September 29, 2025
Kleinfelder Project No. 20203585.001A

Mr. Ramesis Bustamante, PE, Project Manager
City of San Diego
Engineering and Capital Projects
Program and Project Development Division
525 B Street, Suite 750
San Diego, California 92101

SUBJECT: **Geologic Study and Geotechnical Engineering Report**
Design of El Monte Water Transmission Pipeline Rehabilitation
County of San Diego
Cities of Santee, El Cajon, La Mesa and San Diego, California
Contract H197037

Dear Mr. Bustamante:

Kleinfelder has completed the authorized subsurface exploration and geotechnical engineering evaluation for the above-referenced project in general accordance with our contract for the City of San Diego, dated March 8, 2019. The purpose of the geotechnical study was to explore and evaluate the subsurface conditions at selected locations along the pipeline alignment, and to provide recommendations related to the geotechnical aspects of the project design and construction. The attached Kleinfelder report contains a description of the findings of our field exploration and laboratory testing program, our engineering interpretation of the results with respect to the project characteristics, and our design recommendations as well as construction guidelines for the planned project.

We appreciate the opportunity to provide geotechnical engineering services to you on this project. If you have any questions regarding this report or if we can be of further service, please do not hesitate to contact us at 619.831.4600.

Sincerely,

KLEINFELDER

Spencer Floyd, PE
Project Engineer

Scott Rugg, PG, CEG
Principal Engineering Geologist

Robert A. Torres, PE
Senior Program Manager

GEOLOGIC STUDY AND
GEOTECHNICAL ENGINEERING REPORT
DESIGN OF EL MONTE WATER TRANSMISSION PIPELINE REHABILITATION
COUNTY OF SAN DIEGO
CITIES OF SANTEE, EL CAJON, LA MESA, AND SAN DIEGO, CALIFORNIA
KLEINFELDER PROJECT NO. 20203585.001A

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A Report Prepared for:

Mr. Ramesis Bustamante, PE, Project Manager
City of San Diego
Engineering and Capital Projects
Program and Project Development Division
525 B Street, Suite 750
San Diego, California 92101

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DESIGN OF EL MONTE WATER TRANSMISSION PIPELINE REHABILITATION
COUNTY OF SAN DIEGO
CITIES OF SANTEE, EL CAJON, LA MESA AND SAN DIEGO, CALIFORNIA

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1 INTRODUCTION

1.1 GENERAL

Kleinfelder has completed the authorized geotechnical engineering evaluation which included geologic study and subsurface geotechnical exploration for the design of the El Monte Water Transmission Pipeline Rehabilitation project. The project consists of an approximately 12.3-mile-long water pipeline alignment that was constructed between the years of 1942 and 1948. The alignment extends along a northeast to southwest directed trend, with the northeast end located just north of Lake Jennings. The pipeline alignment trends southwesterly from Lake Jennings to the Alvarado water treatment plant at the Lake Murray terminus, as shown on Figure 1, Site Vicinity Map. Our services were provided in general accordance with our contract dated May 5, 2021.

Our report consists of a description of the work performed, description of the general geologic conditions along the entire alignments, a discussion of the general geotechnical conditions observed at the selected exploratory locations spaced along the project alignment, and geotechnical design recommendations developed from our geologic and engineering analyses of field and laboratory data. Discussion is also provided regarding groundwater conditions, geologic/seismic hazards such as liquefaction, excavatability, shoring parameters, grading, suitability of the site soils for reuse as backfill, foundations, and soil corrosivity in recommendation sections of this report.

1.2 PROPOSED CONSTRUCTION

We understand that the County of San Diego is planning to provide information for improvements to the pipeline structure, pipeline appurtenances and access locations, both during construction and as required for future maintenance. The proposed improvements will include, but are not necessarily limited to, construction of access manholes, pipeline vaults, and valve systems such as butterfly valves. Depths of excavations will range from approximately 2 feet to 17 feet at specific locations, corresponding with the existing pipeline invert elevations.

The El Monte Pipeline is within the County of San Diego (unincorporated Lakeside), and the Cities of Santee, El Cajon, San Diego, and La Mesa and was constructed in four distinct segments as indicated on the following table:

Table 1-1: Pipeline Segment Parameters

Segment	Design Drawings – Constructed	Pipe Material	Stations	Length (Miles)
I	1942 – 1946	RCSC – Reinforced Concrete Steel Cylinder	0+00 – 113+93	2
II	1946 – 1948	RCSC – Reinforced Concrete Steel Cylinder	113+93 – 439+20	6
III (Grossmont Tunnel)	1946 – 1948	Cast-in-Place Reinforced Concrete	439+20 – 512+75	1.2
IV	1946 – 1947 (2004)	RCP – Reinforced Concrete Pressure Pipe (original, no cylinder), CML&C Steel Pipe, approximately 400 LF replaced in 2004	512+75 – 658+50	3

The scope of the exploration and engineering evaluation for this study, as well as the conclusions and recommendations in this report, were based on our understanding of the project as described above. This report has been prepared and the recommendations are based on our understanding of the project to date. If pertinent details of the project have changed or otherwise differ from our descriptions, we must be notified and engaged to review the changes and modify our recommendations, if needed.

1.3 SITE DESCRIPTION

The project pipeline alignment is bounded by the cities of Santee and Lakeside to the north, and trends generally southwest from here through Lakeside, Santee, western El Cajon, La Mesa and San Diego. The surrounding areas of the project alignment have become urbanized since the original construction in the 1940's and consisting primarily of light commercial properties and residential neighborhoods with some rural areas located on El Monte Road. The existing alignment is primarily within existing roadways and at some locations on private property, either covered by asphalt concrete, gravel, or bare earth. Existing overhead and underground utilities were noted within the current right-of-way during our field exploration. The approximate locations of our exploratory locations are shown on Figures 2a through 2j, Exploration Map.

1.4 SCOPE OF SERVICES

Our scope of services included drilling ten (10) exploratory borings, associated laboratory testing on soil samples, engineering analyses, and preparation of this geologic study/geotechnical report. Specifically, our scope of work consisted of the following:

- Perform a geologic reconnaissance of the pipeline alignment;
- Review background information including but not limited to available geotechnical reports, geologic maps, groundwater data, and stereoscopic aerial photographs;
- Perform siting and staking of exploratory boring locations, obtained a written approval of the locations from the County of San Diego, and obtain clearance by Underground Service Alert. We also utilized the services of a private utility locator to clear each boring location.
- Drill and log up to 10 borings ranging in depth from 10 to 20 feet below ground surface, using a truck mounted/or track mounted, hollow-stem auger drill rig. Samples were logged and secured at selected intervals within the borings and transported them to our geotechnical laboratory for testing and analysis.
- Provide site specific geologic assessment and visual geotechnical assessment for 1 location where drilling equipment access was not feasible.
- The borings were backfilled using the excavated materials, or bentonite as required by San Diego County Department of Environmental Health (DEH) permit, flush with existing grades where applicable. For those borings located in streets, Aquaphalt™ cold patch asphalt was used as required by the local jurisdictions.
- Prepared localized traffic control plans for those borings located within streets or shoulders. We also obtained encroachment permits for those borings within City/County rights-of-way.
- Preparation of this report including:
    - Boring location map showing the approximate location of each boring.
    - Logs for each boring showing:
        - ✓ Approximate elevation of ground surface.
        - ✓ Location of groundwater table (if encountered).
        - ✓ Number, type, and location of samples.
        - ✓ Visual classification of each soil strata.
    - Laboratory test results.
    - Description of encountered subsurface conditions.
    - Evaluation of potential geologic hazards.
    - Recommendations for foundation subgrade preparation and over-excavation.

- Considerations for dewatering (by others) and temporary excavation support design and construction.
- Recommendations for vault foundation design and construction.
- Discussion of preliminary corrosive soil screening.

This report includes our recommendations related to the geotechnical aspects of the project design and construction. Conclusions and recommendations presented in the report are based on the subsurface information encountered in our borings and the provisions and requirements outlined in the Additional Services and Limitations sections of this report. In addition, a document prepared by the Geoprofessional Business Association (GBA), Important Information About This Geotechnical Engineering Report, has been included in Appendix C. We recommend that all individuals read the report in its entirety along with the included Limitations and GBA document.

2 FIELD AND LABORATORY INVESTIGATION

2.1 FIELD INVESTIGATION

Our field investigation consisted of drilling and sampling ten (10) borings ranging from approximately 10 to 20 feet below ground surface (bgs). The borings were drilled between May 2, 2022 to June 28, 2023 with either a truck mounted (Marl M-10 Yeti), track-mounted (Fraste PLG, Mole), or tripod drill rigs using hollow-stem and solid-stem augers. Our field investigation program is summarized in Table 2-1 below. The approximate locations of the borings are shown on Figures 2a through 2j for boring locations.

Table 2-1: Field Investigation Program

Site No.	Boring No.	Location	Approx. STA	Surface Elev. (ESL, ft)	Boring Depth (ft)
1	B-1	Magnolia Ave & Kenney St	303+00	397	19½
2	B-2	W Bradley Ave & Floyd Smith Dr	345+00	387	19½
3	-	Baltimore Dr	610+50	502	-
4	B-4	Wagner Drive & N Marshall Ave	401+00	433	20
5	B-5	W Main St & Dewane Ave	434+50	562	20
6	B-6	El Monte Road	1+50	440	20
7	B-7	Julian Ave	65+00	426	20
8	B-8	W Main St	439+20	560	20
9	B-9	MTS Amaya Ct	520+00	569	16½
10	B-10	Woodside Ave & Shadow Hill Rd	239+50	662	19
11	B-11	Woodside Terrace	187+35	434	10

Note:
Site 3 was completed as a visual geologic/geotechnical mapping and assessment location.

A Kleinfelder geotechnical engineer supervised the field operations, and the borings were logged by a Kleinfelder geologist or engineer. Selected bulk and drive samples retrieved from the borings were transported to our San Diego laboratory for further evaluation. Additional descriptions of the field exploration program and the logs of borings are presented in Appendix A.

2.2 LABORATORY TESTING

Laboratory tests were performed on selected samples to evaluate pertinent engineering properties of those materials. Laboratory tests were performed in general accordance with applicable standards. Laboratory testing procedures and test results are provided in Appendix B. The laboratory testing program consisted of the following:

- **Visual Classification**, ASTM D2488, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure).
- **Atterberg Limits**, ASTM D4318, Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.
- **Maximum Density**, ASTM D1557, Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort.
- **Moisture Content Tests**, ASTM D2216, Standard Test Method for Laboratory Determination of Water.
- **Dry Unit Weight Tests**, ASTM D2937, Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method.
- **Sieve Analysis**, ASTM D6913, Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis.
- **Direct Shear**, ASTM D3080, Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions.
- **Corrosion Testing**, The samples were tested in general accordance with California Test Methods 643, 417 and 422, for pH and resistivity, and soluble sulfates and soluble chlorides, respectively.
- **R-Value**, ASTM D2844, Standard Test Method for Resistance R-Value and Expansion Pressure of Compacted Soils.

3 SITE CONDITIONS

3.1 REGIONAL GEOLOGIC AND GEOTECHNICAL SETTING

San Diego County resides within the southern portion of California's Peninsular Ranges Geomorphic Province. This province is characterized as an assemblage of north to northwest trending, high-relief ranges stretching south from the southern margin of the Santa Monica Mountains in Los Angeles, through San Diego County and south into Baja, California. Some of the notable ranges of Southern California include the Santa Ana Mountains, the Laguna Mountains and the Cuyamaca Mountains. The development of this mountain systems are closely tied to the transform tectonism of the San Andreas Fault System (SAFS).

The County encompasses three geomorphic sub-zones set in a series of north-to-northwest trending belts, roughly parallel to the coastline. From west to east, these zones are comprised of a relatively narrow, low-relief coastal plain; the central high-relief mountainous zone; and the low-lying desert zone. The coastal plain and mountainous zones are part of the more extensive geomorphic province of the Peninsular Ranges discussed above. The desert zone is part of a larger geomorphic province known as the Colorado Desert.

The project site is located within along the margin between the coastal plan subzone and the central mountainous subzone. The coastal plain subzone west of the site is underlain by Cretaceous to Tertiary age sedimentary rocks consisting of a variety of sandstones, claystones, siltstones and conglomerates. These rocks overlie the basement rocks of the mountainous subzone which are mostly comprised of granitic rock with some metamorphic rock.

During the late Pleistocene, the land surface throughout San Diego County was down-cut and eroded by fluvial processes in response to a world-wide glacially induced drop in sea level. This erosional event resulted in the dissected system of east to west flowing drainages and intervening basins that empty into the Pacific Ocean. Near the coast, these drainages down cut several hundreds of feet below current sea-level elevations. Near the end of the Pleistocene epoch and continuing up to the present, sea-level gradually rose as the continental glaciers receded, which forced in-filling of the eroded drainages with alluvial sediments. The San Diego River is one of the larger drainages in this area and borders the northern margin of the project alignment. Figure 3, Regional Geology, shows the geologic conditions surrounding the project area.

3.2 REGIONAL FAULTING AND SEISMICITY

Southern California straddles the boundary between two global tectonic plates known as the North American Plate (on the east) and the Pacific Plate (on the west). The main plate boundary is represented by the San Andreas fault which stretches northwest from the Gulf of California in Mexico, through the desert region of the Imperial Valley, through the San Bernardino region, and into Northern California where it eventually trends offshore north of San Francisco (Kleinfelder, 2019). Within Southern California, the San Andreas fault is a complex system of numerous faults known as the San Andreas Fault System that span a 150-mile-wide zone from the main San Andreas fault in the Imperial Valley westward to offshore of San Diego (Kleinfelder, 2019). The major faults east of the San Diego region (from east to west) include the San Andreas Fault, the San Jacinto fault, and the Elsinore fault. Major faults west of San Diego include the Palos Verdes-Coronado Bank fault, the San Diego Trough fault, and the San Clemente fault.

The most dominant zone of active faulting within the western portion of San Diego County are several faults associated with the Rose Canyon Fault Zone (RCFZ) to the west and Elsinore Fault Zone (EFZ) to the east, as presented in Figure 4, Regional Fault Map and Earthquake Epicenters. The northeastern end of the alignment is located approximately 18-mile northeast of the Rose Canyon fault and 24 miles southwest of the Elsinore fault. The southwestern end of the project is approximately 8 northeast of the Rose Canyon fault and 34 miles southwest of the Elsinore fault. No active faults are mapped nearby or within the project alignment. Although activity on any of the known and unknown faults within the SAFS affect the seismicity of the San Diego region, activity within both the RCFZ and the EFZ dominates most aspects of the seismic hazard at the project site.

3.3 SITE GEOLOGY

The project alignment crosses through four different geologic terrains. Table 3-1 and Figure 3 identifies the geology and location of each of these terrain areas. These terrains areas are also displayed on the Regional Geologic Map, Figure 3. It should be noted that these four terrain areas are separate from the defined four project segments 1 through 4.

Table 3-1: Project Geologic Terrains

Name	Approximate Stationing	Terrain Geology	Terrain Length (Miles)
Northern Terrain	0+00 to 175+00	Young Alluvial Deposits over Granitic and Metavolcanic Bedrock	3.2
North-Central Terrain	175+00 to 290+00	Granitic Bedrock with Shallow Intervening Alluvial Deposits	2.2
South Central Terrain	290+00 to 425+00	Old Alluvial Deposits Over Variable Depth Granitic Bedrock	2.6
Southern Terrain	425+00 to 654+00	Eocene Age Sedimentary Deposits	4.3

3.3.1 Northern Terrain

The northernmost terrain stretches from the beginning of the project alignment at 0+00 to approximate station 175+00. This segment trends along the southern boundary area of the San Diego River and bedrock hillslopes comprised of Cretaceous age metavolcanic and granitic rock. Our review indicates the majority of this segment is underlain by recent alluvial deposits of the San Diego River drainage. This alluvium typically consists of interbedded poorly graded sand, silty sand, silts and clays. These soils are typically in a relatively medium dense to dense condition, where encountered during our subsurface exploration. The depth of the alluvium along the project alignment will vary from as shallow as 10 feet to in excess of 50 feet. Additionally, there is the potential for near surface groundwater ranging from approximately 10 to 20 feet below ground surface.

Based on the results of a geotechnical investigation we prepared several years ago for the nearby Woodside Avenue Flood Control Project along Woodside Avenue between approximate stations 124+00 to 132+00 (Kleinfelder, 2019), the deepest boring of approximately 50 feet below ground surface (bgs) did not penetrate the bottom of the alluvial deposits. The soils encountered consisted of interbedded silty sands, poorly graded sands and silt with some clay layers. Sampling blow counts correlated with a relatively loose to medium dense condition, and encountered groundwater levels ranged from approximately 13 feet to 26 feet bgs.

We anticipate that the alluvium overlies weathered bedrock of granite and metavolcanic material across the Northern Terrain portion of the project alignment. The depth to bedrock will vary appreciably based both on proximity to bedrock outcrops and variability in the subsurface topographic relief of the bedrock surface. Based on observations of nearby outcrops and our experience in the area, the degree of weathering of the bedrock will likely vary from nearly decomposed to hard and slightly weathered.

Due to the potential of relatively loose to medium dense recent alluvial soils and near surface groundwater below most of the northern terrain, the risk for liquefaction during an earthquake event would be considered moderate to high.

3.3.2 North-Central Terrain

The North-Central terrain extends between approximate stations 175+00 to 290+00. The southwest end is located at the SR52/SR67 connector. This portion of the alignment takes a broad bend to the southwest as it crosses over bedrock hillslopes of Cretaceous age granitic rock. The topography varies along this stretch of the alignment crossing over hillslopes of granitic rock and lower areas of alluvial and or colluvial soils.

Two hillslopes of granitic rock occur from approximately stations 175+00 to 195+00 and stations 262+00 to 274+00. Our experience in this area, as well as observations along this alignment indicates that the weathering of the granitic material varies notably and ranges from being nearly decompose and slightly to moderately weathered.

The low-lying areas between the granitic outcrops are anticipated to be underlain by relatively shallow old alluvial deposits which in-turn overlie variably weathered granitic bedrock similar to the adjacent hillslopes. We reviewed boring log data for the SR52 / SR67 connector which showed that the southern end of this terrain is underlain by either near surface granitic bedrock or shallow alluvium over bedrock as discussed above (CALTRANS, 2007). The alluvium below the connector ranged up to approximately 20 feet in depth.

The pipeline is positioned within the cut area between a cribwall and housing development situated on the east side of Woodside Avenue between approximate stations 221+00 to 224+00. It is our understanding that the wall was installed in conjunction with the construction of the State Route 67. The cribwall shows signs of distress, notably along the upper portions. Should the wall condition continue to degrade, there is the potential for an adverse effect on the pipeline.

3.3.3 South-Central Terrain

South of the SR52/SR67 connector the alignment extends southwest across the El Cajon Valley up to approximate station 440+00. The valley is a relatively flat-lying feature underlain by old alluvial deposit. This unit is designated as Qu on the regional geologic map, which corresponds to undivided quaternary alluvial deposits. This alluvium is typically older than the young alluvial deposits of the San Diego River drainage on the north, being late Pleistocene in age and therefore are typically in a denser condition. Groundwater across this region can vary appreciably. Our review of groundwater data on the Geotracker website indicates that water elevations range from less than 10 feet bgs to in excess of 30 feet bgs (State of California, 2023). However, given the age of these alluvial deposits and the relative density, the potential for liquefaction of these old alluvial deposits is much less than the recent alluvial deposits on the north of the site and would be anticipated to range from low to moderate.

3.3.4 Southern Terrain

The southern portion of the alignment stretches from approximate station 440+00 to the Alvarado Water Treatment Plant at Lake Murray at station 654+00. This portion of the alignment consists of both low relief and rolling hillside topography which is underlain by Eocene age sedimentary rock. From oldest to youngest these sedimentary units consist of the Friars Formation, the Stadium Conglomerate, the Mission Valley Formation and the Pomerado Conglomerate (Todd, 2004). On the northeastern site near station 440+00, the alignment enters a tunnel section that extends to approximate station 513+00. Near the tunnel northeast portal location, the site is underlain by the Friars Formation. The Friars Formation consist of a light greenish gray claystone. This area is part of the Fletcher Hills and is noted for slope instability due to weak soil strength of the Friars Formation.

Beyond the northeast portal area, the tunnel section crosses below units of the Stadium Conglomerate, Pomerado Conglomerate and Mission Valley Formation. At the western portal area of the tunnel near station 513+00, the pipeline exits into material of the Stadium Conglomerate, and continues westward within this unit along the northern drainage margin of Fletcher Parkway. Across this stretch, there are several areas of roadway fill and fill underlain by stream alluvial deposits. At the westernmost end of the pipeline alignment at Lake Murray, the pipeline crosses through sandstone material of the Mission Valley Formation.

Both the Stadium and Pomerado Conglomerates contain up to 50 percent gravel and cobble size clasts with occasional boulder size clasts up to 24 inches in diameter. These clasts are extremely hard and should be considered during design and construction.

Table 3-2: Unit Types and Depths

Site No.	Sta. No.	Boring No.	Surface Elev.[1] (ft)	Artificial Fill Depth Interval (ft)	Alluvium Depth Interval (ft)	Friars Formation Depth Interval (ft)	Stadium Conglomerate Depth Interval (ft)	Decomposed Granitic Rock Depth Interval (ft)
1	303+00	B-1	397	-	0 - 17	-	-	17 – 19½
2	345+00	B-2	387	-	0 – 19½	-	-	-
3[2]	610+50	-	502	-	-	0 - 20	-	-
4	401+00	B-4	433	0 – 3½	-	3½ - 20	-	-
5	434+50	B-5	562	-	-	0 - 20	-	-
6	1+50	B-6	440	-	0 - 20	-	-	-
7	65+00	B-7	426	-	0 - 20	-	-	-
8	439+20	B-8	560	0 - 15	-	15 - 20	-	-
9	520+00	B-9	569	0 - 5	-	-	5 - 16	-
10	239+50	B-10	662	0 - 9	9 – 18½	-	-	18½ - 19
11	187+35	B-11	434	0 - 8	-	-	-	8 - 10

Notes:
[1]Ground surface elevations based on Google Earth imagery using the World Geodetic System 1984 (WGS84) Earth Gravitational Model (EGM96) Geoid vertical datum.
[2]Site 3 completed as a geologic mapping location, depth of Friars formation is estimated to extend full length of proposed construction depths.

