



## **DIRIDON STATION – PROJECT SPARTAN** SAN JOSE, CALIFORNIA

### **LOTS A, B, and C** **PRELIMINARY GEOTECHNICAL ASSESSMENT**

**SUBMITTED TO**  
Google Incorporated  
1600 Amphitheatre Parkway  
Mountain View, CA 94043

**PREPARED BY**  
ENGEO Incorporated

June 25, 2018

**PROJECT NO.**  
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**14121.000.000**

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Mr. Clyde Wright  
Google Incorporated  
1600 Amphitheatre Parkway  
Mountain View, CA 94043

Subject: Diridon Station – Project Spartan  
Lots A, B, and C  
San Jose, California

## PRELIMINARY GEOTECHNICAL ASSESSMENT

Dear Mr. Wright:

We prepared this preliminary geotechnical assessment for Lots A, B, and C of the Spartan Project as outlined in our agreement dated December 1, 2017. This report presents our geotechnical observations, as well as our preliminary conclusions and recommendations for the project. The purpose of this report is to provide geotechnical recommendations for preliminary design purposes.

If you have any questions or comments regarding this report, please call, and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

Oliver Van Rooyen

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## SELECTED REFERENCES

**APPENDIX A** – Project Spartan Geotechnical Story Board

**APPENDIX B** – Exploration Logs

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## 1.0 INTRODUCTION

### 1.1 PURPOSE AND SCOPE

We prepared this preliminary geotechnical report for development of Lots A, B and C for the Spartan Project at Diridon Station in San Jose, California as outlined in our agreement dated December 1, 2017. Google Incorporated authorized us to conduct the following scope of services:

- Review available literature and geologic maps.
- Review available geotechnical explorations.
- Perform subsurface field exploration.
- Analyze preliminary data.
- Provide preliminary recommendations.

This report was prepared for the exclusive use of our client and its consultants for the design of this project. In the event that any changes are made in the character, design or layout of the development, we must be contacted to review the findings of this report to evaluate whether modifications are recommended. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without our express written consent.

### 1.2 SITE LOCATION AND BACKGROUND

The site encompasses Assessor's Parcel Numbers (APN) 259-28-031, 259-28-041, 259-28-044 and 259-28-043, and consists of a surface parking lot at 525 West Santa Clara Street in San Jose, California. The site is generally bounded W Julian Street to the North, N Montgomery Street and the SAP Centre to the East, W Santa Clara Street to the South, and the Southern Pacific Railroad tracks to the West.

The site is elongated and slightly "L"-shaped, orientated in a north-south direction. The ground surface ranges from Elevation 80 to 100 feet (NAVD88). The site is currently a paved parking area that serves the SAP Center, formally called the HP Pavilion. There are three main parking areas at the site, Lot A, Lot B and Lot C each taking up roughly 1/3 of the site. Lot C is the northernmost lot, and Lot A is the southernmost lot. Rail lines are present to the west of the site and service an active passenger and freight rail network. The CalTrain and VTA transit stations are located at Diridon Station to the south of the site.

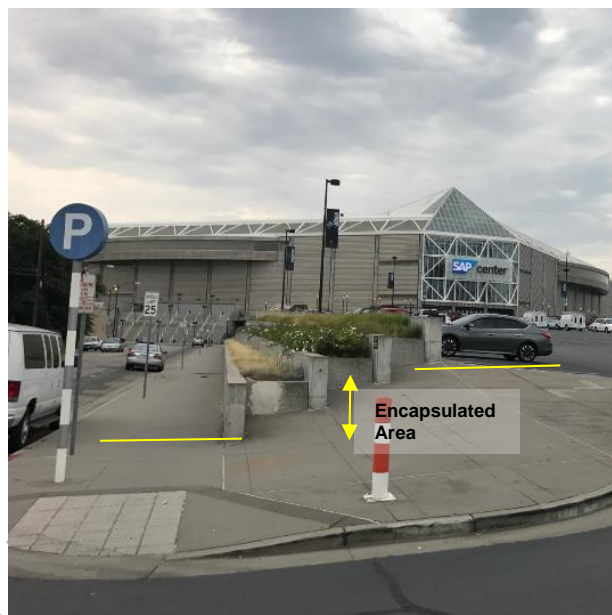
**ILLUSTRATION 1.2-1: Parking Lot Areas**



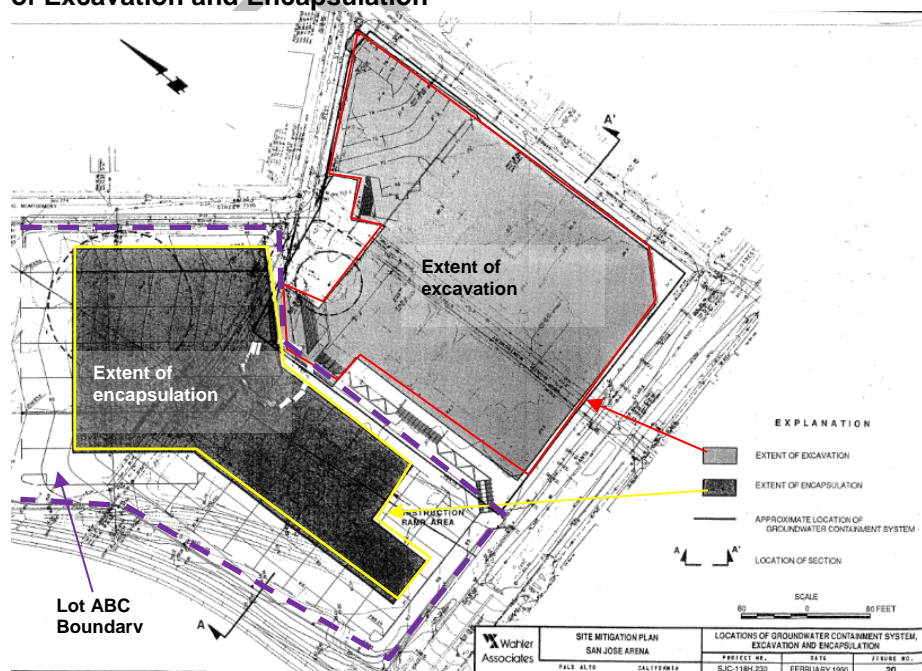
### 1.2.1 Encapsulated Material

During development of the SAP Center to the east of the site, soil was excavated to a depth of 17 feet below the ground surface (bgs) to form a basement. The soil from underneath the SAP center had reported contaminant concentrations of PAHs greater than 3000 mg/kg, lead up to 4000 mg/kg, TPHg at 390 mg/kg, TPHd at 1500 mg/kg, and oil and grease up to 13,000 mg/kg. This excavated soil was placed in Lot A and the southern portion of Lot B, creating an approximately 2- to 15-foot elevation change from the surrounding ground. The excavated material was placed on top of a compacted bottom clay layer and capped by 2 feet of engineered clean fill and asphalt at the surface. The area of capped contaminated material is known as the encapsulated area and is shown in Photo 1.2.1-1 and Illustration 1.2.1-2. The basement of the SAP Center was also surrounded by an impermeable cutoff (slurry) wall to a depth of 35 feet bgs to prevent lateral migration of potentially contaminated material and groundwater from below the SAP Center to the surrounding properties.

**PHOTO 1.2.1-1: Photo of elevation difference at the encapsulation area**



**ILLUSTRATION 1.2.1-1: Wahler Associates Site Mitigation Plan with Extents of Excavation and Encapsulation**



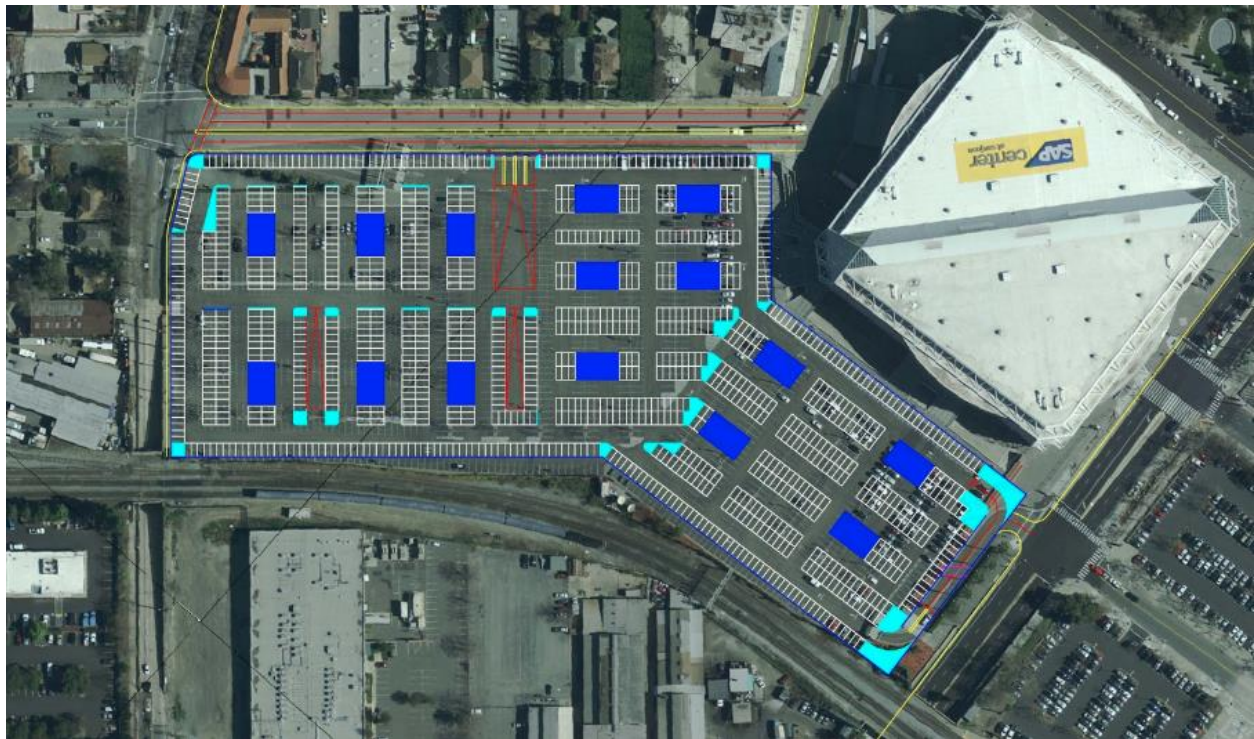


### 1.2.2 Proposed Development

We received schematic development plans by ARUP for Lots A, B, and C. We understand the proposed development will consist of residential and commercial development. Two scenarios are considered as part of this preliminary assessment;

- **Scenario 1 – Below-grade option:** consisting of up to a three-story parking garage below grade with 18 stories above grade (21 stories total). We estimate basement excavations will extend to approximately Elevation 50 feet (NAVD88).
- **Scenario 2 – At-grade option;** 18-story building constructed at current grade with some regrading of the site with retail on the lower level.

#### ILLUSTRATION 1.2.2-1: Lots A, B, and C Development Plans



## 2.0 PREVIOUS STUDIES

We previously completed a subsurface characterization and initial geotechnical assessment of the master Project Spartan area, which included this site. This assessment provided overall characterization of subsurface conditions to provide initial geotechnical recommendations for high-level planning.

- ENGEO (2017); Subsurface Characterization and Initial Geotechnical Assessment, Diridon Station – Project Spartan, San Jose California; DRAFT September 25, 2017; Project No. 14121.000.000.

In addition to the above assessment, we reviewed the following reports relating to the subsurface conditions of Lots A, B and C.

- HMM/Bechtel (2008) Silicon Valley Rapid Transit Project – Central Area Guideway 65% Submittal: Combined Geotechnical Baseline Report; Dated December 2008; Report Number P0503-D300-RPT-GEO-006.
- Earthtech (2006), Silicon Valley Rapid Transit Project, Geotechnical and Seismic Design Criteria Report for Parking Garages and Ancillary Facilities; Dated June 26, 2006, Report Number P0507-D425-RPT-GEO-002
- MACTEC (2003), Geotechnical Investigation Report, Santa Clara to Diridon Corridor Improvement Project, San Jose; July 7, 2003; Project Number 4097030022.

The above reports are presented in Appendix A – Project Spartan Geotechnical Story Board, Previous Geotechnical Reports Tab.

The above reports have over 20 historic exploratory logs within and near the study area. These exploratory logs provide subsurface information extending to depths from 10 to 115 feet bgs. The exploratory logs are dated between 1976 and 2006 and we present the logs in Appendix A – Project Spartan Geotechnical Story Board, Borings Tab.

### 3.0 REGIONAL GEOLOGY AND SEISMICITY

In our previously referenced report, we presented regional geology and seismicity for the site. The following is a summary of this reference.

#### 3.1 REGIONAL GEOLOGY

We reviewed published geologic maps prepared by Wentworth et al. (1999, 2006), Knudsen (2000), Wagner et al. (1991), and Graymer et al. (1998). In addition, we reviewed seismic-hazard maps prepared by the California Geological Survey. Selected maps are presented in Appendix A – Project Spartan Geotechnical Story Board, Geology Tab.

The site is located in San Jose within the Santa Clara Valley. The Santa Clara Valley is an alluvium-filled basin that consists of gently sloping topography formed by coalescing alluvial fans. Natural levee deposits are common along the principal stream channels that generally drain northward to San Francisco Bay. The alluvial deposits are over 1,000 feet thick and are underlain by bedrock that outcrops around the Communication Hill area to the southeast. The origins of the alluvium are described by the US Geological Survey (USGS) as being:

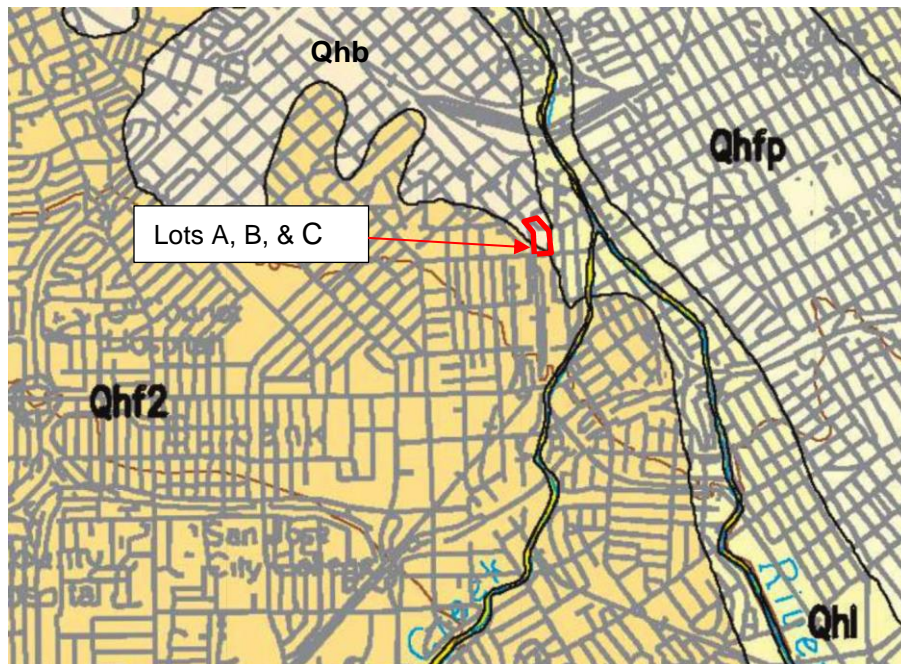
*“...built up by deposits from streams that are very irregular in their flow and from time to time change their courses. For this reason, the nature of the deposits changes rapidly...”*  
by Clark, 1924

The upper soil profile is predominately alluvial fan deposits and alluvium of Holocene age (Qhb, Qhfp, and Qhf2). The Holocene Alluvial Fan Deposits (Qhf) and Holocene Alluvium Basin deposits (Qhb) primarily consist of medium stiff to very stiff silty clays and clayey silts with varying amounts

of sand. The clays and silts are occasionally interbedded with medium dense to dense sand layers that are typically less than 5 feet thick. The Holocene Alluvial Fan Deposits and Holocene Alluvium may be generally categorized with USCS designations of CL, ML, SC, and SM.

The Holocene Alluvial Fan Deposits and Alluvium are generally underlain by Latest Pleistocene Alluvial Fan Deposits (Qpf). The depths of the Pleistocene-age sediments are approximately 50 to 60 feet above Mean Sea Level (MSL) (E. J. Helley, 1990). These deposits are generally similar in composition to the younger Holocene-age deposits above, except that the sand layers are generally more dense. In addition, gravel layers with varying amounts of silt and clay are also common. The Latest Pleistocene Alluvial Fan Deposits may generally be categorized with USCS designations of CL, ML, SP, SM, SC, GP, GM, and GC.

