Appendix E

Geotechnical Investigation Memorandum

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Memorandum

To	Casper van Keppel, Project Manager (AECOM)
Subject	**Geotechnical Investigation Memorandum for the San José Waste Water Treatment Plant Outfall Levee and Bridge**
From	John Tabor, P.E. (AECOM)
Date	July 30, 2018

INTRODUCTION

The following memorandum presents the results or our geotechnical review, field exploration, laboratory testing, and geotechnical recommendations for the San José Santa Clara Regional Wastewater Facility (RWF) Outfall Bridge and Levee Improvement Project. This memorandum follows our Progress Report and Preliminary Results of Field Exploration memorandum, dated June 21, 2018. This report includes the results of our laboratory soils tests on samples obtained from the boring which are used to develop geotechnical design recommendations to support the structural and civil engineering design.

1.0 Project Understanding and Site Conditions

Geologic and Geotechnical Site Conditions Review

AECOM has reviewed the available geologic and geotechnical data and relevant as-built structural drawings for the outfall levee and bridge.

We began by compiling and reviewing existing available geotechnical data. Data sources included:

- Dames & Moore, Soils and Foundation Investigation, Proposed Secondary Sewage Treatment Facilities, San Jose, California, for the City of San Jose, March 16, 1961.
- Dames & Moore, Soil and Foundation Investigation, Proposed Additions to San Jose-Santa Clara Water Pollution Control Plant, San Jose, California, for the City of San Jose, May 29, 1969.
- Geo/Resource Consultants, Inc., Geotechnical Investigation, Main Haul Road & Outfall Road Improvement Project, San Jose/Santa Clara Water Pollution Control Plant San Jose, California, October, 1993.

The levee upon which the boring location lies was reportedly built in the 1930’s as part of the Shoreline Improvement Project and is currently approximately 14 feet deep over the native ground. We understand the project involves development of alternatives for replacement or rehabilitation of existing equipment and structures including the existing 65 foot long pedestrian outfall bridge, Sulfur Dioxide (SO_2) Building, and transformer pad. Based on the available engineering drawings, the concrete weir and bridge, and the SO_2 Building are supported on pile foundations. According to the

1973 as-built drawings, the SO_2 Building is an approximate 9x9 foot wide reinforced concrete structure supported on eight piles at the perimeter of the building. Settlement of the east outfall channel levee has resulted in the development of a void space approximately 8 to-12 inches high between the ground and bottom of the SO_2 Building slab. Soil erosion downslope of the adjacent transformer pad has resulted in some tilting of the transformer slab foundation.

The project will provide increased flood protection for the SO_2 Building and transformer pad by raising the elevation of the surrounding grade. The existing concrete transformer pad is planned be raised some two feet and replaced with a new subbase and concrete pad. We understand the void space beneath the SO_2 Building will be infilled with cement grout and the grade raised around the building to provide positive drainage away from the building.

Geotechnical Design Considerations

The new transformer will involve the demolition of the existing concrete pad and reconstruction of a new mat-slab foundation. Subgrade preparation will involve the removal of existing fill materials and replacement with compacted structural fill subbase to accommodate the new equipment loads and provide uniform bearing for the new transformer pad as described further in the Recommendations section below.

The foundation for a new bridge designed to span the outfall channel will likely require deep foundations such as drilled piers to reduce potential seismic induced liquefaction and consolidation settlements.

2.0 Geotechnical Field Exploration

AECOM performed a subsurface investigation consisting of one (1) geotechnical boring designated as B-1, drilled at the project site on May 23th, 2018 to a depth of 76.5 feet. The boring was drilled on the levee adjacent to the east bridge abutment as presented in Figure 1.

Pitcher Drilling of East Palo Alto, CA, drilled the soil boring using a truck-mounted Failing 1500 drill rig. The exploratory boring was advanced using hand-auger methods from 0.0 to 5.0-feet below the ground surface (bgs). The boring was advanced further using a solid stem auger from 6.0 to 9.0 feet bgs. The remainder of the boring was drilled using rotary wash drilling methods. Drilling was conducted in accordance to ASTM standards (ASTM D5783).

An AECOM engineer representative performed oversight of the drilling, logged, and visually classified the soils encountered during drilling in accordance with the Unified Soil Classification System.

Three types of samplers were used for this study:

- Modified California (MC) Sampler – 2.5-inch I.D., 3.0-inch O.D., split-barrel equipped with brass tube liners.
- Standard Penetration Test Sampler split-barrel (SPT) – 1.4-inch I.D., 2-inch O.D., 24-inch long, split barrel, with a 1.4-in I.D. cutting shoe.
- Shelby Tube Sampler – 2.875-inch I.D., 3-inch O.D., 30-inch long thin-walled sampler

After the borehole was drilled to the specified depth, the sampler mounted on the drill rods was lowered to the bottom, seated, and then driven into the soil with a hammer to retrieve a MC or SPT sample, or pushed by the rig (Shelby Tube). The SPT and MC samplers were driven 18 inches or to refusal (50 blows for 6 inches) into the material at the bottom of the borehole using a 140-pound automatic hammer with a free fall of 30 inches for each blow, and the Shelby was pushed 30 inches and the gauge pressure in pounds per square inch (psi) was recorded. The number of hammer blows required to advance the sampler each of the three successive 6-inch increments was counted in the field. The number of blows required to advance the sampler the last 12 inches was recorded as the penetration resistance (blows-per-foot). These blowcounts were used to evaluate density, soil strength and consistency of the soils and to evaluate the liquefaction hazard at the site. The MC samplers were generally used to obtain drive samples in clayey material. The SPT drive sampler was generally used to sample granular materials. The Shelby Tube was used for soft, saturated fine-grained material.

After completion of the drilling and sampling, the boring was tremie backfilled with neat cement grout and inspected in accordance with the Santa Clara Valley Water District regulations. The grouting of the boring was observed by a represented of the Santa Clara Valley Water District Well Inspection Department. The boring log and key to log is presented in Appendix A.

The MC samples were collected and sealed on both ends with plastic caps. The SPT samples were placed in 1-gallon plastic bags and sealed. The Shelby tubes were capped at both ends and sealed with electrical tape; samples were stored upright until laboratory testing. All samples were carefully labeled. The samples were transported to the Inspection Services Inc. Laboratory in Berkeley, California for further examination and testing. The log of the test boring was prepared based on the soil classification made in the field and verified by the laboratory index test results.

3.0 Geotechnical Laboratory Tests

Laboratory tests were performed on soil samples recovered from the field exploration to evaluate their geotechnical properties. The geotechnical laboratory tests results are provided in Appendix B and shown on the boring log at the appropriate depths. The following soil tests were performed by Inspection Services, Inc. (ISI) of Berkeley, California:

- Four, Sieve Analysis (ASTM Test Method D6913)
- Two, Passing #200 Sieve (ASTM D1140)
- One, Atterberg limits (ASTM Test Method D4318)
- Three, Unconsolidated-Undrained triaxial (UU) (ASTM Test Method D2850)
- Three, Moisture and Density, and
- One, Consolidation Test (ASTM D2435)

4.0 Regional Geology and Seismicity

4.1 Regional Geology

The RWF site is located within the within the geologically complex region of the Coast Ranges geomorphic province of California. This region is characterized by northwest-trending ridges and valleys that generally are parallel to major geologic structures, such as the San Andreas and Hayward fault systems. The project site is in an area mapped as having a moderate potential for liquefaction as shown on the US Geological Survey Liquefaction Hazard Map (2005).

4.2 Site Geotechnical Conditions and Groundwater

The boring encountered levee fill materials from the surface to approximately 10 feet below ground surface (bgs) consisting of layered soft-to-medium dense lean clays and poorly graded gravels with various amounts of sand. Concrete gravel was encountered toward the bottom of the levee at approximately 10 to 14-feet bgs. Beneath the levee fill, Quaternary Young Bay Clay (Bay Mud) was encountered from 14 to 31-feet bgs consisting of very soft, wet, organic fat (highly plastic) clay. The Bay Mud overlies 13 feet of Pleistocene Old Bay Clay from 31 to 44 feet bgs characterized by very stiff sandy lean clay and very loose clayey sand. Loose to medium dense poorly graded sand was encountered from 44.0 to 60.0 feet bgs. Dense-to-very dense poorly graded gravel with varying amounts of clay and sand was encountered below the poorly graded sand to the bottom of boring at 76.5 feet bgs. Similar geologic conditions were encountered in a boring on a levee road approximately 320 feet northwest of B-1 (Boring 4; Dames and Moore, 1961) as shown in Figure 1.

Groundwater was encountered at approximately 7.0 feet below grade during drilling. The groundwater depth will vary with seasonal rain and the channel tides.

4.3 Seismotectonic Setting

The project area is located in a portion of the Coast Ranges that is tectonically and seismically influenced by several major faults, with twenty one (21) known active to potentially active faults that lie within 31 mi (50 km) of the site (see Figure 2). The tectonic setting of the Coast Ranges is influenced by plate boundary interaction between the Pacific and North American lithospheric plates. This interaction occurs along a broad belt of northwest- trending right-lateral strike slip faults. The closest known active fault zone to the project area is the Hayward fault zone located approximately 3.5 miles (6 km) to the east. During the life of the levee and building, it is probable that at least one moderate to severe earthquake will cause ground shaking in the project area. Table 1 lists the faults and their distances from the project site, with fault length, slip rate, and maximum earthquake magnitude (Mmax) estimates.