3.4 GROUNDWATER OBSERVATIONS

Groundwater was encountered in borings B-2 and B-8 between approximately 9½ and 12½ feet bgs, corresponding to variable elevations based on boring locations. The results of laboratory testing for moisture content did not identify soils significantly over optimum. The depth of groundwater encountered during field investigation is summarized in Table 3-3.

Table 3-3: Elevation and Depth of Groundwater

Site No.	Boring No.	Approx. STA	Date Completed	Surface Elev. (ESL, ft)	Boring Depth (ft)	Groundwater Depth (ft)	Groundwater Elevation (ESL, ft)
1	B-1	303+00	5/2/2022	397	19½	-	-
2	B-2	345+00	12/9/2022	387	19½	12½	367½
3[1]	-	610+50	-	502	-	-	-
4	B-4	401+00	9/23/2022	433	20	-	-

Site No.	Boring No.	Approx. STA	Date Completed	Surface Elev. (ESL, ft)	Boring Depth (ft)	Groundwater Depth (ft)	Groundwater Elevation (ESL, ft)
5	B-5	434+50	5/2/2022	562	20	-	-
6	B-6	1+50	12/9/2022	440	20	-	-
7	B-7	65+00	9/21/2022	426	20	-	-
8	B-8	439+20	6/28/2023	560	20	9½	550½
9	B-9	520+00	9/21/2022	569	16½	-	-
10	B-10	239+50	4/11/2023	662	19	-	-
11	B-11	187+35	4/11/2023	434	10	-	-

Note:
[1]Site 3 was completed as a geologic mapping location, groundwater not expected at this location.

It should be noted that groundwater levels at the site can fluctuate with time due to changes in weather, irrigation, construction, groundwater withdrawal or injection, and other influences that were not present at the time the observations were made. Perched water can develop near the interface of the more permeable fill over the less permeable formational materials. In general, the soils encountered were in a moist condition below saturation levels necessary for free groundwater conditions.

4 POTENTIAL GEOLOGIC AND SEISMIC HAZARDS

4.1 SEISMIC SHAKING AND CBC SEISMIC DESIGN PARAMETERS

The overall project site, like most of Southern California, is located in a seismically active area and is likely to experience ground shaking as a result of earthquakes on nearby or more distant faults. Seismic design parameters were developed consistent with the requirements of the 2022 CBC, ASCE/SEI 7-16, and Supplements 1 through 3 of that standard. Based on the results of our field investigation, we recommend that the Sites 1, 4, 6, 7, 8, and 10 be classified as Site Class D and Sites 2, 3, 5, 9, and 11 be classified as Site Class C. Site Class D is defined as a site having a stiff soil profile within the upper 100 feet with an average shear wave velocity between 600 ft/s to 1,200 ft/s, an average SPT N-value between 15 and 50 blows per foot, or an average undrained shear strength between 1,000 and 2,000 psf. Site Class C is defined as a site having a very dense soil and soft rock profile within the upper 100 feet with an average shear wave velocity between 1,200 ft/s to 2,500 ft/s, an average SPT N-value greater than 50 blows per foot, or an average undrained shear strength greater than 2,000 psf.

Based on the Site Class and the mapped spectral accelerations, site-specific ground motion hazard analyses are required for Sites 1, 4, 6, 7, 8, and 10 in accordance with Section 11.4.8 of ASCE/SEI 7-16 Supplement 3 unless exceptions are taken by the structural engineer. For a Site Class D designation, the exception of Section 11.4.8 of ASCE 7-16 Supplement 3 states that a ground motion hazard analysis is not required when the values of S_{M1} and S_{D1} are increased by 50 percent. A F_v value per Table 11.4-2 of ASCE 7-16 Supplement 3 may be used in conjunction with the code-based S_1 value for each site for purposes of calculating the T_S value, determination of the Seismic Design Category, and, when taking the exceptions of Section 11.4.8 of ASCE 7-16 Supplement 3, for calculation of S_{M1} and S_{D1} values. As such, code-based seismic design parameters provided in Table 4-1 for the spectral response acceleration at a period of 1.0 second include the 50 percent increase assuming the exception of Section 11.4.8 of ASCE 7-16 Supplement 3 will be taken. The parameters listed in Table 4-1 are estimated using the Seismic Design Maps website maintained by SEAOC/OSHPD (https://seismicmaps.org) and the requirements of the 2022 CBC/ASCE 7-16.

Table 4-1: General Ground Motion Parameters Based on ASCE 7-16

Design Parameter	Symbol	2022 CBC / (ASCE 7-16) Reference(s)	Recommended Value[1]					
			Site 1	Site 2	Site 3	Site 4	Site 5	Site 6
Site Class	--	Section 1613.3.2 (Section 11.4.2)	D	D	C	D	C	D
Site-Modified Peak Ground Acceleration	PGA_m	(Fig. 22-9))	0.42g	0.42g	0.43g	0.42g	0.40g	0.42g
Site-Modified Design Spectral Response Acceleration (5% damped) at Short Periods	S_{DS}	Section 1613.3.4 (Eq. 11.4-3)	0.61g	0.61g	0.66g	0.61g	0.62g	0.62g
Site-Modified Design Spectral Response Acceleration (5% damped) at 1-Second Period	S_{D1}	Section 1613.3.4 (Eq. 11.4-4)	0.573g	0.273g	0.298g	0.574g	0.283g	0.580g

Notes:

[1]Section 11.4.8 of ASCE/SEI 7-16 Supplement 3 requires a site-specific ground motion hazard analysis be performed for Site Class D sites with S_1 greater than or equal to 0.2g unless an exception is taken in which the values of S_{M1} and S_{D1} are increased by 50 percent. Values of S_{M1} and S_{D1} provided in the table are based on F_v values per Table 11.4-2 of ASCE 7-16 Supplement 3 and include this 50 percent increase assuming the exception will be taken. The value of F_v per Table 11.4-2 of ASCE 7-16 Supplement 3 shall only be used for calculation of T_S., determination of Seismic Design Category, linear interpolation for intermediate values of S_1, and when taking the exceptions under Items 1 and 2 of Section 11.4.8 for the calculation of S_{D1}.

Table 4-1: General Ground Motion Parameters Based on ASCE 7-16 (continued)

Design Parameter	Symbol	2022 CBC / (ASCE 7-16) Reference(s)	Recommended Value[1]				
			Site 7	Site 8	Site 9	Site 10	Site 11
Site Class	--	Section 1613.3.2 (Section 11.4.2)	D	D	C	D	C
Site-Modified Peak Ground Acceleration	PGA_m	(Fig. 22-9))	0.42g	0.42g	0.41g	0.42g	0.39g
Site-Modified Design Spectral Response Acceleration (5% damped) at Short Periods	S_{DS}	Section 1613.3.4 (Eq. 11.4-3)	0.61g	0.61g	0.63g	0.61g	0.61g
Site-Modified Design Spectral Response Acceleration (5% damped) at 1-Second Period	S_{D1}	Section 1613.3.4 (Eq. 11.4-4)	0.577g	0.576g	0.287g	0.574g	0.282g

Notes:
[1]Section 11.4.8 of ASCE/SEI 7-16 Supplement 3 requires a site-specific ground motion hazard analysis be performed for Site Class D sites with S_1 greater than or equal to 0.2g unless an exception is taken in which the values of S_{M1} and S_{D1} are increased by 50 percent. Values of S_{M1} and S_{D1} provided in the table are based on F_v values per Table 11.4-2 of ASCE 7-16 Supplement 3 and include the 50 percent increase assuming the exception will be taken. The values of F_v per Table 11.4-2 of ASCE 7-16 Supplement 3 shall only be used for calculation of T_S., determination of Seismic Design Category, linear interpolation for intermediate values of S_1, and when taking the exceptions under Items 1 and 2 of Section 11.4.8 for the calculation of S_{D1}.

4.2 LIQUEFACTION

The term liquefaction describes a phenomenon in which saturated, cohesionless soils temporarily lose shear strength (liquefy) due to increased pore water pressures induced by strong, cyclic ground motions during an earthquake. Structures founded on or above potentially liquefiable soils may experience bearing capacity failures due to the temporary loss of foundation support, vertical settlements (both total and differential), and may undergo lateral spreading. The factors known to influence liquefaction potential include soil type, relative density, grain size, confinement, depth to groundwater, and the intensity and duration of the seismic ground shaking. Liquefaction is most prevalent in loose to medium dense, sandy and gravely soils below the groundwater table, but can also occur in non-plastic to low plasticity fine-grained soil.

Based on the guidelines provided for liquefaction evaluation in the 2022 CBC/ASCE 7-16, evaluations of potential liquefaction susceptibility based on groundwater level, deposit age, and soil composition were made according to the criteria of Bray and Sancio (2006) and Boulanger and Idriss (2006). Only Sites 2 and 8 encountered groundwater within the depths explored and; therefore, all other sites are considered not susceptible to liquefaction. Borings B-2 and B-8 encountered shallow groundwater during drilling. However, the materials in boring B-2 are considered not susceptible to liquefaction due to the clayey dense materials encountered. At boring B-8, loose artificial fill composed of clayey sands were encountered below groundwater; however, due to the amount of fines in these materials and the medium to high plasticity of the clay particles, the materials in boring B-8 are also considered not susceptible to liquefaction.

4.3 FAULT SURFACE RUPTURE

Review of readily available geologic and fault maps does not show any active or potentially active fault features passing through or nearby any of the sites. An active fault is one which has undergone displacement within the last approximate 11,000 years. A potentially active fault (aka: Pre-Holocene fault) is one in which movement has occurred at sometime between 1.6 million years and 11,000 years before present. The site is not located within an Alquist-Priolo Earthquake Fault Zone. Based on this information, the hazard with respect to fault rupture is considered negligible.

4.4 LANDSLIDES

Landslides are deep-seated ground failures (tens to hundreds of feet deep) in which a large section of a slope slides downhill. Landslides are not to be confused with minor surficial slope failures (slumps), which are usually limited to the topsoil zone and can occur on slopes composed of almost any geologic material. Landslides can cause damage to pipelines and structures both above and below the slide mass. Undermining of structures can damage structures above the slide area. Areas below a slide can be damaged by being overridden and crushed by the failed slope material.

Much of the northern, north-central and south-central terrain areas of the EMPL crosses through relatively flat-lying topography. The north-central segment does cross through two moderate hillslope areas which are composed of granitic rock. It is our opinion that the potential for landsliding or slope stability instability in these areas would be relatively low to moderate. The northern and south-central terrain areas do not have any notable sloping topography and the potential landsliding is considered low.

The southern terrain crosses into moderately sloping hillside areas comprised of Eocene age sedimentary material. One of these units, the Friars Formation on the eastern side of this segment is notorious for instability on moderate to steeply sloping ground surfaces. There are both structures as well as the tunnel section portal in this area. The potential for slope instability in this area would be considered moderate to high.

The remainder of the alignment through this southern terrain is comprised mostly of conglomerate and sandstone of the Pomerado Conglomerate, Stadium Conglomerate and Mission Valley Formation. These units are typically considered relatively stable in regard to landsliding on moderate to steeply sloping surfaces and it is our opinion that the landsliding hazard is considered low.

4.5 COMPRESSIBLE SOILS

Soil types typically prone to compressibility are alluvium. Generally, the younger the alluvium, the more prone to compressibility. This is because older alluvium typically has more time to densify and becomes cemented. Alluvium was encountered along much of the three northernmost terrain areas. The alluvium within the northern terrain area is the youngest alluvium from recent deposition within the San Diego River drainage area and are considered to be the most prone to compressibility being a moderate to high hazard. The older alluvium within the El Cajon Valley area would be considered much less prone to compressibility having a moderate potential.

4.6 FLOOD HAZARD

The flood hazard potential at the site was evaluated based on flood hazard maps available through the FEMA Map Service Center Web site (FEMA Flood Map Service Center, 2023). Based on FEMA Maps, all sites are located within Zone X corresponding to an area of minimal flood hazard.

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

It is our understanding that earthwork to be performed will include general excavation at selected locations along the pipeline alignment ranging from approximately 5 to 20 feet in depth. The recommendations submitted herein are based upon data obtained from our subsurface exploration and geologic review. The nature and extent of subsurface variations that may exist along the project alignment will not become evident until construction. If variations appear evident, the recommendations presented in this report should be evaluated. In the event that any changes in the nature, design, location of the proposed project are planned, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed, and our recommendations modified in writing.

5.2 EARTHWORK

5.2.1 Site Preparation

Site preparation includes removal of existing improvements, pavement and stripping of all vegetation and topsoil from the proposed construction sites. Stripping depths required will likely vary and should be adjusted to remove all vegetation and root systems, if encountered. A Kleinfelder representative should monitor the stripping operations to observe that unsuitable materials have been removed. Soils removed during site stripping operations could be used for final site grading outside the pipeline area. Care should be exercised to separate these materials to avoid incorporation of the organic matter in structural fill sections.

Depending upon the extent of construction, any required tree removal should also be accomplished at this time. Care should be taken to thoroughly remove all root systems from the construction areas. Materials disturbed during removal of stumps should be undercut and replaced with structural fill.

5.2.2 Excavation Characteristics

There are several construction considerations given the geologic conditions along the project alignment. The first of these will be rippability and excavation difficulty in areas underlain by granitic rock. Based on our experience in this area of San Diego County, the degree of weathering and rippability can vary widely even over relatively short distances. The granitic rock generally will range from being nearly decomposed to slightly weathered. Areas of slightly weathered granitic rock may require specialized construction equipment consisting of rock breakers or potentially blasting.

Near surface groundwater in areas underlain by alluvial soils could pose an issue during construction from the presence of water in excavations. Dewatering of the excavations may be required in certain areas.
The southern terrain has appreciable quantities of conglomerate material. The majority of this conglomerate can typically be excavated with conventional equipment, however, there is the potential that some areas are heavily cemented and may pose difficult during excavation.

It is anticipated that excavations for the proposed waterline will be in undocumented fill, colluvial/alluvial soils, formational soils or weathered granitic rock. Excavation of the fill and colluvial/alluvial soils should be possible with appropriately sized conventional equipment such as backhoes. Excavation of the Friars Formation can be excavated with moderate effort, but some heavy ripping may be necessary in high cemented zones. The weathered granitic rock encountered in the borings appear to be rippable. It may have variable excavation effort ranging from moderate in the weakly cemented layers to moderate/heavy effort in the highly cemented layers. The contractor should plan for appropriate excavation equipment and methods.

5.2.3 Foundation Subgrade Preparation and Over Excavation

Excavations will range from approximately 2 feet to 20 feet, where necessary. The maximum vault depth of proposed structure is anticipated to be approximately 20 feet at Site 5. Based on the anticipated excavation depths, shoring will be necessary to reduce potential for caving of proposed excavations. The contractor should develop appropriate shoring means and methods and should follow OSHA guidelines. Recommendations for temporary excavation slopes and temporary shoring are provided in Section 5.2.4 and Section 5.2.5, respectively.

All foundation subgrades should be in a dense, unyielding condition prior to foundation installation. Based on our field investigation, subgrades of proposed structure foundations will be formational material and weathered granitic rock for most of sites. Sites 6, and 8may consist of loose artificial fill or alluvial deposits with relatively low penetration blow counts that are not considered suitable for the support of proposed improvements. Loose materials encountered at the foundation level should either be removed to competent material, or removed and re-compacted to a minimum depth of 2 feet below foundation subgrades.. Sites 1, 7, and 10 are likely to encounter dense alluvial materials at proposed bearing subgrade elevations. These materials are considered suitable for support of proposed foundations; however, they are generally made up of clayey sands. Should high plasticity clays be encountered at the foundation subgrade, these materials may damage foundations due to the potential for expansion and should be removed and replaced with compacted, granular structural fill.

Prior to placing any fill for overexcavated foundation subgrades, the exposed subgrade materials should be scarified to a depth of at least 8 inches, moisture conditioned, and compacted to a minimum of 90 percent of the maximum density per ASTM D1557. Any expansive soils observed within the upper three feet of finished subgrade within proposed structure should be removed and replaced with non-expansive soils and compacted to 90 percent.

Table 5-1: Approximate Depth of Temporary Excavation

Site No.	Boring No.	Approx. STA	Anticipated Depth (ft)	Anticipated Subgrade Material	Groundwater Depth (ft)
1	B-1	303+00	14	Alluvium	-
2	B-2	345+00	14	Alluvium	12½
3	-	610+50	12	Friars Formation	-
4	B-4	401+00	8	Friars Formation	-
5	B-5	434+50	18	Friars Formation	-
6	B-6	1+50	8	Alluvium	-
7	B-7	65+00	13	Alluvium	-
8	B-8	439+20	5	Fill	9½
9	B-9	520+00	20	Stadium Conglomerate	-
10	B-10	239+50	14	Fill / Alluvium	-
11	B-11	188+00	13	Decomposed Granite	-

The following locations are of particular interest as they are to include excavations and construction within either loose fill or alluvial soils, and/or have the presence of potentially saturated soil conditions.

Site 2 (STA. 345+00) / Site 8 (STA.439+20): Site 2 is underlain by alluvial deposits with relatively high blow counts to approximately 19 feet. The invert depth of the existing pipe is approximately 10 feet and the excavated condition is anticipated to consist of mostly very dense to hard sands. Fat clay soil was encountered below the sand at a depth of 14 feet. Should future construction include a permanent structure at this location, it is recommended that the clay be over excavated and replaced with engineering fill.

Groundwater was encountered at approximately 12½ feet bgs for Site 2 and 9½ feet bgs for Site 8. Water intrusion flows could occur in the proposed excavations, which could require dewatering methods/equipment as described in Section 5.2.6. Excavation should be rapidly proceeded as feasible, to limit the time temporary excavations are remaining open. The installation of proposed structures may be impacted by groundwater or wet soil conditions within the sites including Site 2 and Site 8. If the excavation will be conducted in wet conditions, the exposed foundation subgrade should be protected with a rat (mud) slab to facilitate the installation of waterproofing and foundation construction.

Site 6 (STA. 1+50) / Site 8 (STA. 439+20): Sites 6 and 8 are underlain by fill and alluvial deposits with relatively low penetration blow counts to the proposed invert depth. We recommend these loose deposits to be removed to a minimum depth of 2 feet, with the exposed foundation subgrade compacted to a firm and unyielding condition. A 12-inch-thick gravel leveling course may be used to support the vaults in lieu of replacing and re-compacting the excavated material. The gravel should be completely wrapped with a non-woven geofabric. For Site 10, loose sand fill soil is present to a depth of 9 feet, above the alluvium. The anticipated loose soil condition should be considered in the shoring / temporary excavation design.

5.2.4 Temporary Excavation Slopes

As mentioned, the maximum depth of temporary excavations is expected up to 20 feet for portions of the proposed construction. Excavations should be cut to a stable slope inclination or be temporarily braced, depending on the excavation depths and the subsurface conditions encountered. The contractor is responsible for all temporary slopes and trenches excavated at the project sites and the design of any required temporary shoring. Should other excavation alternatives be considered as design progresses, Kleinfelder can provide subsequent recommendations as needed.

Temporary construction slopes and shoring should be designed in compliance with OSHA standards to provide temporary trench stability during construction. The maximum vertical height for an unbraced excavation is approximately 4 feet. Construction slopes should be closely observed for signs of mass movement by the contractor’s competent person: tension cracks near the crest, bulging at the toe, etc. If potential stability problems are observed, the contractor should make appropriate modifications to the excavation and/or shoring systems to provide the required trench wall stability. The responsibility for excavation safety and stability of temporary construction slopes lies solely with the contractor.

If very steep or vertical-sided excavations in excess of 5 feet deep are necessary, we recommend that temporary trench excavations should be laid back or shored in accordance with the California Occupational Safety and Health Administration (Cal-OSHA), Caltrans, and any other applicable regulations.

The contractor should be responsible for the structural design and safety of the temporary shoring system, and we recommend that this design be reviewed by Kleinfelder for conformance with our recommendations.

For preliminary planning of OSHA sloping and shoring requirements, we recommend that fill, saturated materials, and alluvial deposits materials be considered as Type C soils and that native formational materials be considered as Type B soils. The actual OSHA soil type should be determined by the contract's "competent person" based on conditions exposed in the field.

Stockpiles should be placed at least 5 feet away from the edge of the excavation and their height should be controlled so they do not surcharge the sides of the excavation. Surface drainage should be carefully controlled to prevent flow of water into the excavations.

Measures should be taken to stabilize the vertical sides and/or sloping face of the excavations. Such measures may include interception and diversion of surface water, placing a fabric over the material and/or bracing the sides of the excavation. If side slopes are not stabilized, reworking of the excavation should be anticipated.

5.2.5 Dewatering

For Site 2 and Site 8, greater amounts of groundwater flows may occur. Appropriate dewatering methods such as well points may likely be supplemented by sumps and pumps at the bottom of the excavation. An experienced dewatering specialist should be consulted for designing the dewatering system to facilitate the planned excavation if groundwater is encountered during excavation. The system should be designed to lower the immediate water surface to an appropriate depth to maintain the excavation bottom stability. Groundwater should be maintained at this elevation until construction has progressed above the surrounding static groundwater level, and sufficient structural dead load and backfill have been placed to counteract hydrostatic uplift pressures. The system should operate continuously until the construction process has progressed above the surrounding static water table surface.