**ILLUSTRATION 3.1-1: Wentworth, 1999 Geologic Map**



### 3.1.1 Paleontological Resources

The potential of exposing fossils is low to moderate for the project site.

## 3.2 REGIONAL FAULTING AND SEISMICITY

The California Geological Survey defines an active fault as one that has experienced surface displacement within Holocene time (about the last 11,000 years) (SP42 CGS, 2007). Because of the presence of numerous active faults, the San Francisco Bay Region is considered seismically active. Numerous small earthquakes occur every year in the San Francisco Bay Region, and larger (greater than Moment Magnitude 7) earthquakes have been recorded and can be expected to occur in the future. Appendix A – Project Spartan Geotechnical Story Board, Seismic Tab, shows the approximate locations of active and potentially active faults and significant historical earthquake epicenters mapped within the San Francisco Bay Region.



The nearest active fault is the Hayward – South East extension, which is mapped approximately 6.2 miles northeast of the site. This fault is considered capable of a moment magnitude earthquake of 6.4. Other active faults in the region include the Hayward-Rodgers Creek and San Andreas faults approximately 9 and 11 miles away, capable of a moment magnitude of 7.4 and 8.0. For a more complete list of active faults within 100km of the project site, refer to our 2017 report. In addition to the active faults noted above, Bortugno (1991) and Jennings (1994) mapped two concealed faults, the Silver Creek fault and the San Jose fault, in the vicinity of the site; these faults are also discussed in our 2017 report.

### 3.2.1 Ground Rupture Hazard

The site is not located in an Alquist-Priolo Earthquake Fault Zone. The nearest known active or potentially active fault is the San Jose Fault located about 2 miles west of the site; the risk of ground rupture is deemed to be negligible.

### 3.2.2 Ground Shaking Hazard

Multiple active faults capable of a magnitude 7.0 or greater earthquake are located within 20 miles of the site. The risk of strong ground shaking is high at the site.

### 3.2.3 Liquefaction and Lateral-Spreading Hazard

According to the State of California Seismic Hazard Zones Map for the San Jose West Quadrangle (2002), the site is located within an area mapped as being potentially susceptible to liquefaction. We assess liquefaction and lateral spreading at the project site in further sections.

## 4.0 FIELD EXPLORATION

Our field exploration included advancing two Cone Penetration Tests (CPTs) and two Seismic Cone Penetration Tests (S-CPTs) along with dilatometer testing at the site. We performed our field exploration on March 12 and 29, 2018. The location and elevations of our explorations are approximate and were estimated by pacing from features at the site; they should be considered accurate only to the degree implied by the method used.

We retained the services of a subcontractor with a CPT rig to push the CPT and S-CPT to a maximum depth of approximately 114 feet. We performed the CPTs in general accordance with ASTM D-5778. Measurements include the tip resistance to penetration of the cone ( $Q_c$ ), the resistance of the surface sleeve ( $F_s$ ), and pore pressure ( $U$ ) (Robertson and Campanella, 1988).

We also performed Seismic Cone Penetration Testing in conjunction with the CPTs to obtain shear wave soundings with the addition of a horizontally active geophone (28 Hertz) that is rigidly mounted in the body of the cone penetrometer, 0.2 meter behind the cone tip. We generated shear waves by using an impact hammer horizontally striking a beam that is held in place by a normal load. We recorded the shear wave arrival time at the cone.

We also performed Dilatometer Testing (DMT) at CPT-3 and S-CPT-2 locations. The Dilatometer blade is 15 mm thick and 96 mm wide in cross-section and is pushed into the soil at a constant rate of 2 cm/sec. The test is accomplished by advancing the dilatometer to a specified depth and

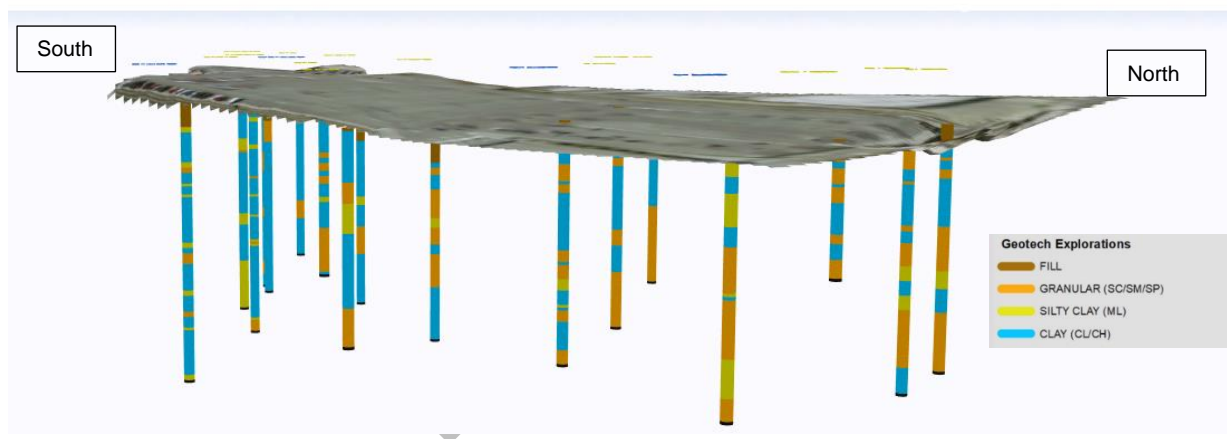
increasing the nitrogen gas pressure until the membrane is flush with the blade. This pressure is noted as the A-pressure, the pressure required to begin to move the membrane against the soil. The membrane is then inflated further until the center of the membrane is displaced 1.1mm into the soil. This pressure is noted as the B-pressure. The blade is then advanced into the ground to the next depth increment and the procedure is repeated.

Shear wave velocities, CPT data, and Dilatometer results are presented in Appendix B and also Appendix A – Project Spartan Geotechnical Story Board, Borings Tab. The CPT holes were backfilled with cement grout upon completion in accordance with the requirements of the Santa Clara Valley Water District.

## 5.0 SUBSURFACE CONDITIONS

We present the subsurface conditions at the site graphically within the Appendix A – Project Spartan Geotechnical Story Board, 3D subsurface explorations Tab. We have simplified the subsurface conditions into major soil unit types in the model; Fill, Granular Soil, Silty Clay, and Clay. We discuss the subsurface conditions in the following sections.

**ILLUSTRATION 5.0-1: Subsurface Aodel at Lots A, B, and C**



### 5.1.1 Existing Fill

The site and the adjacent SAP Center, where the encapsulated fill was derived, was previously occupied by a PG&E coal gasification plant. Potentially contaminated fill is expected at the project site due to past use and the construction of the encapsulated fill. Prior to the placement of the encapsulated material, the site was used for various industrial uses and fill is expected to variable depths from the original ground surface. There are two distinct fill areas.

1. The encapsulated areas and the surrounding areas (Lots A and B).
2. Outside of the encapsulated areas (Lot C).

We discuss these two fill areas in the sections below.



#### ILLUSTRATION 5.1.1–1: Fill Areas within Lots A, B, and C



##### 5.1.1.1 Lots A and B - The Encapsulated and Surrounding Areas

Both clean and potentially contaminated fill from excavations during construction of the SAP Center was placed on the original ground surface at Lots A and B. The height of this fill ranged from 10 to 15 feet in areas adjacent to the SAP Center and tapers to the original ground surface to the north and west. The encapsulation consists of a compacted bottom clay layer, overlain by potentially contaminated material and covered by at least 2 feet of clean fill and asphalt paving. The encapsulation material consists of a mixture of silt, sand and clay. CPTs performed within the encapsulation area indicate that the entire cap was likely compacted during placement.

Beneath the encapsulation, additional fill material was encountered in borings previously advanced by others. This fill varied in thickness but generally ranged between 5 and 20 feet. Environmental and historic explorations along the southwestern edge of the encapsulated area encountered localized fill depths of up to 30 feet. Fill below the encapsulation contained portions of concrete slabs, brick and other construction debris. It is likely that existing subsurface structures such as previous building foundations are still present at the site.

##### 5.1.1.2 Lot C - Outside of the Encapsulated Area

In Lot C, fill depths ranged between 1 and 5 feet below grade, but deeper fill depths may be present locally. The fill may include loose to dense silty sand, sand, gravelly sand, silt, and variable amounts of debris. Since the fill sources are generally not known, we anticipate the fill may vary significantly between locations.

##### 5.1.2 Alluvium

The alluvium consists of clay, silty clay, clayey silt, silt, silty sand, and gravelly sand beneath the fill. The upper alluvium, generally encountered within the upper 40 to 55 feet, consists of fine-grained soil that is typically soft to stiff. The upper alluvium becomes very stiff to hard at depths below about 40 to 55 feet below ground. Interbedded layers of loose to very dense silty and clayey sand, sand, and gravel vary in thickness and consistency within the upper alluvial

layer. The interbedded granular layers generally range from ½ inch to 5 feet thick and are medium dense to dense. The upper alluvium may include layers of moderately compressible clay.

Below about 40 to 55 feet, the lower alluvium consists of sand and gravel interbedded with a clay and silt matrix. The coarse-grained deposits within the lower alluvium are interbedded with very stiff to hard clay and silt. Course-grained deposits in the lower alluvium were encountered up to 32 feet thick in the northern portion of the site. It appears that the sand lenses diminish in thickness towards the south. The alluvial deposits are expected to continue to bedrock. Bedrock is estimated to be over 1,000 feet below ground surface.

### 5.1.3 Holocene and Pleistocene Alluvium interface

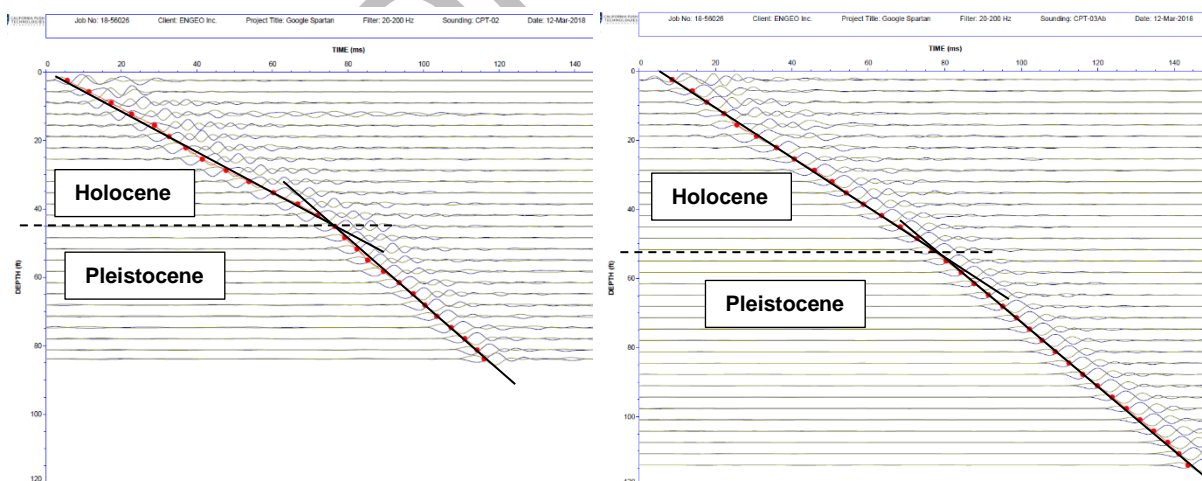
The younger portion of the Alluvium (less than 11,000 years old) is Holocene. Below that, the older alluvium is Pleistocene. Generally, older alluvium is stiffer and denser than Holocene material. We reviewed the shear wave velocities at the site to estimate the depth to the top of the Pleistocene Alluvium. We estimate the interface of these two ages of soil to be where the shear wave velocity appears to increase. We present our findings below.

**TABLE 5.1.3-1: Summary of Top of Pleistocene Depths**

LOCATION	APPROXIMATE SURFACE ELEVATION (FEET NAVD88)	DEPTH TO TOP OF PLEISTOCENE (FEET BELOW GROUND SURFACE)	INFERRED ELEVATION OF THE TOP OF PLEISTOCENE (FEET NAVD88)
S-CPT-2	87	45	42
S-CPT-3A	99	52	47

**ILLUSTRATION 5.1.3-1 (Left): S-CPT-2 Shear Wave Analysis of Holocene/Pleistocene Interface**

**ILLUSTRATION 5.1.3-2 (Right): S-CPT-3A Shear Wave Analysis of Holocene/Pleistocene Interface**



We estimate the top of Pleistocene-age sediments at the site to be between Elevation 40 and 50 feet. This is consistent with published maps (E. J. Helley, 1990), which estimate the top of the Pleistocene Alluvium at around 50 feet above mean sea level in the vicinity of Lots A, B, and C.

## 5.2 HYDROLOGIC CONDITIONS

Based on a report by HMM (HMM, 2008), within the geological profile, there are believed to be three hydraulically distinct aquifer systems: a shallow unconfined aquifer contained within surficial soil and two deeper confined or semi-confined aquifers. The shallower of the two deeper aquifers is referred to as a minor aquifer, and the deeper zones are referred to as a major aquifer (likely to have been assigned based on their economic value and historical usage).

The shallow unconfined aquifer contained within the surficial soil is likely to be hydraulically connected to the upper granular soil layers and closely tied or hydraulically connected to the water level of the nearby Guadalupe River and Los Gatos Creek. Therefore, when these watercourses are at elevated stages, the piezometric head in the underlying sand and gravel layers will rise quickly with a corresponding change in the groundwater level at the site.

During dry weather conditions when the watercourses are low, perched water conditions are typically encountered at the site above low-permeability clay layers.

The deeper major aquifer was historically used for agriculture in the Santa Clara Valley. Over-pumping of groundwater over a period of years caused a lowering of the piezometric head in the aquifer such that artesian pressures were lost, resulting in saline intrusion and land subsidence. A program of groundwater recharge and management has reversed this trend in recent decades, such that artesian pressures have periodically returned to parts of this major aquifer. A summary of the hydrostratigraphy in the vicinity of the site is provided in Table 5.2-1, based on (HMM, 2008).

**TABLE 5.2-1: Generalized Hydrostratigraphy of the Santa Clara Valley Groundwater Basin (HMM, 2008)**

PERIOD	LITHOSTRATIGRAPHY (major component)	HYDRO- STRATIGRAPHY	THICKNESS (feet)	APPROXIMATE DEPTH BELOW GROUND SURFACE TO TOP
Quaternary	Surficial soils: silty sands, sands, and gravelly sands	Unconfined Shallow Aquifer	< 20	n/a
	Clays and silts	Upper Aquitard	50 to 80	<20
Tertiary	Silty sand, sand, gravelly sand, and sandy gravel	Minor aquifer	10 to 45	50 to 80
	Clays and silts	Lower Aquitard	100	80 to 150
	Sand and gravel, interbedded with clay and silts	Major Aquifer	800	200 to 250
Cretaceous to Jurassic	Generally greenstone, Sandstone with shale and conglomerate	Non-aquifer Basement Rock	Bedrock	~1000

Due to the nature of the depositional environment, the hydrostratigraphy (and therefore, available yield) of the aquifer from an individual boring is likely to vary horizontally and vertically.

### 5.2.1 Existing Groundwater Information

Existing groundwater information from historic borings is presented in Appendix A – Project Spartan Geotechnical Story Board, Groundwater Data Tab. This information largely captures the groundwater level within the upper shallow unconfined aquifer. As discussed above, the groundwater level in the shallow unconfined aquifer is generally closely tied or hydraulically connected to the water level of the nearby watercourses. As such, and combined with the variability in the sub-surface profile, there can be large variability in the groundwater levels across the site. Generally, groundwater was encountered between 10 and 25 feet below the ground surface at the site.

The hydrology report (HMM/Bechtel, 2008a) assessed the horizontal permeability of soil surrounding the site in a variety of different ways, including pumping tests, slug tests, laboratory tests, and correlations with particle size distribution. The following horizontal permeability values are suggested according to (HMM/Bechtel, 2008a).

