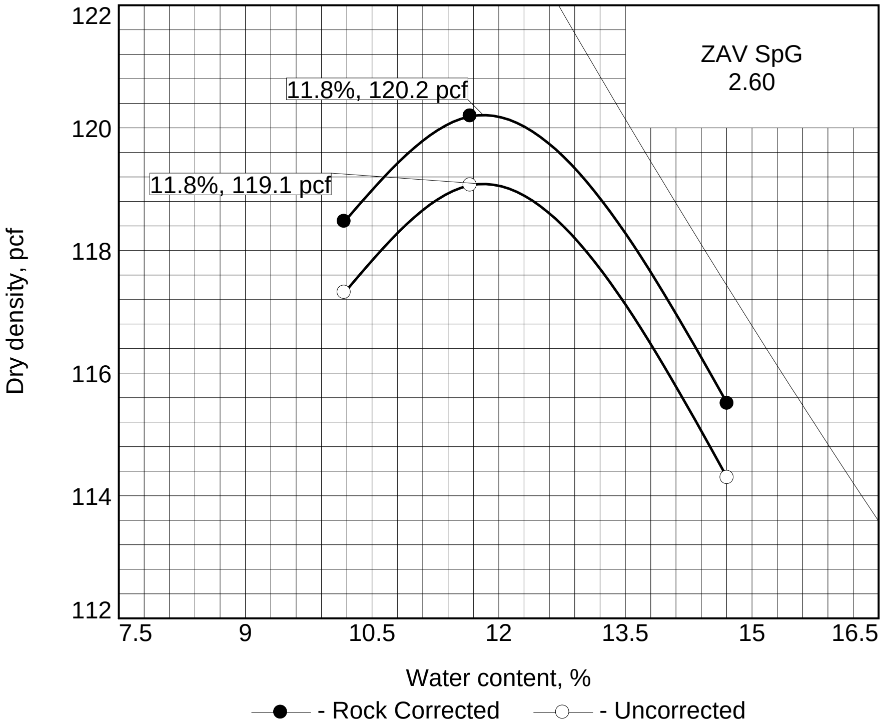
LABORATORY TESTING

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-6;
- Sieve analyses (ASTM D422), and the final test results are plotted as gradation curves on Figures B-7 thru B-10;
- Direct shear test (ASTM D3080). The test results are presented on Figures B-11 thru B-17.

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 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 **LL** **PI**
Sp.G. (ASTM D 854) 2.6
%>#4 3.8 **%<No.200** 15.7
USCS **AASHTO**
Date Sampled 2/23/18
Date Tested
Tested By

TESTING DATA

	1	2	3	4	5	6
WM + WS	5988.0	6044.0	6016.0			
WM	4034.0	4034.0	4034.0			
WW + T #1	485.0	515.2	487.1			
WD + T #1	446.2	467.9	433.4			
TARE #1	64.8	62.4	68.3			
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	10.2	11.7	14.7			
DRY DENSITY	118.5	120.2	115.5			

ROCK CORRECTED TEST RESULTS	UNCORRECTED
Maximum dry density = 120.2 pcf	119.1 pcf
Optimum moisture = 11.8 %	11.8 %

Material Description
Yellow brown, moist, silty sand with scattered gravel and cobbles to 4" in maximum dimension

Remarks:

Project No. 179 GS-16-F **Client:** INFRASTRUCTURE ENGINEERING
Project: SEWER AND STORM DRAIN GROUP 828

○ **Location:** TP-1 **Depth:** 1'-3' **Sample Number:** 1

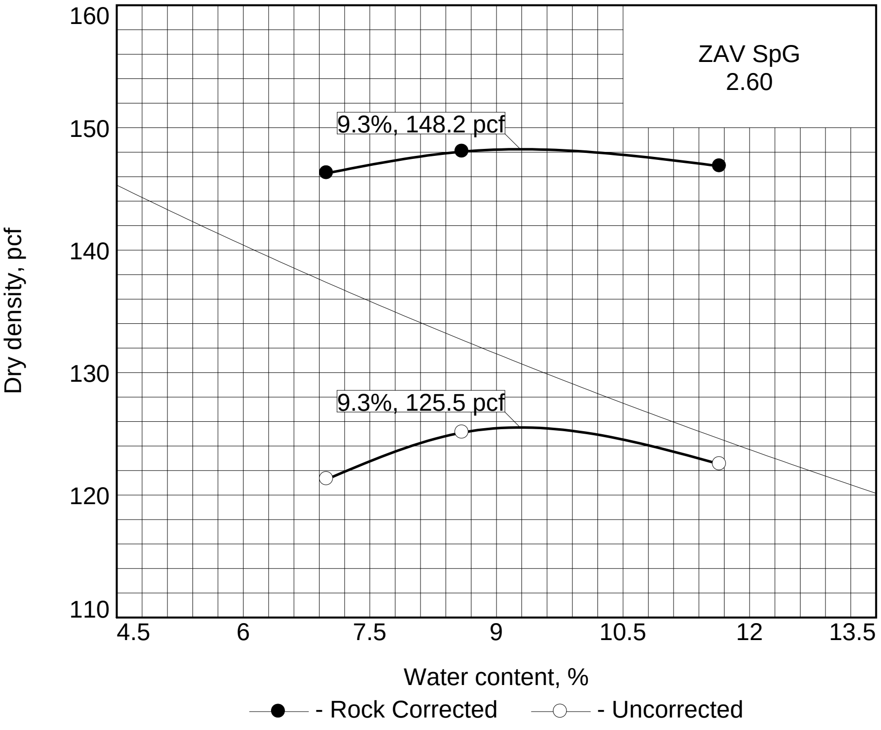
Allied Geotechnical Engineers, Inc.

Santee, CA

Checked by:
Title:

Figure B-1

COMPACTION TEST REPORT



Curve No.

Test Specification:
ASTM D 1557-91 Procedure A Modified
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 ______ **LL** ______ **PI** ______
Sp.G. (ASTM D 854) 2.6
%>#4 67.6 **%<No.200** 2.2
USCS GW **AASHTO** ______
Date Sampled ______
Date Tested ______
Tested By N. Barnes

TESTING DATA

	1	2	3	4	5	6
WM + WS	5996.0	6088.0	6102.0			
WM	4034.0	4034.0	4034.0			
WW + T #1	474.9	445.3	454.2			
WD + T #1	448.2	414.9	413.2			
TARE #1	66.3	61.3	61.2			
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	7.0	8.6	11.6			
DRY DENSITY	146.3	148.0	146.9			

ROCK CORRECTED TEST RESULTS	UNCORRECTED
Maximum dry density = 148.2 pcf	125.5 pcf
Optimum moisture = 9.3 %	9.3 %

Material Description
Dark yellow brown well graded gravel with sand

Remarks:

Project No. 179 GS-16-F **Client:** INFRASTRUCTURE ENGINEERING
Project: SEWER AND STORM DRAIN GROUP 828

○ **Location:** TP-3 **Depth:** 2'-2.5' **Sample Number:** 1

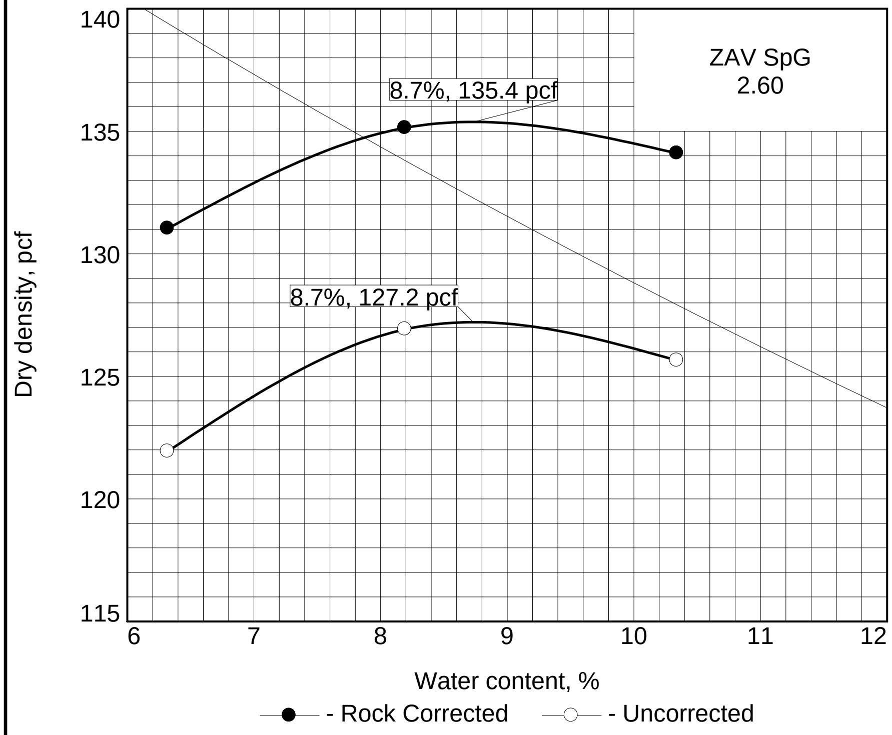
Allied Geotechnical Engineers, Inc.