The dewatering consultant should provide recommendations regarding the effects of the dewatering on adjacent structures. This is a critical task since the proposed improvements are located in a residential area of the County surrounded by many developments. Additionally, the excavation sequence, design, and dewatering should consider environmental activities such as required onsite treatment and disposal or storage of groundwater at/from the site.

5.2.6 Structural Fill

The onsite materials may be used as structural fill, provided they are free of oversized rock, organic materials, deleterious debris, and clay soils. Oversize material in excess of 3 inches in diameter should not be used in engineered fill. Although the optimum lift thickness for fill soils will be dependent on the size and type of compaction equipment utilized, fill should generally be placed in uniform lifts not to exceed approximately 8 inches in loose thickness. The onsite soil placed as engineered fill should be moisture conditioned between 1 and 3 percent above optimum moisture content and compacted to a minimum of 90 percent relative compaction based on ASTM D 1557. In pavement and exterior flatwork areas, the upper 12 inches of subgrade soils should be moisture conditioned to a moisture content above optimum content and compacted to 95 percent of the maximum laboratory dry density, as evaluated by ASTM D 1557, just prior to placement of aggregate base.

If import soil is needed for this project, we recommend the material be clean, granular material that has less than 30 percent passing the #200 sieve, a minimum R-value of 30, and a low expansion index (Expansion Index of 30 or less) as evaluated by ASTM D 4829 (Expansion Index Test). Imported structural fill should be moisture conditioned between 0 to 3 percent above optimum moisture content.

In addition, import material should exhibit a low corrosivity potential. Low corrosivity material is defined as having a minimum resistively of more than 2,000 ohm-cm, a chloride content less than 200 parts per million (ppm), and a sulfate content less than 0.05 percent when tested in accordance with California Tests 643, 422, and 417, respectively unless defined otherwise by the corrosion consultant. Import material should be evaluated by the geotechnical consultant at the borrow site for its suitability as fill prior to importation to the project site.

5.2.7 Pipe Bedding and Backfill

We recommend the pipe be supported on 6 inches of granular bedding materials such as sand, gravel, or crushed aggregate with a sand equivalent (SE) of not less than 30 in accordance with ASTM D2914. If crushed rock is used, a filter fabric should be provided to completely envelope the pipe bedding and pipe zone backfill. Filter fabric should be a non-woven material approved by the geotechnical consultant and should be overlapped 1 foot or more.

Bedding should be extended the full width of the trench for the entire pipe zone, which is the zone from the bottom of the trench to 1 foot above the top of the pipe. We recommend the proposed waterline be supported on 6 or more inches of granular bedding material. Granular pipe bedding should be provided to distribute vertical loads around the pipe. The trench excavations should provide 6 to 8 inches of lateral clearance between the trench wall and the side of the utilities.

The pipe bedding should be densified to 90 percent of the ASTM D 1557 relative compaction prior to backfilling. Densification of pipe bedding should be accomplished by mechanical methods, such as pneumatic hand operated equipment, and not by jetting with water.

Backfill material above the bedding zone may consist of on-site or imported soils. If on-site soils are used, these soils will require moisture conditioning and possibly segregation of oversize materials and potentially expansive clayey soils. Oversize material is defined as 4 inches and may consist of cobbles, boulders, and highly cemented concretions or formational chunks which do not break down with compactive effort.

Backfill placed above the bedding zone should be placed in uniform layers not exceeding 8 inches in loose thickness, moisture conditioned to 1 to 3 percent above optimum, and mechanically compacted. The relative compaction should be 90 percent of the ASTM D 1557 maximum density. If the location of any pipe zone is within paved areas or areas subject to traffic loading, we recommend the upper 12 inches of the finish subgrade be compacted to 95 percent.

5.3 VAULTS

5.3.1 Vault Foundations

Proposed vaults at sites 1, 3, 5, 7, and 10 should be supported on firm, undisturbed alluvium, formational materials, or compacted structural fill as recommended in Section 5.2.6. Any loose soils should be removed prior to placing concrete. A 12" thick gravel leveling course may be used to support the vaults. The gravel should be completely wrapped with a non-woven geofabric.

It is our understanding that proposed footings for vaults at Sites 1, 7, and 10 are to be constructed at depths of about 14 ft bgs bearing on dense alluvial materials. For vault foundations bearing entirely in dense alluvial materials, an allowable bearing capacity of 4,000 pounds per square foot (psf) may be used for design of vault foundations. Note that at these sites, the alluvial materials are primarily made up of clayey sands. If highly plastic sands are encountered at the foundation subgrades, the subgrade should be overexcavated a minimum of 2 feet below subgrade and replaced with properly compacted structural fill. For vault foundations bearing entirely in Friars formation or granitic rock, such as those at Sites 3 and 5, an allowable bearing capacity of 6,000 psf may be used. Should the proposed depths of the vault foundations be shallower than anticipated and do not bear on competent material, or if over-excavation is performed and engineered fill is placed within the footing subgrades, an allowable bearing capacity of 3,000 psf may be used for vault foundations bearing entirely on engineered fill. Vault foundations should bear entirely in one unit type to minimize differential settlements. Total settlements for shallow foundations designed using these recommended allowable bearing pressures are estimated to be less than approximately 1 inch.

Lateral loads may be resisted by lateral passive pressures and/or frictional resistance along the base of vault foundations. Allowable lateral passive pressure values as provided in Table 5-2 may be used for sides of footings embedded in fill, alluvium, or formational or rock materials to a maximum value of ten times the passive pressures values provided in Table 5-2. An allowable base friction coefficient of 0.4 may be used for vaults bearing on formational or rock materials or a gravel leveling pad. An allowable base friction coefficient of 0.3 may be used for vaults bearing on fill and alluvial materials. The allowable passive pressure and friction coefficient values were estimated using a safety factor of 1.5. The allowable passive pressure and frictional resistance can be combined to resist lateral loads. If not using passive pressures to resist lateral loads, the ultimate base friction coefficient may be used.

Footings may experience a reduction in bearing capacity or an increased potential to settle when located in close proximity to existing or future utility trenches. Furthermore, stresses imposed by the footings on the utility lines may cause cracking, collapse, and/or a loss of serviceability. To reduce this risk, utility excavations should not extend below a 2H:1V plane projected downward from 12 inches above the bottom of the outside edge of the footing. Also, no parallel utility excavations should be made within a lateral distance of 18 inches outside the footing.

Prior to placing reinforcing steel or concrete, footing excavations should be cleaned of all debris, loose or soft soil, and water. All footing excavations should be observed by the project geotechnical engineer immediately prior to placement of reinforcing steel and concrete to check that the recommendations contained herein are implemented during construction. The structural design of shallow footings should be designed by a licensed structural engineer in the state of California for the specific loading conditions.

5.3.2 Lateral Earth Pressures for Vaults

The at-rest and passive lateral earth pressures presented in Table 5-2 may be used for permanent design of the proposed vault structures using a triangular pressure distribution. Active earth pressures presented in Table 5-2 may be used for temporary shoring design for the proposed excavations for these improvements using the pressure distributions as shown on Figure 5. The earth pressures presented in Table 5-2 do not include surcharges. Table 5-2 does include hydrostatic pressures for use in vault design as well as where below groundwater. If surcharges are present, such as those from nearby foundations or traffic, the applicable active or at-rest earth pressure coefficient should be multiplied by the surcharge loading for application of a uniform lateral pressure distribution due to surcharges. Due to close proximity to existing roadways, we recommend a traffic surcharge load of 100 psf be included.

Table 5-2: Recommended Lateral Earth Pressure

Site No.	Approx. STA	Material Type[1]	Soil Unit Weight, γ_t (pcf)	Friction Angle, Φ (°)	Active Lateral Earth Pressures			At-Rest Lateral Earth Pressures			Passive Lateral Earth Pressures			Seismic Earth Pressure
					K_a	Drained	Undrained	K_o	Drained	Undrained	K_p	Drained	Undrained	
						EFW (psf/ft)	EFW (psf/ft)		EFW (psf/ft)	EFW (psf/ft)		EFW (psf/ft)	EFW (psf/ft)	EFW (psf/ft)
1	303+00	Alluvium	126	34	0.28	36	80	0.44	56	90	2.30	285	140	11.6
		Decomposed Granite	130	38	0.24	31	78	0.38	50	88	2.82	360	170	11.0
2	345+00	Alluvium	125	33	0.29	37	81	0.46	57	91	2.29	290	140	11.0
3[2]	610+50	Friars	130	34	0.28	37	82	0.44	57	92	2.30	300	150	12.3
4	401+00	Friars	115	35	0.27	31	77	0.43	49	85	2.41	275	125	10.4
5[3]	434+50	Friars	125	36	0.30	38	81	0.51	64	94	2.73	340	160	10.4
6	1+50	Alluvium	115	30	0.33	38	80	0.5	58	89	1.93	220	110	11.8
7	65+00	Alluvium	129	32	0.31	40	83	0.47	61	94	2.13	270	130	12.5
8	439+20	Alluvium	120	28	0.36	43	83	0.53	64	93	2.21	270	125	12.7
		Friars	125	35	0.27	34	79	0.43	53	89	2.41	300	145	11.3
9	520+00	Stadium Cong.	125	36	0.26	32	79	0.41	52	88	2.51	315	150	10.7

Site No.	Approx. STA	Material Type[1]	Soil Unit Weight, γ_t (pcf)	Friction Angle, Φ (°)	Active Lateral Earth Pressures			At-Rest Lateral Earth Pressures			Passive Lateral Earth Pressures			Seismic Earth Pressure
					K_a	Drained	Undrained	K_o	Drained	Undrained	K_p	Drained	Undrained	
						EFW (psf/ft)	EFW (psf/ft)		EFW (psf/ft)	EFW (psf/ft)		EFW (psf/ft)	EFW (psf/ft)	EFW (psf/ft)
10	239+50	Fill	115	28	0.36	41	81	0.53	61	90	1.87	215	105	12.1
		Alluvium	130	34	0.28	37	82	0.44	57	92	2.30	300	150	11.9
11[5]	187+35	Fill	115	28	0.36	41	81	0.53	61	90	1.87	215	105	11.3
		Decomposed Granite	130	38	0.24	31	78	0.38	50	88	2.82	370	175	10.3

Notes:
[1]Geological material type and contact depths are provided in Table 3-2.
[2]Site 3 was completed as geologic mapping, no boring was advanced at this location. Soil parameters based on our experience with similar soil conditions.
[3]Values provided for Site 5 are for an adjacent slope of 4H:1V. Lateral earth pressures for all other sites assume level ground conditions.
[4]Passive resistances (Kp and passive EFW values) in the table are allowable values based on a safety factor of 1.5 and should not exceed 10 times the passive pressure value provided. Passive resistance values are presented as allowable.
[5]Site 11 lateral earth pressure zone of influence is within the level ground. Should temporary shoring or vault design require placement within 15' of the present backslope, revised earth pressures may be necessary.

It is our understanding that vaults will be designed for hydrostatic pressures, therefore undrained conditions are provided in the table. Additionally, Sites 2 and 8 encountered groundwater at 12½ and 9 ½ ft bgs, respectfully, therefore hydrostatic conditions at these locations should be used where applicable. Waterproofing should be used where hydrostatic pressures are used below design groundwater elevations.

Permanent retaining walls greater than 6 feet in retained height should also be designed to resist dynamic earth pressures from earthquake loading. A triangular earth pressure distribution may be used for the seismic earth pressure with the resultant force acting at 1/3H above the bottom of the wall. For restrained wall conditions, the maximum value of either the seismic incremental force added to the active earth pressure or the at-rest earth pressure may be used for seismic design.

Should vaults be designed to not include hydrostatic pressures, a proper wall drainage system should be used. Although some of the site did not encounter groundwater in the anticipated excavation depths, build-up of hydrostatic pressures behind the wall can occur due to landscape drainage, leaking or ruptured nearby water lines, stormwater infiltration, or other sources of infiltrated water if proper wall drainage is not used. Wall backfill materials should be free draining and provisions should be made to collect and remove excess water that may accumulate behind earth retaining structures. Wall drainage may be provided by free-draining gravel surrounded by non-woven synthetic filter fabric or by prefabricated, synthetic drainage panels. In either case, drainage should be collected by perforated pipes at the base of the wall and directed to a storm drain, sump, or other suitable location for disposal away from the wall.

All retaining wall backfill should be placed and compacted in accordance with the recommendations provided in Section 5.2.6 of this report. During grading and backfilling adjacent to any retaining walls, heavy equipment should not be allowed to operate within a lateral distance of 5 feet from the wall, or within a lateral distance equal to the wall height, whichever is greater, to avoid overstressing the wall. Within this backfill zone, only hand-operated equipment, such as “whackers”, vibratory plates, or pneumatic compactors, should be used to compact backfill soils.

5.4 TEMPORARY SHORING

If sloped excavations are not feasible due to existing nearby improvements, right-of-way limits, or other constraints, excavations for the proposed structures should be supported by temporary shoring systems. It is our experience that cantilever shoring with conventional methods such as soldier beam and lagging or sheet piles is typically limited to about 15 feet deep in level ground conditions without large surcharges. Larger or more closely spaced soldier beams or king piles may increase the cantilever height but are more costly. We understand that the proposed improvements will require excavation depths on the order of 10 to 20 feet bgs, braced or tieback systems will likely be required. The selection of the shoring method and design is ultimately the decision of the contractor. Figure 5 provides lateral earth pressure diagrams for cantilever and shoring systems with one row of tiebacks/bracing. Hydrostatic pressures are also shown on Figure 5 and should be used where shoring systems extend below groundwater such as is anticipated at Sites 2 and 8. Furthermore, lateral pressures from surcharges, such as nearby construction stockpiles or permanent foundation loads, shall be added to the lateral pressures for temporary shoring. Site and material-specific active lateral earth pressure coefficients (Ka) are provided in Table 5-2 and may be used in conjunction with the temporary lateral earth pressure diagrams presented in Figure 5 for temporary shoring conditions. Material layering and respective depths for temporary shoring earth pressures may be referenced from Table 3-2.

The selected shoring system should extend to a sufficient depth below the excavation bottom to provide the required lateral resistance. An equivalent fluid unit weight of the values provided in Table 5-2 specific to site location may be used for allowable passive pressure against soldier or sheet piles that extend below the level of excavation. If soldier piles are used, to account for three-dimensional effects, the passive pressure may be assumed to act on an area 2 times the width of the embedded portion of the pile, provided adjacent piles are spaced at least 3 pile diameters center-to-center. Additionally, we recommend a factor of safety of 1.2 be applied to the calculated embedment depth and that the passive pressure be limited to 10 times the values provided in Table 5-2.

Lateral movement of a shored excavation will depend on the type and relative stiffness of the system used, the workmanship of the contractor, and other factors beyond the scope of this study. The shoring engineer should design the system to limit lateral movements and settlements so that effects on adjacent structures, existing utilities and pipes to remain in place, and other existing site improvements are minimized.

In addition to monitoring of existing structures, horizontal and vertical movements of the shoring system should also be monitored by a licensed surveyor. The construction monitoring and performance of the shoring system are ultimately the contractor's responsibility. At a minimum, we recommend that the tops of the soldier beams be surveyed prior to excavation and that the top and bottom of the soldier beams be surveyed on a weekly basis until the foundation is completed. The soldier beams should be surveyed at spacings no greater than 50 feet on-center.

5.5 PRELIMINARY CORROSIVE SOIL SCREENING

Preliminary laboratory corrosive soil screening of the on-site soils was performed on five samples from Sites 1, 3, 5, 8, and 10 from our investigation. The corrosion tests were performed on bulk samples to evaluate the potential effect of the subsurface materials on concrete and ferrous metals. The results of the testing are presented in Appendices B and are summarized in Table 5-3 below.

Table 5-3: Corrosion Test Results

Sample Site	Boring	pH	Minimum Resistivity (ohm-cm)	Sulfate (ppm)	Chloride (ppm)
1	B-1	7.4	320	180	330
3	-	5.3	3200	39	21
5	B-5	7.2	4500	<30	21
8	B-8	8.3	850	69	96
10	B-10	7.5	4400	36	11

The minimum electrical resistivity tests performed are variable with two samples considered to be severely corrosive and four samples considered moderately corrosive to buried unprotected metal objects based on the 1984 National Association of Corrosion Engineers (NACE) correlation between soil resistivity and corrosivity towards unprotected ferrous metals presented in Table 5-4.

Table 5-4: Electrical Resistivity and Unprotected Ferrous Metals Corrosion Potential

Minimum Electrical Resistivity (ohm-cm)	Corrosion Potential
0 to 1,000	Severely Corrosive
1,000 to 2,000	Corrosive
2,000 – 10,000	Moderately Corrosive
Over 10,000	Mildly Corrosive

In general, Caltrans (2015) considers the subsurface conditions at a site to be aggressive to below-grade concrete if one or more of the following conditions exist for representative soil samples taken at a site: chloride concentration is 500 parts per million (ppm) or greater, sulfate concentration is 2,000 ppm or greater, or the pH is 5.5 or less.

The Portland Cement Association (1988), correlates sulfate content to sulfate attack potential as presented in Table 5-5.

Table 5-5: Sulfate Attack Potential

Sulfate Content (ppm)	Sulfate Attack Potential
0 to 1,000	Negligible
1,000 to 2,000	Moderate
2,000 – 20,000	Severe
Over 20,000	Very Severe

Based on the Caltrans Corrosion Guidelines and Portland Cement Association criteria, the exposure of concrete to sulfate attack from the onsite soils is considered to be negligible by the PCA standards. We recommend that Type II or V cement should be used for concrete structures in contact with soil.

Our corrosion tests are from widely spaced borings and are considered preliminary in nature. Additional sampling and testing may be performed during construction. A qualified corrosion engineer may be retained to evaluate the general corrosion potential with respect to the specific construction materials to be used at this site.

6 ADDITIONAL SERVICES

6.1 CONSTRUCTION OBSERVATION AND TESTING

To effectively achieve the intent of the geotechnical recommendations presented in this report and to maintain continuity from design through construction, Kleinfelder should be retained to provide observation and testing services during construction. This will provide Kleinfelder with the opportunity to observe the subsurface conditions encountered during construction, evaluate the applicability of the geotechnical recommendations presented in our report as they relate to the soil conditions encountered, and to provide follow up recommendations if conditions differ from those described in our report.

7 LIMITATIONS

This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder's profession practicing in the same locality, under similar conditions and at the date the services are provided. Our conclusions, opinions, and recommendations are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated. Kleinfelder makes no other representation, guarantee, or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided. The scope of our services did not include any environmental assessment or exploration for the presence of hazardous or toxic materials in the soil, surface water, groundwater or air, on, below or around this site.

This report may be used only by the Client and the registered design professional in responsible charge and only for the purposes stated for this specific engagement within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report. Land use, site conditions (both on-site and off-site), regulations, or other factors may change over time, and additional work may be required with the passage of time. Any party other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party and client agrees to defend, indemnify and hold harmless Kleinfelder from any claim or liability associated with such unauthorized or non-compliance.

The work performed was based on project information provided by Client. If Client does not retain Kleinfelder to review any plans and specifications, including any revisions or modifications to the plans and specifications, Kleinfelder assumes no responsibility for the suitability of our recommendations. In addition, if there are any changes in the field to the plans and specifications, Client must obtain written approval from Kleinfelder's engineer that such changes do not affect our recommendations. Failure to do so will vitiate Kleinfelder's recommendations.