**TABLE 5.2.1-1: Suggested Horizontal Permeability (HMM/Bechtel, 2008a)**

MATERIAL	SUGGESTED PERMEABILITY VALUES (HMM/BECHTEL, 2008a)	
	MIN	MAX
Gravelly sand and sandy gravel	$4.0 \times 10^{-2}$ cm/s	$5.1 \times 10^{-1}$ cm/s
Silty sand	$3.2 \times 10^{-5}$ cm/s	$1.5 \times 10^{-1}$ cm/s
Clay	$2.0 \times 10^{-6}$ cm/s	$3.4 \times 10^{-6}$ cm/s

### 5.2.2 Measured Groundwater Levels During Exploration

We performed pore pressure dissipation tests at various depths within the CPTs during our field exploration to estimate groundwater levels. The purpose of these tests is to determine whether there are any groundwater changes from potential aquifers. We summarize the pore pressure dissipation tests in the table below;

**TABLE 5.2.2-1: Pore Pressure Dissipation Tests (sorted by depth)**

TEST DEPTH (FEET)	EXPLORATION ID	APPROX. DEPTH TO GROUNDWATER (FEET)	APPROX. GROUNDWATER ELEVATION (FEET)
28	CPT-03	Not Achieved	-
30	CPT-03	Not Achieved	-
31	CPT-03	26.2	70.8
32	CPT-01	8.8	73.2
32	CPT-03A	27.1	71.9
45	CPT-02	12.2	74.8
65	CPT-03	24.3	72.7
76	CPT-03A	26.7	72.3
78	CPT-01	8.8	73.2
84	CPT-02	Not Achieved	-

TEST DEPTH (FEET)	EXPLORATION ID	APPROX. DEPTH TO GROUNDWATER (FEET)	APPROX. GROUNDWATER ELEVATION (FEET)
85	CPT-03	23.7	73.3
108	CPT-03	23.1	73.9
114	CPT-03A	25.1	73.9

The groundwater level within our explorations, which extended into both the unconfined shallow aquifer and minor aquifer as suggested by HMM/Bechtel, 2008a, appears to be at a consistent level at the site. We recommend using a preliminary design groundwater elevation of 75 feet NAVD88, which corresponds to a depth of about 10 to 25 feet below ground surface. During dry weather conditions, the stabilized groundwater level could be at much lower elevations. Perched water conditions may also be present.

## 6.0 GEOLOGIC HAZARDS AND DISCUSSIONS

From a geotechnical engineering viewpoint, the site is suitable for the proposed development, provided the geotechnical hazards presented in this report are considered in the project development plan.

The major geotechnical hazards at the site are listed below. The risks associated with these hazards should be addressed with site-specific geotechnical explorations during the project design development.

- Strong seismic ground shaking
- High groundwater level and rapid groundwater flows during basement excavations
- Liquefaction
- Compressible soil
- Undocumented fill
- Expansive soil

These items and other geotechnical issues are discussed in the following sections of this report.

### 6.1 STRONG SEISMIC GROUND SHAKING

Multiple active faults capable of a magnitude 7.0 or greater are located within 20 miles of the project site; therefore, the potential for severe ground shaking should be considered in the design.

To mitigate the shaking effects, all structures should be designed using sound engineering judgment and the current CBC requirements, as a minimum. Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the actual forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural and nonstructural damage. Conformance with current building code provisions does not constitute



any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAO, 1996).

As described in Section 6.3, there is potentially liquefiable soil within the alluvium that could liquefy in an MCE event. Therefore, the site is classified as a Site Class F. Based on the height of the buildings and anticipated high fundamental period, a site-specific horizontal ground motion response spectrum should be developed for future stages of design. We collected shear wave velocities within the upper 100 feet of the site in S-CPT-2 and S-CPT-3A to aid in development of site-specific ground motion response spectra for future phases of the project.

**TABLE 6.1-1: Average Shear Wave Velocities within the Upper 30 Meters**

LOCATION	AVERAGE $V_{s30}$ (ft/s)	AVERAGE $V_{s30}$ (m/s)
S-CPT-2	804 <sup>1</sup>	245 <sup>1</sup>
S-CPT-3A	802	258

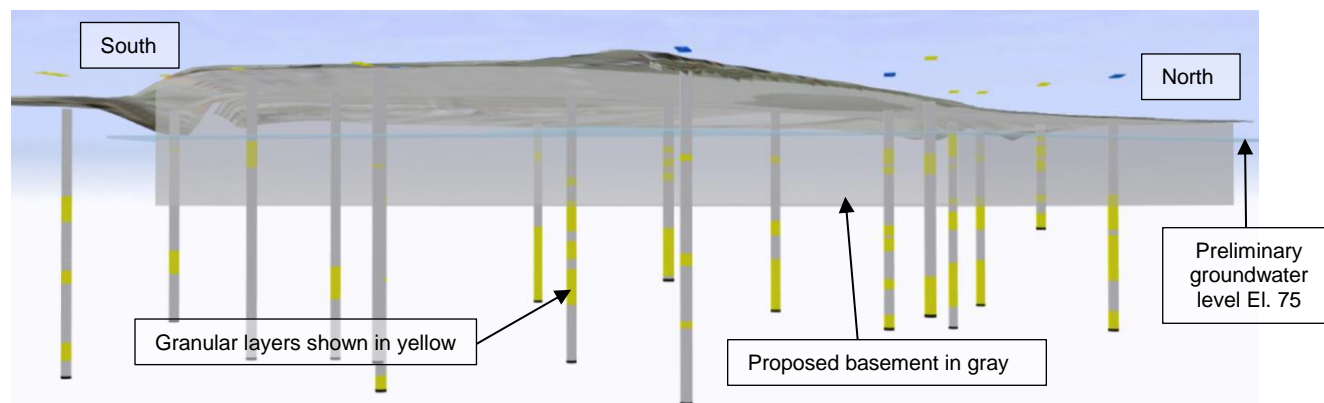
<sup>1</sup>Note due to rig refusal during exploration,  $V_{s30}$  is taken from the upper 80 feet only

## 6.2 HIGH GROUNDWATER LEVEL AND RAPID GROUNDWATER FLOWS

The planned basement excavations for Scenario 1 are approximately 35 to 50 feet below existing grade. As discussed in Section 5.2.2, we recommend a preliminary design groundwater elevation of 75 feet (NAVD88). The base of excavations for any planned basements may be up to 30 feet below the groundwater table.

The granular layers consisting of sand and gravel within the Holocene-age alluvium are likely to be highly permeable, and previous pump tests by others in the vicinity of the site (PARIKH Consultants Inc., 2000) indicate a potential for high groundwater flows in excavations extending below the groundwater table. We mapped the granular layers at the site and present them graphically within Appendix A – Project Spartan Geotechnical Story Board, 3D subsurface explorations Tab. Illustration 6.2-1 shows a cross-section generated from these data with granular layers in yellow and the potential basement excavation depth shown in gray.

**ILLUSTRATION 6.2-1: Graphical Model of Granular Layers**



In general, the northern end of the site has more frequent and thicker granular layers near the base of proposed basement excavations. Due to the presence of thicker granular layers within the Holocene Alluvium at the northern end of the site, there may be higher groundwater flows in excavations in this area. Additional mitigation measures such as localized deeper cut-off walls and/or more intensive and targeted explorations to determine the target depth of cutoff walls are likely to be required. Other localized areas of thicker granular layers with the potential for high groundwater flows may also be present. CPT-1 encountered the thickest granular deposits between elevation 55 and 25 (30 feet thick). We present recommendations for cutoff wall options for basement excavations in Section 7.1.2.

Groundwater levels should be monitored during future phases of the project to assess site-specific groundwater conditions over extended periods of time along with a specific hydrological assessment if Scenario 1 design is progressed.

### 6.3 LIQUEFACTION

Soil liquefaction results from loss of strength during cyclic loading, such as that imposed by an earthquake. The soil considered most susceptible to liquefaction is clean, loose, saturated, uniformly graded, fine-grained sand. Empirical evidence indicates that loose to medium dense gravel, silty sand, low-plasticity silt, and some low-plasticity clay is also potentially liquefiable. Based on the Seismic Hazard Zones map (San Jose West Quadrangle CGS, 2002), the site lies within an area classified as having susceptibility to liquefaction.

We performed a preliminary liquefaction potential analysis of the CPT data using the computer software CLiq Version 2.1 developed by GeoLogismiki. We performed our analysis using the method by Idris and Boulanger, (2014) a Peak Ground Acceleration ( $PGA_M$ ) value of 0.5g as outlined in the 2016 CBC, and a moment magnitude of 8.0 based on the nearby San Andreas fault. We evaluated liquefaction potential of the Holocene alluvium deposits; we consider the Pleistocene deposits to have low potential for liquefaction due to depth, density and aging effects.

We identified lenses of liquefiable loose silt and sand in the CPTs and previous geotechnical data at the site, including liquefiable deposits generally ranging from ½ inch to 6 feet thick within the medium dense to dense, stiff interbedded silt/sand Holocene strata. Earthtech (2006) also identified an area to the southeast of the site with the potential for liquefaction.

Based on our preliminary assessment, the risk for liquefaction is moderate in localized areas of the site. The planned basement excavation will remove some of the potentially liquefiable layers, reducing the effects of liquefaction on development at the site. Nevertheless, both Scenarios 1 and 2 will need to consider some limited liquefaction-induced total and differential settlement (presented below), with the amount depending on the final basement elevations and proposed foundation types. In the event residual computed settlements exceed tolerances for the planned uses, ground improvement may be appropriate under some building foundations.

#### 6.3.1 Liquefaction-Induced Settlement

We calculated potential liquefaction-induced settlement estimates using the program Clmq based on the methods published by G. Zhang, P.K. Robertson, and R. Brachman (2002). Due to the variability in predicting liquefaction-induced settlements, we also compared the above method with other commonly accepted methods.

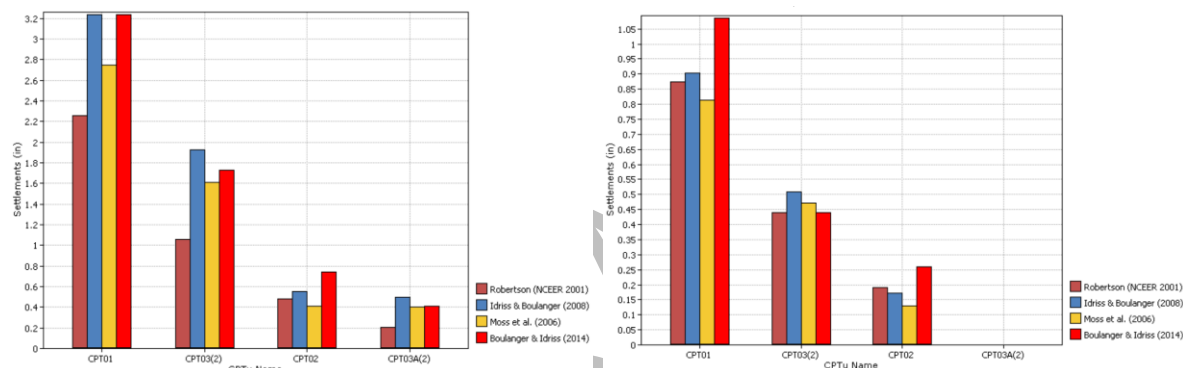
We estimate that liquefaction-induced settlement varies across the site. For preliminary design purposes, we estimate the total liquefaction-induced settlement across the site to be less than 3 inches with differential settlement of 1½ inches over a horizontal distance of 40 feet (SCEC, 1999).

For Scenario 1, where a basement is proposed, the excavation will remove some of the potentially liquefiable material. In this case, for preliminary design, we estimate the total liquefaction-induced settlement across the site to be less than 1 inch with differential settlement of ½ inch over a horizontal distance of 40 feet (SCEC, 1999).

This study is only a preliminary assessment and localized locations of thicker liquefiable material may be present at the site. Further evaluation of this hazard should be performed as part of final design.

**ILLUSTRATION 6.3.1-1 (Left): Estimated liquefaction settlement without basement**

**ILLUSTRATION 6.3.1-2 (Right): Estimated liquefaction settlement with basement excavation**



## 6.3.2 Ground Lurching and Lateral Spreading

Ground lurching is a result of the rolling motion imparted to the ground surface during an earthquake. Such rolling motion can cause ground cracks to form in weaker soils. The potential for the formation of these cracks is considered greater at contacts between deep alluvium and bedrock. Such an occurrence is possible in the Bay Area region, but based on the site location, the offset will likely be minor.

Lateral spreading involves horizontal ground movements caused by seismic shaking. These lateral ground movements are often associated with a weakening or failure of an embankment or soil mass overlying a layer of liquefied sand or weak soil. The Guadalupe River is situated to the east of the site. The closest portion of the site to the Guadalupe River is over 730 feet away. This distance is over 50 times the height of the creek bank, and due to the relatively flat site topography and depth and thickness of liquefiable material, lateral spreading is not likely at the site.

## 6.4 COMPRESSIBLE SOILS

We performed Dilatometer Testing (DMT) in the vicinity of CPT-2 and CPT-3 to assess the compressibility of the soil. The soft to medium-stiff clay at this site is slightly overconsolidated with isolated areas of normally consolidated clay. The cumulative thickness of the normally

consolidated clay is approximately 5 feet. Some clayey soil within the upper alluvium may be moderately compressible.

Compressible clay can compress under new fill or building loads and cause potentially damaging settlement. We discuss the impact on compressible clay on building foundation types in Section 7.0. We summarize the DTM results in the table below, and DTM outputs are in Appendix B.

The planned basement excavations will result in the removal of soil and a decrease in overburden pressure on the underlying soil; thus, the effects of load-induced settlement are expected to be minimal under the building foundations for Scenario 1, where basements are proposed. Scenario 2 will likely be founded on deep foundations and new loads on compressible soil are expected to be negligible. Compressible soil hazards at the site are applicable to at-grade auxiliary structures and locations of increased fill height due to grading.

**TABLE 6.4-1: Summary of DTM Results**

CPT-02					
TEST DEPTH (Feet)	TEST Elevation (Feet)	Over Consolidation Ratio (OCR)		Undraed shear strength Su (psf)	Vertical Drained Constrained Modulus M (ksf)
		(Mayne, 1995)	(Marchetti et al., 2001)		
10	77	N/A - Granular			1048
11	76	N/A - Granular			330
12	75	N/A - Granular			164
13	74	N/A - Granular			152
14	73	1.3	1.47	381	151
15	72	N/A - Granular			147
27	60	1.57	1.97	638	132
28	59	1.6	2.02	662	139
29	58	1.17	1.25	495	60
30	57	1.23	1.35	531	59
31	56	1.34	1.54	590	133
32	55	1.46	1.76	655	161
33	54	1.43	1.69	652	122
34	53	1.34	1.54	625	115
35	52	1.8	2.43	855	300
36	51	1.69	2.2	820	265
38	49	1.8	2.43	909	300
39	48	1.71	2.25	880	204
40	47	1.55	1.93	814	264
41	46	1.88	2.61	1007	292
42	45	1.97	2.81	1076	396

CPT-03					
TEST DEPTH (Feet)	TEST Elevation (Feet)	Over Consolidation Ratio (OCR)		Undraed shear strength Su (psf)	Vertical Drained Constrained Modulus M (ksf)
		(Mayne, 1995)	(Marchetti et al., 2001)		
23	74	1.16	1.23	545	64
24	73	N/A - Granular			146
25	72	1.01	0.99	518	159
26	71	0.92	0.85	489	74
27	70	1.08	1.09	586	97
41	56	1.43	1.70	976	242
42	55	1.56	1.94	1079	215
43	54	N/A - Granular			2167
44	53	1.36	1.57	974	268
45	52	1.51	1.85	1096	272
46	51	1.39	1.62	1020	222
47	50	0.94	0.88	700	162
48	49	1.17	1.24	883	352
49	48	N/A - Granular			984
50	47	N/A - Granular			685
51	46	N/A - Granular			1149

## 6.5 UNDOCUMENTED FILL

### 6.5.1 Lot A and B - Within the Encapsulated Area

Historic plans and documents indicate that the encapsulated material placed during construction of the SAP Center was placed as compacted engineered fill. We performed two CPTs within the encapsulated material, and the estimated shear strengths are consistent with those expected of a compacted engineered fill. For preliminary design, we assume the cap material to be compacted engineered fill. The encapsulated material is considered contaminated and specific environmental mitigation measures will be required with any disturbance of the encapsulated material.

During our explorations, we also encountered subsurface obstructions beneath the encapsulated material in both CPT-3 and CPT-3A. Environmental explorations conducted at the site at the same time as our geotechnical explorations also encountered obstructions below the encapsulated material at approximately 60 percent of the locations. Drill cuttings from coring through the obstructions (Photos 6.5.1-1 and 6.5.1-2) indicate that obstructions include brick, historic concrete foundations, pavements, and construction debris. The thickness of fill beneath the encapsulated area varies between 5 and 20 feet. Locations of localized deeper areas of fill were encountered on the western side of Lot A and also in the eastern corner of Lot B. Concrete subsurface obstructions within the middle of Lot B were consistent with historical aerial photos in the locations of existing buildings.

**PHOTO 6.5.1-1 (Left): Debris within the fill below encapsulated material (Environmental explorations)**

**PHOTO 6.5.1-2 (Right): Debris within the fill below encapsulated material (Environmental explorations)**



Non-engineered fill, subsurface obstructions, and localized areas of deep fill should be expected below the encapsulated material. Non-engineered fill may undergo excessive settlement, especially under new fill or building loads. The presence of existing subsurface obstructions may also lead to damaging differential settlement. As discussed in Section 6.5.1, building foundation scenarios assessed in this report either include removal of surface soils or do not derive capacities from surface soil. We provide mitigation options for undocumented fill for at-grade structures in Section 7.3.