Santee, CA

Checked by:
Title:

Figure B-2

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 **LL** **PI**
Sp.G. (ASTM D 854) 2.6
%>#4 27.9 **%<No.200** 16.7
USCS **AASHTO**
Date Sampled
Date Tested
Tested By N. Barnes

TESTING DATA

	1	2	3	4	5	6
WM + WS	5994.0	6110.0	6130.0			
WM	4034.0	4034.0	4034.0			
WW + T #1	498.6	424.2	432.7			
WD + T #1	472.9	397.3	398.0			
TARE #1	66.2	69.0	62.4			
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	6.3	8.2	10.3			
DRY DENSITY	131.0	135.1	134.1			

ROCK CORRECTED TEST RESULTS	UNCORRECTED
Maximum dry density = 135.4 pcf	127.2 pcf
Optimum moisture = 8.7 %	8.7 %

Material Description

Brown, damp, silty sand with abundant gravel

Remarks:

Project No. 179 GS-16-F **Client:** INFRASTRUCTURE ENGINEERING
Project: SEWER AND STORM DRAIN GROUP 828

○ **Location:** TP-5 **Depth:** 2'-3' **Sample Number:** 1

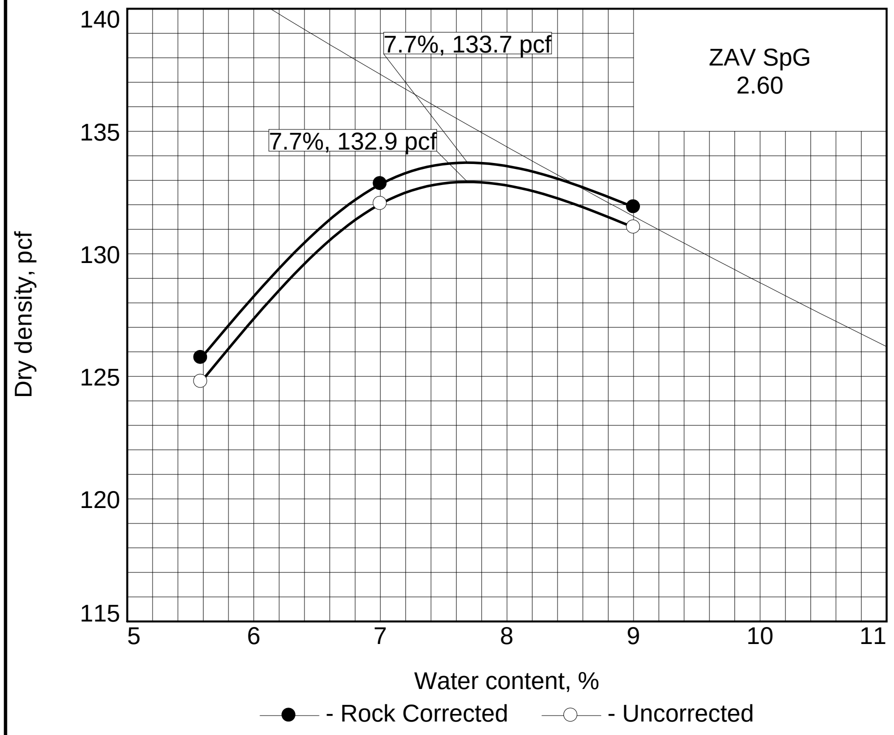
Allied Geotechnical Engineers, Inc.

Santee, CA

Checked by:
Title:

Figure B-3

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 8.7 **LL** **PI**

Sp.G. (ASTM D 854) 2.60

%>#4 3.7 **%<No.200** 39.8

USCS **AASHTO**

Date Sampled 4/17/18

Date Tested

Tested By

TESTING DATA

	1	2	3	4	5	6
WM + WS	6026.0	6170.0	6194.0			
WM	4034.0	4034.0	4034.0			
WW + T #1	430.0	465.1	422.1			
WD + T #1	410.5	438.8	392.3			
TARE #1	61.3	63.2	61.3			
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	5.6	7.0	9.0			
DRY DENSITY	125.8	132.8	131.9			

ROCK CORRECTED TEST RESULTS	UNCORRECTED
Maximum dry density = 133.7 pcf	132.9 pcf
Optimum moisture = 7.7 %	7.7 %

Material Description

Light yellow brown to pale olive, damp, micaceous silty fine-grained sand (SM) with fractured gravel and cobbles.

Remarks:

Project No. 179 GS-16-F **Client:** INFRASTRUCTURE ENGINEERING

Project: SEWER AND STORM DRAIN GROUP 828

○ **Source of Sample:** B-1 **Depth:** 25 **Sample Number:** 6

Allied Geotechnical Engineers, Inc.

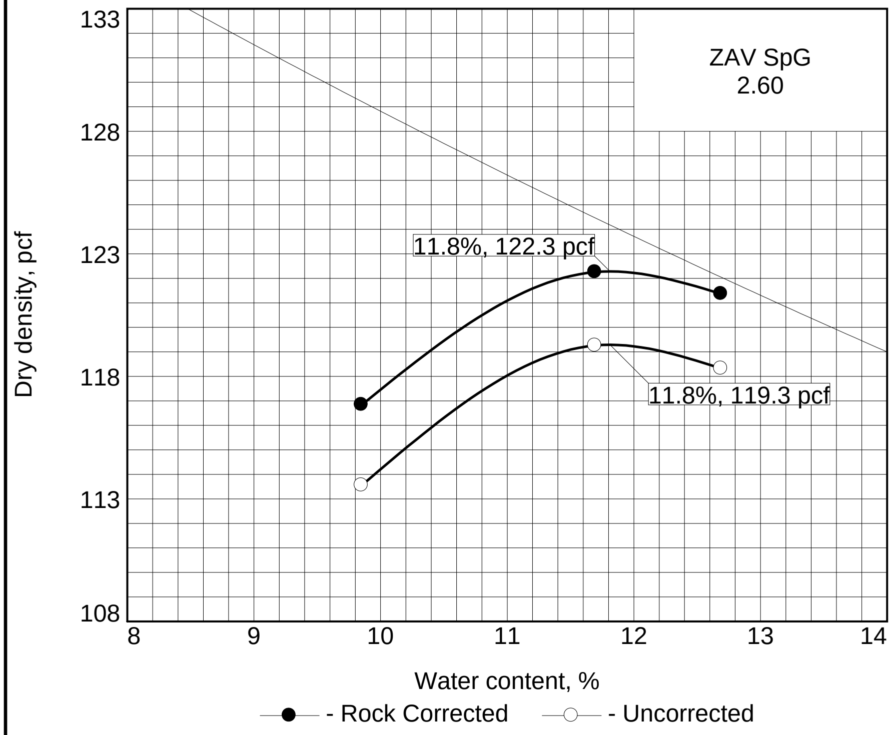
Santee, CA

Checked by:

Title:

Figure B-4

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 **LL** **PI**
Sp.G. (ASTM D 854) 2.6
%>#4 10.0 **%<No.200** 33.2
USCS **AASHTO**
Date Sampled
Date Tested
Tested By N. Barnes

TESTING DATA

	1	2	3	4	5	6
WM + WS	5920.0	6048.0	6050.0			
WM	4034.0	4034.0	4034.0			
WW + T #1	440.2	425.2	374.3			
WD + T #1	407.2	387.9	339.9			
TARE #1	72.2	68.9	68.8			
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	9.9	11.7	12.7			
DRY DENSITY	116.8	122.3	121.4			

ROCK CORRECTED TEST RESULTS	UNCORRECTED
Maximum dry density = 122.3 pcf	119.3 pcf
Optimum moisture = 11.8 %	11.8 %

Material Description
Reddish yellow, damp, silty sand (SM) with scattered to abundant sub-rounded gravel and cobbles

Remarks:

Project No. 179 GS-16-F **Client:** INFRASTRUCTURE ENGINEERING
Project: SEWER AND STORM DRAIN GROUP 828