In general, earthquakes occur as a result of movement along faults. For the purpose of activity classification, faults are generally grouped into the following categories:

- Active: Holocene - displacement has occurred within the last 10,000 to 11,000 years.
- Potentially Active: Late Quaternary - displacement has occurred within the last 700,000 years, but evidence of Holocene activity is lacking.
- Potentially Active: Evidence of Quaternary displacement within the last 1.6 million years, but evidence of Holocene activity is lacking.
- Inactive: Pre-quaternary - no recognized evidence of displacement in the last 1.6 million years.

Generally, faults with Holocene movement are considered to be “active” while faults with late Quaternary to Quaternary movement are considered to be “potentially active”.

Table 1. Major Faults in the Project Vicinity

Fault	Distance from Project Site (mi)	Length (mi)	Slip rate (mm/yr)	Mmax
Silver Creek	0.1	30.0	0.1	6.9
Hayward (north+south)	3.5	66.9	9.0	7.3
San Jose	5.1	28.0	0.1	6.8
Calaveras (north+central)	8.1	70.1	15.0	7.25
Monte Vista-Shannon	9.9	37.3	0.6	7.1
San Andreas	14.0	288.0	24.0	7.9
Greenville (north+south)	22.4	49.4	3.0	7.2

5.0 Seismic Design Parameters

The seismic design of the outfall bridge and building improvements shall be performed in accordance with CBC 2016 and the provisions of ASCE 7-10 with 2013 errata. For the seismic design, we recommend using a Site Class E with site coefficient values F_a and F_v of 0.9 and 2.4, respectively. Table 2 presents the spectral acceleration parameters for the project.

Table 2. Spectral Acceleration Response Parameters

Seismic Parameter	Value
Site Class	E
F_a	0.9
F_v	2.4
S_S (g)	1.50 g
S_1 (g)	0.60 g
S_{MS} (g)	1.35 g
S_{M1} (g)	1.44 g
S_{DS} (g)	0.90 g
S_{D1} (g)	0.96 g
PGA_M (g)	0.55 g

Notes:
S_S = mapped Maximum Considered Earthquake (MCE), spectral response acceleration parameter at short periods.
S_1 = mapped MCE spectral response acceleration parameter at a period of 1 second(s).
S_{MS} = F_a x S_s, the MCE spectral response acceleration parameter at short periods adjusted or site class effects.
S_{M1} = F_v x S_1, the MCE spectral response acceleration parameter at a period of 1s adjusted for site class effects.
S_{DS} = 2/3 x S_{MS}, design spectral response acceleration parameter at short periods.
S_{D1} = 2/3 x S_{M1}, design spectral response acceleration parameter at 1s.

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6.0 Liquefaction and Lateral Spreading

Liquefaction is a phenomenon whereby soil deposits temporarily lose shear strength and collapse. This condition is caused by cyclic loading during earthquake shaking that generates high pore water pressures within the soil deposits. The soil type most susceptible to liquefaction is loose, cohesionless, granular soil below the water table and within about 50 feet of the ground surface. Liquefaction can result in a loss of foundation support and settlement of overlying structures, ground subsidence and translation due to lateral spreading, lurch cracking, and differential settlement of affected deposits.

The levee is comprised of layers of lean clay and gravel with variable amounts of sand and fine sands and silt. Based upon the subsurface information from boring B-1, the poorly graded levee gravel from 10 to14-feet bgs is considered to be moderately-to-highly liquefiable during a major earthquake. The granular soil below the Old Bay Clay from 44 to 60-feet consists of variable density silty sand and sandy silt and is considered to have a high potential for liquefaction during a major earthquake.

We performed liquefaction triggering analyses based on the information obtained from boring B-1 using methodology by Boulanger & Idriss (2014). In accordance with the provisions of ASCE 7-10 with 2013 errata and the United States Geologic Survey (USGS) seismic design maps, we used a Peak Ground Acceleration (PGA) of 0.49g and an earthquake magnitude of 7.0 in the analyses.

Based on our liquefaction analyses, we estimate the free field volumetric settlements due to liquefaction to be on the order of 6 to 9 inches. Liquefaction due to strong ground shaking will impact the stability of existing levee resulting in deformations including of slope failure or sloughing, differential settlement, and/or lurch cracking.

7.0 Elastic and Consolidation Settlement

The long term consolidation settlement of the Young Bay Clay and other compressible clay layers within the levee itself that has occurred over the course of its history are estimated to be on the order of 4 to 5-feet. Assuming the construction of the levee dates back some 50 to 60 years, we estimate additional continued settlement without added loading to be less than 1-inch. The consolidation parameters for the calculations were developed from the consolidation test results and typical soil parameters for Young Bay Clay.

Although we are not aware of the new transformer loads at this time, immediate (elastic) and consolidation settlements associated with the placement of the proposed new concrete transformer pad placed over a new raised aggregate base pad will be on the order of 2 to 3-inches. In addition, we estimate that the infilling of the void beneath the SO_2 Building and raising the grade surrounding the building will result in some local additional long term consolidation settlement.

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8.0 Foundation Design and Recommendations

Various foundation alternatives including isolated shallow foundations as well as deep foundations such as drilled piers and driven piles were considered to support a proposed new bridge structure. However, the shallow foundation alternative was eliminated from evaluation considering the potential settlement expected due to the consolidation settlement of the levee clay and Young Bay Clay at the site should surface loads be applied. As part of the deep foundation design, we evaluated 24-inch diameter casted-in-place reinforced drilled piers as described below. Note that the axial and lateral piles capacities of deep foundation alternatives can be increased by increasing the diameter of the piers.

Ultimate Axial Pier Capacity

In our design, we considered only the frictional soil resistance to calculate the axial pier capacity. Downdrag forces are expected to be generated when the soil around the pier undergoes settlement due to the liquefaction settlement of the liquefiable zones identified in our analysis. These downdrag forces were included as negative loads on the piers. We also assumed that the center-to-center pier spacing is at a minimum of three pier diameters, and the top of the pier is at the new grade level. For a new fully spanning bridge we anticipate two piers and pier cap at each side of the outfall channel. We recommend a minimum pier depth of 80 feet from the top of levee, assuming a ground surface elevation of 10 feet (NAVD 88) at the surface, corresponding to a tip elevation of minus (-) 70 feet, in order to embedment the bottoms of piers into the medium-to-very dense granular materials for stability.

Axial capacity of a 24-inch diameter drilled piers was calculated using methods recommended by FHWA-NHI-10-016. The calculated ultimate axial capacity is presented in Figure 3. Downdrag due to consolidation settlement was not included in the analysis given our understanding of the age of the levee. We therefore anticipate minimal continued settlement due to long term consolidation to impact pier capacities. Note that the downdrag forces due to liquefaction induced settlement are shown as negative loads in Figures 3.

9.0 Site Preparation and New Fill Placement

The proposed construction locations, particularly at the new transformer pad and surrounding area, should be cleared of all obstructions including buried utilities, old foundations, concrete slabs, and asphalt-concrete pavement. Voids resulting from the removal of all obstructions should be backfilled and compacted in accordance with the guidelines provided below, or backfilled with an approved controlled density backfill material (slurry cement backfill). Areas of softer soils should be over excavated and backfilled in accordance with the guidelines below. We recommend that all removal of underground obstructions and deleterious materials and backfilling of resulting voids be performed under the observation of the geotechnical representative during construction.

The existing soil material beneath the new reinforced concrete pad should be removed a minimum of 12 inches deep and minimum of 2 feet beyond the perimeter of the concrete pad. The pad should be located a minimum of 3 feet from the top of levee slope. The subgrade to receive fill materials should be scarified a minimum of 6 inches in depth and compacted to a minimum 95 percent relative

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compaction in accordance with ASTM D1557 prior to placement of fill materials. In addition to being compacted to the required density, the subgrade should also be stable, i.e., not exhibit “pumping" behavior. Where soft subgrade soils are encountered as determined by the geotechnical field representative we recommend that a geogrid product such as Tensar BX-1200 or equal be placed over the subgrade to receive new engineered fill.

The proposed fill material shall be Caltrans Class 2 aggregate base (AB). Fill and backfill materials shall be placed in loose lifts not exceeding 8-inches. Each loose lift shall be compacted with the appropriate equipment to the specified degree of compaction, minimum relative compaction of 95 percent relative compaction. The moisture content shall be controlled within 2 percent of the optimum water content. All compaction criteria refer to the maximum dry density and optimum moisture content determined in accordance with ASTM D1557 test method. In addition to being compacted to the required density, the engineered fill should also be stable, i.e., not exhibit “pumping" behavior.