8 REFERENCES

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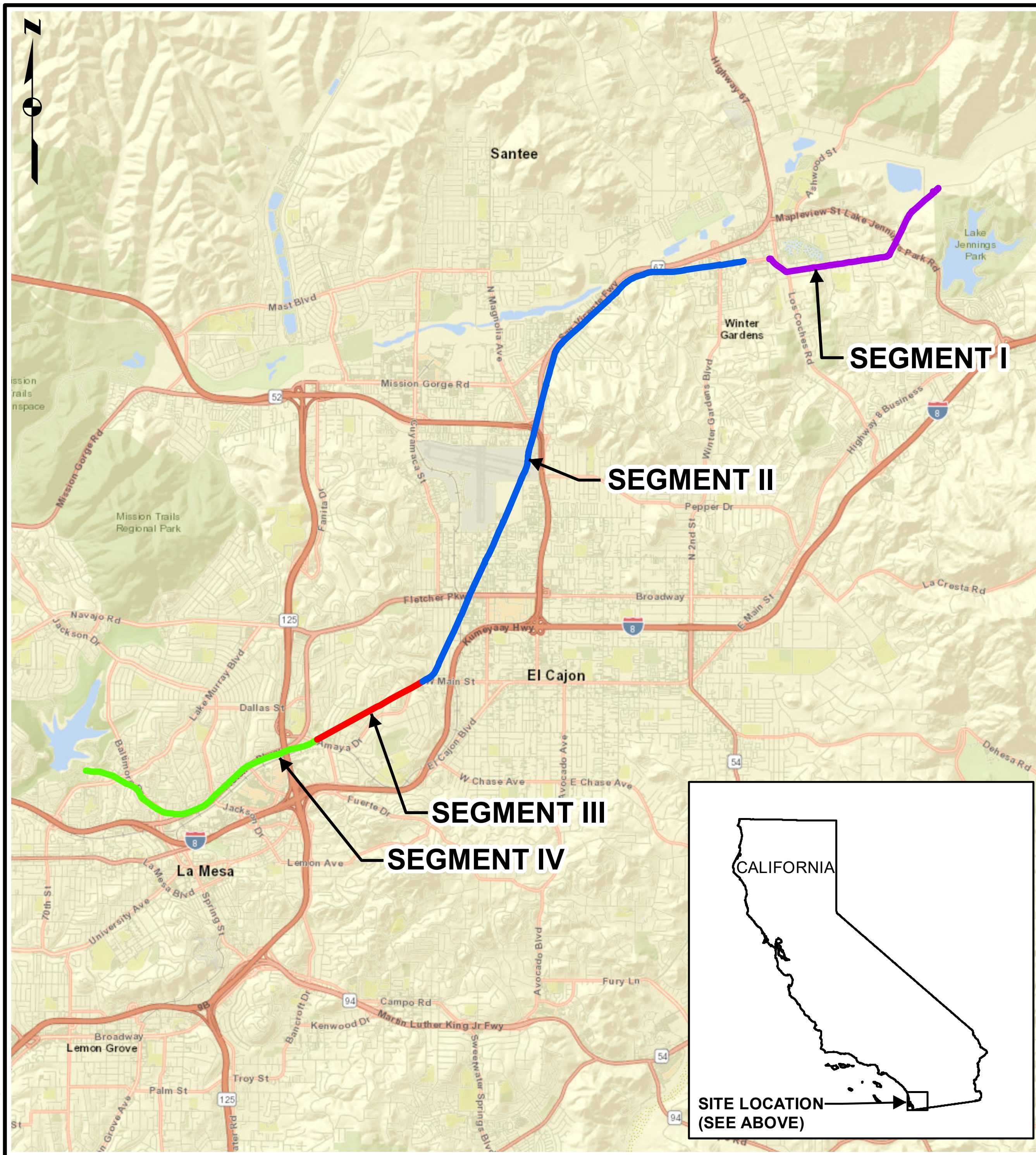
FIGURES



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SITE VICINITY MAP

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FIGURE

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EXPLORATION MAP

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FIGURE

2a

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FIGURE

2b

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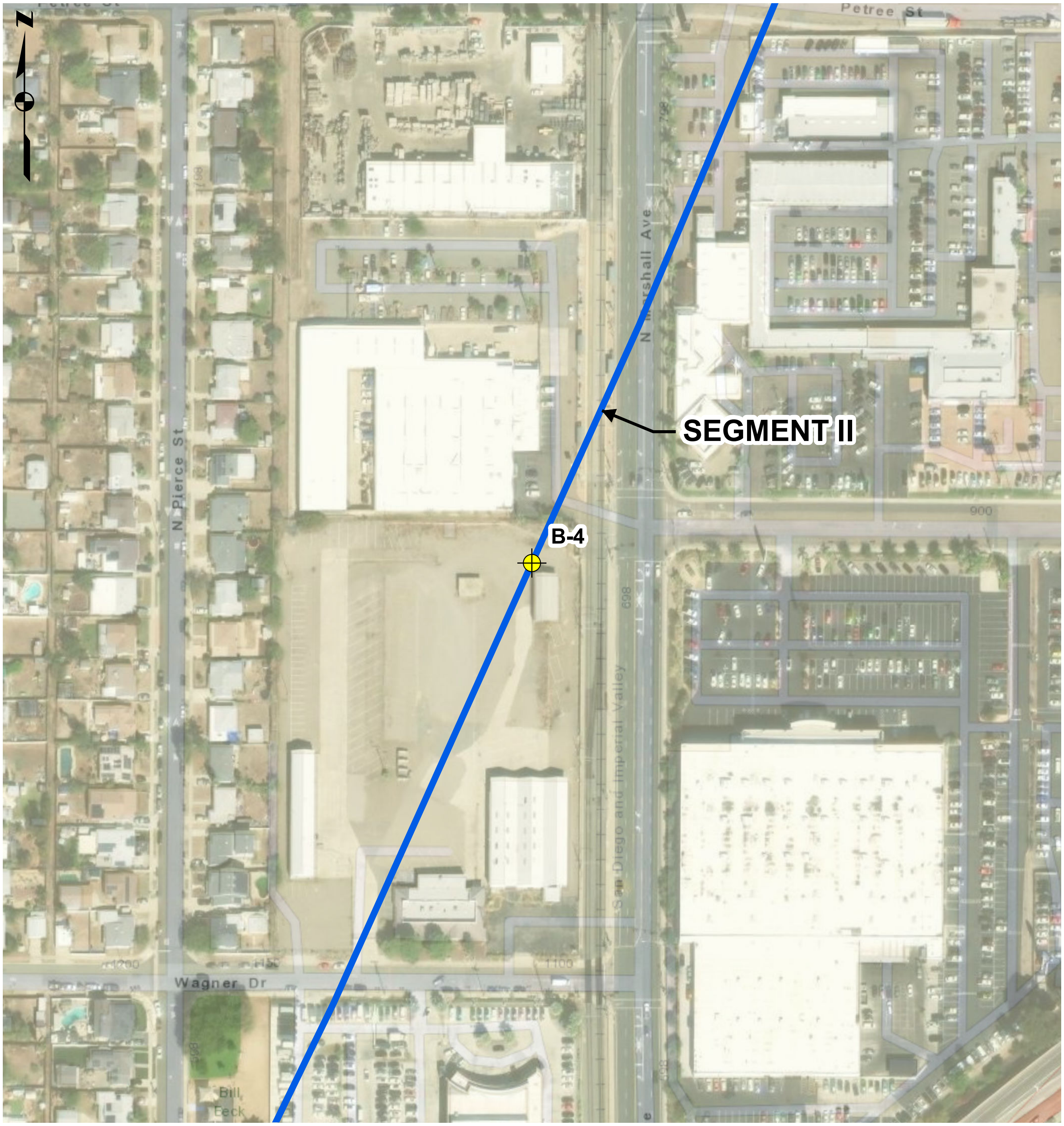

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FIGURE

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EXPLORATION MAP

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FIGURE
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FIGURE

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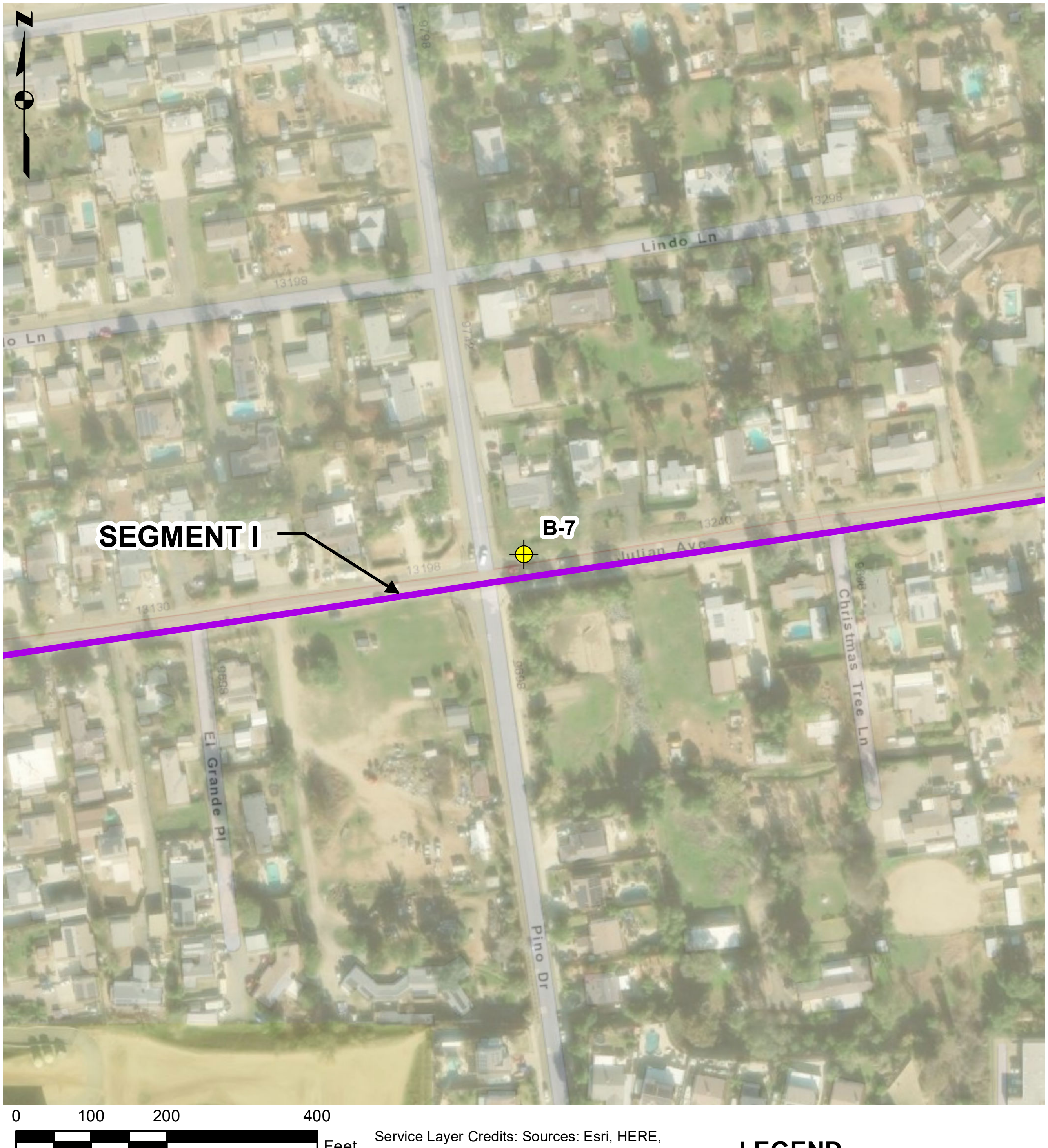

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FIGURE

2f

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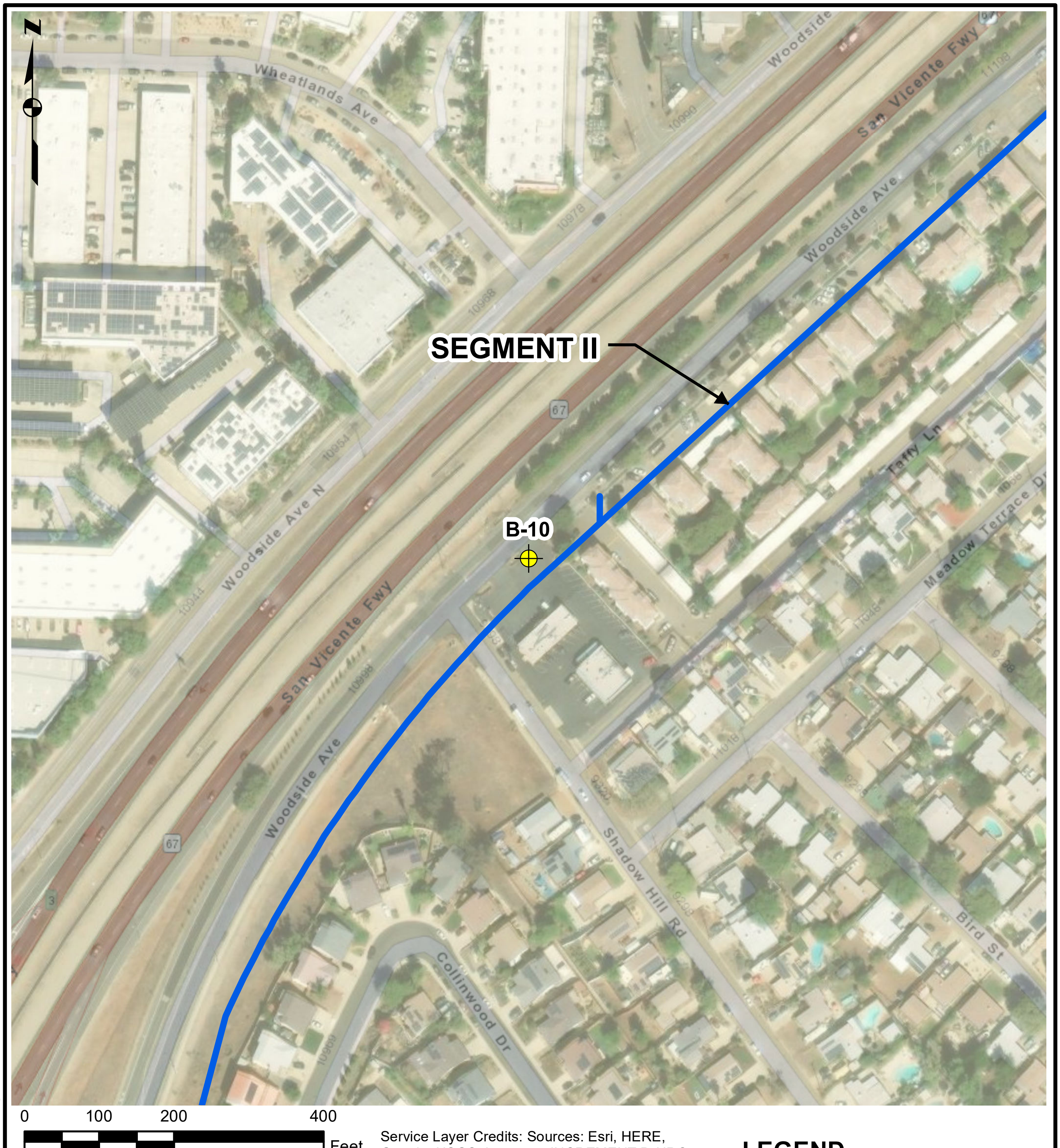

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EXPLORATION MAP

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FIGURE
2i

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FIGURE
2j

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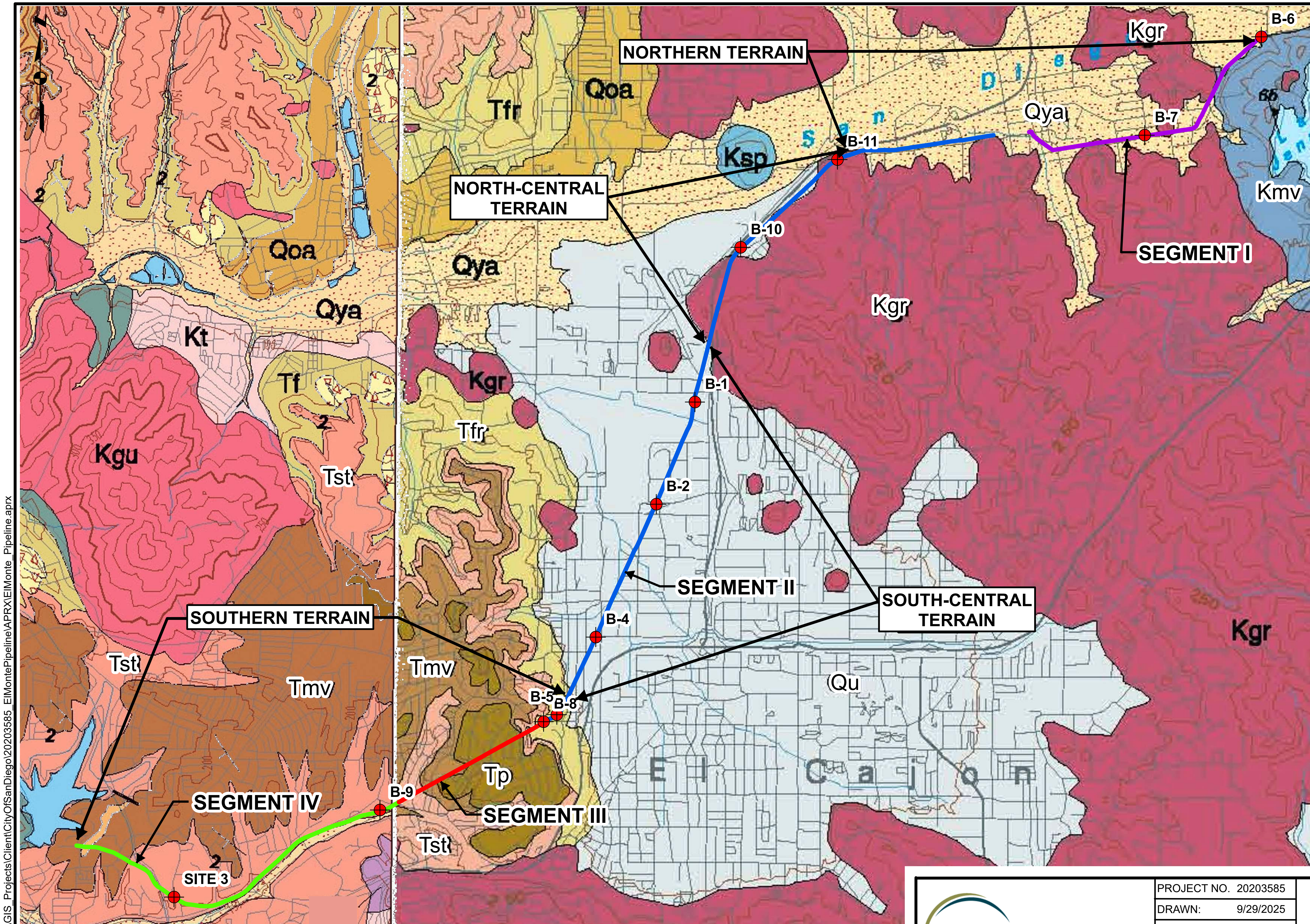

LEGEND

- Qya - YOUNG ALLUVIUM
- Qu - ALLUVIUM AND COLLUVIUM, UNDIVIDED
- Tp - POMERADO CONGLOMERATE
- Tmv - MISSION VALLEY FORMATION
- Tst - STADIUM CONGLOMERATE
- Tfr - FRIARS FORMATION
- Kgr - GRANITOID ROCKS
- Kmv - METAVOLCANIC ROCKS
- APPROXIMATE SOIL SAMPLE LOCATION

0 850 1,700 3,400 5,100 6,800 8,500 Feet

SOURCE:
GEOLOGIC MAP OF THE SAN DIEGO 30' X 60' QUADRANGLE, CALIFORNIA; BY KENNEDY AND TAN; 2005; AND THE PRELIMINARY GEOLOGIC MAP OF THE EL CAJON 30' X 60' QUADRANGLE, SOUTHERN CALIFORNIA; BY TODD; 2004

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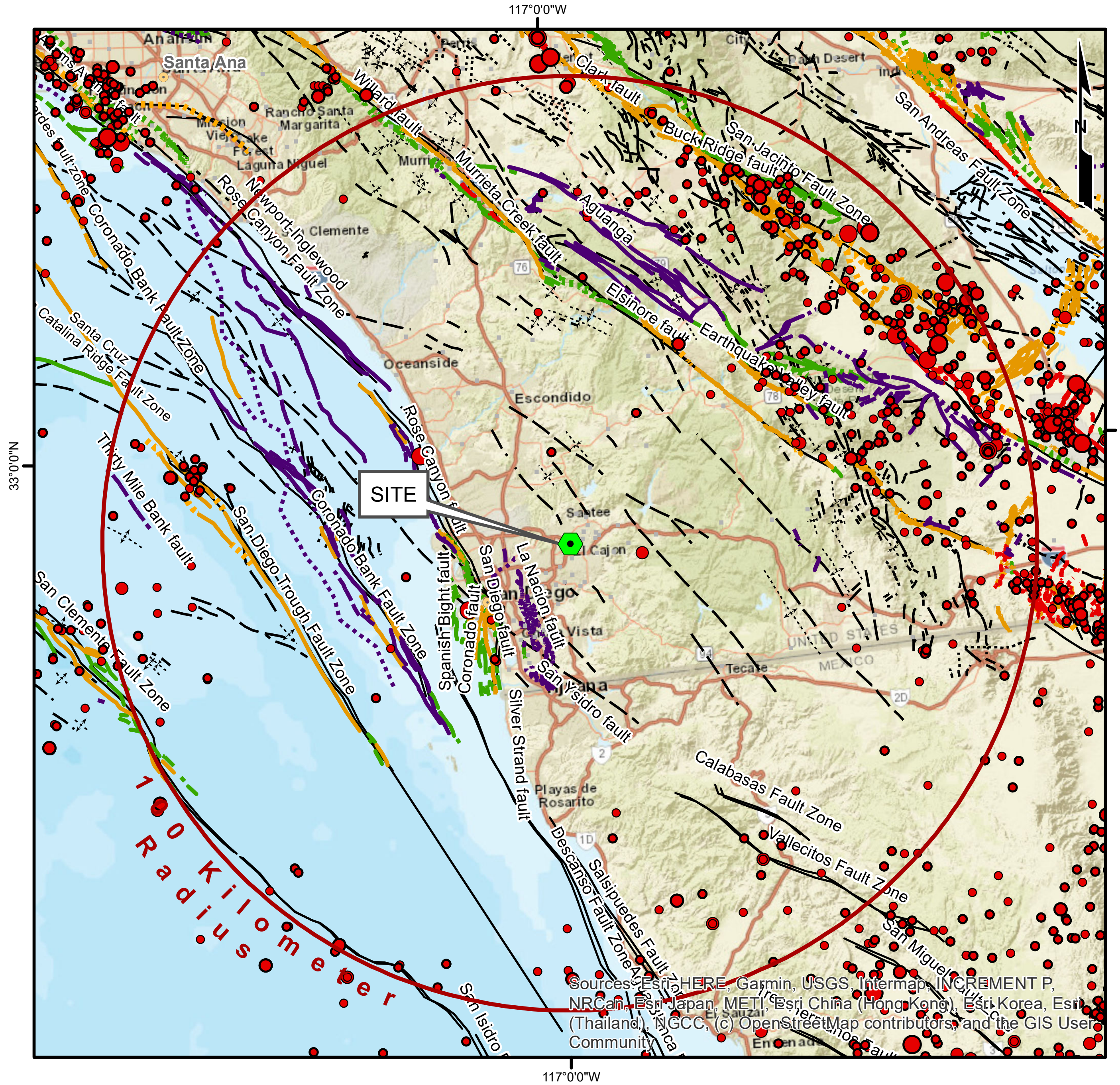
PROJECT NO.	20203585
DRAWN:	9/29/2025
DRAWN BY:	JP/RA
CHECKED BY:	AR
FILE NAME:	ElMonte_Pipeline.mxd