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

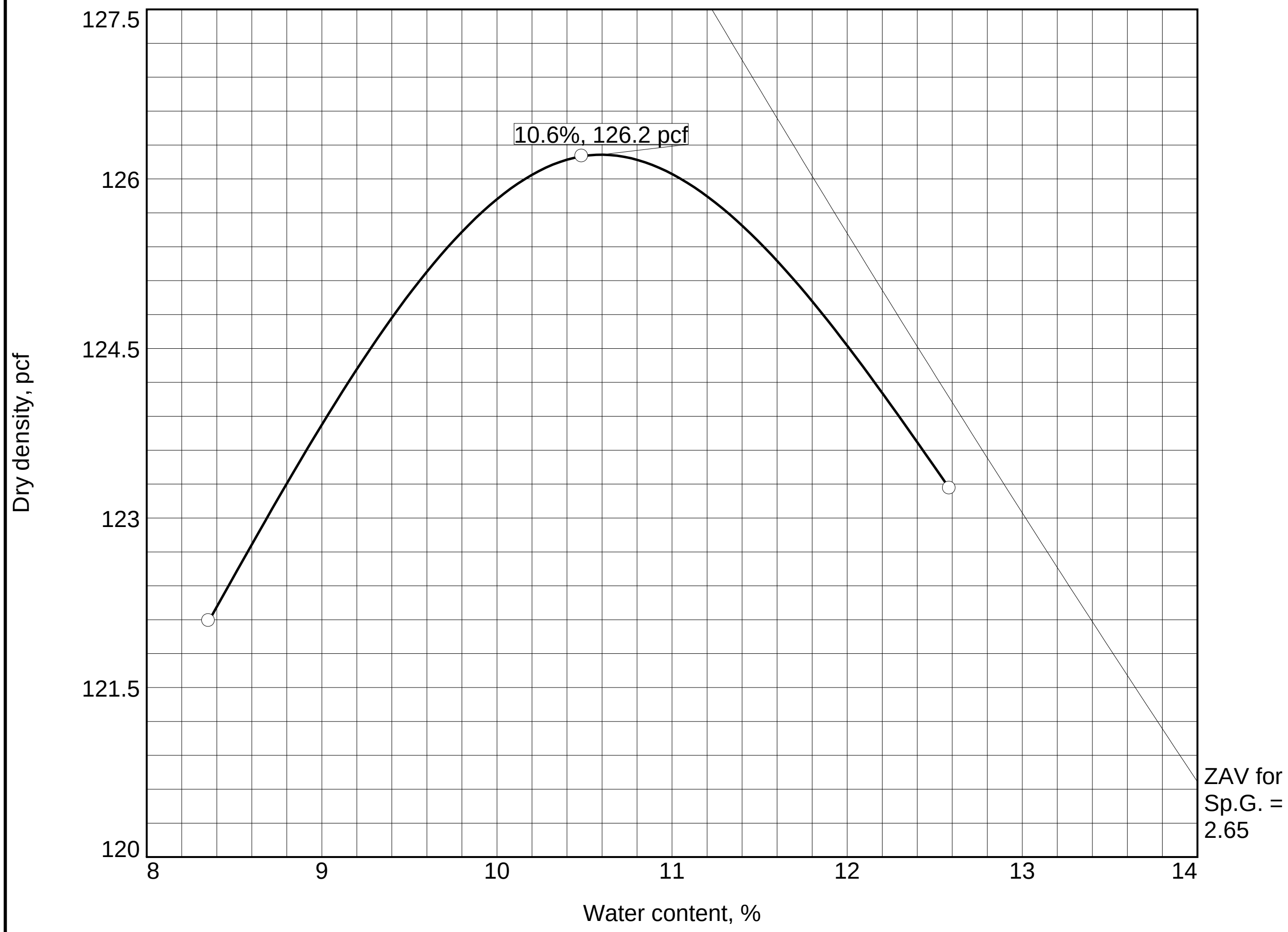
Allied Geotechnical Engineers, Inc.

Santee, CA

Checked by:
Title:

Figure B-5

COMPACTION TEST REPORT



Test specification: ASTM D 1557-91 Procedure A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
20	SM		11.4	2.65			0	

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 126.2 pcf Optimum moisture = 10.6 %	

Project No. 179 GS-16-F **Client:** INFRASTRUCTURE ENGINEERING
Project: SEWER AND STORM DRAIN GROUP 828

○**Source of Sample:** B-3 **Sample Number:** 5

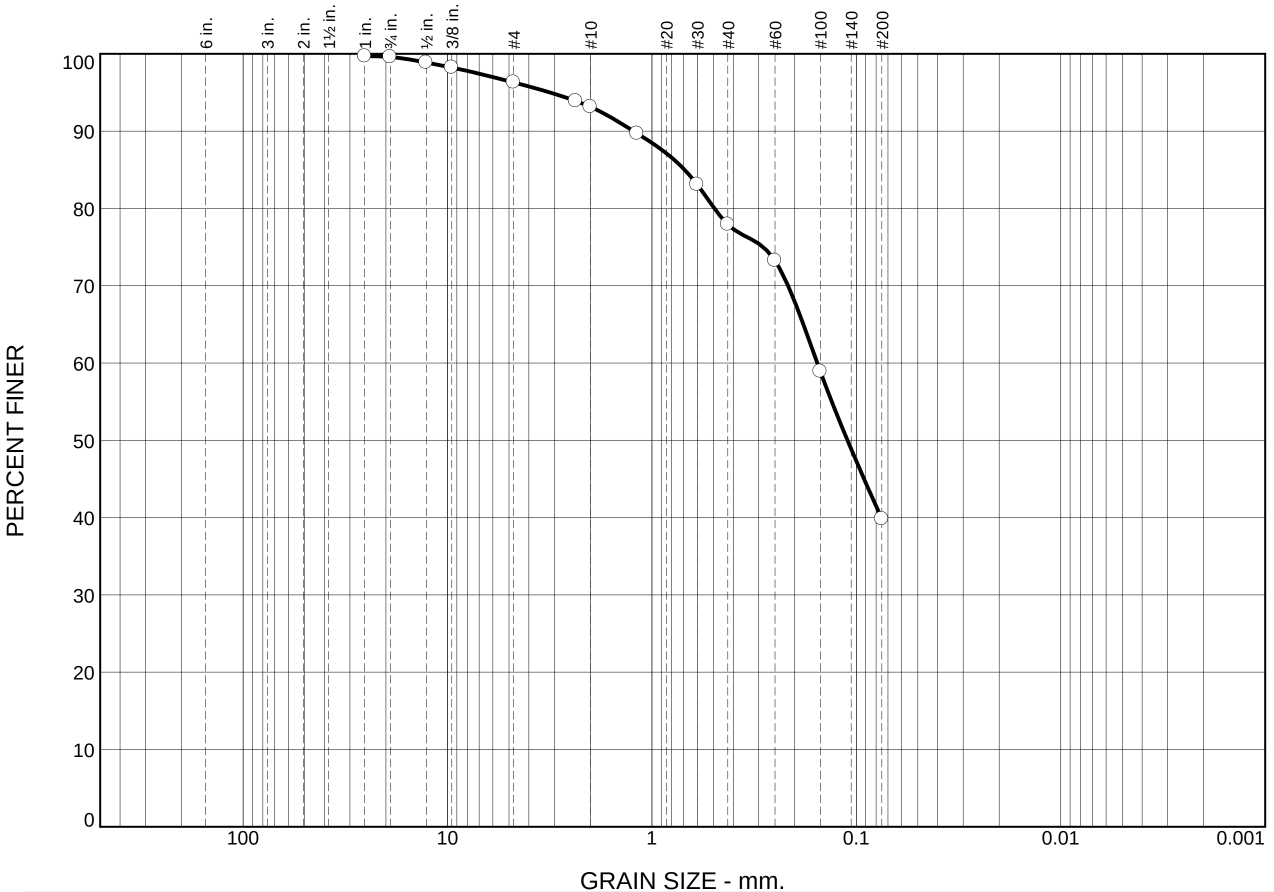
Remarks:

Allied Geotechnical Engineers, Inc.

Santee, CA

Figure B-6

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○			3.3	3.2	15.2	38.1	39.8	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-1	6	25	Light yellow brown to pale olive, damp, micaceous silty fine-grained sand (SM).	SM

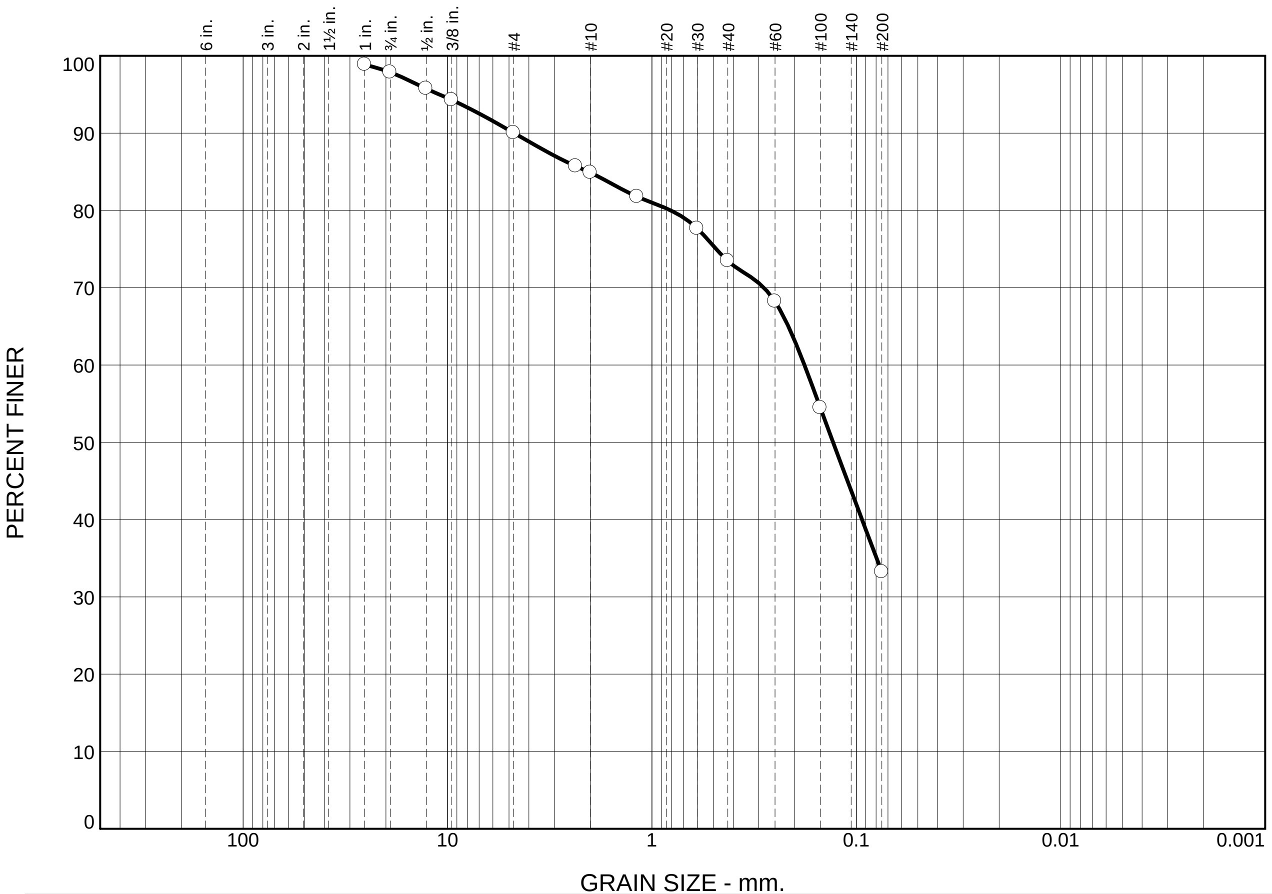
Allied Geotechnical Engineers, Inc.