10.0 Drilled Pier Construction Consideration

It is the responsibility of the Contractor to ensure that the drilled pier excavations are stable prior to placement of steel reinforcement and concrete. We anticipate that temporary casing will be required in approximately the upper 35-feet from the ground surface to stabilize the pier hole within the upper levee fill materials and YBM. Pier shafts should be cleaned of loose rock and debris before placing steel reinforcement and concrete. Owner’s geotechnical field representative should visually inspect completed pier excavations prior to placing steel and concrete. If an installation problem arises during pier excavation, the depth of the pier may need to be deepened in order to develop the equivalent design capacity. Groundwater should be anticipated in all drilled pier holes. The concrete should be tremied cast in a continuous pour from the bottom of the pier to the pier head.

11.0 Construction Monitoring

An AECOM representative of the geotechnical engineer of record shall inspect the subgrade preparation for fill placement and any over-excavation to verify that the subsurface conditions encountered are consistent with the anticipated subsurface conditions presented in this letter report, and to verify that the recommendations for drilled piers presented above are followed to achieve the allowable design capacity. In addition, a copy of the foundation plans and specifications shall be submitted to AECOM for review prior to construction.

12.0 Limitations and Closure

These recommendations have been provided in accordance with the standard of care commonly used as state-of-the-practice in the profession. No other warranties are either expressed or implied. The recommendations presented in this report are based on the assumption that the soil conditions do not vary significantly from those encountered in our subsurface explorations near the site. Should differing conditions be discovered during construction, we should be advised and will revise these recommendations accordingly.

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Attachments

Figures:	1	Boring Location and Site Plan
	2	Regional Faults
	3	Ultimate Capacity for 2-foot Drilled Shaft
Attachments:	A	Boring Log and Key
	B	Laboratory Tests

B-1: Boring by AECOM, May 2018
Boring 4: Boring by Dames & Moore, October, 1960
Source: SCC, 2018

AECOM
San Jose Waste Water Treatment Plant
Santa Clara County, CA

Figure 1 Boring Location and Site

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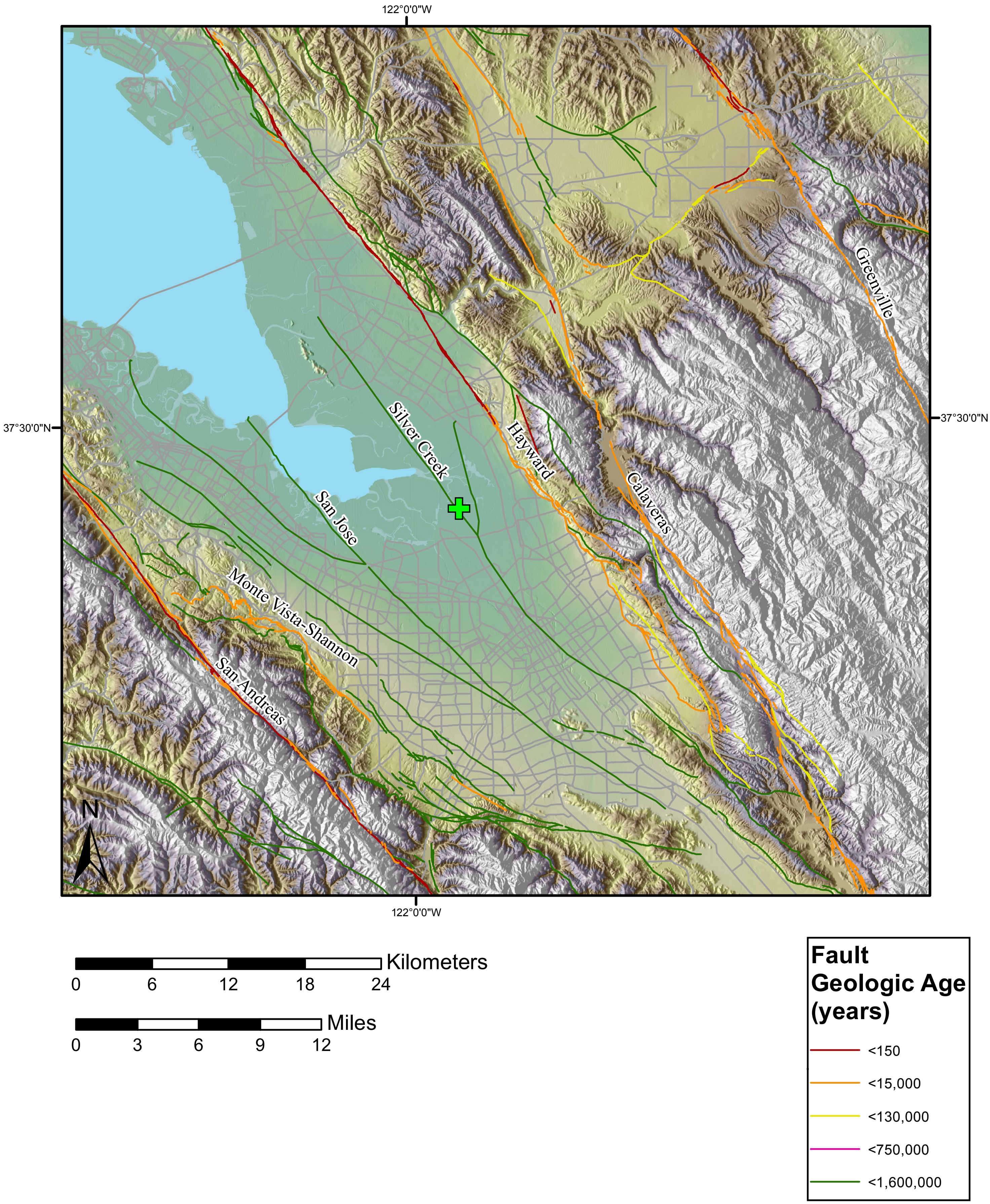

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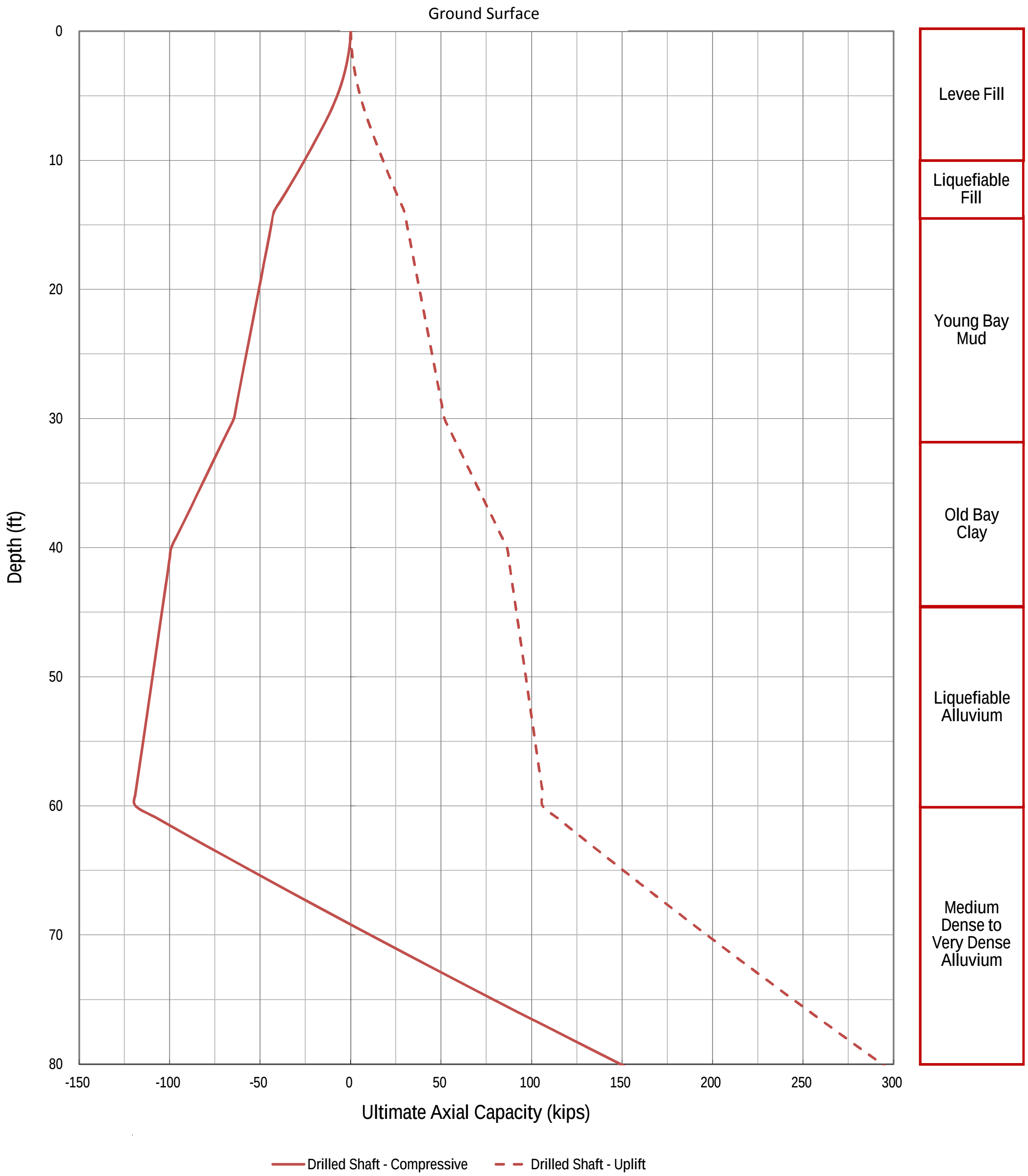
Project No. 60569842

San Jose Waste Water Treatment Plant Outfall Levee

MAP OF REGIONAL SEISMIC SOURCES

Figure 2



Note

1. Ultimate axial capacities were calculated based on FHWA Design and Construction Methods using recent and historical geotechnical investigation data.
2. Elevations are based on NAVD88 Datum.
3. Capacities are based on the assumption that pile center to center spacing is larger than 3 times pile diameter.
4. A factor of safety of at least 2 should be applied to the ultimate capacity curves to determine allowable compressive loads.
5. A factor of safety of at least 3 should be applied to the ultimate capacity curves for uplift conditions. Weight or Buoyant weight of pile may be added for the design uplift capacity.
6. Allowable axial design loads in compression and tension may be increased by one-third under transient loading condition.