REGIONAL GEOLOGY MAP

GEOLOGIC STUDY / GEOTECHNICAL ENGINEERING REPORT
DESIGN OF THE EL MONTE TRANSMISSION PIPELINE REHABILITATION
COUNTY OF SAN DIEGO
CITIES OF SANTEE, EL CAJON, LA MESA, AND SAN DIEGO, CALIFORNIA

FIGURE
3



Faulting Legend

Quaternary Faults (Bryant, 2005; USGS, 2018)

Historic displacement (< 150 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Holocene displacement (< 15,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Late Quaternary displacement (< 750,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Quaternary & unspecified displacement (< 1,600,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Pre-Quaternary Geologic Structures (CGS, 2000)
- fault, approx. located
- fault, approx. located, queried
- fault, certain
- fault, concealed
- fault, concealed, queried
- fault, inferred, queried

ANSS Earthquakes

Magnitude
- 4.0 - 4.9
- 5.0 - 5.9
- 6.0 - 6.9
- 7.0 - 7.9
- 8.0 - 8.9

35 17.5 0 Kilometers

\\azrgisstorp03\GIS_Projects\Client\CityOfSanDiego\20203585_ElMontePipeline\MXD\20203585_EQ.mxd

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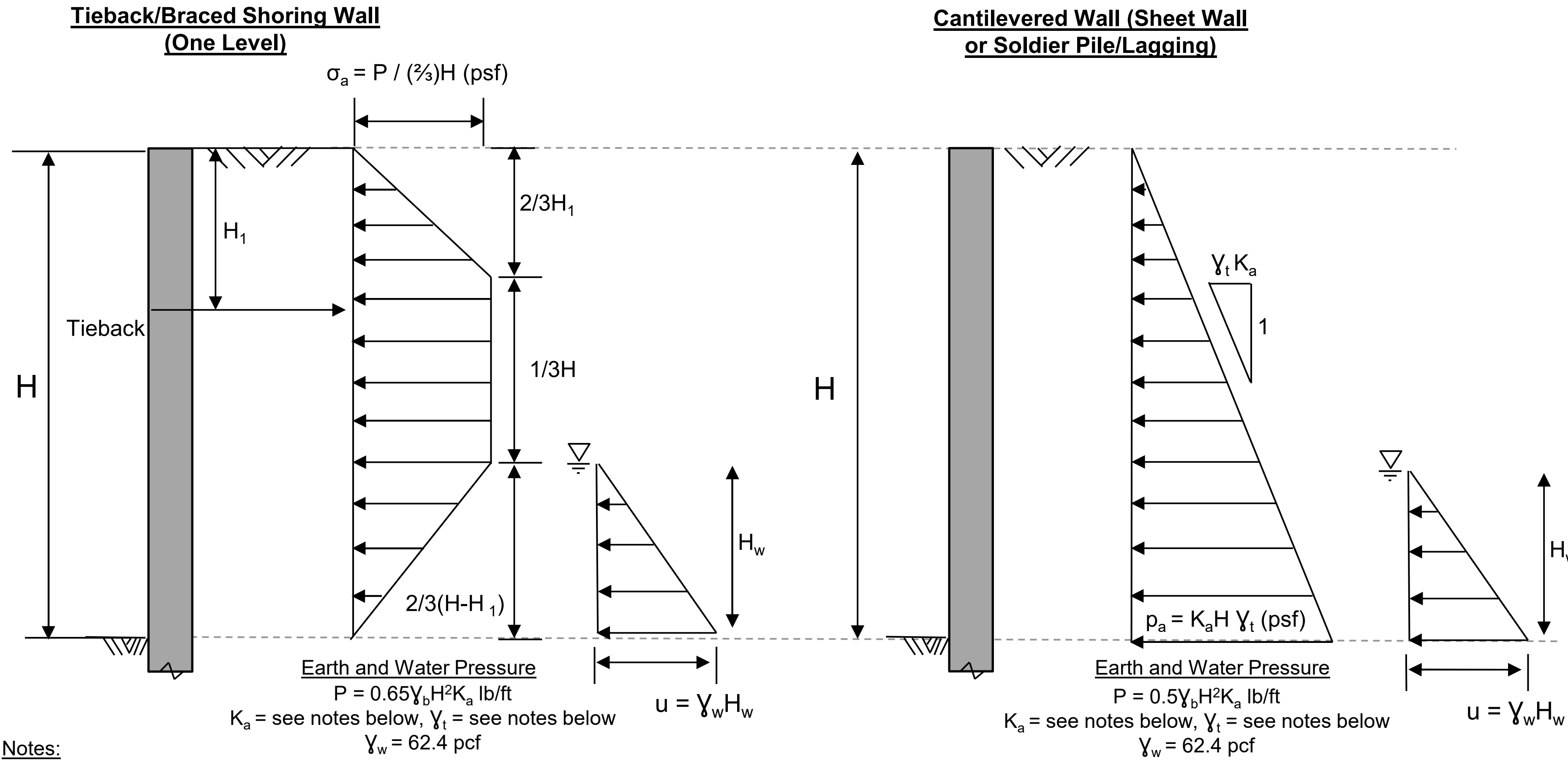
PROJECT NO.	20203585
DRAWN:	8/3/2023
DRAWN BY:	JP
CHECKED BY:	SF
FILE NAME:	20203585_EQ.MXD

REGIONAL FAULT MAP AND EARTHQUAKE EPICENTERS (1800 - APRIL 2023)

GEOLOGIC STUDY / GEOTECHNICAL ENGINEERING REPORT
DESIGN OF THE EL MONTE TRANSMISSION PIPELINE REHABILITATION
COUNTY OF SAN DIEGO
CITIES OF SANTEE, EL CAJON, LA MESA, AND SAN DIEGO, CALIFORNIA

FIGURE
4



Notes:

1) The earth pressures shown should be used in conjunction with the recommendations of Kleinfelder's Geotechnical Investigation Report for the project dated October 2023.
2) The earth pressures shown are based on the 2011 Caltrans Trenching and Shoring Manual assuming cantilever and one level of tieback anchors or bracing used for the shoring wall and level backslope conditions.
3) The H shown herein is the height of the retained soil for each soil type in units of feet. The H_1 is the distance from the top of wall to the tieback anchor in units of feet. Site specific depths of material types are provided in Table 3-2 of the geotechnical report. Site and material specific K_a and total unit weight (γ_t) values are provided in Table 5-2 of the geotechnical report
4) Active and passive pressures should be applied to the unexposed portion of the wall in accordance with Caltrans Memo to Designers 5-12.
5) Pressure diagrams do not include surcharge pressures. Surcharge pressures should be added to the pressure diagrams provided herein in accordance with the recommendations provided in the geotechnical report.
6) The wall configuration and lateral earth pressures shown are not to scale.
7) Buoyant unit weight (γ_b) is equal to the saturated unit weight minus the unit weight of water. A design groundwater depth of 12.5 and 9.5 feet bgs should be used for Site 2 and 8, respectively. All other sites for the project are not anticipated to encounter groundwater within the depth of the excavations.

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PROJECT NO.	20203585.001A
DATE:	9/26/2023
BY:	SF
CHECKED:	RT
File Name:	Tieback Wall.ppt

LATERAL EARTH PRESSSURE DIAGRAMS - HYDROSTATIC

El Monte Water Pipeline Rehabilitation
County of San Diego

FIGURE

5

APPENDIX A
FIELD EXPLORATION PROGRAM

APPENDIX A
FIELD EXPLORATION PROGRAM

Kleinfelder conducted the field exploration for this study between May 5, 2022 to June 28, 2023 by drilling 10 borings ranging from approximately 10 to 20 feet below ground surface. The borings were performed with a truck mounted (Marl M-10), track-mounted (Fraste PLG), and tripod drill rigs using continuous flight and hollow-stem auger techniques. Soil samples were obtained by California split barrel sampler lined with 6-inch-long stainless-steel sleeves. The Standard and California samplers were driven a distance of 18-inches (or until "refusal") in general conformance with ASTM D6066 and ASTM D1586. The sampler is driven into the bottom of the boring over an 18-inch sampling interval by a 140-pound hammer that is dropped a distance of 30 inches. The total number of hammer blows required to drive the sampler the final 12 inches is termed the blow count and is recorded on the Logs of Borings.

A Kleinfelder staff engineer coordinated the field exploration activities, logged the soil conditions exposed in the borings and collected soil samples for laboratory testing. Soil was classified in the field according to the Unified Soil Classification System (USCS) using the visual-manual procedure in accordance with ASTM D 2488. Field descriptions and classifications were reviewed against the laboratory descriptions (ASTM D2487) and adjusted where laboratory data was available.

The Logs of Borings describe the earth materials encountered, samples obtained, and show field and laboratory tests performed. The logs also show the approximate elevation, boring number, drilling date, and the names of the logger and drilling subcontractor. Elevations for boring locations noted on the boring logs are approximate and are based on the provided topographic plan. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Bulk and intact samples of representative earth materials were obtained from the borings. Upon their completion, the borings were backfilled with soil cuttings and patched with cold patch asphalt, where necessary.

The locations of all subsurface explorations (Figures 2a through 2j) are approximate and were estimated by our engineer. A Unified Soil Classification System (USCS) chart and a Boring Log Legend are presented as Figures A-1 and A-2, respectively. The Logs of Borings are presented as Figures A-3 through A-12.

SAMPLE/SAMPLER TYPE GRAPHICS

- BULK SAMPLE
- CALIFORNIA SAMPLER (3 in. (76.2 mm.) outer diameter)
- GRAB SAMPLE
- STANDARD PENETRATION SPLIT SPOON SAMPLER (2 in. (50.8 mm.) outer diameter and 1-3/8 in. (34.9 mm.) inner diameter)

GROUND WATER GRAPHICS

- WATER LEVEL (level where first observed)
- WATER LEVEL (level after exploration completion)
- WATER LEVEL (additional levels after exploration)
- OBSERVED SEEPAGE

NOTES

- The report and graphics key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report.
- Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual or differ from those shown.
- No warranty is provided as to the continuity of soil or rock conditions between individual sample locations.
- Logs represent general soil or rock conditions observed at the point of exploration on the date indicated.
- In general, Unified Soil Classification System designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index property testing.
- Fine grained soils that plot within the hatched area on the Plasticity Chart, and coarse grained soils with between 5% and 12% passing the No. 200 sieve require dual USCS symbols, ie., GW-GM, GP-GM, GW-GC, GP-GC, GC-GM, SW-SM, SP-SM, SW-SC, SP-SC, SC-SM.
- If sampler is not able to be driven at least 6 inches then 50/X indicates number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

ABBREVIATIONS

WOH - Weight of Hammer
WOR - Weight of Rod

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	**GRAVELS** (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVEL WITH <5% FINES	$Cu \geq 4$ and $1 \leq Cc \leq 3$	**GW**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			$Cu < 4$ and/ or $1 > Cc > 3$	**GP**	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5% TO 12% FINES	$Cu \geq 4$ and $1 \leq Cc \leq 3$	**GW-GM**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			$Cu < 4$ and/ or $1 > Cc > 3$	**GP-GM**	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GP-GC	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH > 12% FINES		**GM**	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
				GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (Half or more of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	$Cu \geq 6$ and $1 \leq Cc \leq 3$	**SW**	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			$Cu < 6$ and/ or $1 > Cc > 3$	**SP**	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH 5% TO 12% FINES	$Cu \geq 6$ and $1 \leq Cc \leq 3$	**SW-SM**	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			$Cu < 6$ and/ or $1 > Cc > 3$	**SP-SM**	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SP-SC	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH > 12% FINES		**SM**	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
				SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES

FINE GRAINED SOILS (Half or more of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)	**ML**	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid Limit 50 or greater)	**MH**	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

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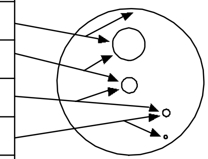
PROJECT NO.: 20203585.001A		GRAPHICS KEY	FIGURE
DRAWN BY:	AB	El Monte Water Pipeline Rehabilitation County of San Diego	A-1
CHECKED BY:	SF		
DATE:	8/2/2023		

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12 in. (304.8 mm.)	>12 in. (304.8 mm.)	Larger than basketball-sized
Cobbles		3 - 12 in. (76.2 - 304.8 mm.)	3 - 12 in. (76.2 - 304.8 mm.)	Fist-sized to basketball-sized
Gravel	coarse	3/4 -3 in. (19 - 76.2 mm.)	3/4 -3 in. (19 - 76.2 mm.)	Thumb-sized to fist-sized
	fine	#4 - 3/4 in. (#4 - 19 mm.)	0.19 - 0.75 in. (4.8 - 19 mm.)	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19 in. (2 - 4.9 mm.)	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079 in. (0.43 - 2 mm.)	Sugar-sized to rock salt-sized
	fine	#200 - #40	0.0029 - 0.017 in. (0.07 - 0.43 mm.)	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029 in. (<0.07 mm.)	Flour-sized and smaller



SECONDARY CONSTITUENT

	AMOUNT	
Term of Use	Secondary Constituent is Fine Grained	Secondary Constituent is Coarse Grained
Trace	<5%	<15%
With	≥5 to <15%	≥15 to <30%
Modifier	≥15%	≥30%

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT - N_{60} (# blows / ft)	Pocket Pen (tsf)	UNCONFINED COMPRESSIVE STRENGTH (Q_u)(psf)	VISUAL / MANUAL CRITERIA
Very Soft	<2	PP < 0.25	<500	Thumb will penetrate more than 1 inch (25 mm). Extrudes between fingers when squeezed.
Soft	2 - 4	0.25 ≤ PP <0.5	500 - 1000	Thumb will penetrate soil about 1 inch (25 mm). Remolded by light finger pressure.
Medium Stiff	4 - 8	0.5 ≤ PP <1	1000 - 2000	Thumb will penetrate soil about 1/4 inch (6 mm). Remolded by strong finger pressure.
Stiff	8 - 15	1 ≤ PP <2	2000 - 4000	Can be imprinted with considerable pressure from thumb.
Very Stiff	15 - 30	2 ≤ PP <4	4000 - 8000	Thumb will not indent soil but readily indented with thumbnail.
Hard	>30	4 ≤ PP	>8000	Thumbnail will not indent soil.

FROM TERZAGHI AND PECK, 1948; LAMBE AND WHITMAN, 1969; FHWA, 2002; AND ASTM D2488

REACTION WITH HYDROCHLORIC ACID

DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT-N_{60} (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	<4	<5	0 - 15
Loose	4 - 10	5 - 12	5 - 15	15 - 35
Medium Dense	10 - 30	12 - 35	15 - 40	35 - 65
Dense	30 - 50	35 - 60	40 - 70	65 - 85
Very Dense	>50	>60	>70	85 - 100

FROM TERZAGHI AND PECK, 1948

PLASTICITY

DESCRIPTION	LL	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm.) thread cannot be rolled at any water content.
Low (L)	< 30	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	30 - 50	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit.
High (H)	> 50	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit.

STRUCTURE

DESCRIPTION	CRITERIA
Stratified	Alternating layers of varying material or color with layers at least 1/4-in. thick, note thickness.
Laminated	Alternating layers of varying material or color with the layer less than 1/4-in. thick, note thickness.
Fissured	Breaks along definite planes of fracture with little resistance to fracturing.
Slickensided	Fracture planes appear polished or glossy, sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness.

ANGULARITY

DESCRIPTION	CRITERIA
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces.
Subangular	Particles are similar to angular description but have rounded edges.
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges.
Rounded	Particles have smoothly curved sides and no edges.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20203585.001A DRAWN BY: AB CHECKED BY: SF DATE: 8/2/2023	SOIL DESCRIPTION KEY El Monte Water Pipeline Rehabilitation County of San Diego	FIGURE A-2

PLOTTED: 08/03/2023 10:19 AM BY: ABriggs
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PROJECT NUMBER: 20203585.001A
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BORING LOG B-1

Date Begin - End:	5/02/2022	**Drilling Company:**	Pacific Drilling
Logged By:	A. Rekani	**Drill Crew:**	Gerardo & Robert
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	Fraste Multidrill XL
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger
Weather:	sunny	**Exploration Diameter:**	6 in. O.D.

Hammer Type - Drop: 140 lb. Auto - 30 in.

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 32.82856°
Longitude: -116.96387°
Approximate Ground Surface Elevation (ft.): 397.00
Location Offset: Magnolia Ave & Kenney St
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	0	**Alluvium (Qal)** **Silty Clayey SAND (SC-SM)**: fine to medium-grained with few coarse grained sand, medium plasticity, dark reddish brown, moist, micaceous	S1				SC-SM			100	34	22	7	**pH=** 7.4 **Resistivity=** 320 **Sulfates=** .018 **Chlorides=** .033
395														
	5	- becomes light brown to dark reddish brown, dense	S2		BC=8 13 27	18"								Hand auger to 5 ft **Direct Shear=** Peak Cohesion: 233 ksf Peak Friction Angle: 46.2°
390														Increase in density
	10	- becomes very dense, some iron oxide staining	S3		BC=12 50/6"	12"		12.6	107.0					
385														
	15	- subangular particles, trace mafic minerals	S4		BC=6 26 48	18"		13.5	116.4					
380		**Decomposed Granitic Rock (Kgr)** **Silty SAND (SM)**: fine to medium-grained with few coarse grained sand, non-plastic, dry, very dense, micaceous, iron oxide staining, subangular grains	S5		BC=24 50/4"	10"		11.7	139.5					
	20													
375														

The boring was terminated at approximately 19.5 ft. below ground surface. The boring was backfilled with auger cuttings on May 02, 2022.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Google Earth.

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PROJECT NO.: 20203585.001A		BORING LOG B-1	FIGURE A-3
DRAWN BY:	SF	El Monte Water Pipeline Rehabilitation County of San Diego	
CHECKED BY:	SHR		
DATE:	8/2/2023		PAGE: 1 of 1

Date Begin - End:	12/09/2023	Drilling Company:	Pacific Drilling		BORING LOG B-2
Logged By:	S. Floyd	Drill Crew:	Roby & Ryan		
Hor.-Vert. Datum:	WGS84	Drilling Equipment:	Tripod Rig	Hammer Type - Drop:	140 lb. Cathead - 30 in.
Plunge:	-90 degrees	Drilling Method:	Continuous Flight Auger		
Weather:	sunny	Exploration Diameter:	6 in. O.D.		

Latitude: 32.81781°
Longitude: -116.96858°
Approximate Ground Surface Elevation (ft.): 387.00
Location Offset: W Bradley Ave & Floyd Smith Dr
Surface Condition: Bare Earth

Approximate Elevation (feet)	Depth (feet)	Graphical Log	FIELD EXPLORATION: Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	LABORATORY RESULTS: USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
385			**Alluvium (Qal)** **Clayey SAND (SC)**: fine to medium-grained sand, low plasticity, reddish brown (5YR 4/3), moist, angular gravel, slightly micaceous - yellowish red (5YR 5/6)	S1			60"								Hand auger to 1' bgs, hard
380	5		**Silty SAND (SM)**: fine to medium-grained sand, low plasticity, reddish yellow (7.5YR 6/6), moist, very dense, iron oxide staining, subangular gravel, slightly micaceous	S2		BC=16 40 50/4"	16"		5.3	112.7					
375	10		**Poorly Graded SAND with Silt and Clay (SP-SC)**: medium-grained sand, low plasticity, dark reddish brown (5YR 3/4), moist, very dense, moderately cemented, iron oxide staining, subangular gravel, mottled, slightly micaceous	S3		BC=48 50/6"	12"		12.0	120.4					Resistance change
370	15		**Fat CLAY (CH)**: fine-grained sand, high plasticity, pale olive (5Y 6/4), wet, hard, iron oxide staining	S4		BC=25 32 32 PP=4.0	18"		18.1	113.0			34	17	
			- high plasticity, pale yellow (5Y 7/4), wet, hard, iron oxide staining, some red streaking, slightly micaceous	S5		BC=38 50/5" PP=3.0	10"								Hole cave following CAL
365	20														

The boring was terminated at approximately 19.5 ft. below ground surface. The boring was backfilled with auger cuttings and bentonite on December 09, 2023.

GROUNDWATER LEVEL INFORMATION:
Groundwater was observed at approximately 12.5 ft. below ground surface during drilling.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Google Earth.

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PROJECT NO.: 20203585.001A	BORING LOG B-2	FIGURE
DRAWN BY: SF	El Monte Water Pipeline Rehabilitation County of San Diego	A-4
CHECKED BY: SHR		
DATE: 8/2/2023		PAGE: 1 of 1

gINT FILE: Klf_gint_master_2020 PROJECT NUMBER: 20203585.001A OFFICE FILTER: SAN DIEGO PLOTTED: 08/03/2023 01:49 PM BY: ABriggs
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2020.GLB [_KLF_BORING/TEST PIT SOIL LOG]

BORING LOG B-4

Date Begin - End:	9/23/2022	**Drilling Company:**	Pacific Drilling
Logged By:	S. Rugg	**Drill Crew:**	Gerardo & Robert
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	Fraste Multidrill XL
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger
Weather:	sunny	**Exploration Diameter:**	6 in. O.D.

Hammer Type - Drop: 140 lb. Auto - 30 in.

Latitude: 32.80381°
Longitude: -116.97599°
Approximate Ground Surface Elevation (ft.): 433.00
Location Offset: Wagner Dr & N Marshall Ave
Surface Condition: Gravel

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			GRAVEL: (6 inches)												
			<u>Artificial Fill (af)</u> **Clayey SAND with Gravel (SC)**: medium to coarse-grained sand, gray (10Y 5/1), moist, medium dense												
430				S1		BC=16 7 5 PP=1.5		SC	8.3	126.0	100	65			
			<u>Friars Formation (Tf)</u> **Lean CLAY (CL)**: high plasticity, dark greenish gray (10Y 4/1), moist, stiff, homogenized												
	5			S3		BC=5 11 17 PP=3.5							58	37	
			CLAYSTONE excavates as Lean Clay (CL): fine-grained sand, medium to high plasticity, light greenish gray (10Y 7/1), moist, stiff to very stiff												
425				S4		BC=11 15 17 PP=3.5			29.0	88.1					
	10			S5		BC=8 12 19 PP=3.0			24.5	94.7					
420															
	15		- hard	S6		BC=12 21 43 PP=5.0									
415				S7		BC=13 17 40 PP=4.5									
	20														
410															

The boring was terminated at approximately 20 ft. below ground surface. The boring was backfilled with auger cuttings on September 23, 2022.