Santee, CA

Client: INFRASTRUCTURE ENGINEERING CORPORATION
Project: SEWER AND STORM DRAIN GROUP 828

Project No.: 179 GS-16-F **Figure** B-7

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○			7.9	5.1	11.4	40.3	33.2	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-2		1	Reddish yellow, damp, silty sand (SM) with scattered to abundant	SM
				sub-rounded gravel and cobbles	

Allied Geotechnical Engineers, Inc.

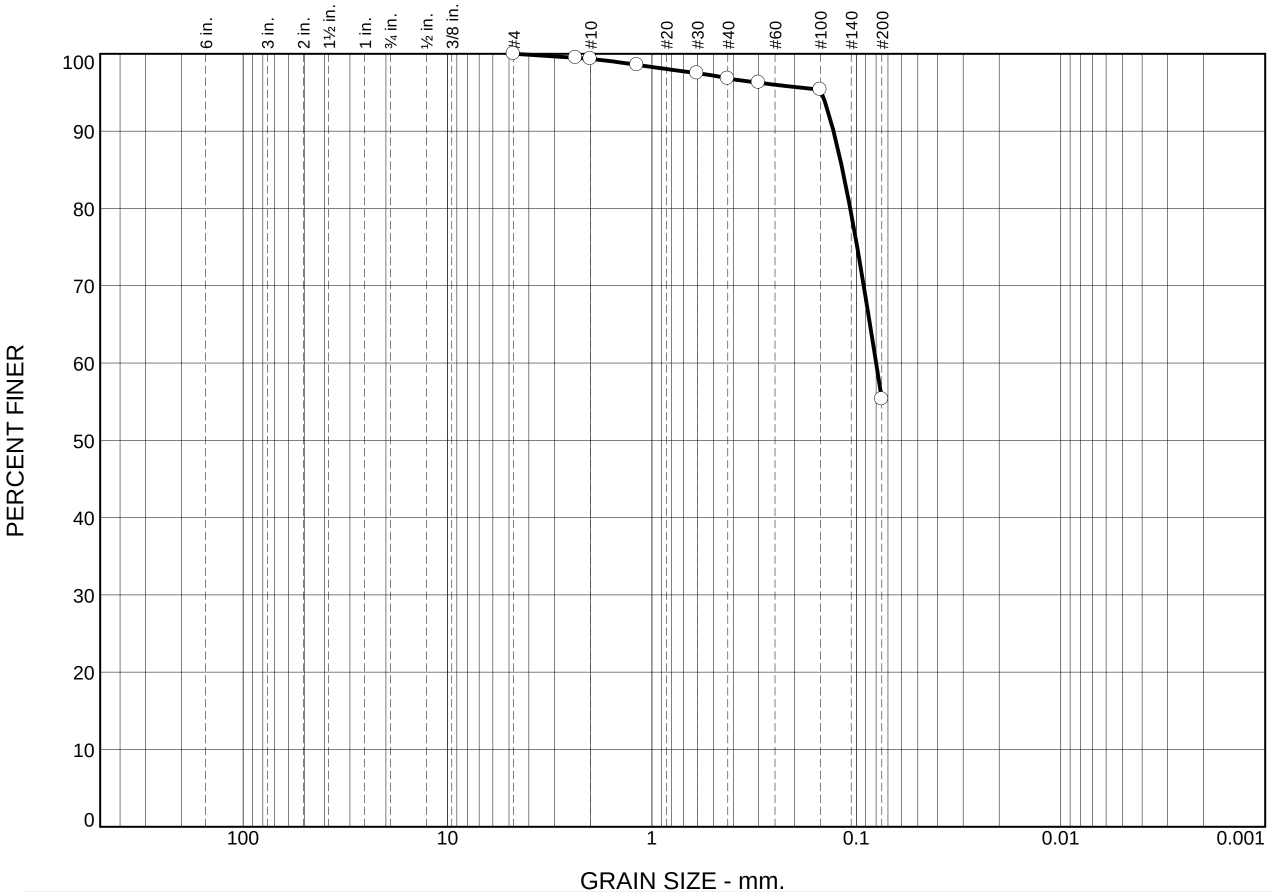
Santee, CA

Client: INFRASTRUCTURE ENGINEERING CORPORATION

Project: SEWER AND STORM DRAIN GROUP 828

Project No.: 179 GS-16-F **Figure** B-8

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	0.0	0.7	2.5	41.5	55.3	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-3	4	15	Light olive gray to olive, damp, stiff, sandy siltstone (ML).	ML

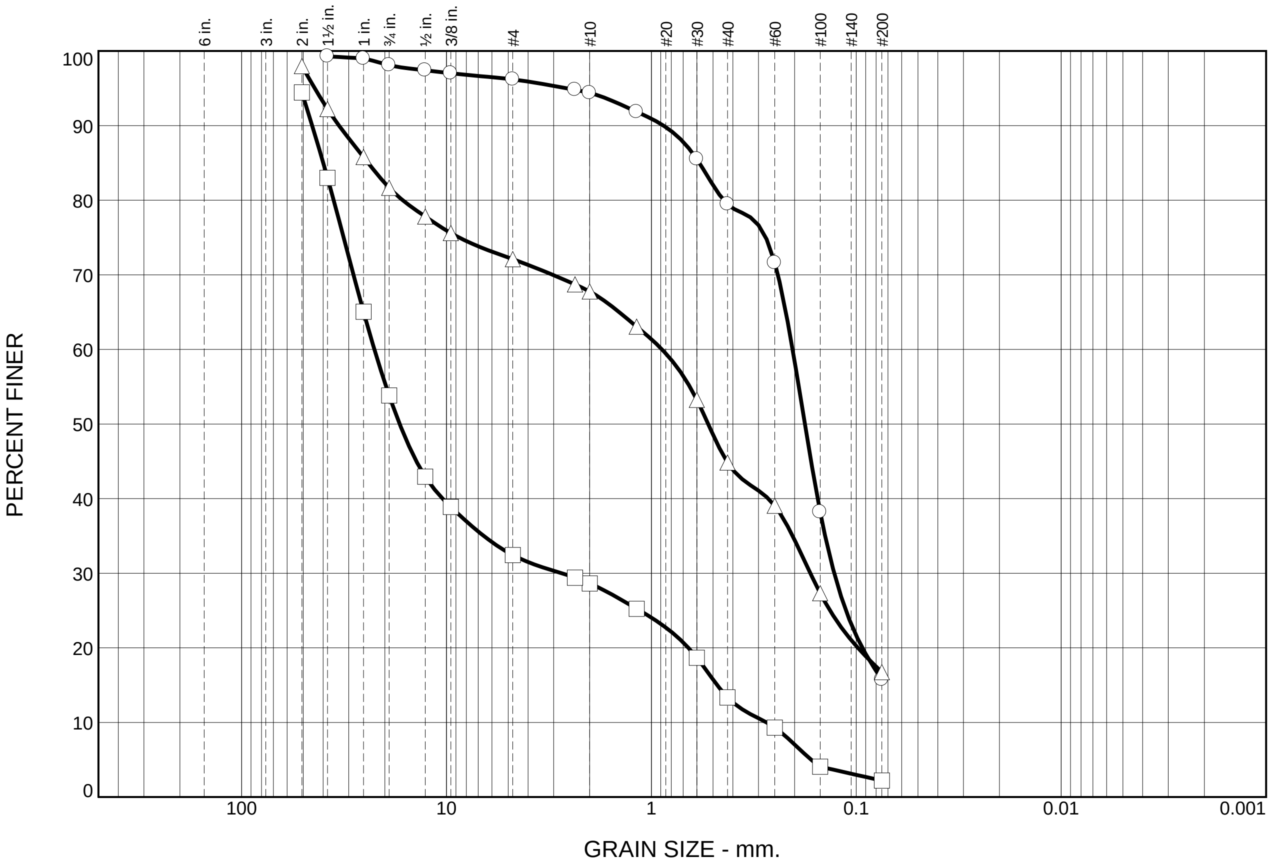
Allied Geotechnical Engineers, Inc.