AECOM	Project Number: 60569842 San Jose Wastewater Facility	Ultimate Capacity for 2-foot Drilled Shaft	Figure 3

Appendix A

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Project: San Jose Waste Water Treatment Plant
Project Location: San Jose, CA
Project Number: 60569842

Key to Log of Soil Boring

Sheet 1 of 1

Elevation feet	Depth, feet	SAMPLES: Type	SAMPLES: Number	SAMPLES: Sampling Resistance blows/foot	SAMPLES: Recovery, %	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Plasticity Index	Dry Unit Weight, pcf	REMARKS AND OTHER TESTS
1	2	3	4	5	6	7	8	9	10	11	12

COLUMN DESCRIPTIONS

1 **Elevation:** Elevation in feet referenced to specified datum.

2 **Depth:** Depth in feet below the ground surface.

3 **Sample Type:** Type of soil sample collected at depth interval shown; sampler symbols are explained below.

4 **Sample Number:** Sample identification number.

5 **Sampling Resistance:** Number of blows required to advance driven sampler 12 inches beyond first 6-inch interval, or distance noted, using a 140-lb hammer with a 30-inch drop; or down-pressure for pushed sampler.

6 **Recovery:** Percentage of driven or pushed sample length recovered; "NA" indicates data not recorded.

7 **Graphic Log:** Graphic depiction of subsurface material encountered; typical symbols are explained below.

8 **Material Description:** Description of material encountered; may include density/consistency, moisture, color, and grain size.

9 **Water Content:** Water content of soil sample measured in laboratory, expressed as percentage of dry weight of specimen.

10 **Plasticity Index:** Difference between Liquid Limit and Plastic Limit (Atterberg limits)

11 **Dry Unit Weight (pcf)** Dry weight per unit volume of soil measured in laboratory, expressed in pounds per cubic feet (pcf).

12 **Remarks and Other Tests:** Comments and observations regarding drilling or sampling made by driller or field personnel.

- **pp=** Pocket Penetromer reading [tsf]
- **SA:** Sieve Analysis: G=Gravel, S=Sand, F=Fines [%] ASTM D422
- **WA:** Wash on #200, F=Fines [%] ASTM D1140
- **UU:** Unconsolidated Undrained max deviator stress [psf] ASTM D2850
- **CONSOL:** Consolidation Test ASTM D 2435
- **LL: PL:** Liquid Limit [%];Plastic Limit [%] ASTM 4318

TYPICAL MATERIAL GRAPHIC SYMBOLS

- ASPHALT
- ORGANIC FAT CLAY (OH)
- LEAN CLAY (CL)
- LEAN CLAY with GRAVEL (CL)
- SANDY LEAN CLAY (CL)
- CLAYEY GRAVEL (GC)
- POORLY GRADED GRAVEL with CLAY (GP-GC)
- CLAYEY GRAVEL with SAND (GC)
- SILTY SAND with GRAVEL (SM)
- SILTY SAND (SM)
- SILTY SAND (SM)/SANDY SILT (ML)

TYPICAL SAMPLER GRAPHIC SYMBOLS

- GRAB SAMPLE
- STANDARD PENETRATION TEST (SPT)
- 2.5-IN ID MODIFIED CALIFORNIA
- 2.8-IN ID SHELBY TUBE

OTHER GRAPHIC SYMBOLS

- First water encountered at time of drilling
- Static water as measured
- Change in material properties within a stratum
- Inferred or transitional contact

GENERAL NOTES

1. Soil descriptions and contact lines are interpretive. Field descriptions may have been modified to reflect results of lab tests.
2. Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times.
3. Coordinates listed are California State Plane Zone 3 (feet). Elevations were surveyed by the San Jose Waste Water Treatment Plant (SCC, feet).

Project: San Jose Waste Water Treatment Plant Project Location: San Jose, CA Project Number: 60569842	# Log of Soil Boring B-1

Date(s) Drilled	**5/23/2018**	Logged By	**K. Zeiger**	Checked By	**J. Tabor**
Drilling Method	**Rotary Wash**	Drill Bit Size/Type	**4" Tricone**	Total Depth of Borehole	**76.5 feet**
Drill Rig Type	**Truck-mounted Failing 1500**	Drilling Contractor	**Pitcher Drilling Company**	NAVD 88 Ground Surface Elevation	**10-ft**
Groundwater Level(s)	**7.0-ft.**	Sampling Method(s)	**SPT, ModCal, Shelby Tube**	Hammer Data	**Automatic hammer; 140 lbs, 30-inch drop**
Borehole Backfill	**Neat cement grout to ground surface**	Borehole Location	**East Outfall Levee**	Coordinate Location	**N 6138417.130 E 1985949.990**

Elevation, feet	Depth, feet	SAMPLES: Type	SAMPLES: Number	SAMPLES: Sampling Resistance	SAMPLES: Recovery, %	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Plasticity Index	Dry Unit Weight, pcf	REMARKS AND OTHER TESTS
10	0						1" ASPHALT				Hand Auger 0.0-5.0 feet
							4" GRAVEL ROAD BASE				
							CLAYEY GRAVEL with SAND (GC); very dark grayish brown (2.5Y 3/2); 40% angular GRAVEL to 1"; 30% medium grained SAND; 30% low plasticty FINES; dry; noncohesive --LEVEE FILL--				
			S-1								pp = 1.25 tsf
							LEAN CLAY (CL); very dark gray (2.5Y 3/1); moist; very soft; cohesive				
5	5						CLAYEY GRAVEL with SAND (GC); brown (10YR 4/3); 50% angular GRAVEL to 1/2"; 30% low plasticty FINES; 20% medium grained SAND; moist; cohesive				
			S-2	0 0 0	87		LEAN CLAY with GRAVEL (CL); very dark gray (10YR 3/1); 80% low plasticity FINES; 20% rounded GRAVEL to 1/2"; moist; very soft; cohesive ∇				S-2 two liners retained 5.5-6.0 feet; 6.0-6.5 feet Water Level at time of drilling 7.0 feet
			S-3	0 0 4	87		LEAN CLAY with SAND (CL); brown (10YR 4/3); 74% medium plasticity FINES; 24% fine to coarse grained SAND; 2% rounded GRAVEL to 1/4"; wet; soft; cohesive		13		SA: F=2%, S=24%, F=74% LL=31, PL=18 Switch to Rotary Wash Drilling at 9.0 feet
0	10		S-4	10 8 7	7		POORLY GRADED GRAVEL with CLAY (GP-GC); brown (10 YR 4/3); 90% angular GRAVEL to 1.5"; 10% low plasticity FINES; wet; medium dense; gravel is concrete				Some fluid return lost at 10.0 feet
			S-5	7 9 11	13						
							ORGANIC FAT CLAY (OH); very dark greenish gray (10Y 3/1); high plasticity FINES; wet; very soft; cohesive --YOUNG BAY MUD--				Advanced 5" casing to 15.0 feet
-5	15							70		58	CONSOL
			S-6	75 psi	100						
-10	20		S-7	0 0 0	33						S-7 one liner retained: 20.0-20.5 feet
-15	25										
			S-8	0 to 75 psi	100						
-20	30										

Project: San Jose Waste Water Treatment Plant
Project Location: San Jose, CA
Project Number: 60569842

Log of Soil Boring B-1

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Sampling Resistance	SAMPLES Recovery, %	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Plasticity Index	Dry Unit Weight, pcf	REMARKS AND OTHER TESTS
-20	30		S-9	0 5 3	87		SANDY LEAN CLAY (CL); very dark greenish gray (10Y 3/1); 56% low plasticty FINES; 44% fine to medium grained SAND; wet; medium stiff; with abundant shells and shell fragments SANDY LEAN CLAY (CL); olive brown (2.5Y 4/3); 65% low plasticity FINES; 35% fine grained SAND; moist; very stiff; cohesive --OLD BAY CLAY--				WA: F=56% S-9 two liners retained: 30.5-31.0 feet, 31.0-31.5 feet
-25	35		S-10	5 8 13	53			25		99	UU=2315 psf S-10 two liners retained: 35.0-35.5 feet, 35.5-36.0 feet
-30	40		S-11	0 0 3	100		becomes mottled olive brown (2.5Y 4/3) and greenish gray (10GY 5/1); soft	27		102	WA: F=66%, UU=1420 psf S-11 two liners retained: 40.0-40.5 feet, 40.5-41.0 feet
-35	45		S-12	8 6 10	80		SILTY SAND (SM)/SANDY SILT (ML); brown (10YR 4/3); fine to medium grained SAND; no plasticity FINES; wet; medium dense; noncohesive --ALLUVIUM--				
-40	50		S-13	4 2 4	100		becomes loose				SA: G=0%, S=48%, F=52%
-45	55		S-14	4 3 8	100		becomes very dark greenish gray (10YR 3/2); medium to coarse grained; medium dense				
-50	60		S-15	11 21 32	100		SILTY SAND with GRAVEL (SP); very dark gray (N3); 65% fine to coarse SAND; 22% rounded GRAVEL to 1/2"; 13% no plasticity FINES; wet; very dense; noncohesive becomes medium dense				SA: G=22%, S=65%, F=13%
-55	65										