## 6.5.2 Lot C - Outside of the Cap Area

As discussed in Section 5.1.1.2, existing fill is present up to 5 feet below grade and should be considered non-engineered. Due to the existing historic site use, localized areas of non-engineered fill deeper than 5 feet below grade may be present. Non-engineered fill can undergo excessive settlement, especially under new fill or building loads. Existing foundations or debris may also be present in the fill. Building foundation scenarios assessed in this report either remove surface soils or do not derive capacities from surface soil. As such, mitigation of non-engineered fill is only required for areas such as sidewalks, utility corridors, and roadways outside the building footprint. We provide mitigation options for undocumented fill for at-grade structures in Section 7.3.



## 6.6 EXPANSIVE AND CORROSIVE SOIL

Potentially expansive soil was encountered in many of the borings reviewed. Previous laboratory testing on clayey soil indicates a plasticity index range between 10 and 40, which indicates a low to very high expansion potential. Expansive soil can change in volume with changes in moisture. It can shrink or swell and cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations.

We anticipate that the weight of proposed buildings or the removal of expansive soil for basement excavations will mitigate expansive soil hazards under building footprints. Sidewalks, slabs-on-grade, and lighter improvements along the perimeter of the proposed buildings could be damaged by volume changes associated with expansive soil. Damage can be reduced by (1) removal and replacement with non-expansive soil, (2) deepening the footings or edges of slabs to below the zone of seasonal moisture fluctuation, or (3) using deep foundations such as cast-in-drilled-hole piles. We present expansive soil mitigation options for at-grade structures in Section 7.3.

### 6.6.1 Corrosivity Testing

We reviewed corrosion testing results from the Earthtech (2006) report, which included results of testing on four samples collected near the site ranging from 5 to 40 feet below grade. The pH of the soil ranged from 7.36 to 7.71, soil sulfate content ranged from 28.9 to 68.5 (ppm), and chloride content ranged from 7.4 to 48.3 (ppm). TRC (2016) measured resistivity at 9 feet below grade as 3,050 ohm-cm.

The chloride and sulfate ion concentrations and pH appear to be secondary considerations in affecting corrosion potential and are likely to be negligible for the native subsurface materials. Based on the soil resistivity, the native soil may be characterized as moderately corrosive to buried metallic improvements. Further assessments will be required in future design stages.

## 7.0 PRELIMINARY RECOMMENDATIONS

Based on review of the geotechnical hazards above, We present the following preliminary recommendations for the two scenarios presented in Section 1.2.2 and also at-grade structures in the sections below. Additional site-specific geotechnical investigation will be needed to obtain design-level geotechnical parameters.

### 7.1 SCENARIO 1 – BELOW-GRADE SOLUTION

Scenario 1 considers a three-story below-grade basement excavation with a total building height of 21 stories (18 stories above grade). We have assessed excavation down to Elevation 50 feet NAVD88, approximately 30 to 50 feet below existing grades. We consider the most feasible foundation system for this scenario to be a reinforced structural mat foundation. Basement excavation will need to consider dewatering and construction requirements.

#### 7.1.1 Reinforced Structural Mat Foundation

Structural mat foundations are appropriate for Scenario 1. We discuss the following items relating the reinforced structural mat foundation preliminary design.

## **A. Mat Foundation Capacities**

A conventionally reinforced structural mat or stiffened rib mat would be appropriate for building support. For preliminary design, mats founded in areas not underlain by compressible soil may be designed for an average allowable bearing pressure up to 5,000 psf with maximum localized bearing pressure of 6,000 psf at column or wall loads. Bearing pressures should be refined based on modulus of subgrade reaction in the footprint of the proposed mat during detailed design. For preliminary design, we recommend a subgrade modulus of 150 psi/in. Allowable bearing pressures can be increased by one-third for wind or seismic loads.

Resistance to lateral loads is generally provided by frictional resistance between the foundation concrete and the subgrade soil and by earth pressure acting against the basement walls.

## **B. Liquefaction-Induced Settlement**

Mat foundation design should consider total and differential seismically induced settlements presented in Section 6.3.1. We estimate the total liquefaction-induced settlement to be less than 1 inch with differential settlement of ½ inch over a horizontal distance of 40 feet (SCEC, 1999) following basement excavation.

## **C. Consolidation Settlement**

The planned basement excavations will result in the removal of soil and a decrease in overburden pressure on the underlying soils; thus, the effects of load-induced settlement are expected to be minimal under the building foundations. Load-induced settlement analyses should be performed as part of the design-level analysis.

## **D. Mat Thickness**

Mat thickness should be determined by the structural engineer depending on loading and other requirements. Based on our experience, structural mat thicknesses are expected to be between 2 and 5 feet.

## **E. Mat Subgrade**

To stabilize the bottom of the excavation prior to mat construction, a 2- to 4-inch-thick layer of non-structural concrete can be constructed. Alternatively, the upper 12 to 18 inches of the subgrade can be lime treated or scarified and recompacted. Subgrade preparation is dependent on dewatering conditions and should be assessed following selection of a preferred dewatering type; refer to Section 7.1.2 for more information.

## **F. Excavation Subgrade Heave**

During construction of deep excavations, the overburden pressure on the soil below the planned structures will be reduced by the removal of soil for the below-grade parking area. This may cause the excavation subgrade to heave during excavation and recompress during foundation construction. The shoring wall should be designed to reduce the risk of subgrade heave on the project. This should be assessed during design-level studies in conjunction with the shoring wall design.

### 7.1.2 Tie-down Anchors

The weight of the building will need to be assessed to determine whether it has sufficient weight to offset the hydrostatic uplift with a design groundwater elevation. We understand that there may be a plan to construct the basement levels in a different phase from the above-ground building construction. For preliminary purposes and due to the possibility of staging the garage and building separately, it can be assumed that the below-grade structure only will not have sufficient weight to resist the hydrostatic uplift forces prior to construction of upper levels, and tie-down anchors will be required. This assumption can be reviewed during further design stages.

Tie-down anchors are designed to resist the hydrostatic uplift forces in excess of the building weight. These anchors typically consist of driven piles or relatively small-diameter, drilled, grout-filled shafts with steel bars or tendons embedded in grout. Tie-down anchors should be connected to the building foundations. The shear strengths presented in Section 7.2.1.1 can be used for preliminary design of tie-down anchors, if required.

For preliminary cost-estimating purposes, we recommend tie-down piles comprising 25-foot-long, 24-inch-square driven concrete piles, with one per 100 square feet of basement area. Alternative tie-down types and sizing can be revised during later design stages.

### 7.1.3 Excavation Shoring and Dewatering

#### 7.1.3.1 Construction Dewatering

Section 5.2 outlines the groundwater conditions at the site. Dewatering will be required for excavations extending below the groundwater table to allow for construction under dry conditions. As discussed in Section 6.2, granular layers consisting of sand and gravel within the Holocene-age alluvium are likely to be highly permeable and have the potential for high groundwater flows during basement excavations. To reduce the risk of excessive dewatering during construction, we recommend a relatively impervious shoring system and dewatering from inside the excavation. However, no shoring system is perfectly impermeable and some drawdown of groundwater should be anticipated outside of the excavation. The impacts of this potential dewatering on surrounding improvements should be considered when designing the shoring and dewatering system.

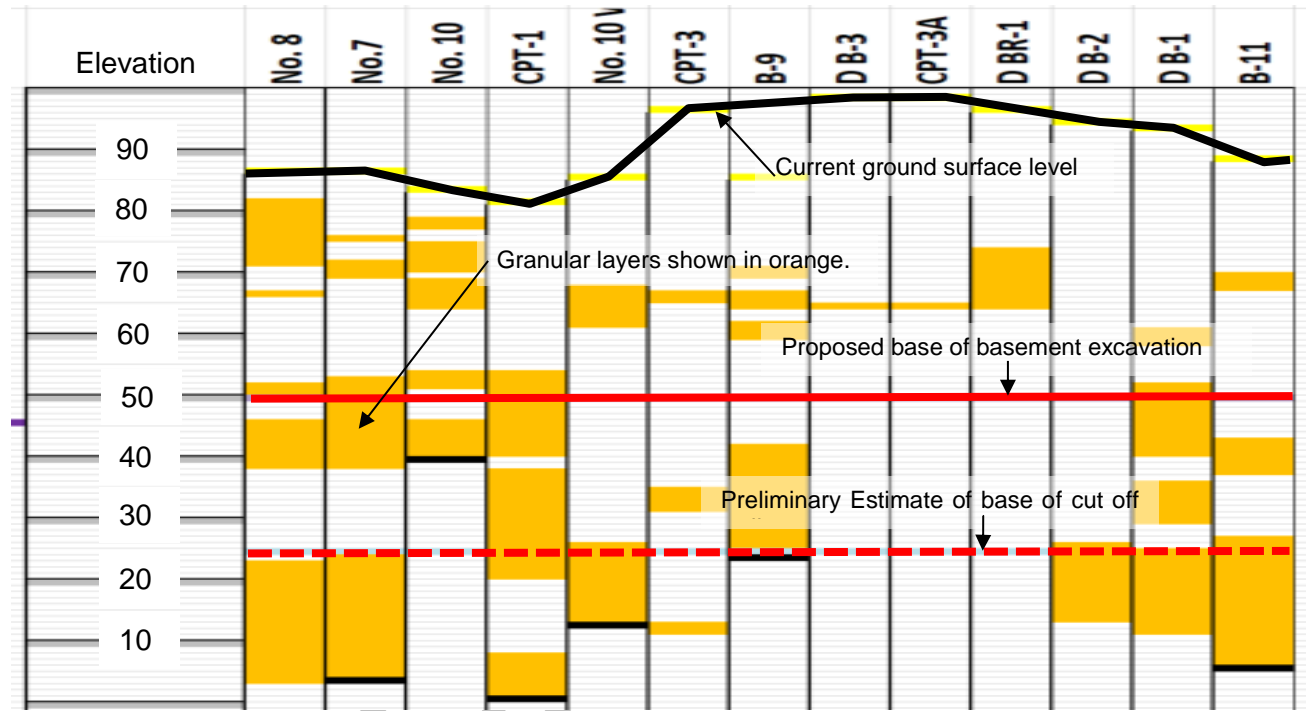
Excavation, dewatering and shoring are typically the responsibility of the contractor to design, install, maintain and monitor. An experienced shoring and dewatering system designer should be retained to select and design these systems. Following are some general considerations that should be incorporated into shoring and dewatering system design. Geotechnical shoring design recommendations are dependent on performance criteria, the type of system selected, and construction sequencing.

For preliminary design purposes, the following shoring systems may be considered suitable for the subject site: Secant piles, diaphragm wall, and cement deep soil mixing (CDSM). We recommend the use of cement-deep-soil-mixed (CDSM) walls for preliminary cost estimating.

The CDSM walls should extend into a low-permeability clay layer. We recommend at least 10 to 15 feet of embedment into low-permeability clay layers below the bottom of the deep excavation for preliminary design. We mapped the granular layers encountered in explorations around the

perimeter of the proposed excavation and recommend the bottom of cut-off wall extend to Elevation 25 feet (NAVD88). The cut-off wall may need to be deeper locally around the areas of CPT-1 due to the presence of thick granular layers below the bottom of the excavation in these areas. The depth and extent of low-permeability clay layers around the perimeter of the site should be explored as part of the site-specific exploration program during detailed design of the shoring system.

**ILLUSTRATION 7.1.3.1-1: Cut-Off Wall and With Granular Layers**



Upon completion of the CDSM walls, groundwater should be pumped from the interior of the planned excavations with a series of wells and pumps working together to remove water in a controlled matter. The dewatering system should be designed and implemented by an experienced dewatering contractor. Regulatory agencies will generally require pumped water to be detained in baker tanks and tested before discharge. We recommend installing perimeter piezometers to monitor the groundwater drawdown around the exterior of the excavation. Excessive groundwater drawdown could result in subsidence and settlement in the immediate area of the active dewatering. Contaminated groundwater extraction and processing should also be considered in dewatering design.

The minor aquifer is estimated to be 50 to 80 feet below the ground surface. Excavation into the minor aquifer may create significant groundwater flows. Our explorations did not encounter significant changes in groundwater elevations at 50 to 80 feet below the ground surface. Groundwater modeling and the evaluation of the proximity of the basement excavations to this aquifer will need to be conducted during detailed design.

### 7.1.3.2 Shoring System

Due to the depth of the excavation, the CDSM may need tieback anchors or internal bracing to be installed to retain the sidewalls and reduce deformations around the excavations. The shoring systems should be designed to limit lateral deformations to existing streets, rail tracks and neighboring structures around the excavation perimeter.

If adequate room is available, cut slopes can be made above the shoring system to reduce the vertically retained height. The slope height, inclination, and temporary excavation depths should not exceed those specified in local, state, or federal safety regulations. Large cut slopes with benches instead of a shoring system will likely be impractical for the proposed development. The specific design of shoring system and type will be required next to sensitive structures, such as the historic Diridon Station, critical utilities, train tracks, pavements, and roadways. We provide parameters for preliminary shoring design below.

### 7.1.3.3 Soil Pressures and Ground Anchors

Shoring walls are commonly designed and constructed in accordance with the Federal Highway Administration (FHWA) Geotechnical Engineering Circular No. 4 (FHWA-IF-99-015). The temporary shoring may be designed for active lateral equivalent fluid pressures provided in the table below. When permanent shoring systems are planned, at-rest pressures plus a hydrostatic pressure of 30 pcf should be considered for wall sections below the design groundwater level (El. 75 feet). For thickness and depth of soil layers, refer to Section 5.0.

**TABLE 7.1.3.3-1: Temporary and Permanent Shoring Preliminary Design Parameters**

SOIL LAYER	AT-REST UNDRAINED PRESSURES (pcf)	ACTIVE UNDRAINED PRESSURES (pcf)	PASSIVE EARTH PRESSURES (pcf)
Fill	110	100	N/A
Holocene	100	90	250
Pleistocene	100	80	300

Where seismic evaluation is performed, permanent shoring should be designed with an additional dynamic increment combined with active equivalent pressures and can be calculated as follows:

$$\Delta P = 23 \times H^2$$

Where H is the design height of the wall (in feet) and  $\Delta P$  is the active incremental seismic force in pounds per foot of wall length. If at-rest pressure plus hydrostatic pressure exceeds the seismic pressure, the permanent shoring wall can be designed for at-rest pressure plus hydrostatic pressure only.

Surcharge loads from structures, stockpiles, and vehicles should be included in shoring design if the surcharge loading is within 20 feet of the top of the excavation or within a 1:1 line of projection extending from the bottom of the excavation, whichever is farther. The surcharge should be taken as one-half of any vertical surcharge loads and should be applied as a uniform lateral load. A minimum lateral surcharge load equal to 72 psf, as prescribed in the Caltrans Trenching and Shoring Manual, should be considered for traffic loading, where applicable.



Ground anchors, also commonly referred to as tiebacks, are structural elements installed in grout-filled holes drilled into soil and are used to transmit applied tensile loads into the ground. The drilling method used for the installation of ground anchors should consider the potential for caving of the drilled holes. Typical tieback inclinations range between 15 and 30 degrees below the horizontal. Ground anchor inclinations up to 45 degrees below the horizontal can generally be installed by most contractors. For preliminary design and cost estimating, the bonded zone of the ground anchors can be assumed to be behind a potential failure plane, drawn from the heel of the wall at a 30-degree angle from vertical. This plane roughly corresponds to the active earth pressure wedge for the site alluvial deposits. The vertical position of ground anchors will depend on capacity requirements and constructability. The horizontal spacing of the ground anchors should be large enough to avoid group effects of anchors. When tiebacks extend beyond the property limits, authorization from neighboring property owners will be required prior to construction.

## 7.2 SCENARIO 2 – AT-GRADE SOLUTION

Scenario 2 considers an approximately 18-story building at grade. We assume that the encapsulated material will remain and buildings will be constructed at existing grade, on top of the encapsulated material. Due to the site topography, it is likely that the building will need to be stepped and imported fill will be required to level the site for each floor level.

We have not been provided specific building loads, but for a building 18 stories tall on a shallow foundation, we estimate the settlement will exceed tolerable amounts and bearing capacity of the upper alluvial soil will be exceeded; we therefore recommend the proposed building for Scenario 2 be supported on deep foundations.

### 7.2.1 Pile Foundations

Deep foundations will gain support through skin friction and end bearing in the alluvial soil. There are many deep foundation systems available, including driven precast or steel piles, CIDH, EDTTEX, Tubex or torque-down piles.