Santee, CA

Client: INFRASTRUCTURE ENGINEERING CORPORATION
Project: SEWER AND STORM DRAIN GROUP 828

Project No.: 179 GS-16-F **Figure** B-9

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○			1.9	1.8	14.9	63.8	15.7	
□			21.4	3.8	15.2	11.2	2.2	
△			9.6	4.3	23.0	28.1	16.7	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○		1	1'-3'	Yellow brown, moist, silty sand (SM)	SM
□		1	2'-2.5'	Dark yellow brown well graded gravel with sand (GW)	GW
△		1	2'-3'	Brown, damp, silty sand with abundant gravel (SM)	SM

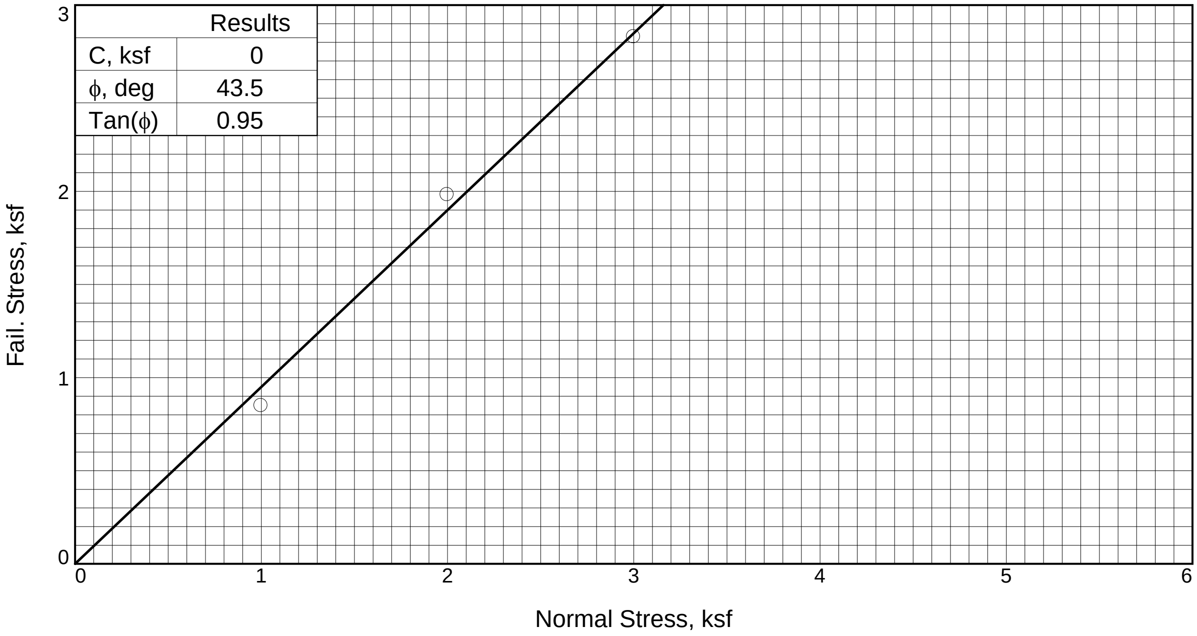
Allied Geotechnical Engineers, Inc.

Santee, CA

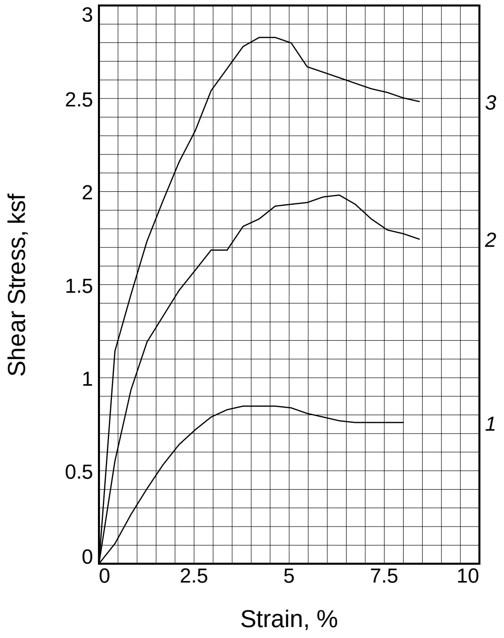
Client: INFRASTRUCTURE ENGINEERING CORPORATION
Project: SEWER AND STORM DRAIN GROUP 828

Project No.: 179 GS-16-F **Figure** B-10

Tested By: N. Barnes



	Results
C, ksf	0
ϕ, deg	43.5
Tan(ϕ)	0.95



	Sample No.	1	2	3
Initial	Water Content, %	12.4	12.5	12.9
	Dry Density, pcf	108.2	111.3	104.4
	Saturation, %	64.4	71.0	60.6
	Void Ratio	0.5004	0.4587	0.5548
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	18.9	18.9	20.3
	Dry Density, pcf	108.4	111.3	104.9
	Saturation, %	98.9	106.9	96.3
	Void Ratio	0.4974	0.4587	0.5470
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
Normal Stress, ksf		1.00	2.00	3.00
Fail. Stress, ksf		0.85	1.98	2.83
Strain, %		3.8	6.3	4.2
Ult. Stress, ksf				
Strain, %				
Strain rate, in./min.		0.006	0.006	0.006

Sample Type: Ring

Description: Light yellow brown to pale olive, damp, micaceous silty fine-grained sand (SM).

LL= NV **PI=** NP

Specific Gravity= 2.6

Remarks:

Figure B-11

Client: INFRASTRUCTURE ENGINEERING CORPORATION

Project: SEWER AND STORM DRAIN GROUP 828

Source of Sample: B-1 **Depth:** 25

Sample Number: 6

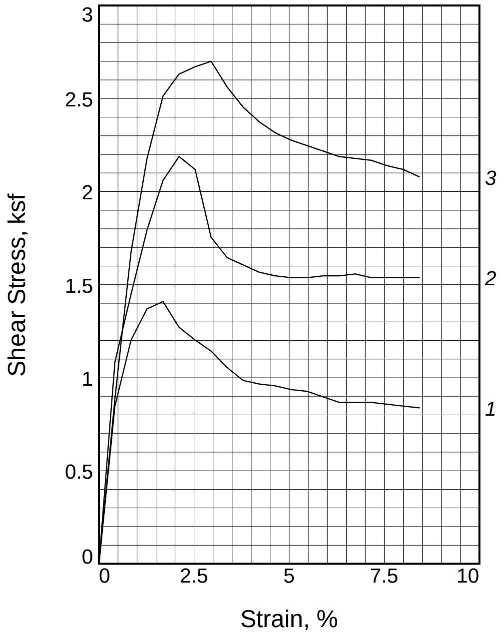
Proj. No.: 179 GS-16-F **Date Sampled:** 4/17/18

DIRECT SHEAR TEST REPORT

Allied Geotechnical Engineers, Inc.
Santee, CA

Tested By: N. Barnes

	Results
C, ksf	0.81
ϕ, deg	32.8
Tan(ϕ)	0.65



Sample No.		1	2	3
Initial	Water Content, %	21.7	19.2	22.1
	Dry Density, pcf	96.1	98.7	100.2
	Saturation, %	79.5	75.1	90.1
	Void Ratio	0.7223	0.6758	0.6513
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	27.4	24.7	25.5
	Dry Density, pcf	96.6	98.7	99.9
	Saturation, %	102.0	96.7	102.9
	Void Ratio	0.7120	0.6758	0.6563
	Diameter, in.	2.38	2.38	2.38
	Height, in.	0.99	1.00	1.00
Normal Stress, ksf		1.00	2.00	3.00
Fail. Stress, ksf		1.41	2.19	2.70
Strain, %		1.7	2.1	2.9
Ult. Stress, ksf				
Strain, %				
Strain rate, in./min.		0.006	0.007	0.006

Sample Type: Ring
Description:

Specific Gravity= 2.65
Remarks:

Figure B-12

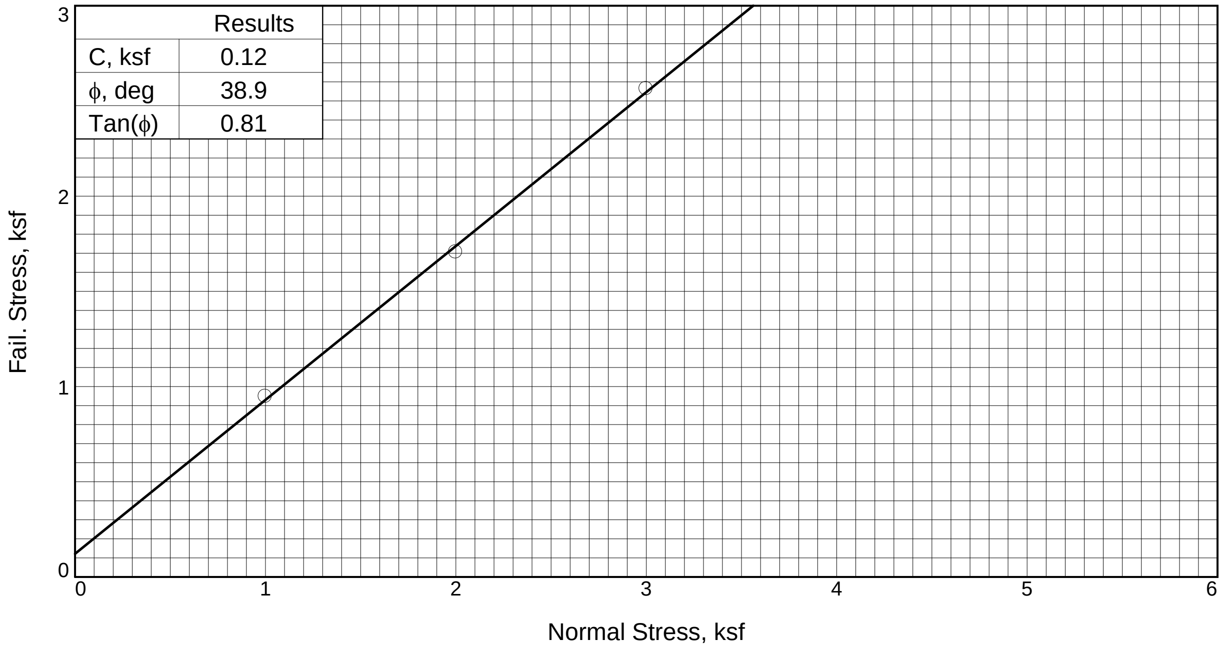
Client: INFRASTRUCTURE ENGINEERING CORPORATION