Project: San Jose Waste Water Treatment Plant
Project Location: San Jose, CA
Project Number: 60569842

Log of Soil Boring B-1

Elevation feet	Depth, feet	SAMPLES Type	Number	Sampling Resistance	Recovery, %	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Plasticity Index	Dry Unit Weight, pcf	REMARKS AND OTHER TESTS
-55	65		S-16	8 5 15	100		SILTY SAND with GRAVEL (SP); very dark gray (N3); 65% fine to coarse SAND; 22% rounded GRAVEL to 1/2"; 13% no plasticity FINES; wet; medium dense; noncohesive --ALLUVIUM, cont'd--				
-60	70		S-17	7 6 9	40		CLAYEY GRAVEL (GC); brown (10YR 4/3); 55% rounded GRAVEL to 1/2"; 45% low plasticity FINES; wet; medium dense; cohesive				
							SILTY SAND (SM); brown (10YR 4/3); 74% fine to coarse grained SAND; 21% no plasticity FINES; 5% fine GRAVEL; wet; very dense; noncohesive				Driller notes change in material at 73.5 feet
-65	75		S-18	15 22 31	100						SA: G=5%, S=74%, F=21%
							TOTAL DEPTH = 76.5 FEET				
-70	80										
-75	85										
-80	90										
-85	95										
-90	100										

Appendix B

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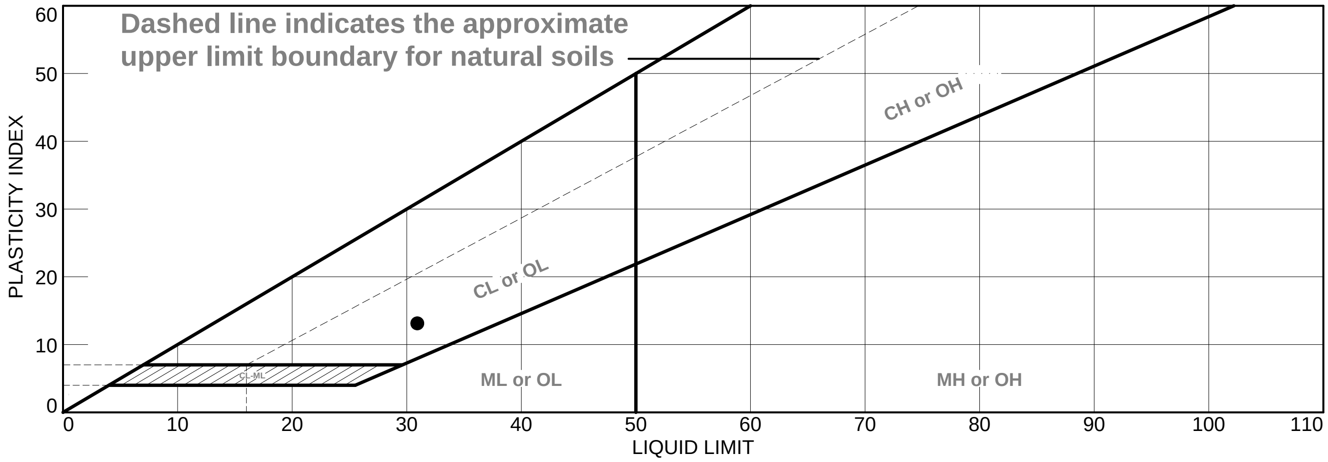
ASTM D-1140
PERCENT PASSING NO. 200 SIEVE REPORT

Method A
Specimens Soaked Overnight without Deflocculating Agent
Dry Mass Determined Directly

Client Name AECOM
Project Name San Jose Santa Clara Outfall
Project Number 60569842

Boring Number	B-1	B-1			
Sample Number	S-9	S-11			
Depth (ft)	30-30.5	40-41.5			
Percent of Soil Finer than No. 200 Sieve	56.3	66.1			
Visual Classification	Gray sandy clay with shells	Grayish brown sandy clay			
Date	06/20/18	06/21/18			
Weight of Dry Soil + Pan (before wash)	474.1	200.7			
Weight of Dry Soil + Pan (after wash)	312.8	101.6			
Weight of Pan	187.6	50.9			

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Brown clay with sand	31	18	13	94	74	CL

Project No. 2301-064.0 **Client:** AECOM

Project: San Jose Santa Clara Outfall
60569842

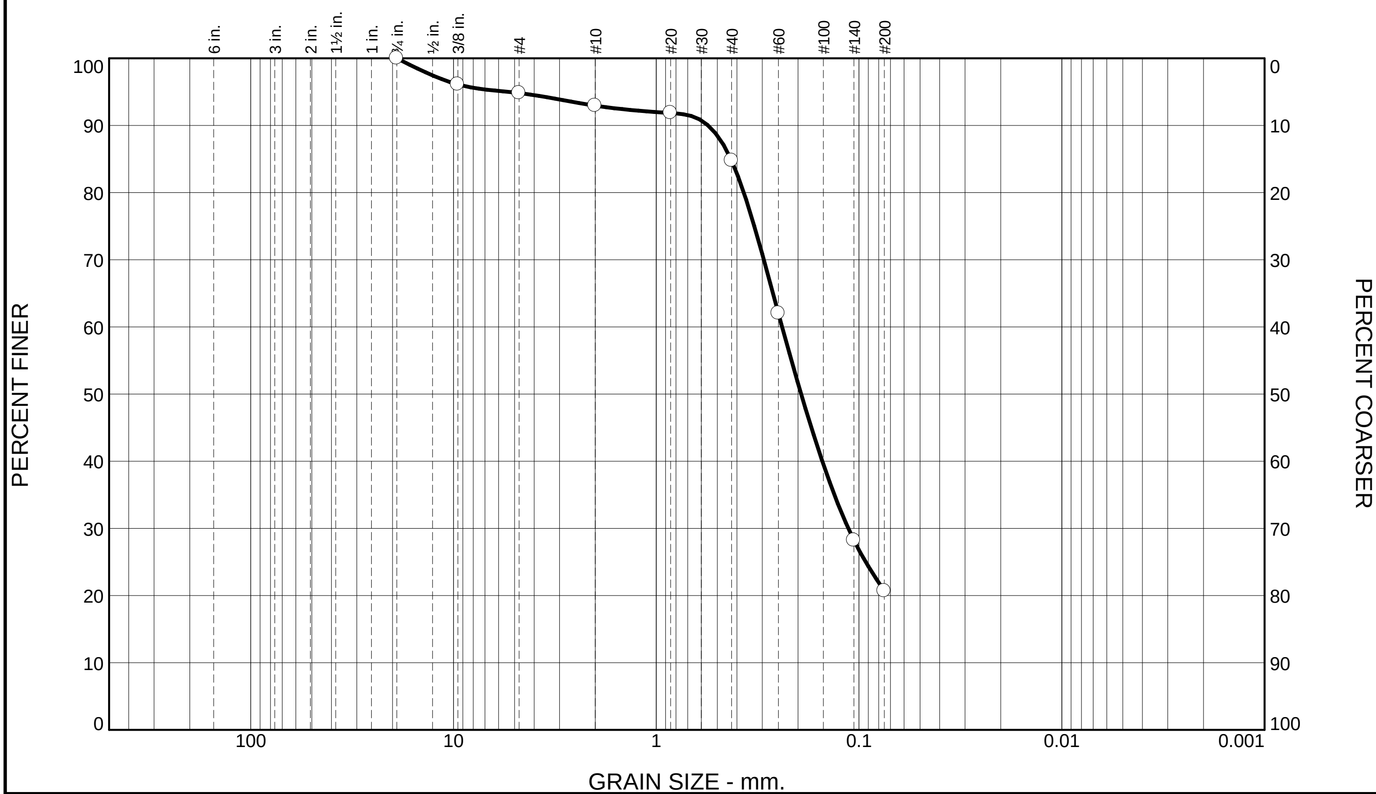
● **Source of Sample:** B-1 **Depth:** 7.5-9 **Sample Number:** S-3

Remarks:

Figure

Tested By: JH **Checked By:** JH

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	5	2	8	64	21	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100		
3/8	96		
#4	95		
#10	93		
#20	92		
#40	85		
#60	62		
#140	28		
#200	21		

* (no specification provided)

Soil Description

Brown silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.5555 D_{85}= 0.4287 D_{60}= 0.2397
D_{50}= 0.1929 D_{30}= 0.1130 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