We present preliminary design for driven piles and displacement auger-cast piles. Typically, driven concrete piles are the most common and most cost effective, but the presence of dense to very dense sand and gravel at variable depths may cause the pile lengths to vary considerably during driving.

#### 7.2.1.1 Preliminary Design Parameters

Preliminary skin friction values for pile design are presented in Table 7.2.1.1-1. Piles should be spaced at least three pile diameters center to center. Axial capacity reduction for pile groups should be considered if piles are spaced closer than three pile diameters. Appropriate factors of safety or resistance/load factors need to be applied.

**TABLE 7.2.1.1-1: Preliminary CIDH Pile Vertical Capacity Design Parameters**

GEOLOGIC UNITS	APPROXIMATE TOP OF UNIT ELEVATION (FEET)	GENERALIZED SOIL TYPE	AVERAGE SKIN FRICTION (PSF)
Fill	Surface (100 to 82)	Ignore for preliminary design	
Holocene	81 to 75	CL/SM, CL-CH, CL	750
Pleistocene	50 to 40	CL, SC, SM/ML	2,000

### 7.2.1.2 Vertical Capacities

For our preliminary analyses, we considered 16-inch driven square concrete piles, and 18-inch-diameter displacement auger-cast piles (ACP) to support the buildings. The ability to achieve embedment into the alluvial soils is dependent on the contractor's equipment and technique.

**TABLE 7.2.1.2-1: Estimate of Preliminary Vertical Capacities**

DEEP FOUNDATION TYPE	ESTIMATED PILE LENGTH BELOW ELEVATION 82 (FEET)	PRELIMINARY ALLOWABLE COMPRESSION AXIAL CAPACITY (KIPS) DEAD PLUS LIVE LOAD
16-inch driven square concrete piles	80	170
	100	240
	120	300
18-inch-diameter displacement auger-cast piles	80	150
	100	210
	120	260

For our analysis, we used a Factor of Safety value of 2.0 for skin friction to calculate the allowable capacities. Allowable axial compression capacities can be increased by one-third for the short-term effects of wind or seismic loading.

Prior to construction, if driven piles are anticipated, we recommend that indicator piles be driven to:

- Develop production driving criteria.
- More accurately estimate production pile lengths.
- Evaluate appropriate predrill depths.
- Evaluate the contractor's pile driving system.

At least one of the indicator piles should be subject to a static load test and dynamic pile driving analysis should be performed on the indicator piles. Both the indicator and production piles should be driven with the same pile driving system. We also recommend that the contractor perform a wave equation analysis to assess the compatibility and drivability of the pile driving system with the pile type and soil conditions at the site. The wave equation analysis will also help in assessing the pile driving stresses.

### 7.2.1.3 Lateral Capacity

Lateral load resistance for pile-supported structures is developed through pile-bending/soil interaction. The magnitude of the lateral load resistance is dependent upon several factors, including pile stiffness, pile embedment length, conditions of fixity at the pile cap, the physical properties of surrounding soil, and the magnitude of lateral deflections.

We developed recommended soil properties for estimation of preliminary lateral pile capacities. The soil properties summarized in the following table are for use with the computer program LPILE to estimate lateral pile loads and deflections along with bending moments and points of fixity.

**TABLE 7.2.1.3-1: Preliminary Lateral Pile Analysis Parameters**

LAYER	SOIL TYPE	EFFECTIVE UNIT WEIGHT (pci)	FRICTION ANGLE (degrees)	MODULUS, K (pci)	UNDRAINED COHESION, C (psi)	STRAIN FACTOR E <sub>50</sub>
Fill	Stiff Clay w/o Free Water (Reese)	0.069	-	-	14	0.005
Holocene	Soft Clay (Matlock)	0.069	-	-	3.5	0.02
Pleistocene	Stiff Clay w/o Free Water (Reese)	0.036	-	-	8.3	0.007

Research indicates that the lateral capacity of a group of piles is generally less than the sum of capacities for single piles for spacing less than 6 to 8 pile diameters. We recommend the multipliers provided in Table 7.2.1.3-2 be considered in the LPILE/GROUP analysis. These multipliers are based on AASHTO LRFD BDS where “D” is pile diameter.

**TABLE 7.2.1.3-2: Preliminary Design P-Multiplier Values By Row Position**

	DESIGN P-MULTIPLIER			
Pile Spacing (Center-to-Center)	3D	5D	7D	8D
Lead Row	0.75	1.0	1.0	1.0
Second Row	0.55	0.85	1.0	1.0
Third and Higher Rows	0.40	0.70	0.90	1.0

### 7.2.1.4 Casing Through Contaminated Material

Due to the presence of potentially compacted material within the encapsulated material, additional mitigation measures may be required by the Department of Toxic Substance Control (DTSC). Piles may need to be cased through the area of contamination to prevent migration of contaminated material deeper within the stratigraphy. We recommend assuming that casing will be required in areas of contamination for preliminary design.

### 7.2.1.5 Small to Medium Buildings

Smaller buildings such as wood-framed buildings five stories or less may be supportable on shallow foundations at grade. Specific assessment of this case would be required if this option were to be considered.

## 7.3 AUXILIARY FOUNDATIONS AND RETAINING WALLS

We understand that for both scenarios auxiliary structures will be founded on either a podium structure or a floor slab supported on piles. Auxiliary foundations will be structural and tied into the structure of the building.

### 7.3.1 Retaining Structures

Scenario 2 may require retaining walls for regrading of the site. Provided that adequate drainage is included, retaining walls resisting lateral earth pressures imposed by engineered fill may be designed for the following equivalent fluid pressures (static case).

**TABLE 7.3.1-1: Preliminary Retaining Wall Design Load (Static Case)**

BACKFILL SLOPE CONDITION	EQUIVALENT FLUID PRESSURE (PCF)	
	UNRESTRAINED	RESTRAINED
Level	50	70
4:1	60	75
3:1	70	80

The above lateral earth pressures assume sufficient drainage behind the walls to prevent any build-up of hydrostatic pressures from surface water infiltration and/or a rise in the groundwater level. Dampproofing of the walls should be included in areas where wall moisture would be problematic.

Seismic conditions need to be considered in the design of retaining walls that are greater than 6 feet in height and walls that are parts of buildings. Under seismic conditions, the active incremental seismic force along the face of a retaining wall should be added to the static active pressures (even in restrained wall conditions, the seismic load should be added to the active pressures for that wall height and backfill slope), and can be calculated as follows:

$$\Delta P = 23 \times H^2$$

Where H is the design height of the wall (in feet) and  $\Delta P$  is the active incremental seismic force in pounds per foot of wall length. This force has a horizontal direction and should be applied at  $0.3 \times H$  from the base of the wall.

The effect of any surcharge (dead or live load) should be added to the lateral earth pressures. When traffic loads are anticipated within a horizontal distance from the top of the structure equal to one-half of the wall height, a surcharge load should be applied to the retaining wall. Additional earth pressure from permanent surcharge (dead load) may be analyzed by applying a lateral pressure equivalent to 40 to 60 percent of the total surcharge load onto the wall.

Since fill is being imported, the use of mechanically stabilized earth (MSE) retaining structures may be beneficial. Selection of appropriate retaining wall type should be assessed during detailed design in final layout of the site.

### 7.3.2 Consolidation of Imported Fill

Where new fill is placed for site regrading in Scenario 2, the site soil may compress under new fill or building loads. As discussed in Section 6.4, some of the soft to medium-stiff clay below the site is slightly over-consolidated to normally consolidated. We predicted that 3 feet of new fill placed above the existing grade, with a 5-foot thickness of compressible soil, may result in approximately ½ to 1 inch of settlement. The rate of post-construction settlement is highly dependent on the thickness of the compressible soil and drainage conditions. We estimate that the time to achieve 90 percent total settlement would be less than 1 month and may be managed during construction, before construction of settlement-sensitive structures. Further analysis of compressible soil should be completed once conceptual grading plans and locations of grade-raising are confirmed during detailed design.

## 8.0 CONSTRUCTION MONITORING CONSIDERATIONS

The following construction monitoring should be undertaken during grading and excavation of the below-grade levels.

- Pre-construction survey (photograph and video) of structures within 100 feet of the site prior to any construction work.
- Establishment of deep benchmarks and reference points. These provide a reference to monitor vertical and horizontal movement during and after construction.
- Regular surveys of buildings and sensitive structures within 100 feet from the site during construction and following completion of the works. Crack gauges or other monitoring devices should be used in areas of existing damage. These may include:
  - Surface deflection transect lines
  - Utility monitoring points and markers.
  - Building deflection markers.
  - Shoring deflection markers and inclinometers to monitor shoring deflections with excavations.
- Installation and monitoring of observation wells and vibrating-wire piezometers to monitor groundwater levels during construction and de-watering. Safe working groundwater levels should be established during design.
- Anchor load measurements if tieback soil anchors are used in shoring systems.
- Environmental monitoring such as air, dust, groundwater discharge and stormwater pollution prevention (SWPPP) monitoring.

We also recommend an automated data acquisition system, including an integrated hardware system of data loggers, intermediate data storage units, and a data transmission network to record and monitor construction tolerances on a near-real-time basis.



## 9.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents our preliminary geotechnical assessment of Lots A, B and C of the Spartan Project site. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to, developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than two years from the date of report issuance.

We strive to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the area; no warranty is expressed or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks; therefore, we are unable to guarantee or warrant the results of our services.

This report is based upon field and other conditions discovered at the time of report preparation. We developed this report with limited subsurface exploration data. We assumed that subsurface exploration data researched and reviewed is representative of the actual subsurface conditions across the site. Considering possible underground variability of soil, rock, stockpiled material, and groundwater, additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, ENGEO should be notified immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

Our services did not include excavation sloping or shoring, soil volume change factors, flood potential, or a detailed geotechnical exploration program. This document must not be subject to unauthorized reuse, that is, reuse without written authorization of ENGEO. Such authorization is essential because it requires ENGEO to evaluate the document's applicability given new circumstances, not the least of which is passage of time.

Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to ENGEO's documents. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds.

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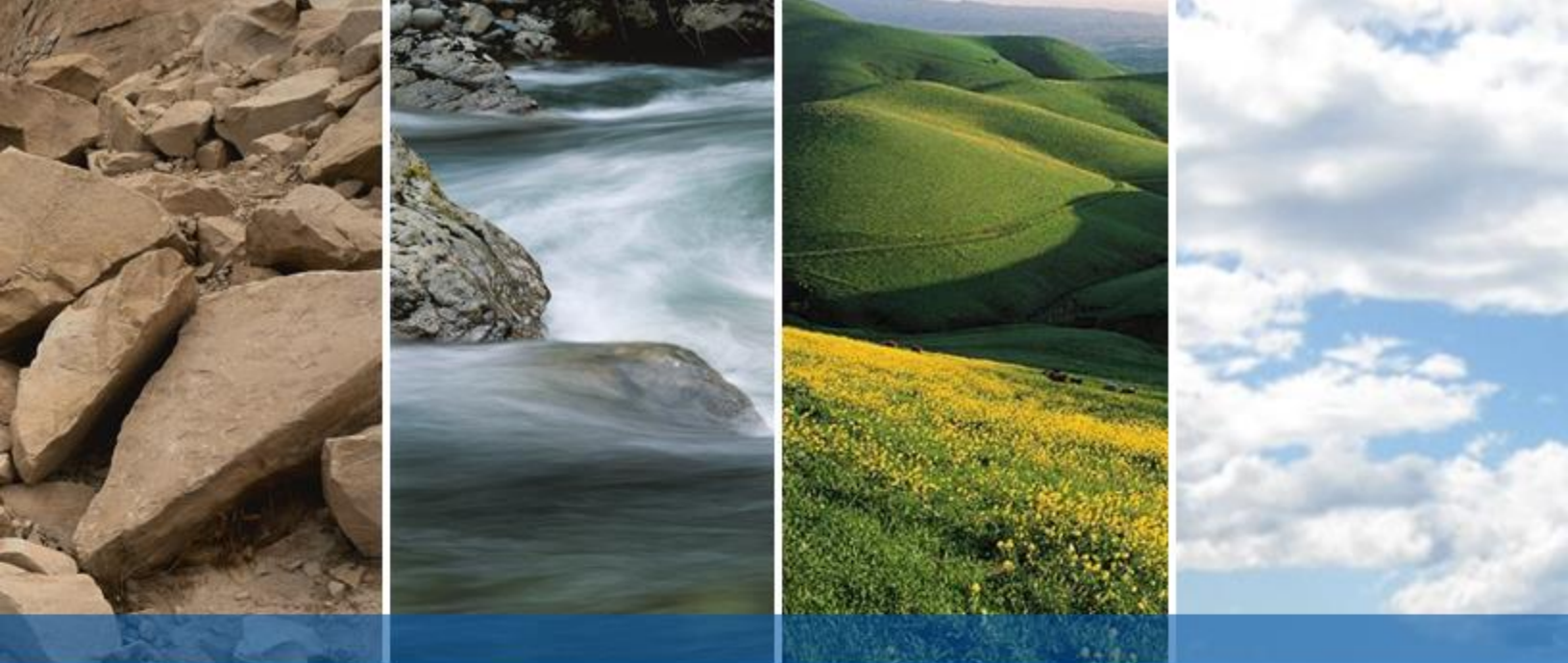
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## **APPENDIX A**

### **Project Spartan Geotechnical Story Board**

The following link provides access to the Project Spartan Geotechnical Story Board

Link:

<https://myengeo.maps.arcgis.com/apps/MapSeries/index.html?appid=83db3c73dc0d4db6be2057c3db90d57c>

Username: GoogleSpartan

P W: Spartan321 (Caps sensitive)

For best viewing results use Google Chrome Web-browser.

The following provides the Table of Contents for the Story Board.

Tabs;

1. Site Plan
2. Property Records
3. Geologic Maps
4. Seismic
5. Previous Geotechnical Reports
6. Borings
7. Groundwater Data
8. Subsurface Data Gaps
9. 3D Ground Model

Screenshots of each tab are presented in the following pages.

Note that the Geotechnical Story Board is a working environment and may be updated after issuance of this report.



## PROJECT SPARTAN GEOTECHNICAL STORY BOARD TAB 1 - Site Plan

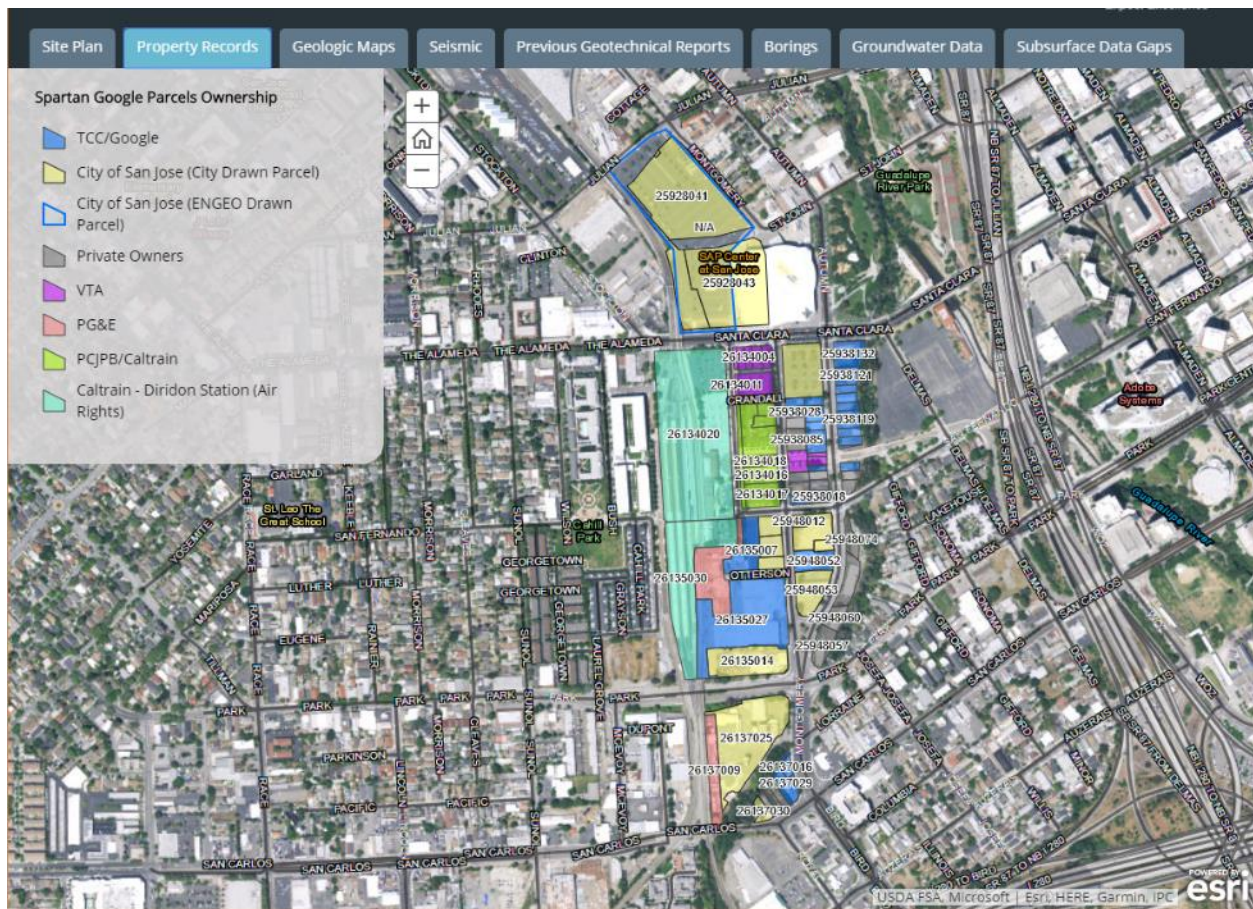


CPRA Confidential Treatment Request Not for Public Release – CPRA Exempt Confidential and Proprietary Business / Siting  
Information Pre-Decision Draft – For Review Only