Project: SEWER AND STORM DRAIN GROUP 828

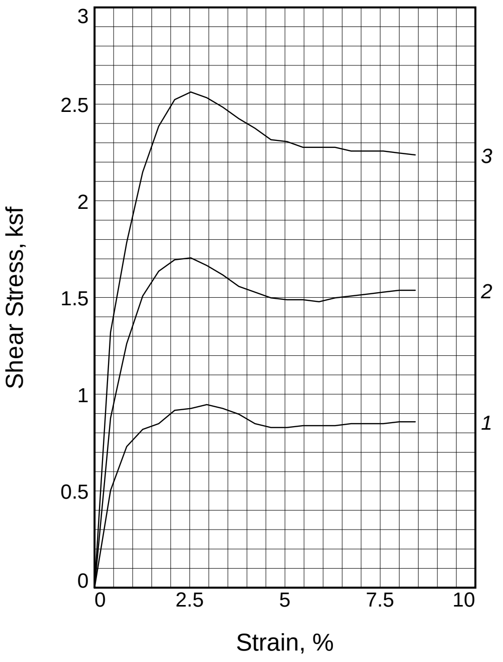
Source of Sample: B-3 **Depth:** 10
Sample Number: 3
Proj. No.: 179 GS-16-F **Date Sampled:** 7-30-18

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

Tested By: William Hayes **Checked By:** Sani Sutanto



	Results
C, ksf	0.12
ϕ, deg	38.9
Tan(ϕ)	0.81



Sample No.		1	2	3
Initial	Water Content, %	12.3	12.2	11.6
	Dry Density, pcf	107.3	107.4	108.6
	Saturation, %	62.6	61.9	60.8
	Void Ratio	0.5124	0.5112	0.4945
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	18.6	17.5	17.1
	Dry Density, pcf	107.1	107.5	108.7
	Saturation, %	93.8	89.4	90.2
	Void Ratio	0.5155	0.5097	0.4930
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
Normal Stress, ksf		1.00	2.00	3.00
Fail. Stress, ksf		0.95	1.70	2.56
Strain, %		2.9	2.5	2.5
Ult. Stress, ksf				
Strain, %				
Strain rate, in./min.		0.007	0.007	0.007

Sample Type: Remold ring

Description: Yellow brown, moist, silty sand (SM)

LL= NV

Specific Gravity= 2.6

Remarks:

Figure B-13

Client: INFRASTRUCTURE ENGINEERING CORPORATION

Project: SEWER AND STORM DRAIN GROUP 828

Location: TP-1

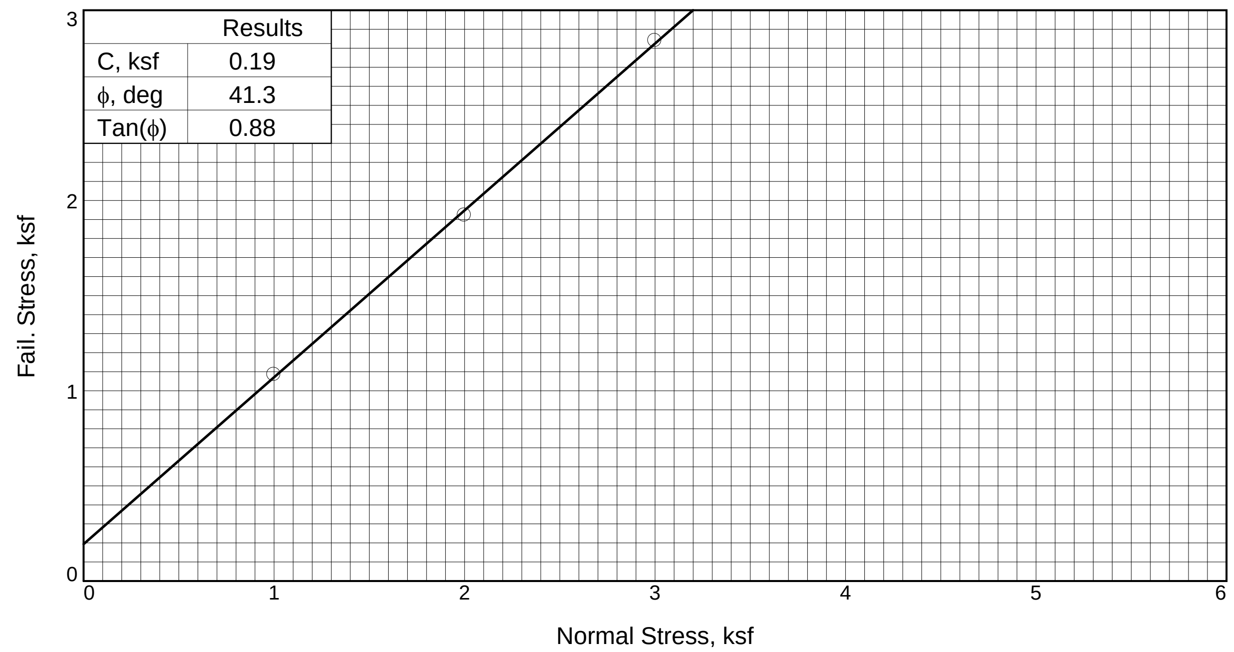
Sample Number: 1 **Depth:** 1'-3'

Proj. No.: 179 GS-16-F **Date Sampled:** 2/23/18

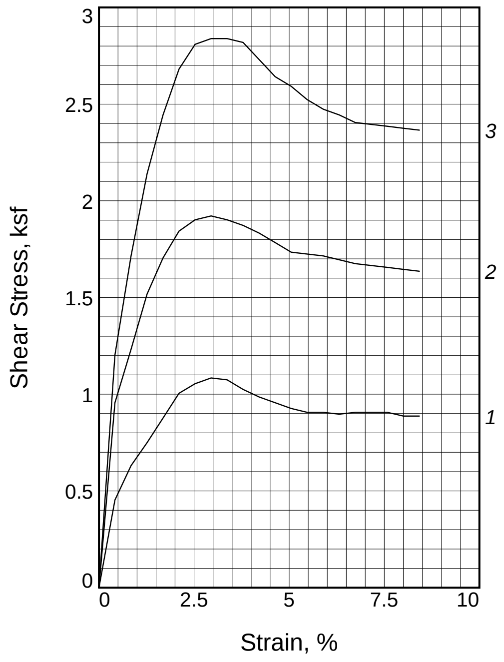
DIRECT SHEAR TEST REPORT

Allied Geotechnical Engineers, Inc.

Santee, CA



Results	
C, ksf	0.19
ϕ, deg	41.3
Tan(ϕ)	0.88



	Sample No.	1	2	3
Initial	Water Content, %	9.4	9.4	9.0
	Dry Density, pcf	114.4	114.5	115.0
	Saturation, %	58.3	58.5	56.7
	Void Ratio	0.4192	0.4171	0.4118
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	14.9	14.3	14.0
	Dry Density, pcf	114.5	115.9	116.1
	Saturation, %	92.7	92.7	91.5
	Void Ratio	0.4178	0.4001	0.3976
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	0.99	0.99
Normal Stress, ksf		1.00	2.00	3.00
Fail. Stress, ksf		1.08	1.92	2.84
Strain, %		2.9	2.9	2.9
Ult. Stress, ksf				
Strain, %				
Strain rate, in./min.		0.007	0.007	0.007

Sample Type: Remold ring

Description: Dark yellow brown well graded gravel with sand (GW)

LL= NV

Specific Gravity= 2.6

Remarks:

Figure B-14

Client: INFRASTRUCTURE ENGINEERING CORPORATION

Project: SEWER AND STORM DRAIN GROUP 828

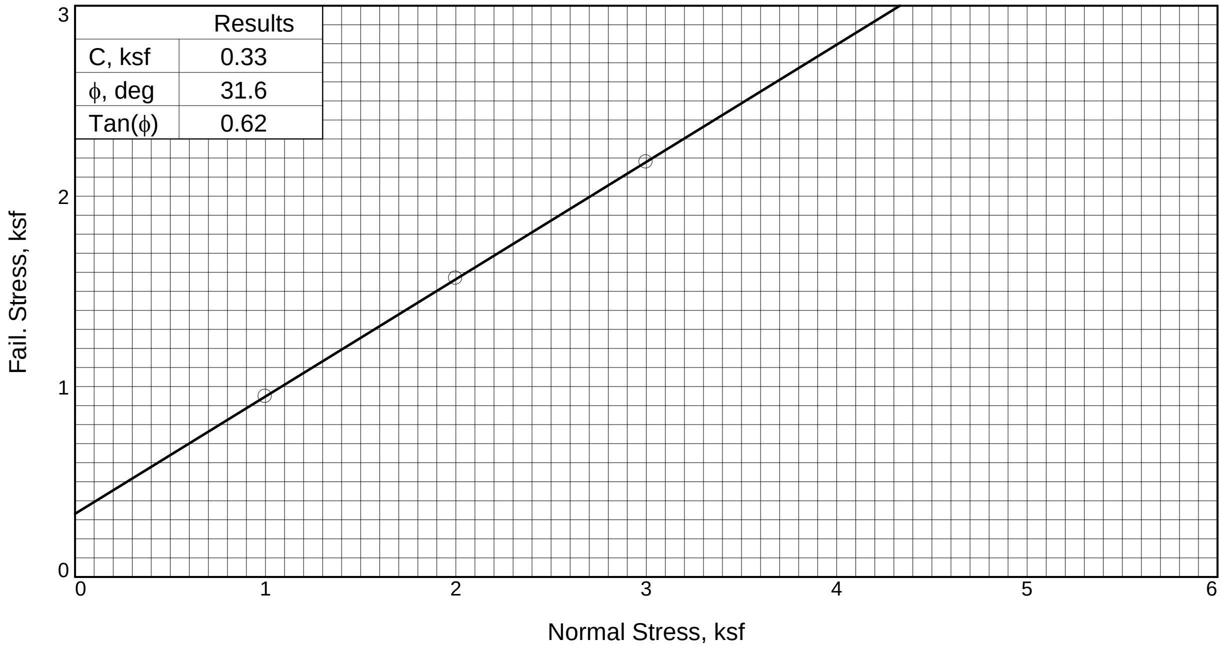
Location: TP-3

Sample Number: 1 **Depth:** 2'-2.5'

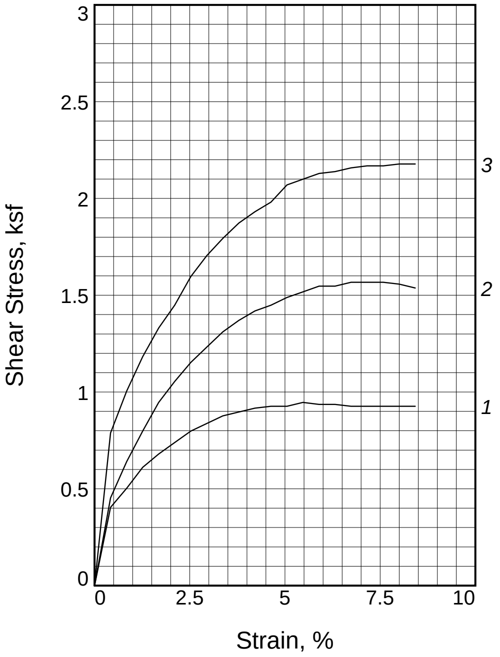
Proj. No.: 179 GS-16-F **Date Sampled:**

DIRECT SHEAR TEST REPORT

Allied Geotechnical Engineers, Inc.
Santee, CA



	Results
C, ksf	0.33
φ, deg	31.6
Tan(φ)	0.62



	Sample No.	1	2	3
Initial	Water Content, %	12.1	12.6	14.7
	Dry Density, pcf	94.2	93.8	94.0
	Saturation, %	43.3	45.0	52.7
	Void Ratio	0.7238	0.7301	0.7269
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	28.3	29.8	30.4
	Dry Density, pcf	94.3	93.2	93.9
	Saturation, %	101.9	104.4	108.4
	Void Ratio	0.7220	0.7422	0.7286
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.01	1.00
	Normal Stress, ksf	1.00	2.00	3.00
	Fail. Stress, ksf	0.95	1.57	2.18
	Strain, %	5.5	6.7	8.0
	Ult. Stress, ksf			
	Strain, %			
	Strain rate, in./min.	0.007	0.005	0.005

Sample Type: Ring

Description: Olive green, damp, dense, fine silty sandstone

Specific Gravity= 2.6

Remarks:

Figure B-15

Client: INFRASTRUCTURE ENGINEERING CORPORATION

Project: SEWER AND STORM DRAIN GROUP 828

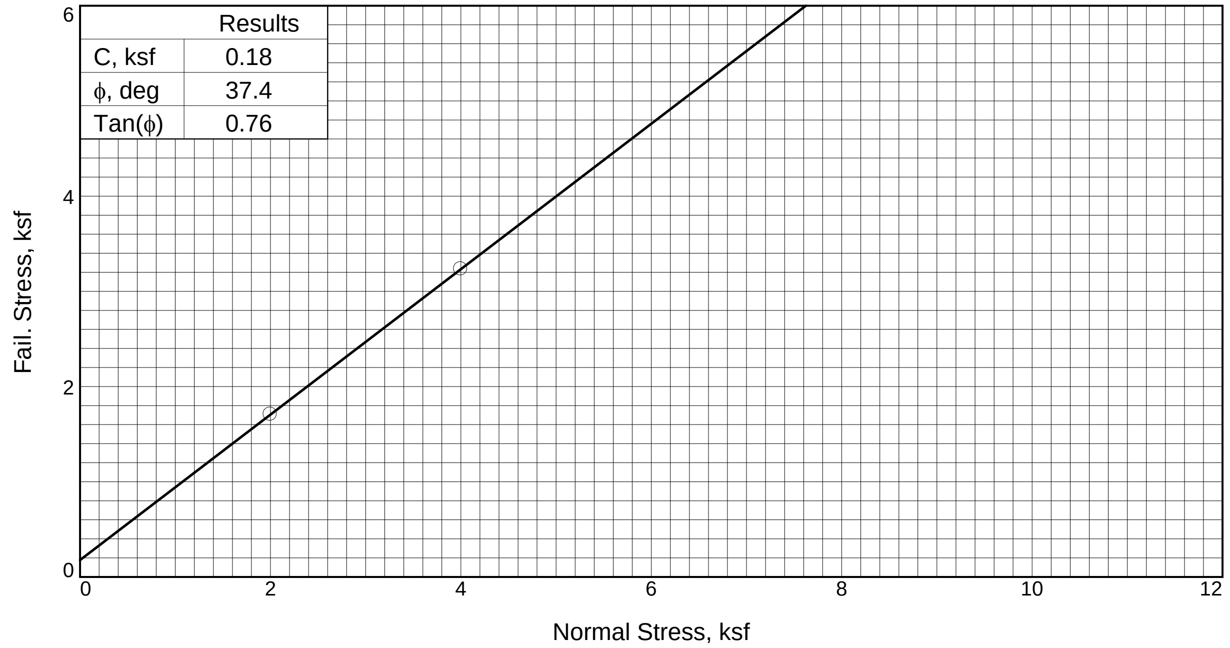
Location: TP-3

Sample Number: 3 **Depth:** 5.5'-6'

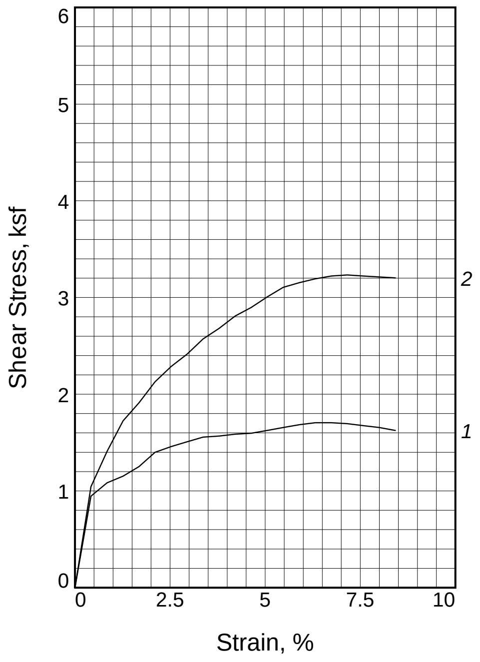
Proj. No.: 179 GS-16-F **Date Sampled:**

DIRECT SHEAR TEST REPORT

Allied Geotechnical Engineers, Inc.
Santee, CA



Results	
C, ksf	0.18
ϕ, deg	37.4
Tan(ϕ)	0.76



Sample No.		1	2
Initial	Water Content, %	9.9	10.4
	Dry Density, pcf	102.2	100.7
	Saturation, %	43.9	44.3
	Void Ratio	0.5875	0.6119
	Diameter, in.	2.38	2.38
	Height, in.	1.00	1.00
At Test	Water Content, %	20.1	21.9
	Dry Density, pcf	102.1	100.5
	Saturation, %	88.7	92.4
	Void Ratio	0.5891	0.6151
	Diameter, in.	2.38	2.38
	Height, in.	1.00	1.00
Normal Stress, ksf		2.00	4.00
Fail. Stress, ksf		1.70	3.23
Strain, %		6.3	7.2
Ult. Stress, ksf			
Strain, %			
Strain rate, in./min.		0.008	0.006