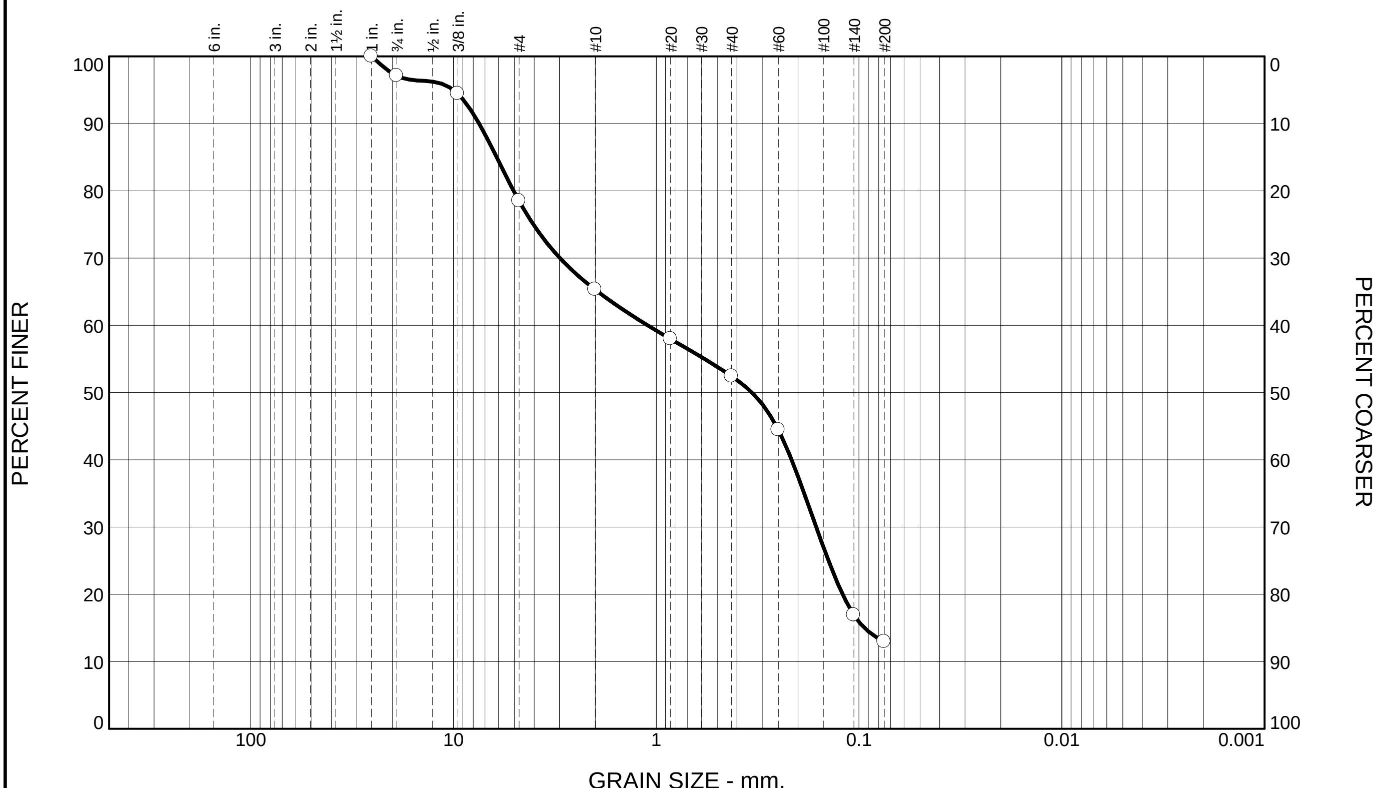
Source of Sample: B-1 **Depth:** 75-76.5
Sample Number: S-18

Date: 6-25-18

Client: AECOM
Project: San Jose Santa Clara Outfall
60569842
Project No: 2301-064.0 **Figure**

Tested By: JH **Checked By:** JH

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	3	19	13	13	39	13	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100		
3/4	97		
3/8	94		
#4	78		
#10	65		
#20	58		
#40	52		
#60	44		
#140	17		
#200	13		

* (no specification provided)

Soil Description

Gray silty sand with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 7.4853 D_{85}= 6.1300 D_{60}= 1.1040
D_{50}= 0.3375 D_{30}= 0.1627 D_{15}= 0.0941
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Source of Sample: B-1 **Depth:** 60-61.5
Sample Number: S-15

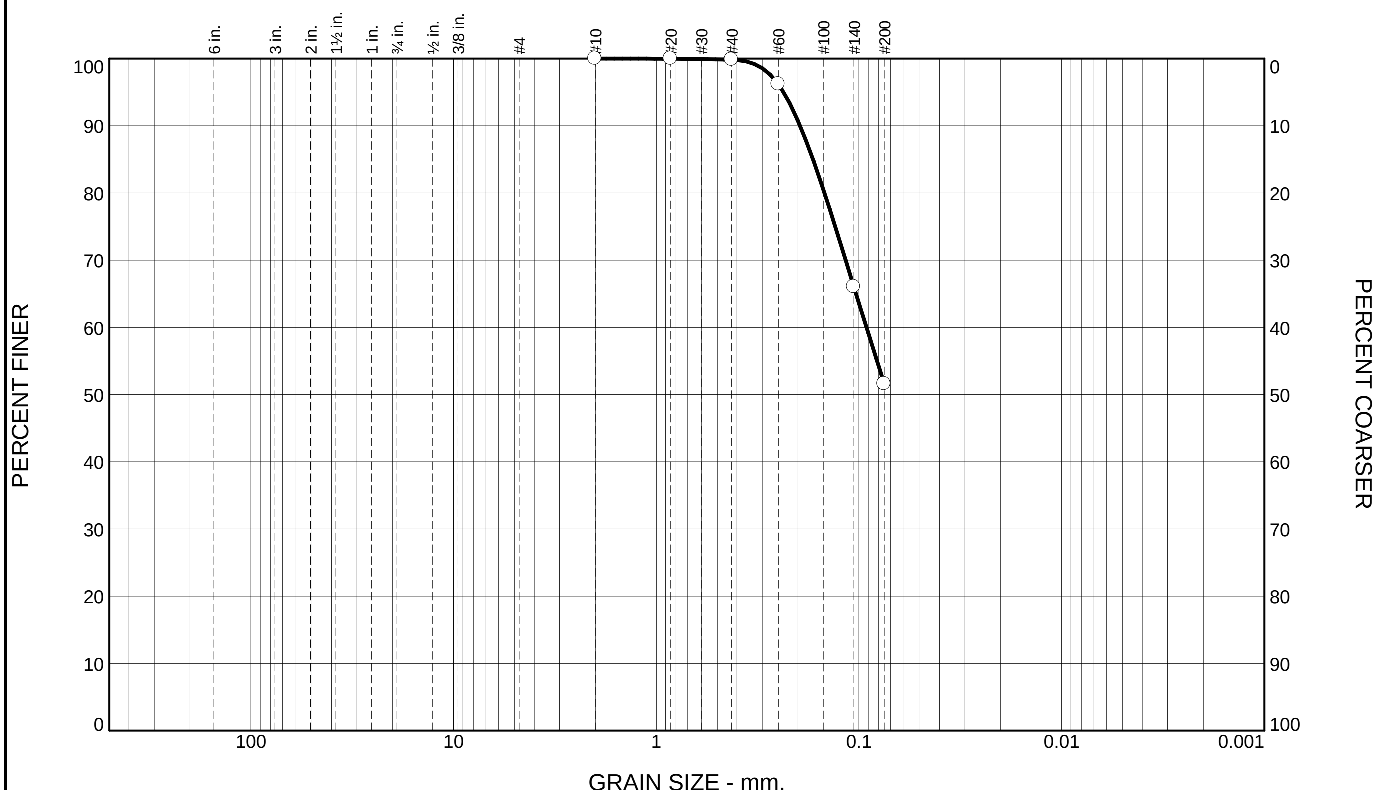
Date: 6-25-18

isi

Client: AECOM
Project: San Jose Santa Clara Outfall
60569842
Project No: 2301-064.0 **Figure**

Tested By: JH **Checked By:** JH

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	0	0	48	52	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100		
#20	100		
#40	100		
#60	96		
#140	66		
#200	52		

* (no specification provided)

Soil Description

Gray sandy silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.1954 D_{85}= 0.1687 D_{60}= 0.0919
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Source of Sample: B-1 **Depth:** 50-51.5
Sample Number: S-13