<u>GROUNDWATER LEVEL INFORMATION:</u>
Groundwater was not observed during drilling or after completion.

<u>GENERAL NOTES:</u>
The exploration location and elevation are approximate and were estimated by Google Earth.

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PROJECT NO.: 20203585.001A		BORING LOG B-4	FIGURE
DRAWN BY:	SF	El Monte Water Pipeline Rehabilitation County of San Diego	A-5
CHECKED BY:	SHR		
DATE:	8/2/2023		PAGE: 1 of 1

Date Begin - End:	5/02/2022	**Drilling Company:**	Pacific Drilling		**BORING LOG B-5**
Logged By:	A. Rekani	**Drill Crew:**	Gerardo & Robert		
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	Fraste Multidrill XL	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	overcast	**Exploration Diameter:**	6 in. O.D.		

FIELD EXPLORATION

Latitude: 32.79564°
Longitude: -116.98077°
Approximate Ground Surface Elevation (ft.): 562.00
Location Offset: W Main St & Dewane Ave
Surface Condition: Bare Earth

LABORATORY RESULTS

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			Topsoil: dark grayish brown	S1				CH			100	57	55	30	**pH=** 7.2 **Resistivity=** 4500 **Sulfates=** <0.003 **Chlorides=** 0.002 Auger chatter on cobbles
560			**Friars Formation (Tf)** **Fat CLAY (CH)**: fine to medium-grained, high plasticity, light brown, moist, very stiff, scattered cobbles												
	5		- hard	S2		BC=17 20 27	NR								
555															
	10		**Lean CLAY with Sand (CL)**: fine to medium-grained, light brown to light reddish brown, trace black mineralization	S3		BC=15 22 28	18"								
550															
	15		- becomes light brown, dry	S4		BC=17 32 50/5"	17"		19.7	101.6					
545															
			- becomes light olive brown	S5		BC=21 30 50/5"	17"		22.1	104.8					
	20														
540															

The boring was terminated at approximately 20 ft. below ground surface. The boring was backfilled with auger cuttings on May 02, 2022.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Google Earth.

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PROJECT NO.: 20203585.001A		BORING LOG B-5	FIGURE
DRAWN BY:	SF	El Monte Water Pipeline Rehabilitation County of San Diego	A-6
CHECKED BY:	SHR		
DATE:	8/2/2023		PAGE: 1 of 1

Date Begin - End:	12/09/2022	**Drilling Company:**	Pacific Drilling		**BORING LOG B-6**
Logged By:	S. Floyd	**Drill Crew:**	Roby & Ryan		
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	Fraste Multidrill XL	**Hammer Type - Drop:**	140 lb. Cathead - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	sunny	**Exploration Diameter:**	8 in. O.D.		

Latitude: 32.86728°
Longitude: -116.89372°
Approximate Ground Surface Elevation (ft.): 440.00
Location Offset: 14042 El Monte Rd
Surface Condition: Bare Earth

Approximate Elevation (feet)	Depth (feet)	Graphical Log	FIELD EXPLORATION: Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	LABORATORY RESULTS: USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			Young Alluvium (Qya) **Clayey SAND (SC)**: fine to medium-grained sand, low plasticity, dark reddish brown (2.5YR 3/3), moist, subangular gravel, micaceous	S1				SC			97	42			Easy consistent resistance
435	5		- reddish brown (2.5YR 4/3), loose, iron oxide staining	S2		BC=4 5 5	18"		12.6	101.0					
			- reddish brown (5YR 4/3), subangular to subrounded gravel, micaceous	S3		BC=5 5 7	18"		9.0	107.4					
430	10														Resistance increase
			- dark grayish brown (10YR 4/2), medium dense, iron oxide staining, subrounded gravel, micaceous	S4		BC=8 15 15	18"								
425	15														Resistance increase
420	20		**Sandy SILT (ML)**: fine-grained sand, low plasticity, grayish brown (2.5Y 5/2), moist, hard, moderately cemented, subrounded gravel, micaceous, slight organics	S5		BC=16 32 39 PP=1.5	18"								

The boring was terminated at approximately 20 ft. below ground surface. The boring was backfilled with auger cuttings on December 09, 2022.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Google Earth.

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PROJECT NO.: 20203585.001A		BORING LOG B-6	FIGURE
DRAWN BY:	SF	El Monte Water Pipeline Rehabilitation County of San Diego	A-7
CHECKED BY:	SHR		
DATE:	8/2/2023		PAGE: 1 of 1

Date Begin - End:	9/21/2022	**Drilling Company:**	Pacific Drilling		**BORING LOG B-7**
Logged By:	S. Floyd	**Drill Crew:**	Miguel & Rory		
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	Yeti M10	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	sunny	**Exploration Diameter:**	6 in. O.D.		

Latitude: 32.85688°
Longitude: -116.90815°
Approximate Ground Surface Elevation (ft.): 426.00
Location Offset: Julian Ave
Surface Condition: Bare Earth

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Sample Number	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
425	0	**Young Alluvium (Qya)** **Clayey SAND (SC)**: fine to medium-grained sand, medium plasticity, olive yellow (2.5Y 6/6), dry, slightly angular gravel, slight iron oxide staining	S1			SC			89	44			
420	5	- medium plasticity, yellowish brown (10YR 5/6), moist, very dense, iron oxide staining, micaceous, subangular gravel	S2	BC=15 24 30	18"		12.5	114.9					Hand auger to 5 ft
415	10	- angular to subangular gravel	S3	BC=16 26 50/6"	18"		11.1	115.0					
410	15	**Silty Clayey SAND (SC-SM)**: fine-grained sand, medium plasticity, olive brown (2.5Y 4/3), moist, dense, iron oxide staining, subangular to subrounded gravel, micaceous	S4	BC=13 14 21	18"								
		Poorly Graded SAND with Silt (SP-SM): fine to coarse-grained sand, low plasticity, moist, dense, iron oxide staining, subangular to rounded gravel, micaceous	S5	BC=5 15 20	18"								
405	20												

The boring was terminated at approximately 20 ft. below ground surface. The boring was backfilled with auger cuttings on September 21, 2022.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Google Earth.

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PROJECT NO.: 20203585.001A		BORING LOG B-7	FIGURE
DRAWN BY:	SF	El Monte Water Pipeline Rehabilitation County of San Diego	A-8
CHECKED BY:	SHR		
DATE:	8/2/2023		PAGE: 1 of 1

PLOTTED: 08/03/2023 01:50 PM BY: ABriggs
OFFICE FILTER: SAN DIEGO
PROJECT NUMBER: 20203585.001A
gINT FILE: Klf_gint_master_2020
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2020.GLB [_KLF_BORING/TEST PIT SOIL LOG]

Date Begin - End:	6/28/2023	**Drilling Company:**	Pacific Drilling		**BORING LOG B-8**
Logged By:	A. Rekani	**Drill Crew:**	Brian & Toby		
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	Sabercat MTXD	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Exploration Diameter:**	6 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 32.79489°
Longitude: -116.98241°
Approximate Ground Surface Elevation (ft.): 561.00
Location Offset: W Main Street
Surface Condition: Bare Earth

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
560	0	**Artificial Fill (af)** **Clayey SAND (SC)**: fine to medium-grained sand, brown (7.5 YR 4/3), moist, mottled with gray (7.5 YR 5/1), abundant gravel up to 3 inches, few cobbles, some roots, few coarse sand grains	S1											**pH=** 8.3 **Resistivity=** 850 **Sulfates=** .007 **Chlorides=** .010 Auger chatter
555	5	- medium plasticity, medium dense, few pieces of gravel up to 2 inches	S2		BC=5 4 6	18"	SC	23.9	95.1	95	41	48	24	
550	10	- wet, very loose	S3		BC=4 2 1	4"								**Direct Shear=** Peak Cohesion: 164.5 psf Peak Friction Angle: 34.7°
545	15	**Friars Formation (Tf)** **Clayey SAND (SC)**: fine to medium-grained sand, low plasticity, very pale brown (10 YR 7/3), moist, very dense, moderately cemented, subangular sand, trace mica, homogenous	S4		BC=19 24 29	18"	SC	17.5	109.2		34			
		Elastic SILT (MH) : medium plasticity, reddish brown (5 YR 4/4), moist, very stiff, moderately cemented, homogeneous, trace fine sand, trace mica	S5		BC=21 24 28	18"	MH	27.6	96.3		67	56	20	
540	20													

The boring was terminated at approximately 20 ft. below ground surface. The boring was backfilled with auger cuttings and bentonite on June 28, 2023.

GROUNDWATER LEVEL INFORMATION:
∇ Groundwater was observed at approximately 10 ft. below ground surface during drilling.
▼ Groundwater was observed at approximately 9.5 ft. below ground surface at the end of drilling.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Google Earth.

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PROJECT NO.: 20203585.001A		BORING LOG B-8	FIGURE
DRAWN BY:	AB	El Monte Water Pipeline Rehabilitation County of San Diego	A-9
CHECKED BY:	AR		
DATE:	8/2/2023		PAGE: 1 of 1

Date Begin - End:	9/21/2022	**Drilling Company:**	Pacific Drilling		**BORING LOG B-9**
Logged By:	S. Floyd	**Drill Crew:**	Miguel & Rory		
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	Yeti M10	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	sunny	**Exploration Diameter:**	6 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 32.78550°
Longitude: -117.00267°
Approximate Ground Surface Elevation (ft.): 569.00
Location Offset: MTS Amaya Ct
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			ASPHALT: (4 inches)												
			BASE: (5.5 inches)												
565			**Artificial Fill (af)** **Silty Clayey SAND with Gravel (SC)**: fine to medium-grained sand, medium plasticity, olive yellow (2.5Y 6/6), moist, subangular to subrounded gravel	S1				SC			89	31			**ASTM D1557 Method B=** Max. Dry Unit Wt.: 133.2 pcf Opt. Water Content: 7.5% **R-Value=** 9
560	5		**Stadium Conglomerate (Tst)** **Silty SAND with Gravel (SM)**: fine to medium-grained sand, low plasticity, olive yellow (2.5Y 4/3)), moist, very dense, subangular to subrounded gravel	S2		BC=28 50/2.9"	NR								Hand auger to 5 ft
555	10			S3		BC=50/6"	3"		6.8	91.4					
	15		fine to coarse-grained sand, non-plastic, olive (5Y 5/4), dry, iron oxide staining, slightly micaceous	S4		BC=50/1"	NR								Grab sample at 15.1 ft
				S5											
				S6		BC=50/4"									SPT at 16 ft
550	20														
545															

The boring was terminated because of practical auger refusal (↑) at approximately 16.5 ft. below ground surface on cobbles. The boring was backfilled with auger cuttings and patched at surface with aquapatch on September 21, 2022.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Google Earth.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20203585.001A	BORING LOG B-9	FIGURE
	DRAWN BY: SF	El Monte Water Pipeline Rehabilitation County of San Diego	A-10
	CHECKED BY: SHR		
	DATE: 8/2/2023		PAGE: 1 of 1

BORING LOG B-10

Date Begin - End:	4/11/2023	**Drilling Company:**	Pacific Drilling
Logged By:	S. Floyd	**Drill Crew:**	Miguel & Gerardo
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	Yeti M10
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger
Weather:	overcast	**Exploration Diameter:**	6 in. O.D.

Hammer Type - Drop: 140 lb. Auto - 30 in.

FIELD EXPLORATION

Latitude: 32.84483°
Longitude: -116.95831°
Approximate Ground Surface Elevation (ft.): 363.00
Location Offset: Woodside & Shadow Hill
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		ASPHALT: (3 inches)												
		Artificial Fill (af) **Clayey SAND (SC)**: fine to medium-grained sand, medium plasticity, very dusky red (2.5YR 2.5/2), moist, subangular gravel	S1				SC	12.6		99	46			**pH=** 7.5 **Resistivity=** 4400 **Sulfates=** .004 **Chlorides=** .001 **ASTM D1557 Method B=** Max. Dry Unit Wt.: 131.2 pcf Opt. Water Content: 8.2% **R-Value=** 40 Hand auger to 5' bgs
360														
	5	- very loose, slight iron oxide staining	S2		BC=2 2 2	15"								
														Direct Shear= Peak Cohesion: 553.5 psf Peak Friction Angle: 41.3°
355														
		Old Alluvium (Qoa)												Increase in resistance
	10	**Clayey SAND (SC)**: fine to coarse-grained sand, low plasticity, dark red (7.5R 3/6), moist, very dense, strongly cemented, calcium carbonate speckling, black mottling, slight yellow brown banding	S3		BC=11 23 49	16"	SC	13.2	124.7	100	34			
350														
	15	- medium plasticity, reddish brown (2.5YR 4/4), mottled calcium carbonate and black tonalite, slight iron oxide staining	S4		BC=14 25 49	15"		15.8	120.1					
345														
		Weathered Granite (Kgt) **Excavates as Silty SAND (SM)**: fine to coarse-grained sand, low plasticity, pale brown (10YR 6/3), moist, very dense, strongly cemented, yellow brown streaking, tonalite grains, calcium carbonate speckling	S5		BC=50/6"	6"								
	20													
340														

The boring was terminated at approximately 19 ft. below ground surface. The boring was backfilled with auger cuttings and patched at surface with aquapatch on April 11, 2023.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Google Earth.

BORING LOG B-11

Date Begin - End:	4/11/2023	**Drilling Company:**	Pacific Drilling		
Logged By:	S. Floyd	**Drill Crew:**	Miguel & Gerardo		
Hor.-Vert. Datum:	WGS84	**Drilling Equipment:**	Yeti M10	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	partly cloudy	**Exploration Diameter:**	6 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 32.85411°
Longitude: -116.94635°
Approximate Ground Surface Elevation (ft.): 434.00
Location Offset: Woodside Terrace
Surface Condition: Bare Earth

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Number	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			Artificial Fill (af) **Silty SAND (SM)**: fine to coarse-grained sand, non-plastic, brown (7.5YR 4/3), moist, iron oxide staining, some clay infills, micaceous	S1				SM	5.8		99	13			Hand auger to 5' bgs Location on fill part of cut/fill slope
430			- increase in fines												
	5		- dark brown (7.5YR 3/3), loose	S2		BC=2 1 5	14"								**Direct Shear=** Peak Cohesion: 943.5 psf Peak Friction Angle: 39.5° Increase in resistance at 8' bgs
425			**Decomposed Granite (Kggd)** **Excavates as Silty SAND (SM)**: fine to medium-grained sand, non-plastic, light olive gray (5Y 6/2), moist, very dense, strongly cemented, slight calcium carbonate, slightly micaceous, slight tonalite minerals												Rig chatter on rock at 9.5' bgs No auger advance at 10' bgs, grab sample of cuttings
	10			S3 S4		BC=50/0"	NR		3.3						
420															
	15														
415															
	20														
410															

The boring was terminated because of practical auger refusal (↑) at approximately 10 ft. below ground surface on granitic rock. The boring was backfilled with auger cuttings on April 11, 2023.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Google Earth.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20203585.001A	BORING LOG B-11	FIGURE
	DRAWN BY: SF CHECKED BY: SHR	El Monte Water Pipeline Rehabilitation County of San Diego	A-12
	DATE: 8/2/2023		PAGE: 1 of 1

APPENDIX B
LABORATORY TESTING PROGRAM

APPENDIX B
LABORATORY TESTING PROGRAM

GENERAL
Laboratory tests were performed on selected, samples as an aid in classifying the soils and to evaluate physical properties of the soils which may affect foundation design and construction procedures. A description of the laboratory testing program is presented below, and the laboratory test results are summarized on Table B-1.

LABORATORY MOISTURE AND DENSITY DETERMINATIONS
Natural moisture content and dry density tests were performed on selected intact samples collected and moisture content was performed on selected disturbed samples. Moisture content was evaluated in general accordance with ASTM Test Method D 2216; dry unit weight was evaluated using procedures similar to ASTM Test Method D 2937. This data is included on the boring logs in Appendix A and on Table B-1.

GRADATION ANALYSIS
Sieve and hydrometer analyses were performed on samples from the site to evaluate the gradation characteristics of the soil and to aid in its classification. The tests were performed in general accordance with ASTM Test Methods D6913 and ASTM D7928 The results of the sieve analyses are shown in Appendix B-4 through B-6, with the fines content summarized on the boring logs.

ATTEBERG LIMITS TEST
Liquid and plastic limits tests were conducted on selected samples. The test was performed in general accordance with ASTM D 4318. In addition, the values are shown on the boring logs in Appendix A and Appendix B-7.

DIRECT SHEAR TEST
Direct shear strength testing was performed on four soil samples. The test procedure was in general accordance with ASTM Test Method D3080. Results are presented in Appendix B-8 through B-11.

SOIL CORROSIVITY TEST
Five samples of the on-site soils were tested for pH, resistivity, soluble sulfates, and soluble chlorides to evaluate the potential for corrosion of concrete and steel. The samples were tested in general accordance with California Test Methods 643, 417 and 422, for pH and resistivity, and soluble sulfates and soluble

chlorides, respectively. A summary of these chemical tests is given in Table B-1 and are also shown on the boring logs in Appendix A.

COMPACTION CURVE

Two tests were performed on representative soil samples to evaluate the laboratory compaction characteristics of the soil using the modified effort for compaction. The test procedure was in general accordance with the ASTM D 1557. The test results are presented on Appendix B-12 and B-13.

R-VALUE

R-value test was performed on two selected soil samples to evaluate resistance value of the near surface soils. The test was performed in general accordance with ASTM Test Method D2844. The results are presented in Table B-1.

Exploration ID	Depth (ft.)	Sample No.	Sample Description	Water Content (%)	Dry Unit Wt. (pcf)	Sieve Analysis (%)			Atterberg Limits			Additional Tests
						Passing 3/4"	Passing #4	Passing #200	Liquid Limit	Plastic Limit	Plasticity Index	
B-1	0.0 - 4.0	S5	SILTY, CLAYEY SAND (SC-SM)			100	100	34	22	15	7	**pH=** 7.4
												Resistivity= 320
												Sulfates= .018
												Chlorides= .033
B-1	6.0	S6	SILTY, CLAYEY SAND (SC-SM)									**Direct Shear=**
												Peak Cohesion: 233 ksf
												Peak Friction Angle: 46.2°
B-1	10.5	S7	SILTY, CLAYEY SAND (SC-SM)	12.6	107.0							
B-1	16.0	S8	SILTY, CLAYEY SAND (SC-SM)	13.5	116.4							
B-1	19.0	S9	SILTY SAND (SM)	11.7	139.5							
B-2	6.0	S2	SILTY SAND (SM)	5.3	112.7							
B-2	10.5	S3	POORLY GRANDED SAND WITH SILTY AND CLAY (SP-SC)	12.0	120.4							
B-2	16.0	S4	FAT CLAY (CH)	18.1	113.0				34	17	17	
SITE 3	0.0											**pH=** 5.3
												Resistivity= 3200
												Sulfates= .004
												Chlorides= .002
B-4	3.5	S1	CLAYEY SAND WITH GRAVEL (SC-SM)	8.3	126.0	100	100	65				
B-4	6.0	S3	FAT CLAY (CL)						58	21	37	
B-4	8.5	S4	LEAN CLAY (CL)	29.0	88.1							
B-4	10.0	S5	LEAN CLAY (CL)	24.5	94.7							
B-5	0.0 - 4.0	S1	SANDY FAT CLAY (CH)			100	100	57	55	25	30	**pH=** 7.2
												Resistivity= 4500
												Sulfates= <0.003
												Chlorides= 0.002
B-5	16.0	S4	LEAN CLAY (CL)	19.7	101.6							
B-5	19.5	S5	LEAN CLAY (CL)	22.1	104.8							
B-6	0.0	S1	SILTY SAND (SC)			100	97	42				

Refer to the Geotechnical Evaluation Report or the supplemental plates for the method used for the testing performed above.
NP = NonPlastic
NA = Not Available

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PROJECT NO.: 20203585.001A
DRAWN BY: AB
CHECKED BY: SF
DATE: 8/2/2023

LABORATORY TEST RESULT SUMMARY

El Monte Water Pipeline Rehabilitation
County of San Diego

TABLE

B-1

Exploration ID	Depth (ft.)	Sample No.	Sample Description	Water Content (%)	Dry Unit Wt. (pcf)	Sieve Analysis (%)			Atterberg Limits			Additional Tests
						Passing 3/4"	Passing #4	Passing #200	Liquid Limit	Plastic Limit	Plasticity Index	
B-6	5.0	S2	SILTY SAND (SC)	12.6	101.0							
B-6	9.0	S3	SILTY SAND (SC)	9.0	107.4							
B-7	0.0 - 5.0	S1	CLAYEY SAND (SC)			96	89	44				
B-7	6.0	S2	CLAYEY SAND (SC)	12.5	114.9							
B-7	11.0	S3	CLAYEY SAND (SC)	11.1	115.0							
B-8	0.0	S1	CLAYEY SAND (SC)									**pH=** 8.3
												Resistivity= 850
												Sulfates= .007
												Chlorides= .010
B-8	5.0	S2	CLAYEY SAND (SC)	23.9	95.1		95	41	48	24	24	
B-8	11.0	S3	CLAYEY SAND (SC)									**Direct Shear=**
												Peak Cohesion: 164.5 psf
												Peak Friction Angle: 34.7°
B-8	15.0	S4	CLAYEY SAND (SC)	17.5	109.2			34				
B-8	18.5	S5	ELASTIC SILT (MH)	27.6	96.3			67	56	36	20	
B-9	0.0	S1	SILTY CLAYEY SAND WITH GRAVEL (SC-SM)									**R-Value=** 9
												ASTM D1557 Method B=
												Maximum Dry Unit Weight: 133.2 pcf
												Optimum Water Content: 7.5%
B-9	1.0	S1	SILTY CLAYEY SAND WITH GRAVEL (SC-SM)			100	89	31				
B-9	10.0	S3	SILTY SAND WITH GRAVEL (SM)	6.8	91.4							