## PROJECT SPARTAN GEOTECHNICAL STORY BOARD

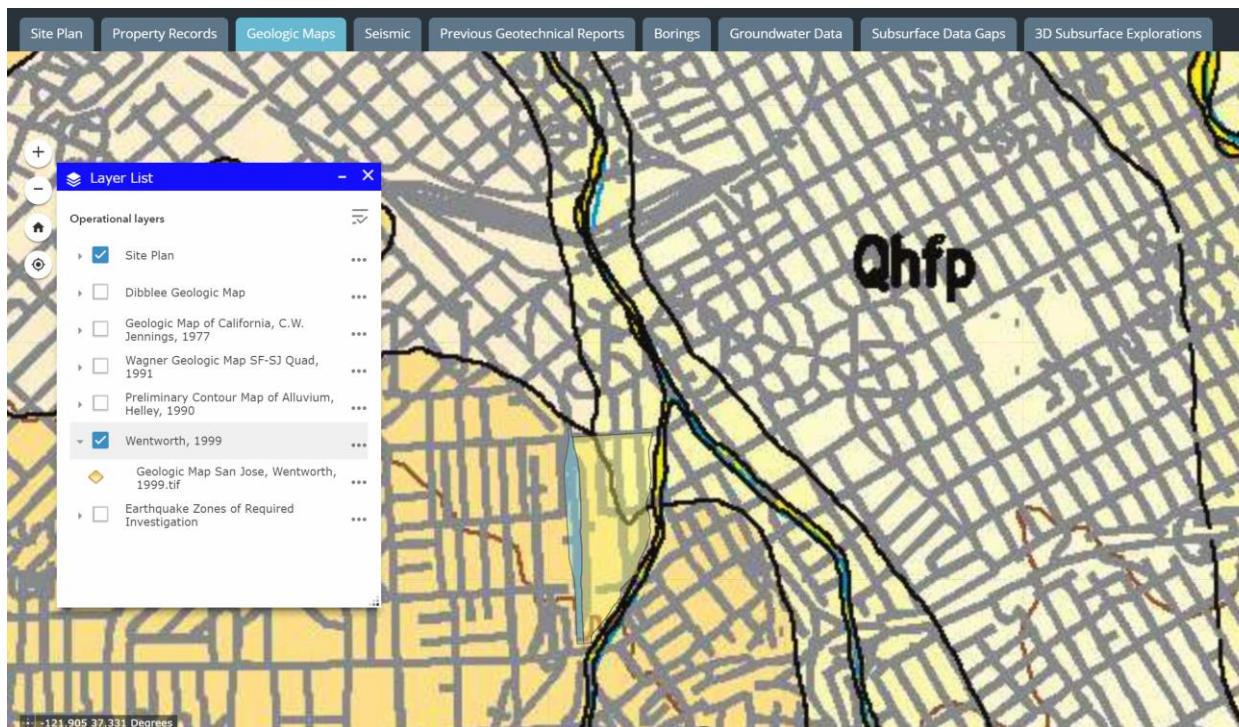
### Tab 2 - Property Records



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Information Pre-Decision Draft – For Review Only

## PROJECT SPARTAN GEOTECHNICAL STORY BOARD

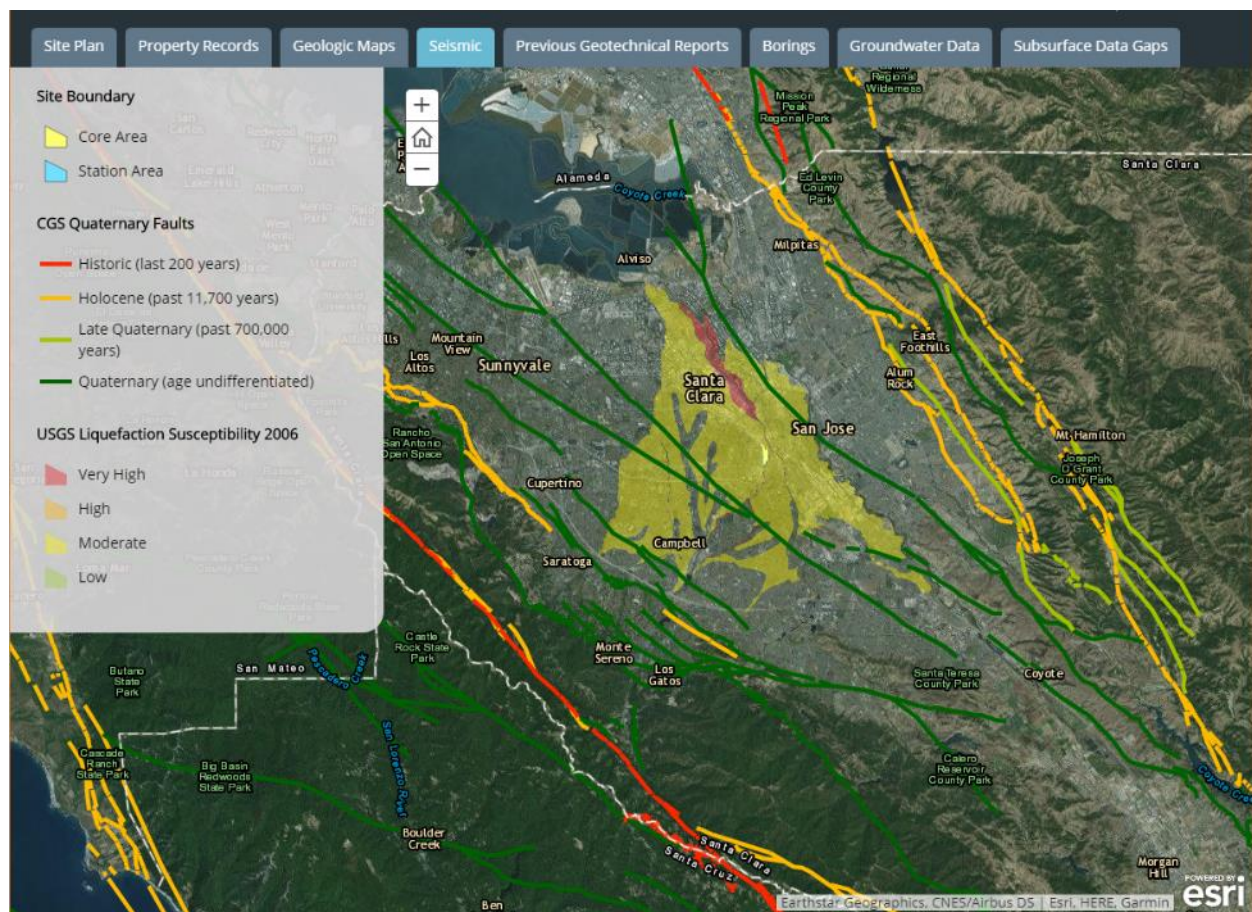
### Tab 3 - Geologic Maps





## PROJECT SPARTAN GEOTECHNICAL STORY BOARD

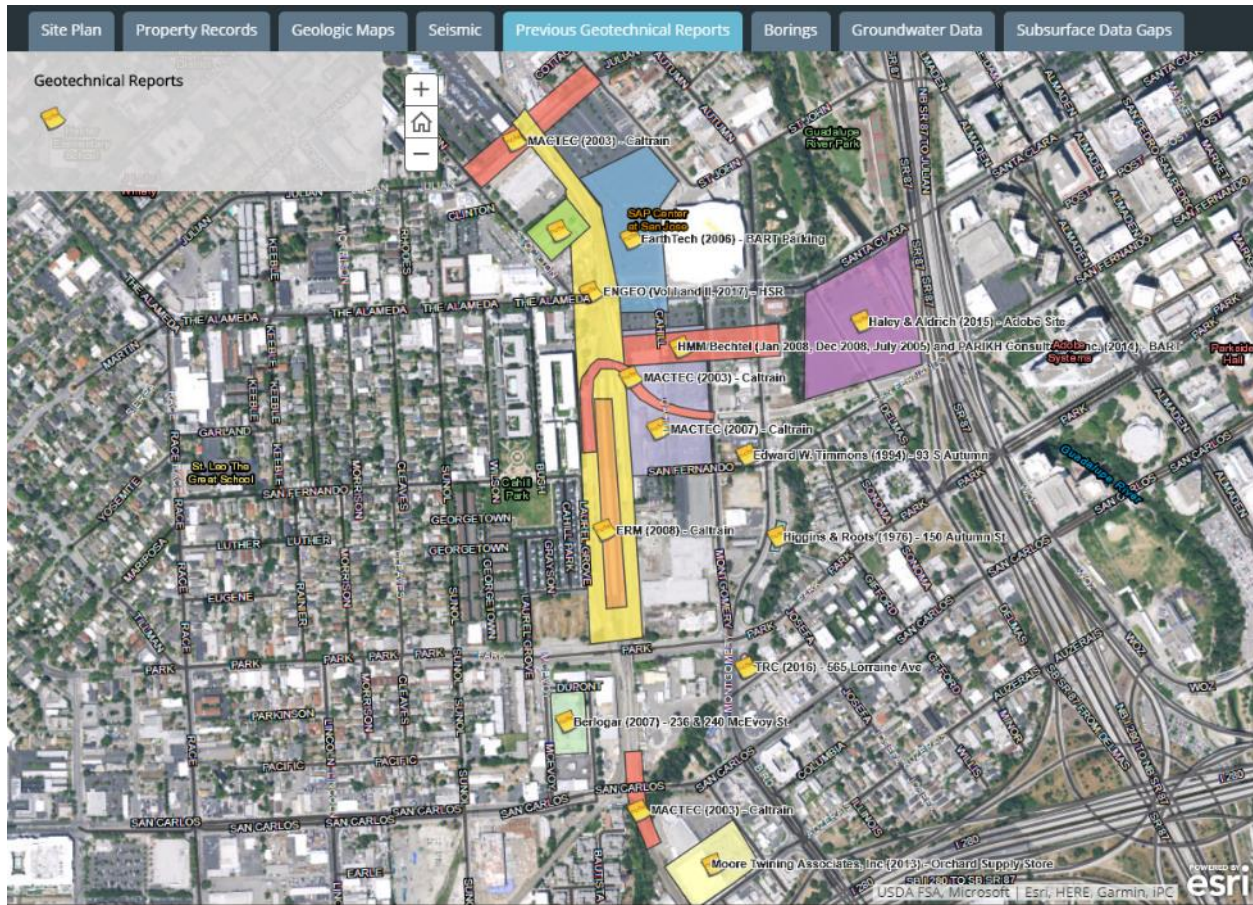
### Tab 4 - Seismic



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Information Pre-Decision Draft – For Review Only

## PROJECT SPARTAN GEOTECHNICAL STORY BOARD

### Tab 5 - Previous Geotechnical Reports

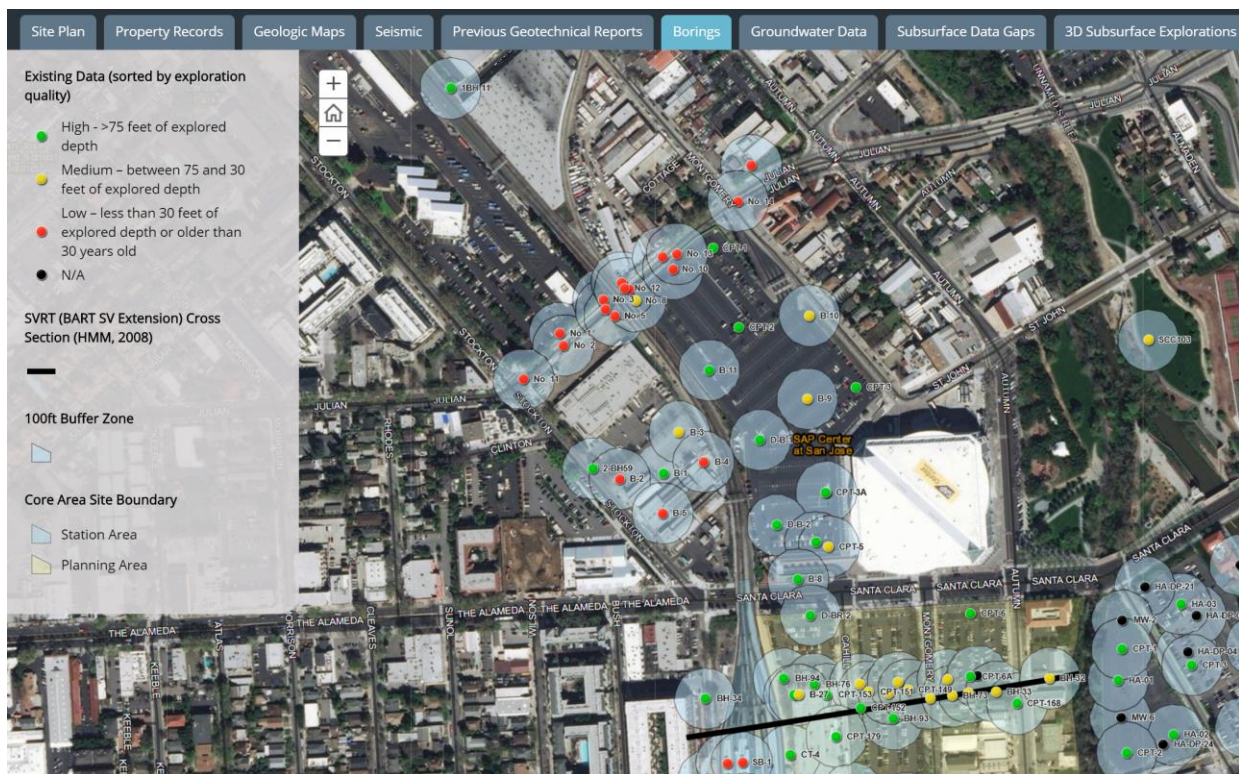


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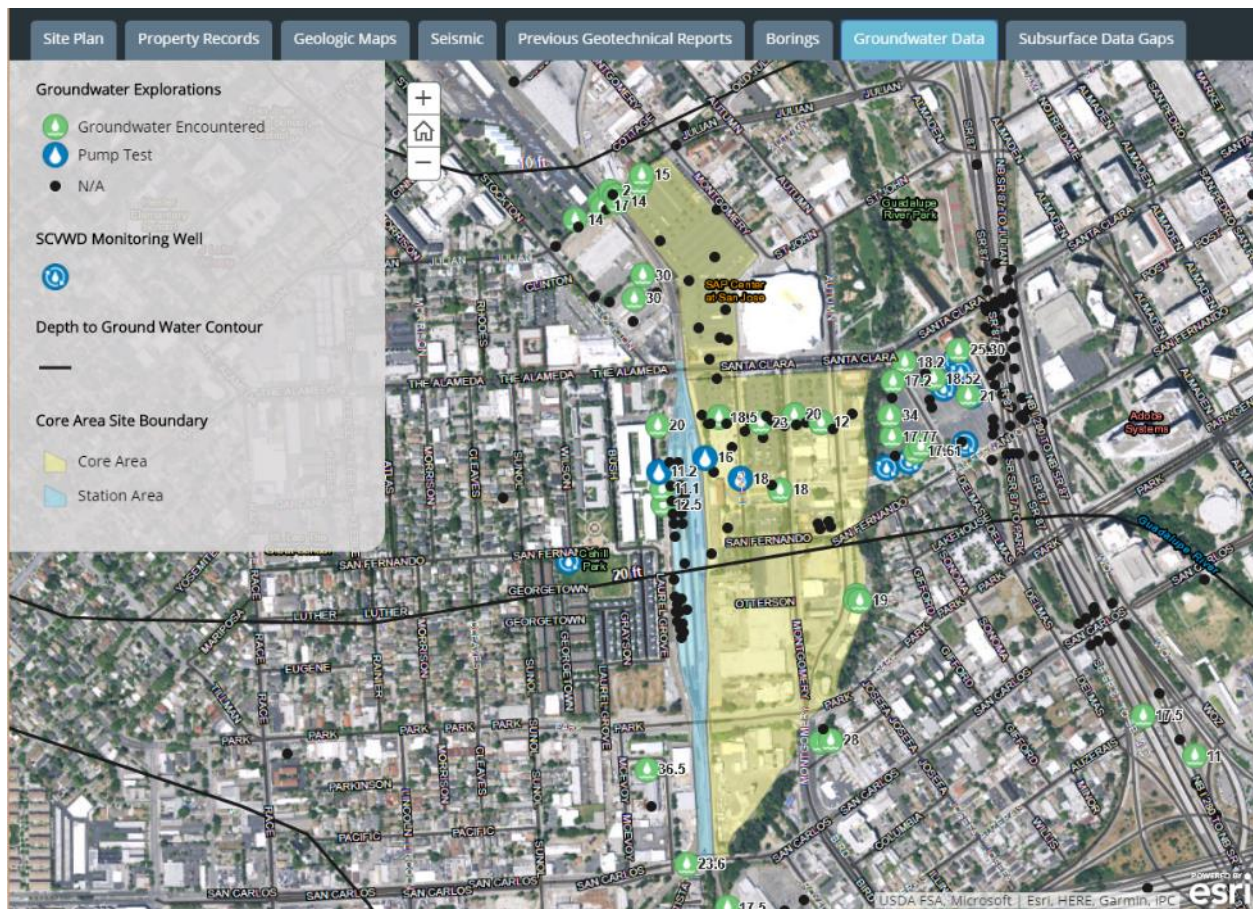
### Tab 6 - Borings