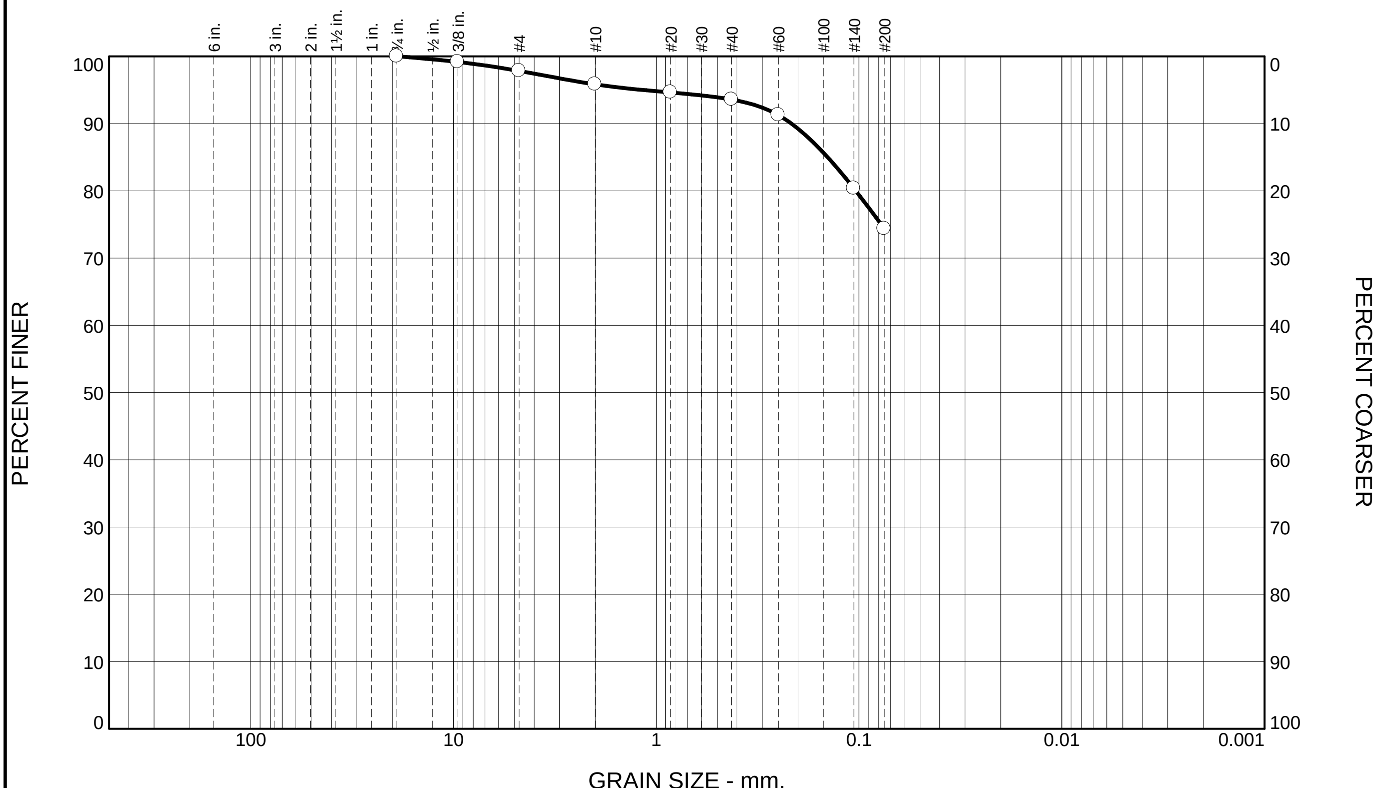
Date: 6-25-18

isi

Client: AECOM
Project: San Jose Santa Clara Outfall
60569842
Project No: 2301-064.0 **Figure**

Tested By: JH **Checked By:** JH

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	2	2	2	20	74	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100		
3/8	99		
#4	98		
#10	96		
#20	95		
#40	94		
#60	91		
#140	80		
#200	74		

* (no specification provided)

Soil Description

Brown clay with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2157 D_{85}= 0.1429 D_{60}=

D_{50}= D_{30}= D_{15}=

D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Source of Sample: B-1 **Depth:** 7.5-9
Sample Number: S-3

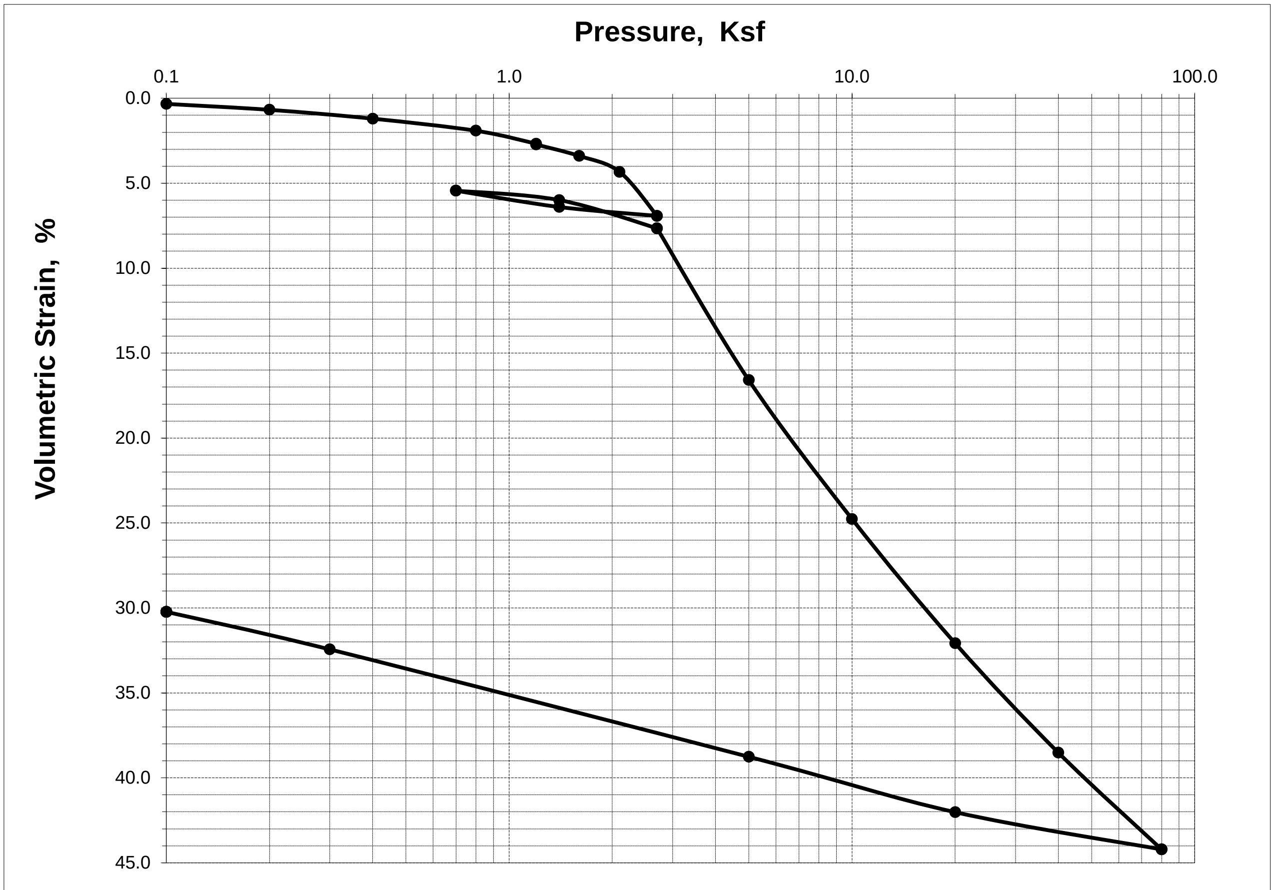
Date: 6-25-18

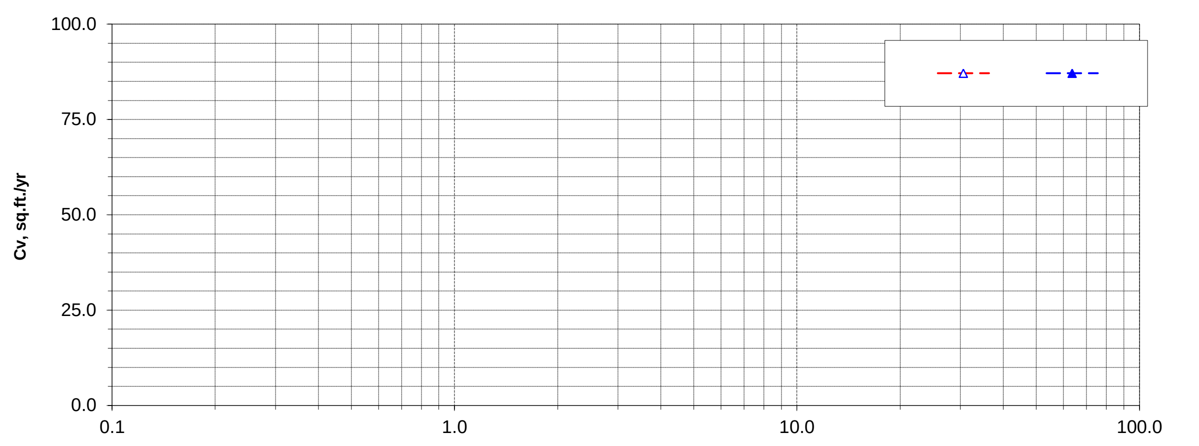
Client: AECOM
Project: San Jose Santa Clara Outfall
60569842
Project No: 2301-064.0 **Figure**