Sample Type: ring

Description: Dark brown, moist, fine to medium silty sand (SM)

Specific Gravity= 2.6

Remarks:

Figure B-16

Client: INFRASTRUCTURE ENGINEERING CORPORATION

Project: SEWER AND STORM DRAIN GROUP 828

Location: TP-4

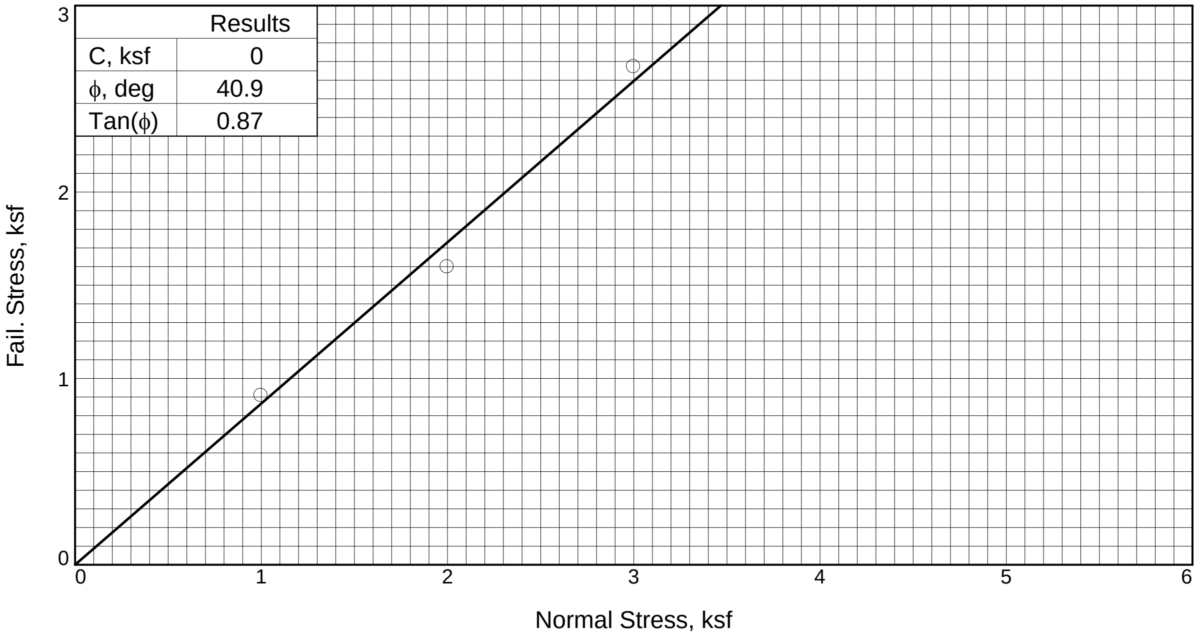
Sample Number: 3 **Depth:** 5'-5.5'

Proj. No.: 179 GS-16-F **Date Sampled:** 2/23/18

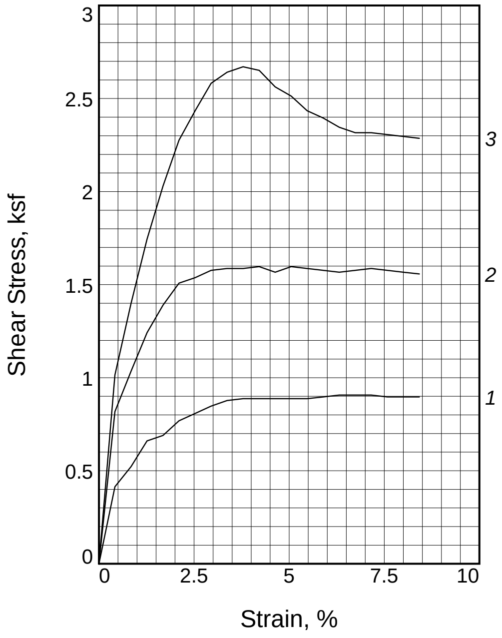
DIRECT SHEAR TEST REPORT

Allied Geotechnical Engineers, Inc.

Santee, CA



Results	
C, ksf	0
ϕ, deg	40.9
Tan(ϕ)	0.87



	Sample No.	1	2	3
Initial	Water Content, %	8.8	10.6	11.7
	Dry Density, pcf	105.9	107.4	105.1
	Saturation, %	43.2	53.7	55.9
	Void Ratio	0.5321	0.5112	0.5446
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	19.7	19.1	20.3
	Dry Density, pcf	106.0	107.3	104.8
	Saturation, %	96.7	97.0	96.1
	Void Ratio	0.5305	0.5127	0.5492
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
	Normal Stress, ksf	1.00	2.00	3.00
	Fail. Stress, ksf	0.91	1.60	2.67
	Strain, %	6.3	4.2	3.8
	Ult. Stress, ksf			
	Strain, %			
	Strain rate, in./min.	0.007	0.007	0.007

Sample Type: ring

Description: Yellow brown, damp, fine silty sandstone (SM)

Specific Gravity= 2.6

Remarks:

Figure B-17

Client: INFRASTRUCTURE ENGINEERING CORPORATION

Project: SEWER AND STORM DRAIN GROUP 828

Location: TP-6

Sample Number: 4 **Depth:** 5'-5.5'

Proj. No.: 179 GS-16-F **Date Sampled:** 2/22/18

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

Tested By: N. Barnes

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: April 11, 2018
Purchase Order Number: 179GS16-A
Sales Order Number: 39719
Account Number: ALLG
To:

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

Laboratory Number: SO6826-3 Customers Phone: 449-5900
Fax: 449-5902
Sample Designation:

One soil sample received on 04/10/18 at 9:00am, taken from Genesee Avenue Sidewalk Project marked as B-3 #1 @ 4'-5'.

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

pH 6.0

Water Added (ml)	Resistivity (ohm-cm)
10	3200
5	2300
5	1800
5	1400
5	1600
5	1800

14 years to perforation for a 16 gauge metal culvert.
18 years to perforation for a 14 gauge metal culvert.
25 years to perforation for a 12 gauge metal culvert.
32 years to perforation for a 10 gauge metal culvert.
39 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417	0.005% (48ppm)
Water Soluble Chloride Calif. Test 422	0.005% (53ppm)
Bicarbonate (as $CaCO_3$) (on a 1:3 water extraction)	6ppm

Laura Torres

Laura Torres
LT/ram

LABORATORY REPORT

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

Date: April 11, 2018
Purchase Order Number: 179GS16-A
Sales Order Number: 39719
Account Number: ALLG
To:

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

Laboratory Number: SO6826-2 Customers Phone: 449-5900
Fax: 449-5902
Sample Designation:

One soil sample received on 04/10/18 at 9:00am, taken from Genesee Avenue Sidewalk Project marked as B-2 #1 @ 4'-5'.

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

pH 8.6

Water Added (ml)	Resistivity (ohm-cm)
10	740
5	600
5	590
5	610
5	640

25 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.
57 years to perforation for a 10 gauge metal culvert.
69 years to perforation for a 8 gauge metal culvert.

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

Water Soluble Chloride Calif. Test 422 0.025% (250ppm)

Bicarbonate (as $CaCO_3$) 40ppm
(on a 1:3 water extraction)

Laura Torres

Laura Torres
LT/ram

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: April 11, 2018
Purchase Order Number: 179GS16-A
Sales Order Number: 39719
Account Number: ALLG
To:

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

Laboratory Number: SO6826-1 Customers Phone: 449-5900
Fax: 449-5902
Sample Designation:

One soil sample received on 04/10/18 at 9:00am,
taken from Genesee Avenue Sidewalk Project
marked as B-1 #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 8.7

Water Added (ml)	Resistivity (ohm-cm)
10	2000
5	1100
5	960
5	970
5	990

30 years to perforation for a 16 gauge metal culvert.
39 years to perforation for a 14 gauge metal culvert.
54 years to perforation for a 12 gauge metal culvert.
69 years to perforation for a 10 gauge metal culvert.
84 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417 0.011% (110ppm)

Water Soluble Chloride Calif. Test 422 0.003% (32ppm)

Bicarbonate (as $CaCO_3$) 70ppm
(on a 1:3 water extraction)

Laura Torres

Laura Torres
LT/ram

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: 179 GS 16-F
Sales Order Number: 41098
Account Number: ALLG

To:

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

Laboratory Number: SO6959 Customers Phone: 449-5900
Fax: 449-5902

Sample Designation:

One soil sample received on 08/02/18 at 9:00am, taken from Group Job#828 Project marked as B-3 #7 @ 23'-24'.

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

pH 8.9

Water Added (ml)	Resistivity (ohm-cm)
10	4100
5	2800
5	2500
5	2600
5	2700

45 years to perforation for a 16 gauge metal culvert.
58 years to perforation for a 14 gauge metal culvert.
80 years to perforation for a 12 gauge metal culvert.
102 years to perforation for a 10 gauge metal culvert.
125 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417	0.010% (100ppm)
Water Soluble Chloride Calif. Test 422	0.002% (21ppm)
Bicarbonate (as $CaCO_3$) (on a 1:3 water extraction)	66ppm

Laura Torres

Laura Torres
LT/ilv