Refer to the Geotechnical Evaluation Report or the supplemental plates for the method used for the testing performed above.
NP = NonPlastic
NA = Not Available

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PROJECT NO.: 20203585.001A
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CHECKED BY: SF
DATE: 8/2/2023

LABORATORY TEST RESULT SUMMARY

El Monte Water Pipeline Rehabilitation
County of San Diego

TABLE
B-2

Exploration ID	Depth (ft.)	Sample No.	Sample Description	Water Content (%)	Dry Unit Wt. (pcf)	Sieve Analysis (%)			Atterberg Limits			Additional Tests
						Passing 3/4"	Passing #4	Passing #200	Liquid Limit	Plastic Limit	Plasticity Index	
B-10	0.0	S1	CLAYEY SAND (SC)									**pH=** 7.5
												Resistivity= 4400
												Sulfates= .004
												Chlorides= .001
												R-Value= 40
												ASTM D1557 Method B=
												Maximum Dry Unit Weight: 131.2 pcf
												Optimum Water Content: 8.2%
B-10	0.25	S1	CLAYEY SAND (SC)	12.6			99	46				
B-10	6.0	S2	CLAYEY SAND (SC)									**Direct Shear=**
												Peak Cohesion: 553.5 psf
												Peak Friction Angle: 41.3°
B-10	11.0	S3	CLAYEY SAND (SC)	13.2	124.7		100	34				
B-10	16.0	S4	CLAYEY SAND (SC)	15.8	120.1							
B-11	0.0	S1	SILTY SAND (SM)	5.8			99	13				
B-11	6.0	S2	SILTY SAND (SM)									**Direct Shear=**
												Peak Cohesion: 943.5 psf
												Peak Friction Angle: 39.5°
B-11	10.0	S3	SILTY SAND (SM)	3.3								

Refer to the Geotechnical Evaluation Report or the supplemental plates for the method used for the testing performed above.
NP = NonPlastic
NA = Not Available

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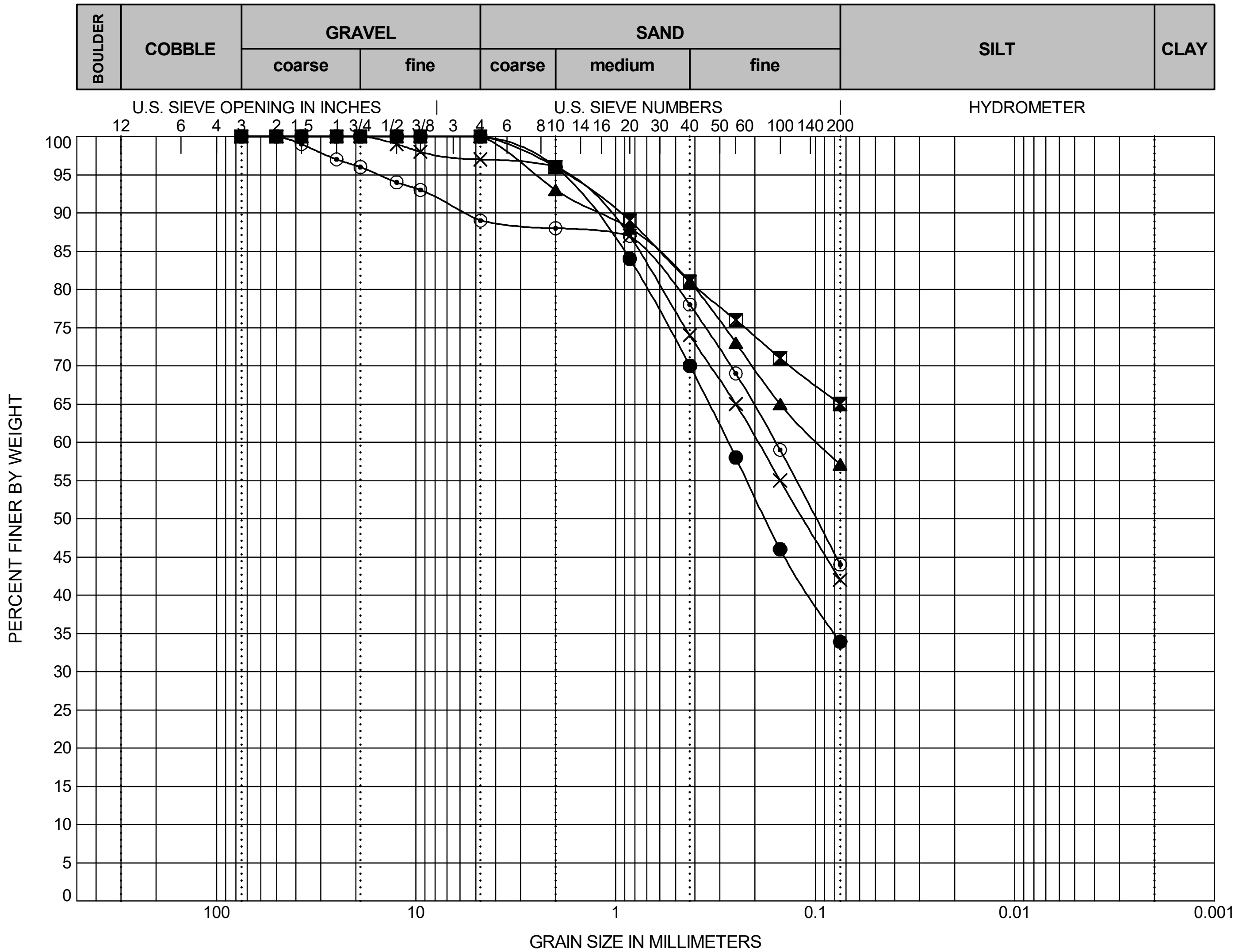
LABORATORY TEST RESULT SUMMARY

El Monte Water Pipeline Rehabilitation
County of San Diego

TABLE
B-3



Exploration ID	Depth (ft.)	Sample Number	Sample Description	LL	PL	PI
● B-1	0 - 4	S-5	SILTY, CLAYEY SAND (SC-SM)	22	15	7
⧗ B-4	3.5 - 4	NA	CLAYEY SAND WITH GRAVEL (SC-SM)	NM	NM	NM
▲ B-5	0 - 4	S1	SANDY FAT CLAY (CH)	55	25	30
× B-6	0 - 4	S1	SILTY SAND (SC)	NM	NM	NM
⊙ B-7	0 - 5	S1	CLAYEY SAND (SC)	NM	NM	NM

Exploration ID	Depth (ft.)	D_{100}	D_{60}	D_{30}	D_{10}	Cc	Cu	Passing 3/4"	Passing #4	Passing #200	%Silt*	%Clay*
● B-1	0 - 4	75	0.273	NM	NM	NM	NM	100	100	34	NM	NM
⧗ B-4	3.5 - 4	75	NM	NM	NM	NM	NM	100	100	65	NM	NM
▲ B-5	0 - 4	75	0.097	NM	NM	NM	NM	100	100	57	NM	NM
× B-6	0 - 4	75	0.194	NM	NM	NM	NM	100	97	42	NM	NM
⊙ B-7	0 - 5	75	0.158	NM	NM	NM	NM	96	89	44	NM	NM

*These numbers represent silt-sized and clay-sized content but may not indicate the percentage of the material with the engineering properties of silt or clay.
Sieve Analysis and Hydrometer Analysis testing performed in general accordance with ASTM D6913(Sieve Analysis) and ASTM D7928 (Hydrometer Analysis).
NP = Nonplastic
NA = Not Available
NM = Not Measured

Coefficients of Uniformity - $C_u = D_{60} / D_{10}$
Coefficients of Curvature - $C_C = (D_{30})^2 / D_{60}\,D_{10}$
D_{60} = Grain diameter at 60% passing
D_{30} = Grain diameter at 30% passing
D_{10} = Grain diameter at 10% passing

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PROJECT NO.: 20203585.001A
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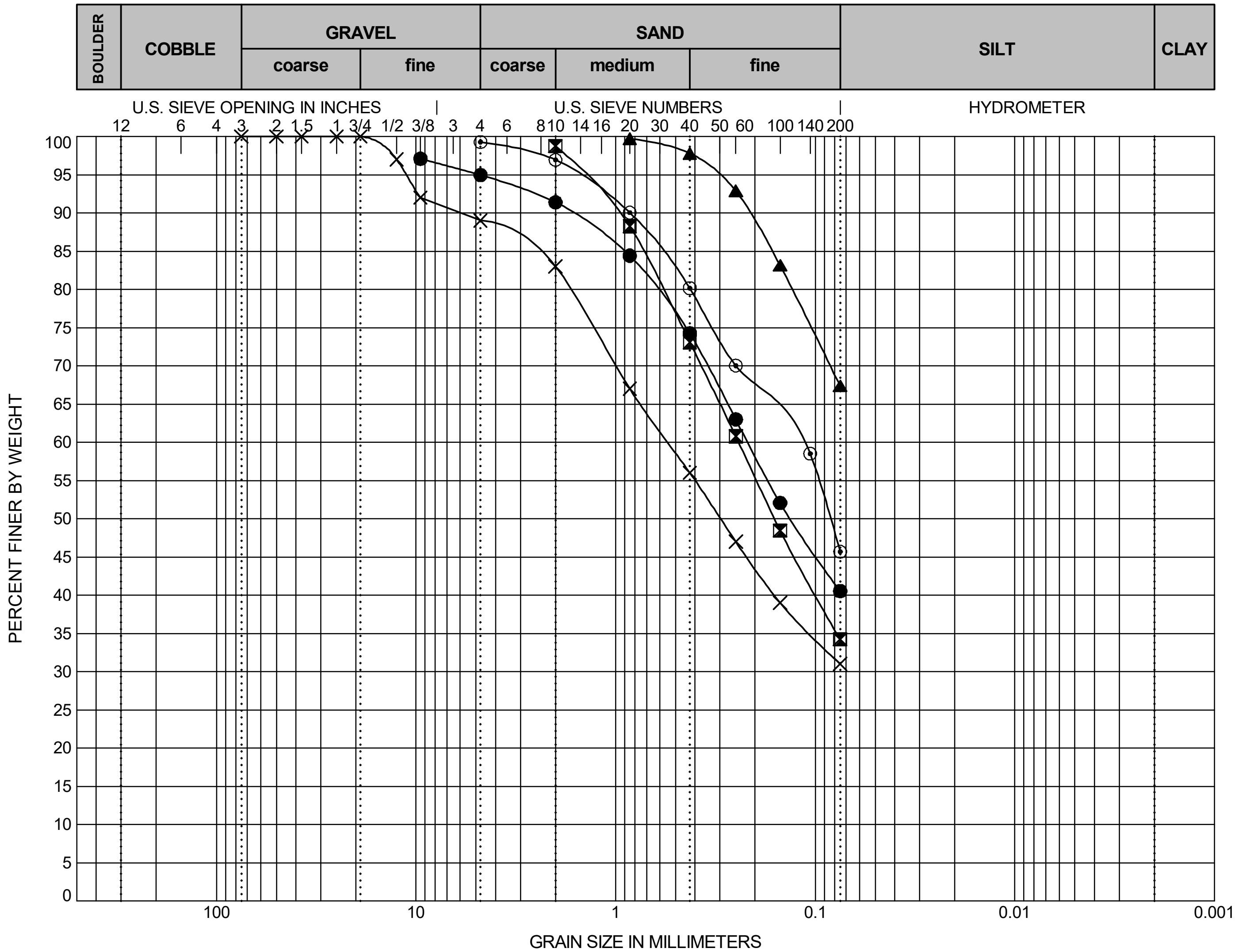
DATE: 8/2/2023

SIEVE ANALYSIS

El Monte Water Pipeline Rehabilitation
County of San Diego

APPENDIX

B-4



	Exploration ID	Depth (ft.)	Sample Number	Sample Description	LL	PL	PI
●	B-8	5	S2	CLAYEY SAND (SC)	48	24	24
⊠	B-8	15	S4	CLAYEY SAND (SC)	NM	NM	NM
▲	B-8	18.5	S5	LEAN CLAY (CL)	56	36	20
×	B-9	1 - 5	NA	SILTY CLAYEY SAND WITH GRAVEL (SC-SM)	NM	NM	NM
⊙	B-10	0.25 - 5	S1	CLAYEY SAND (SC)	NM	NM	NM

	Exploration ID	Depth (ft.)	D_{100}	D_{60}	D_{30}	D_{10}	Cc	Cu	Passing 3/4"	Passing #4	Passing #200	%Silt*	%Clay*
●	B-8	5	9.5	0.217	NM	NM	NM	NM		95	41	NM	NM
⊠	B-8	15	2	0.242	NM	NM	NM	NM			34	NM	NM
▲	B-8	18.5	0.85	NM	NM	NM	NM	NM			67	NM	NM
×	B-9	1 - 5	75	0.547	NM	NM	NM	NM	100	89	31	NM	NM
⊙	B-10	0.25 - 5	4.75	0.119	NM	NM	NM	NM		99	46	NM	NM

*These numbers represent silt-sized and clay-sized content but may not indicate the percentage of the material with the engineering properties of silt or clay.
Sieve Analysis and Hydrometer Analysis testing performed in general accordance with ASTM D6913(Sieve Analysis) and ASTM D7928 (Hydrometer Analysis).
NP = Nonplastic
NA = Not Available
NM = Not Measured

Coefficients of Uniformity - $C_u = D_{60} / D_{10}$
Coefficients of Curvature - $C_C = (D_{30})^2 / D_{60} D_{10}$
D_{60} = Grain diameter at 60% passing
D_{30} = Grain diameter at 30% passing
D_{10} = Grain diameter at 10% passing

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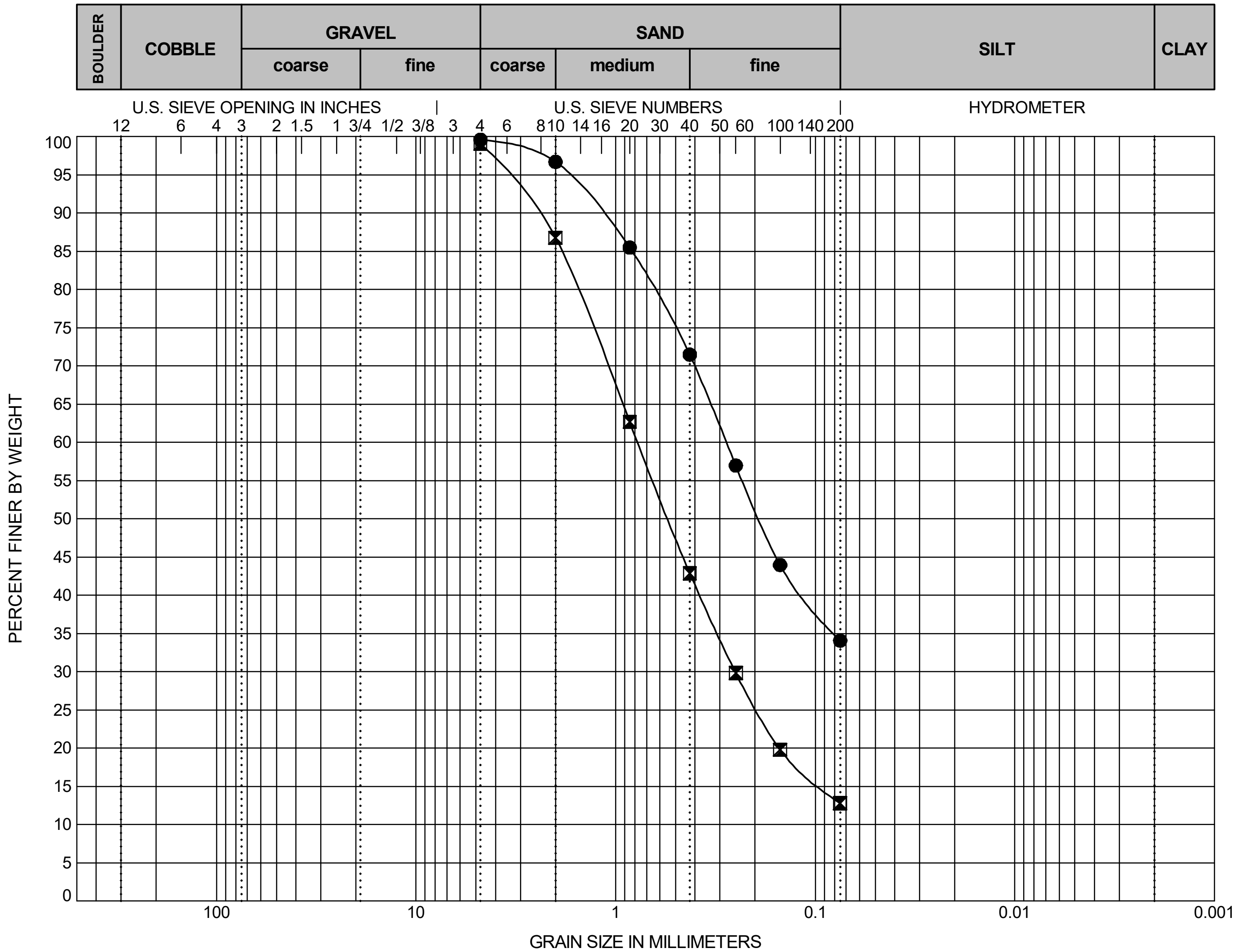
SIEVE ANALYSIS

El Monte Water Pipeline Rehabilitation
County of San Diego

APPENDIX

B-5



Exploration ID		Depth (ft.)	Sample Number	Sample Description	LL	PL	PI
●	B-10	11 - 11.5	NA	CLAYEY SAND (SC)	NM	NM	NM
⊠	B-11	0 - 5	S1	SILTY SAND (SM)	NM	NM	NM

Exploration ID		Depth (ft.)	D_{100}	D_{60}	D_{30}	D_{10}	Cc	Cu	Passing 3/4"	Passing #4	Passing #200	%Silt*	%Clay*
●	B-10	11 - 11.5	4.75	0.28	NM	NM	NM	NM		100	34	NM	NM
⊠	B-11	0 - 5	4.75	0.774	0.252	NM	NM	NM		99	13	NM	NM

*These numbers represent silt-sized and clay-sized content but may not indicate the percentage of the material with the engineering properties of silt or clay.
Sieve Analysis and Hydrometer Analysis testing performed in general accordance with ASTM D6913(Sieve Analysis) and ASTM D7928 (Hydrometer Analysis).
NP = Nonplastic
NA = Not Available
NM = Not Measured

Coefficients of Uniformity - $C_u = D_{60} / D_{10}$
Coefficients of Curvature - $C_C = (D_{30})^2 / D_{60}\, D_{10}$
D_{60} = Grain diameter at 60% passing
D_{30} = Grain diameter at 30% passing
D_{10} = Grain diameter at 10% passing

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PROJECT NO.: 20203585.001A		SIEVE ANALYSIS	APPENDIX
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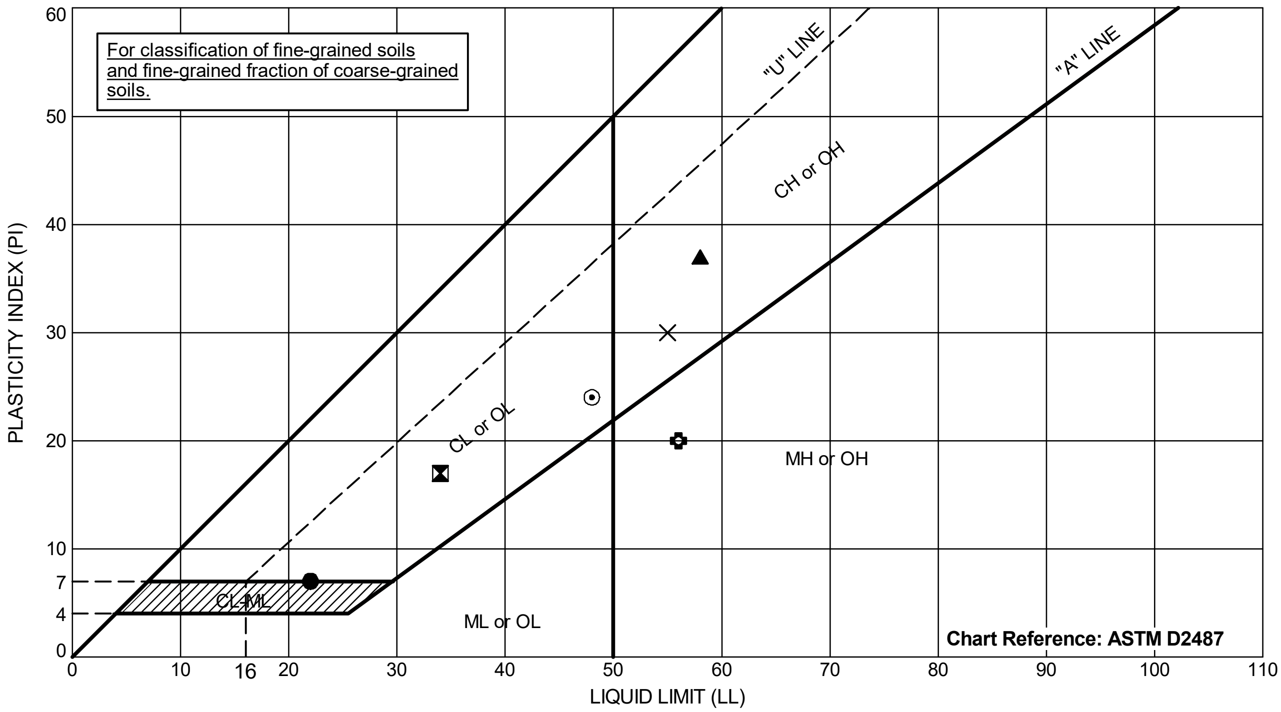
CHECKED BY:	SHR		
DATE:	8/2/2023		