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## PROJECT SPARTAN GEOTECHNICAL STORY BOARD

### Tab 7 - Groundwater Data

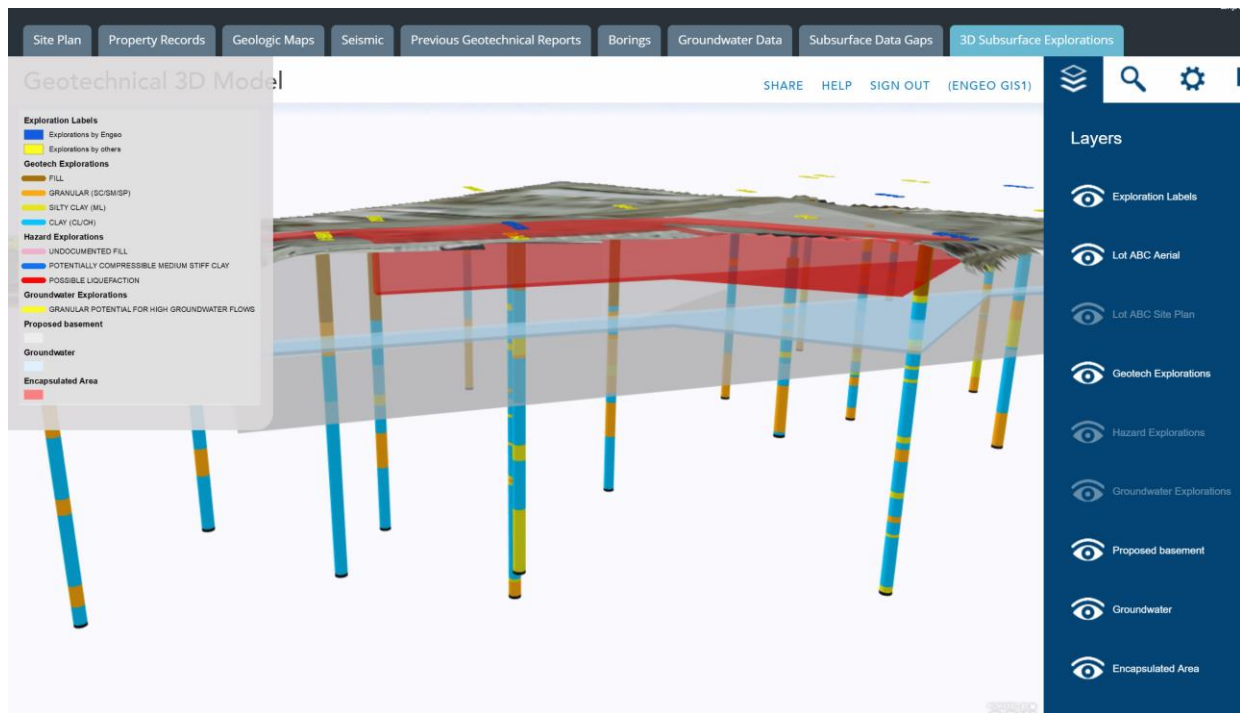


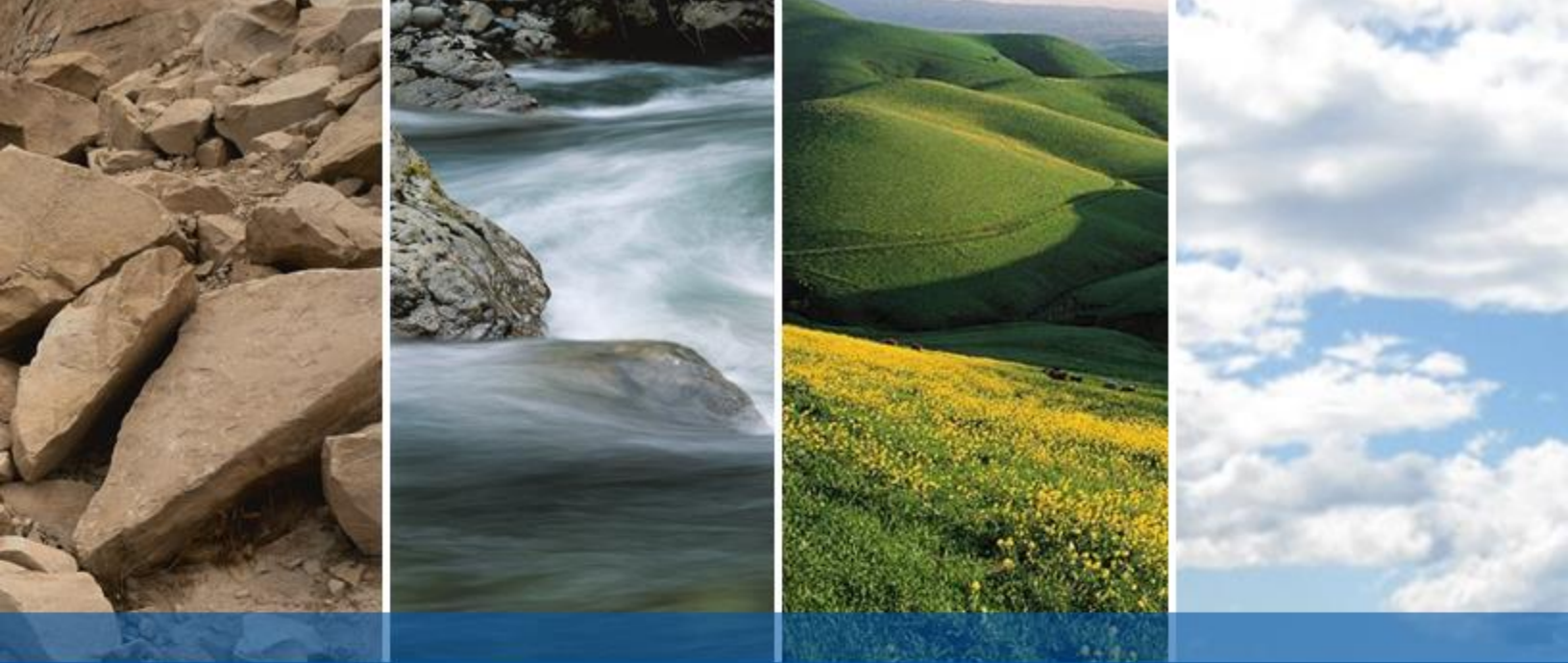
CPRA Confidential Treatment Request Not for Public Release – CPRA Exempt Confidential and Proprietary Business / Siting Information Pre-Decision Draft – For Review Only



## PROJECT SPARTAN GEOTECHNICAL STORY BOARD

### Tab 9 – 3D Subsurface Explorations





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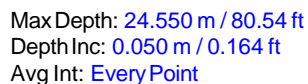
## **APPENDIX B**

### **Exploration Logs**



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Cone: 483:T1500F15U500

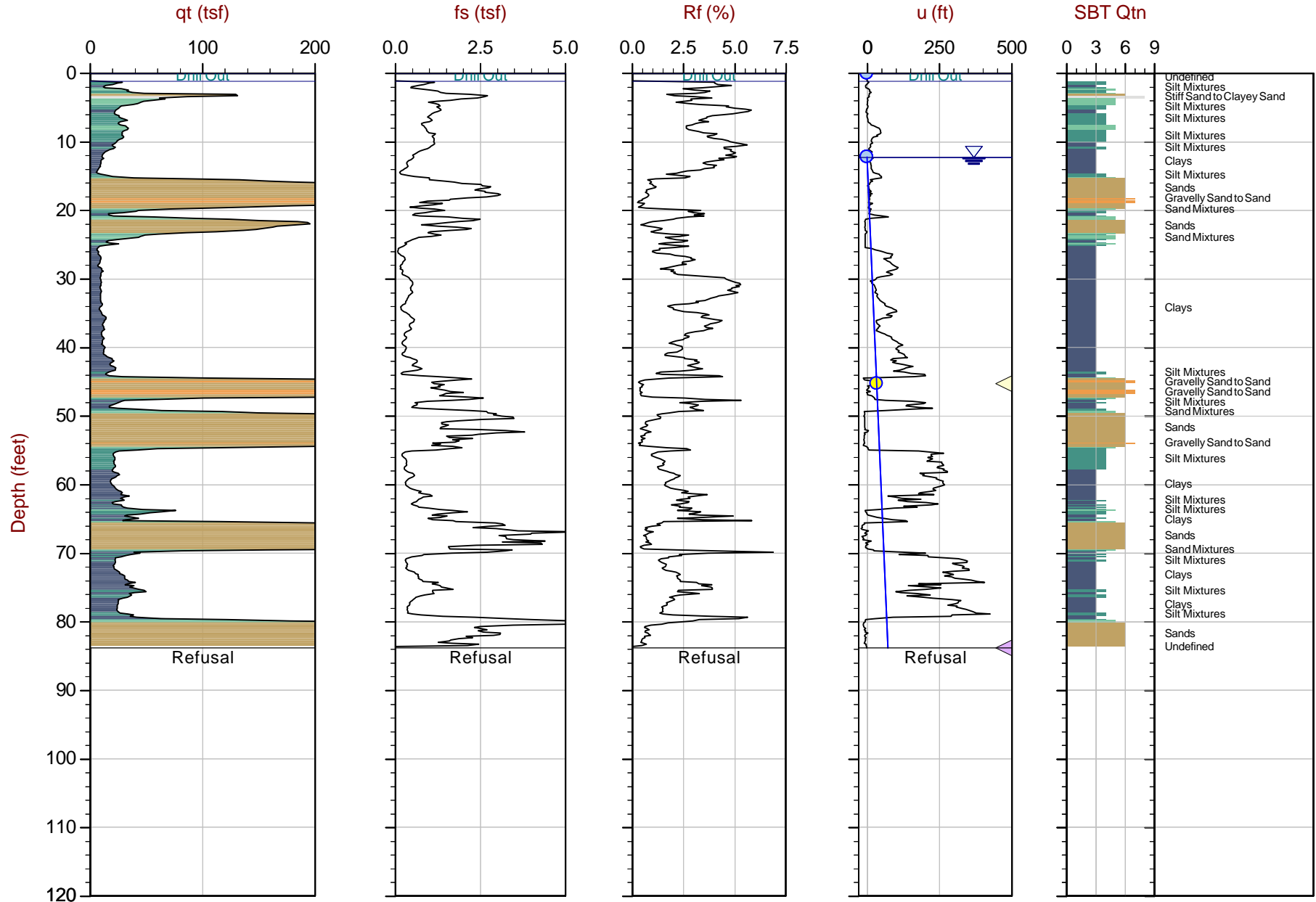


SBT: [Robertson, 2009 and 2010](#)  
 Coords: [UTM10N:4132608mE:597124m](#)  
 SheetNo: 1 of 1

● Equilibrium Pore Pressure (Ueq)    
 ● Assumed Ueq    
 ◀ Dissipation, Ueq achieved    
 ◀ Dissipation, Ueq not achieved    
 — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.





Max Depth: 25.550 m / 83.82 ft

Depth Inc: 0.050 m / 0.164 ft

Avg Int: Every Point

File: 18-56026\_SP02.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM10N N:4132510m E:597150m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)

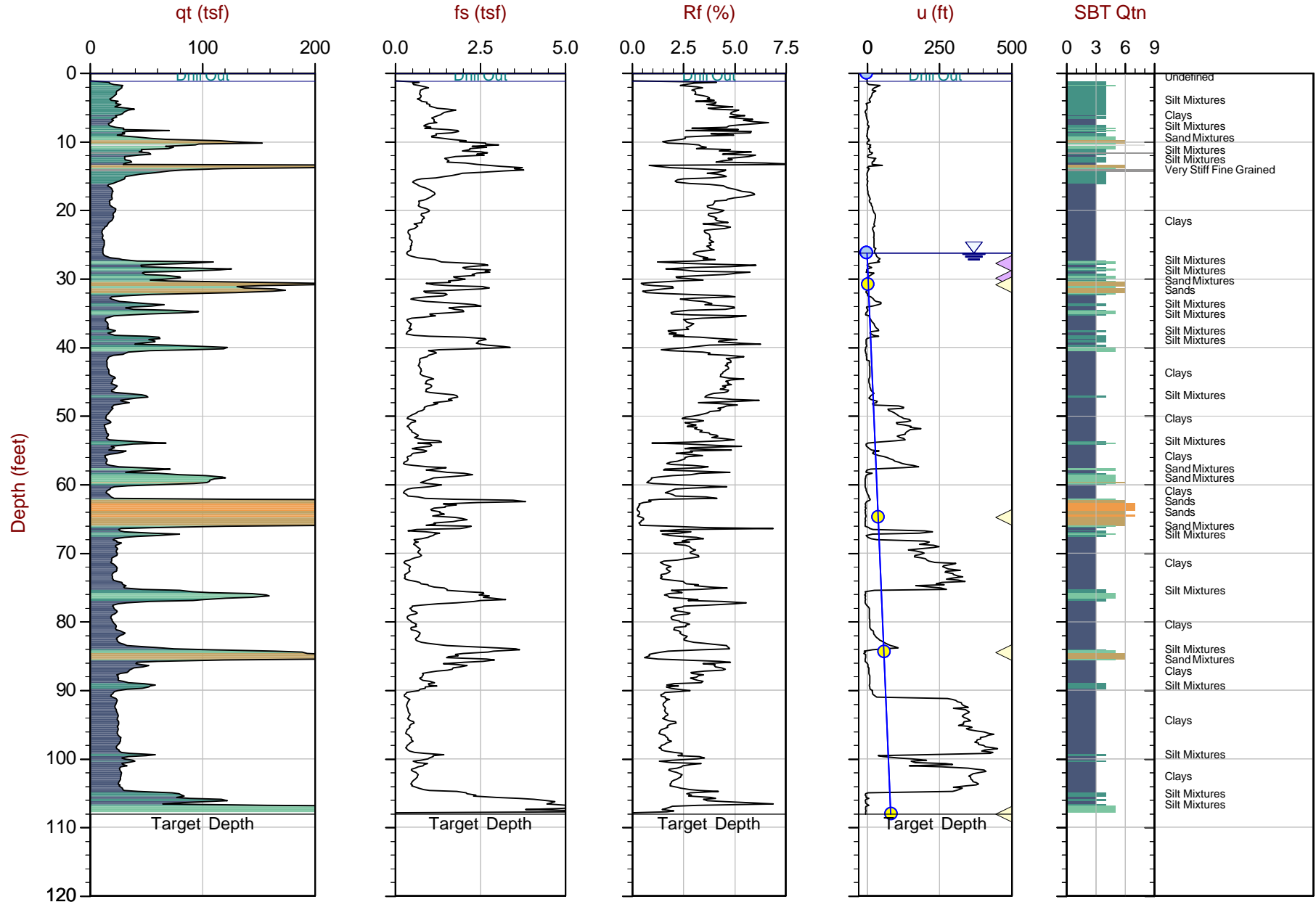
● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Max Depth: 32.950 m / 108.10 ft