Tested By: JH **Checked By:** JH

CONSOLIDATION TEST ASTM D - 2435

Boring Number	B-1	Sample Number	S-6	Depth (ft)	15-17.5				
Soil Description	Gray clay with organics								
	Water Content, %	Total Wet Unit Weight, pcf	Void Ratio	Saturation %	Height in	Diameter in	Specific Gravity	Liquid Limit, %	Plasticity Index, %
Initial	69.7	98.5	1.904	98.9	1.00	2.420	(assumed) 2.70		
Final	38.1	114.8	1.027	100.1	0.698				





UNCONSOLIDATED UNDRAINED COMPRESSION TEST - ASTM D2850

Client : AECOM
Project : San Jose Santa Clara Outfall
Job # : 60569842
Boring # B-1
Sample # : S-11
Depth (ft) : 40-41.5
Date tested : 06/21/18
Soil : Grayish brown sandy clay (soft)

Data Reduction:

Dial factor = 1.0 in/unit
Load factor = 1.0 lb/unit

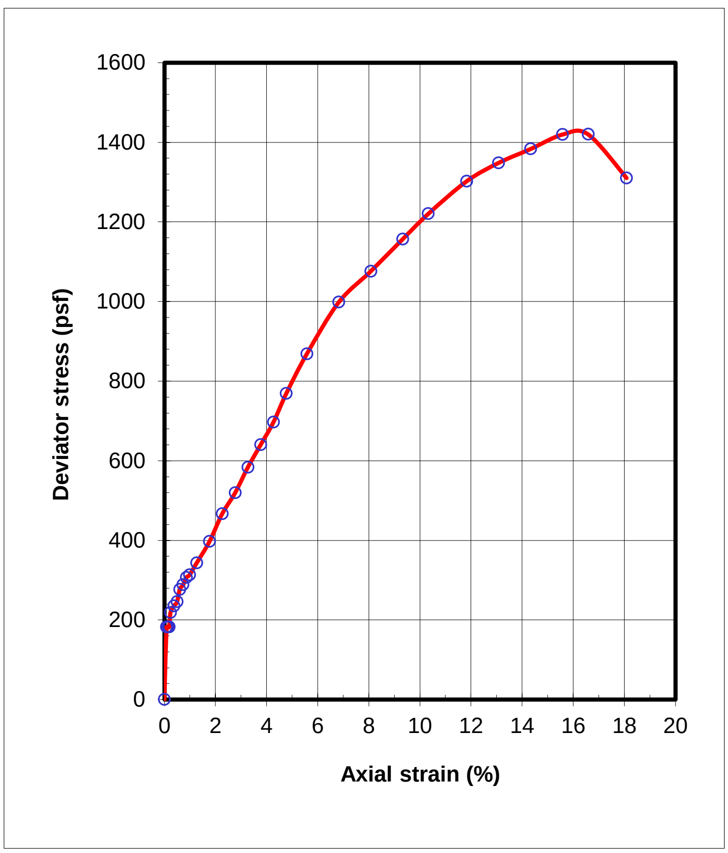
Specimen:

Total wt. =	806.1	gms
Ht. =	5.240	in
Ave dia. =	2.397	in
Area =	4.513	sq.in
Volume =	387.5	c.c.
Shearing rate =	0.03	inch/min
Shearing rate =	0.5	%/min
Gs (assumed) =	2.70	

Test Report:

Void ratio =	0.649	
Ht/Dia ratio =	2.19	
Moisture =	27.1	%
Total density =	129.8	pcf
Dry density =	102.1	pcf
Saturation =	112.5	%
Chamber pressure =	3000	psf
Max. deviator stress =	1420	psf
Strain @ failure =	16.59	%

Dial Read.	Load Read.	Axial Strain (%)	Deviator Stress (psf)
-0.002		0.00	0.0
0.003	5.7	0.08	182.4
0.005	5.7	0.14	182.3
0.008	5.7	0.19	182.2
0.011	6.9	0.24	218.7
0.018	7.4	0.37	235.1
0.024	7.7	0.50	245.8
0.030	8.7	0.60	276.8
0.037	9.1	0.73	288.2
0.043	9.7	0.86	306.9
0.050	9.9	0.99	313.3
0.064	10.9	1.26	343.2
0.091	12.7	1.77	397.1
0.117	15.0	2.26	466.9
0.143	16.7	2.77	519.6
0.169	18.9	3.27	583.6
0.196	20.9	3.77	640.4
0.222	22.8	4.27	697.1
0.248	25.3	4.77	769.1
0.290	28.8	5.57	868.2
0.356	33.6	6.83	998.2
0.421	36.7	8.08	1075.9
0.487	40.0	9.33	1156.8
0.540	42.7	10.33	1220.4
0.618	46.3	11.83	1302.2
0.684	48.6	13.08	1348.3
0.749	50.6	14.33	1383.1
0.815	52.7	15.59	1419.6
0.867	53.3	16.59	1419.8
0.946	50.1	18.09	1310.3



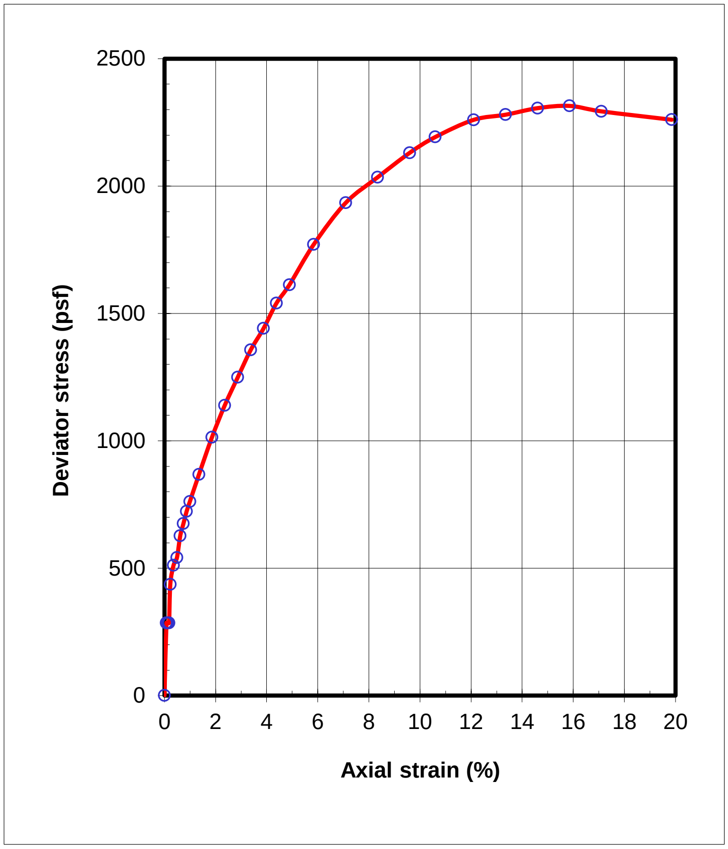
INSPECTION SERVICES, INC. BERKELEY, CA

UNCONSOLIDATED UNDRAINED COMPRESSION TEST - ASTM D2850

Client : AECOM
Project : San Jose Santa Clara Outfall
Job # : 60569842
Boring # B-1
Sample # : S-10
Depth (ft) : 35-36.5
Date tested : 06/21/18
Soil : Brown clay

Specimen:			
Total wt.	=	814.5	gms
Ht.	=	5.530	in
Ave dia.	=	2.410	in
Area	=	4.564	sq.in
Volume	=	413.5	c.c.
Shearing rate	=	0.04	inch/min
Shearing rate	=	0.75	%/min
Gs (assumed)	=	2.70	

Test Report:		
Void ratio =	0.708	
Ht/Dia ratio =	2.29	
Moisture =	24.6	%
Total density =	122.9	pcf
Dry density =	98.6	pcf
Saturation =	93.8	%
Chamber pressure =	2700	psf
Max. deviator stress =	2315	psf
Strain @ failure =	15.85	%



Data Reduction:

Dial factor = 1.0 in/unit
Load factor = 1.0 lb/unit

Dial Read.	Load Read.	Axial Strain (%)	Deviator Stress (psf)
-0.002		0.00	0.0
0.003	9.1	0.08	285.6
0.005	9.1	0.12	285.4
0.008	9.1	0.18	285.3
0.011	13.9	0.23	436.3
0.018	16.3	0.35	511.4
0.026	17.3	0.49	541.7
0.033	20.0	0.62	627.1
0.039	21.5	0.74	674.9
0.046	23.1	0.87	723.2
0.053	24.4	0.99	761.6
0.073	27.9	1.35	868.2
0.101	32.7	1.86	1013.6
0.129	37.0	2.36	1138.9
0.157	40.8	2.87	1249.3
0.185	44.5	3.38	1356.7
0.213	47.5	3.88	1440.9
0.241	51.0	4.38	1539.7
0.269	53.7	4.89	1611.8
0.322	59.6	5.84	1770.9
0.391	66.0	7.09	1934.2
0.460	70.4	8.35	2034.8
0.529	74.7	9.60	2130.4
0.585	77.7	10.60	2192.5
0.668	81.5	12.10	2259.9
0.737	83.4	13.35	2280.1
0.806	85.6	14.60	2305.8
0.875	87.2	15.85	2314.7
0.944	87.7	17.10	2293.3
1.096	89.4	19.85	2260.4

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