Exploration ID		Depth (ft.)	Sample Number	Sample Description	Passing #200	LL	PL	PI
●	B-1	0 - 4	S-5	SILTY, CLAYEY SAND (SC-SM)	34	22	15	7
⊠	B-2	16 - 16.5	NA	FAT CLAY (CH)	NM	34	17	17
▲	B-4	6 - 6.5	NA	LEAN CLAY (CL)	NM	58	21	37
×	B-5	0 - 4	S1	SANDY FAT CLAY (CH)	57	55	25	30
⊙	B-8	5	S2	CLAYEY SAND (SC)	41	48	24	24
✪	B-8	18.5	S5	LEAN CLAY (CL)	67	56	36	20

Testing performed in general accordance with ASTM D4318.
NP = Nonplastic
NA = Not Available
NM = Not Measured

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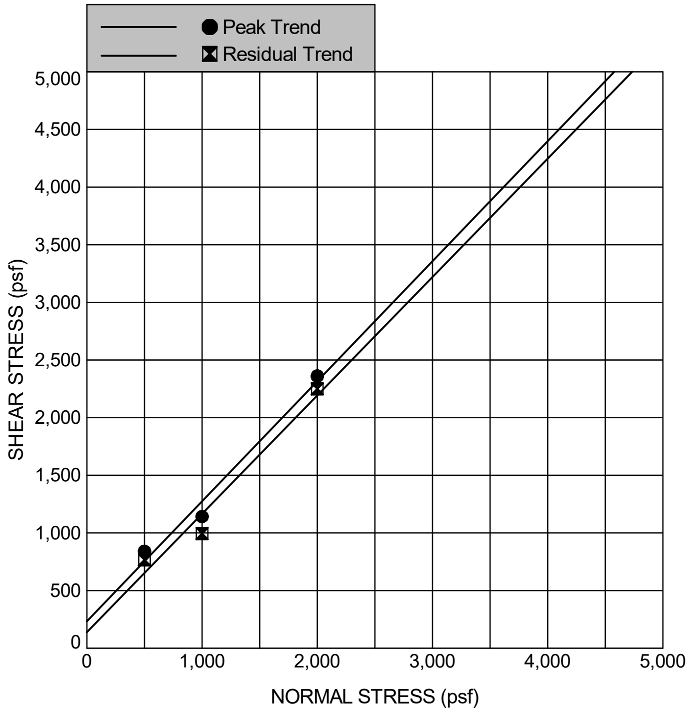
PROJECT NO.: 20203585.001A
DRAWN BY: AB
CHECKED BY: AR
DATE: 8/2/2023

ATTERBERG LIMITS

El Monte Water Pipeline Rehabilitation
County of San Diego

APPENDIX
B-7



Exploration ID	Depth (ft.)	Sample Number	Sample Description
B-1	6 - 6.5	NA	SILTY, CLAYEY SAND (SC-SM)

Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plastic Limit	Plasticity Index	Specific Gravity
NM	NM	NM	NM	NM	

	Specimen No.	Water Content (%)	Dry Unit Weight (pcf)	Saturation (%)	Void Ratio	Area (in^2)	Height (in)
Initial	1						
	2						
	3						
At Test	Specimen No.	Water Content (%)	Dry Unit Weight (pcf)	Saturation (%)	Void Ratio	Area (in^2)	Height (in)
	1						
	2						
	3						

Specimen No.	Peak Shear Stress (psf)	Residual Shear Stress (psf)	Horizontal Displacement (in)	Normal Stress (psf)	Strain Rate (in/min)
1	841	766		500	
2	1142	992		1000	
3	2360	2250		2000	

Results	Cohesion (psf)	Friction ϕ (deg)	Tan ϕ (deg)
Peak	232	46.18	
Residual			

Testing perfomed in general accordance with ASTM D3080.
NP = Nonplastic
NA = Not Available
NM = Not Measured

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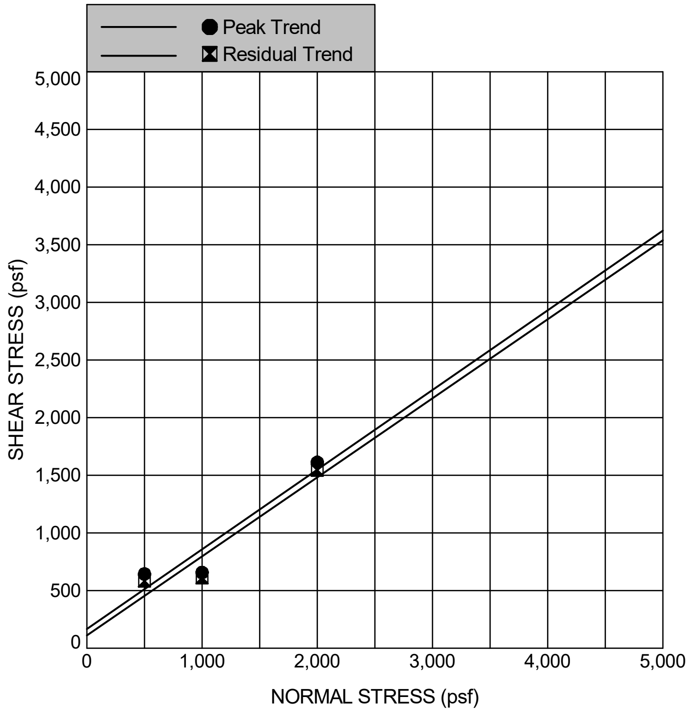
CHECKED BY: SHR
DATE: 8/2/2023

DIRECT SHEAR

El Monte Water Pipeline Rehabilitation
County of San Diego

APPENDIX

B-8



Exploration ID	Depth (ft.)	Sample Number	Sample Description
B-8	11 - 11.5	NA	CLAYEY SAND (SC)

Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plastic Limit	Plasticity Index	Specific Gravity
NM	NM	NM	NM	NM	

	Specimen No.	Water Content (%)	Dry Unit Weight (pcf)	Saturation (%)	Void Ratio	Area (in^2)	Height (in)
Initial	1						
	2						
	3						
At Test	1						
	2						
	3						

Specimen No.	Peak Shear Stress (psf)	Residual Shear Stress (psf)	Horizontal Displacement (in)	Normal Stress (psf)	Strain Rate (in/min)
1	643	581		500	
2	657	609		1000	
3	1614	1546		2000	

Results	Cohesion (psf)	Friction ϕ (deg)	Tan ϕ (deg)
Peak	164.5	34.67	
Residual			

Testing perfomed in general accordance with ASTM D3080.
NP = Nonplastic
NA = Not Available
NM = Not Measured

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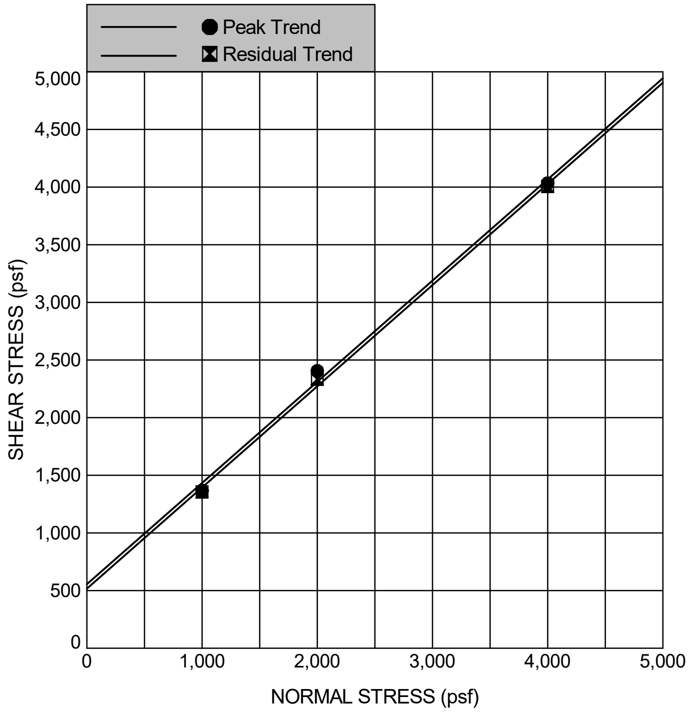
DATE: 8/2/2023

DIRECT SHEAR

El Monte Water Pipeline Rehabilitation
County of San Diego

APPENDIX

B-9



Exploration ID	Depth (ft.)	Sample Number	Sample Description
B-10	6 - 6.5	NA	CLAYEY SAND (SC)

Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plastic Limit	Plasticity Index	Specific Gravity
NM	NM	NM	NM	NM	

	Specimen No.	Water Content (%)	Dry Unit Weight (pcf)	Saturation (%)	Void Ratio	Area (in^2)	Height (in)
Initial	1						
	2						
	3						
At Test	1						
	2						
	3						

Specimen No.	Peak Shear Stress (psf)	Residual Shear Stress (psf)	Horizontal Displacement (in)	Normal Stress (psf)	Strain Rate (in/min)
1	1367	1354		1000	
2	2407	2331		2000	
3	4034	4006		4000	

Results	Cohesion (psf)	Friction ϕ (deg)	Tan ϕ (deg)
Peak	553.5	41.29	
Residual			

Testing perfomed in general accordance with ASTM D3080.
NP = Nonplastic
NA = Not Available
NM = Not Measured

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DRAWN BY: SF
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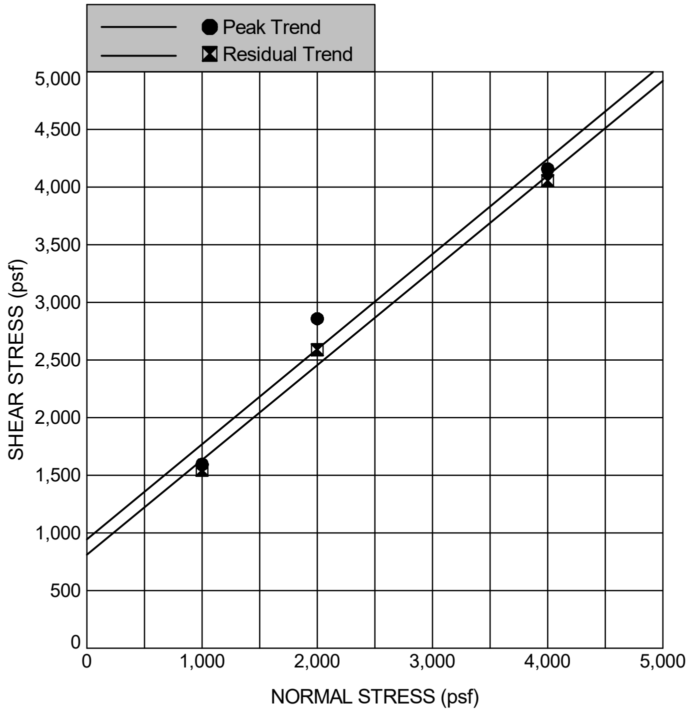
DATE: 8/2/2023

DIRECT SHEAR

El Monte Water Pipeline Rehabilitation
County of San Diego

APPENDIX

B-10



Exploration ID	Depth (ft.)	Sample Number	Sample Description
B-11	6 - 6.5	NA	SILTY SAND (SM)

Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plastic Limit	Plasticity Index	Specific Gravity
NM	NM	NM	NM	NM	

	Specimen No.	Water Content (%)	Dry Unit Weight (pcf)	Saturation (%)	Void Ratio	Area (in^2)	Height (in)
Initial	1						
	2						
	3						
At Test	1						
	2						
	3						

Specimen No.	Peak Shear Stress (psf)	Residual Shear Stress (psf)	Horizontal Displacement (in)	Normal Stress (psf)	Strain Rate (in/min)
1	1593	1545		1000	
2	2858	2591		2000	
3	4157	4054		4000	

Results	Cohesion (psf)	Friction ϕ (deg)	Tan ϕ (deg)
Peak	943.5	39.53	
Residual			

Testing perfomed in general accordance with ASTM D3080.
NP = Nonplastic
NA = Not Available
NM = Not Measured

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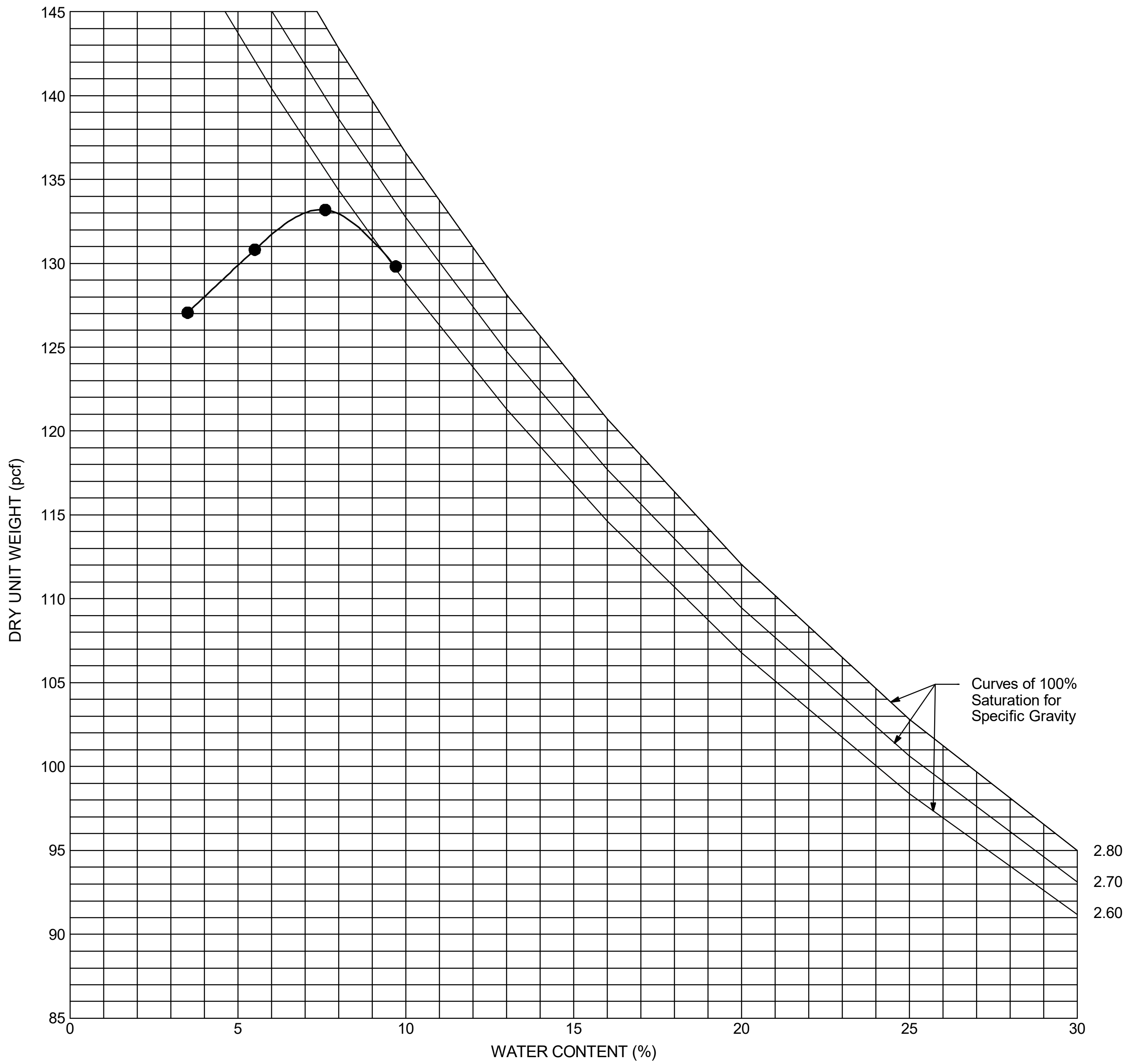

Exploration ID	Depth (ft.)	Sample Number	Sample Description
● B-9	0 - 5	NA	SILTY CLAYEY SAND WITH GRAVEL (SC-SM)

Passing 3/4"	Passing #4	Passing #200	LL	PL	PI	Maximum Dry Unit Weight (pcf)	Optimum Water Content (%)
NM	NM	NM	NM	NM	NM	133.2	7.5

Testing performed in general accordance with ASTM D1557 Method B.
NP = Nonplastic
NA = Not Available
NM = Not Measured

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PROJECT NO.: 20203585.001A	COMPACTION CURVE	APPENDIX
DRAWN BY: SF	El Monte Water Pipeline Rehabilitation County of San Diego	B-12
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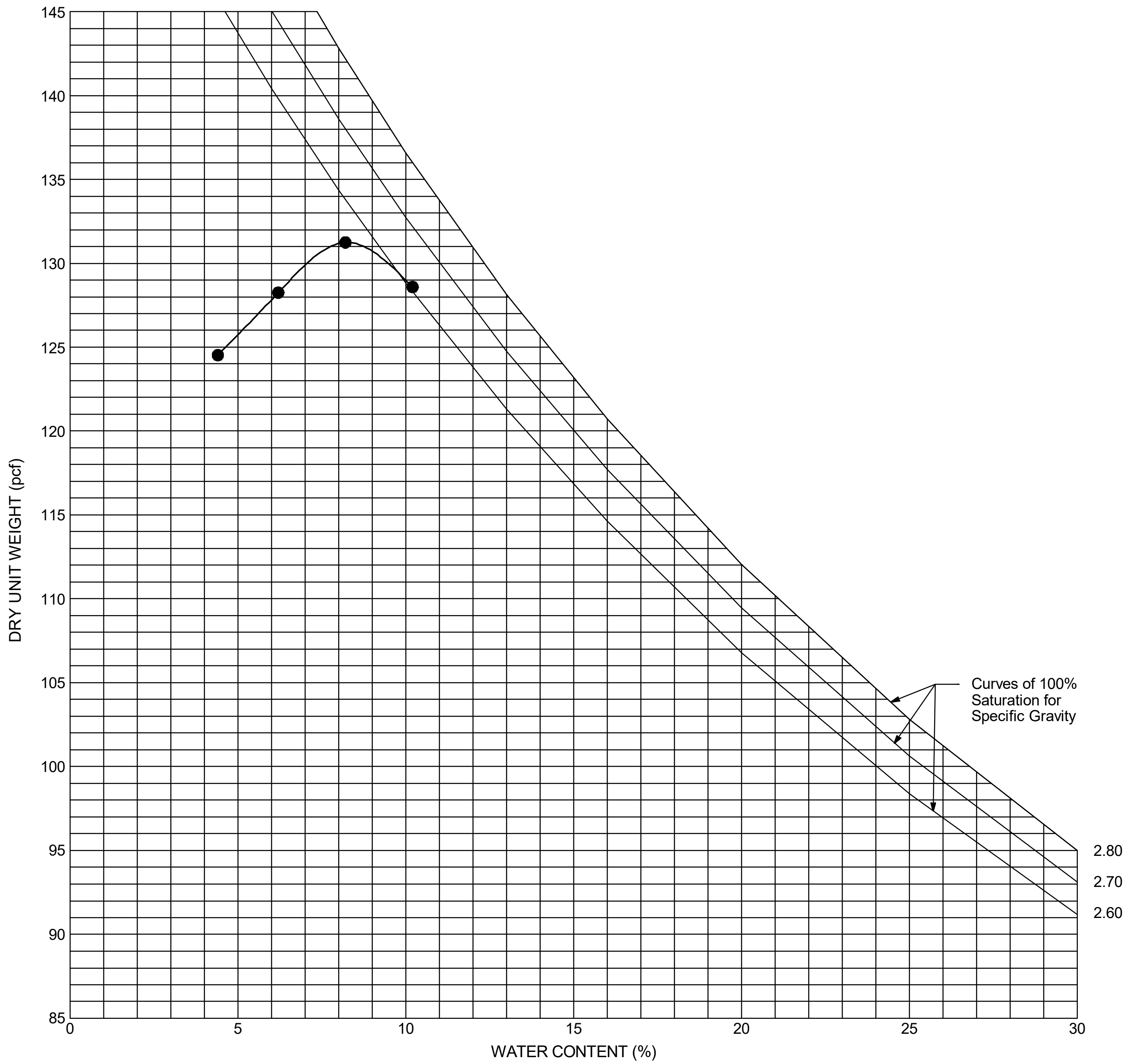
DATE: 8/2/2023		



Exploration ID		Depth (ft.)		Sample Number		Sample Description	
● B-10		0 - 5		NA		CLAYEY SAND (SC)	
Passing 3/4"	**Passing #4**	**Passing #200**	**LL**	**PL**	**PI**	**Maximum Dry Unit Weight (pcf)**	**Optimum Water Content (%)**
NM	NM	NM	NM	NM	NM	131.2	8.2

Testing performed in general accordance with ASTM D1557 Method B.
NP = Nonplastic
NA = Not Available
NM = Not Measured

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20203585.001A	COMPACTION CURVE	APPENDIX
	DRAWN BY: SF	El Monte Water Pipeline Rehabilitation County of San Diego	B-13
	CHECKED BY: SHR		
	DATE: 8/2/2023		

APPENDIX C
GEOTECHNICAL BUSINESS ASSOCIATION DOCUMENT

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:
- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:
- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are <u>not</u> final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are <u>Not</u> Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will <u>not</u> of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are <u>not</u> building-envelope or mold specialists.***

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