Depth Inc: 0.050 m / 0.164 ft

Avg Int: Every Point

File: 18-56026\_CP03B.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM10N: 4132464m E: 597261m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq)

● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

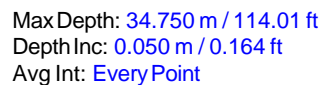
— Hydrostatic Line

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Site: Google Spartan

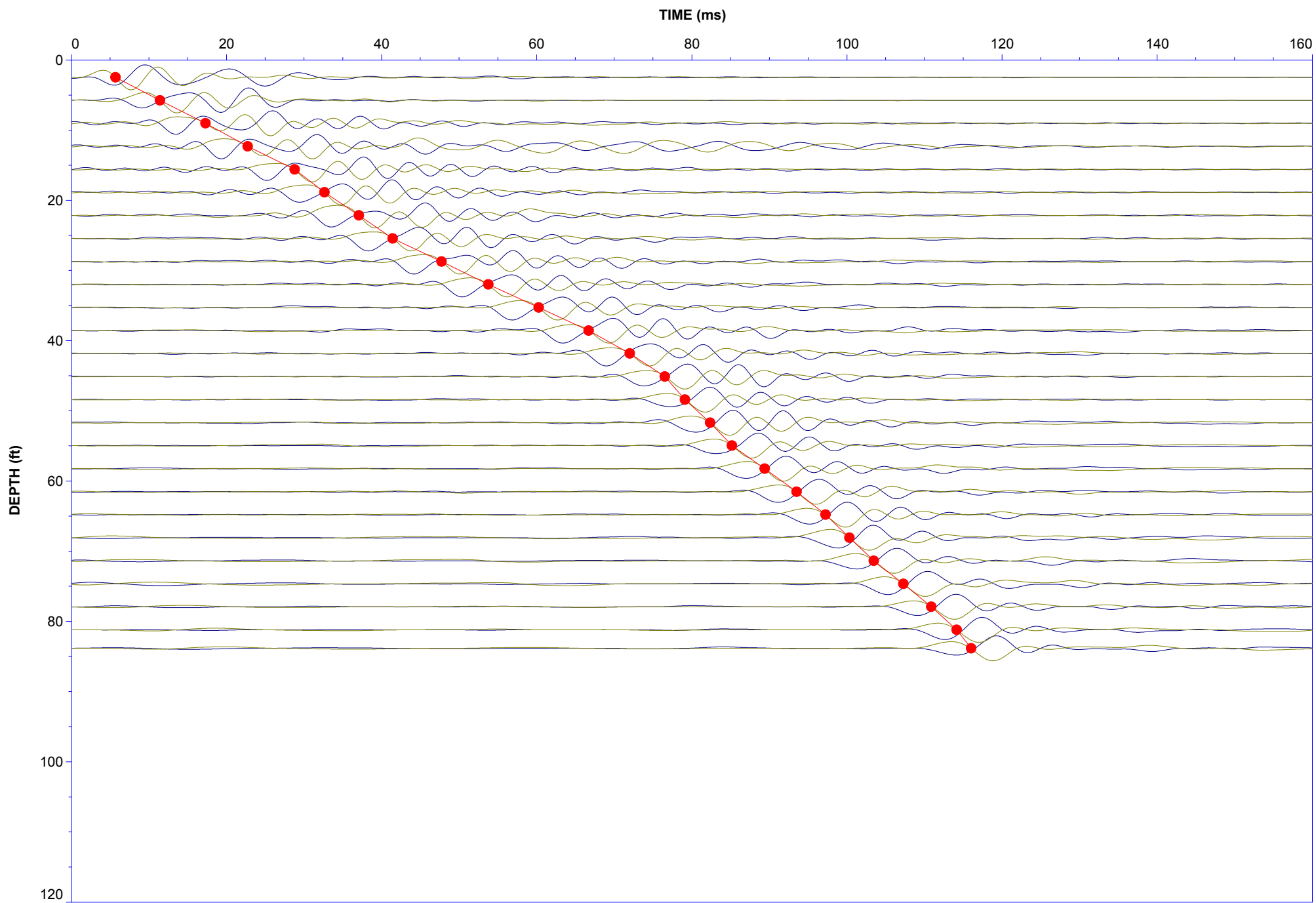
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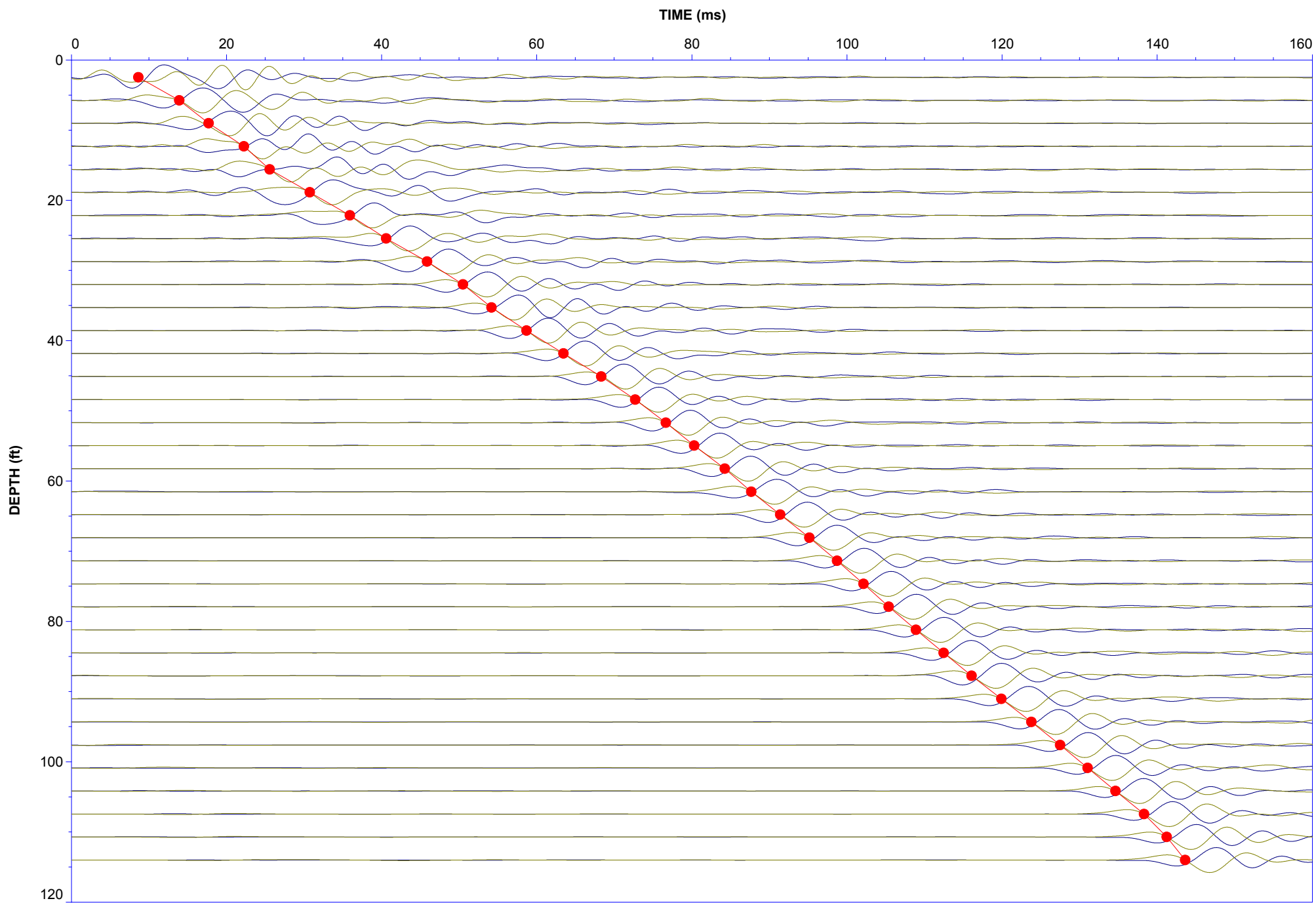


SBT: [Robertson, 2009 and 2010](#)  
 Coords: [UTM10N:4132364mE:597236m](#)  
 SheetNo: 1 of 1

 Equilibrium Pore Pressure (Ueq)    
  Assumed Ueq    
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  Dissipation, Ueq not achieved    
  Hydrostatic Line

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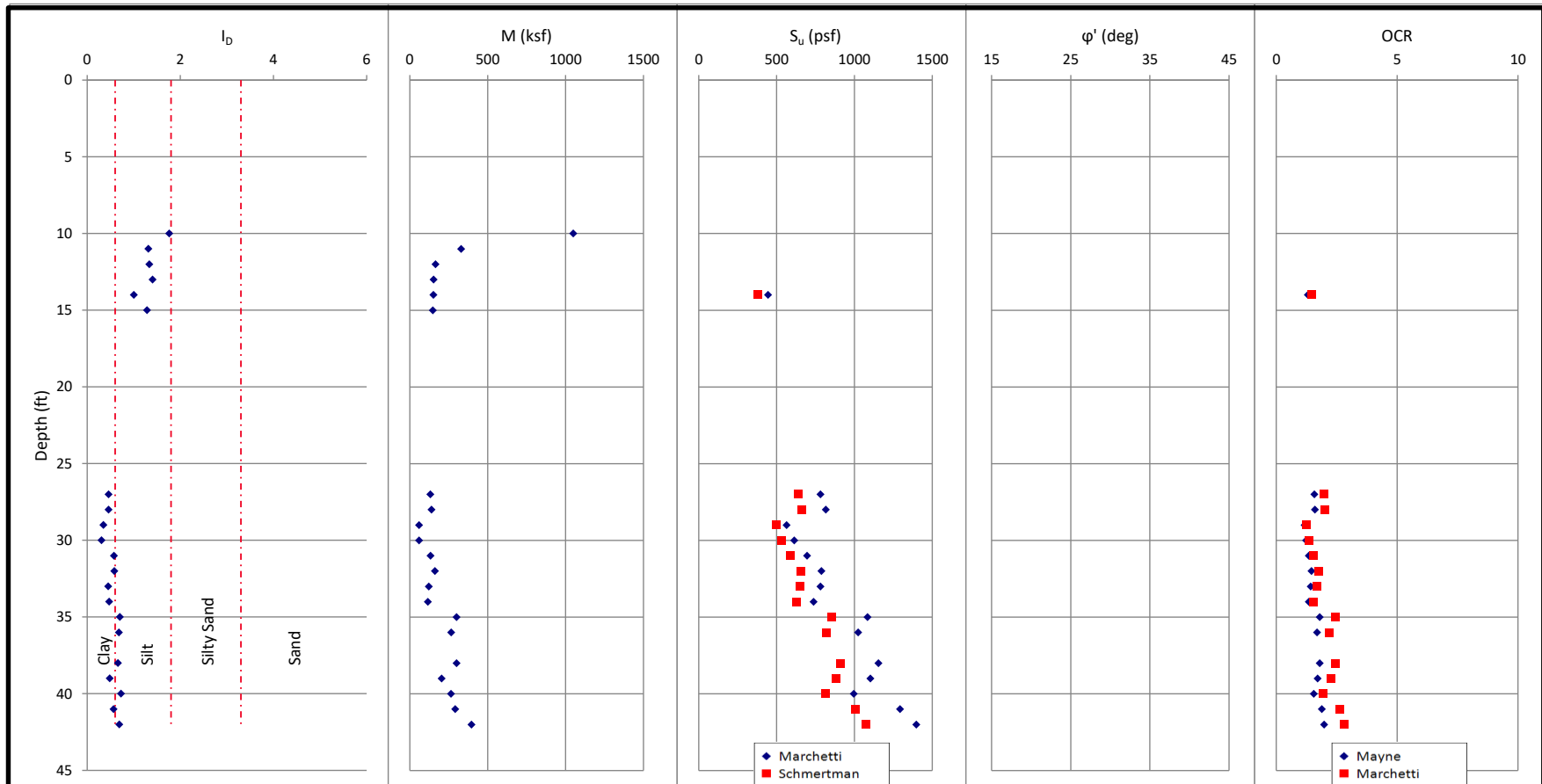




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Client ENGEO Inc.  
Sounding ID 1-DMT02  
Date Mar 29, 2018

Coordinate System WGS 84 / UTM zone 10N  
Northing (m) 4132510  
Easting (m) 597150  
Ground Water Depth (ft) 12.2  
Gauge Zero Offset (bar) 0

### DILATOMETER TEST RESULTS



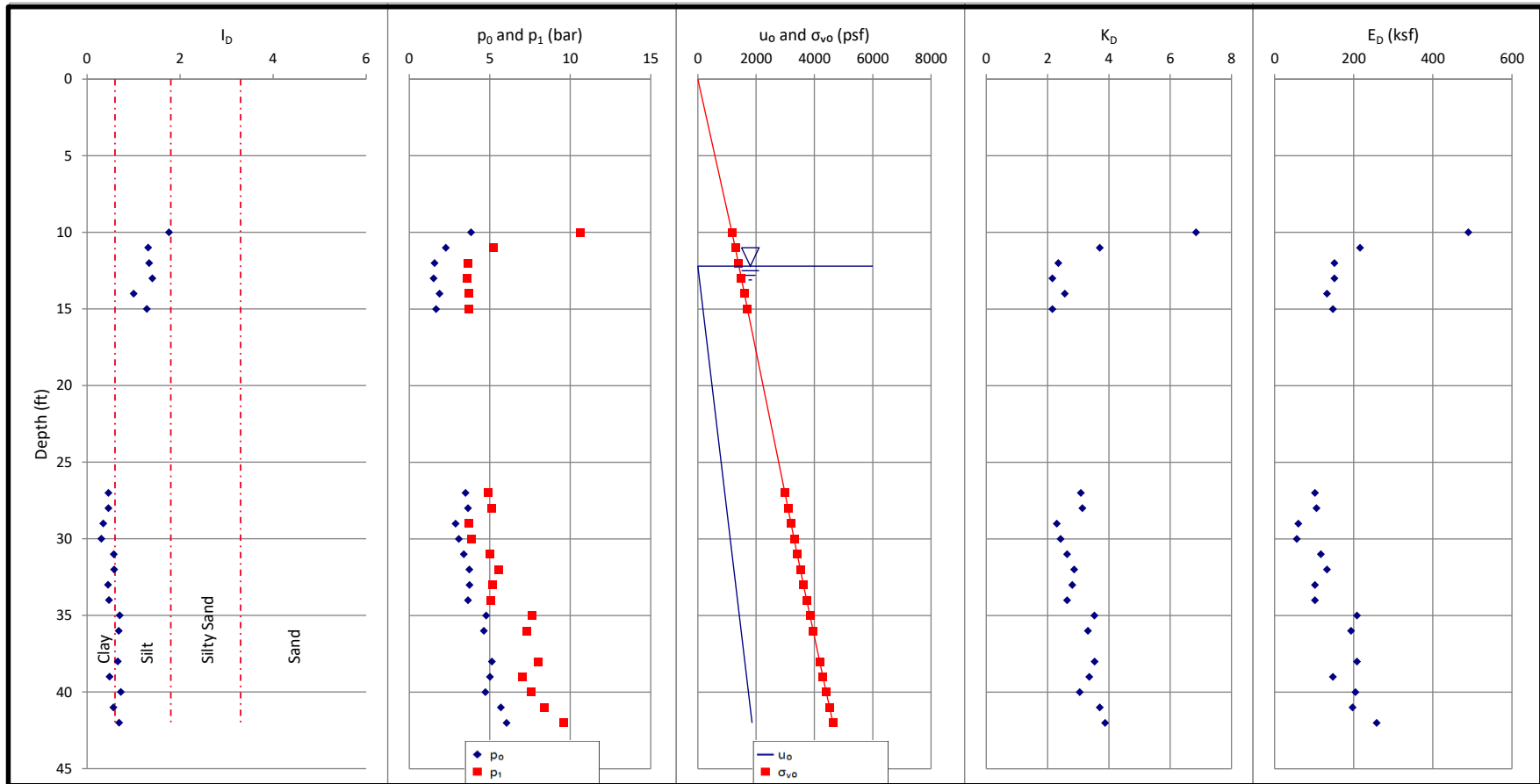


CALIFORNIA PUSH  
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Job No. 18-56026  
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Client ENGEO Inc.  
Sounding ID 1-DMT02  
Date Mar 29, 2018

Coordinate System WGS 84 / UTM zone 10N  
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Easting (m) 597150  
Ground Water Depth (ft) 12.2  
Gauge Zero Offset (bar) 0

### DILATOMETER TEST RESULTS



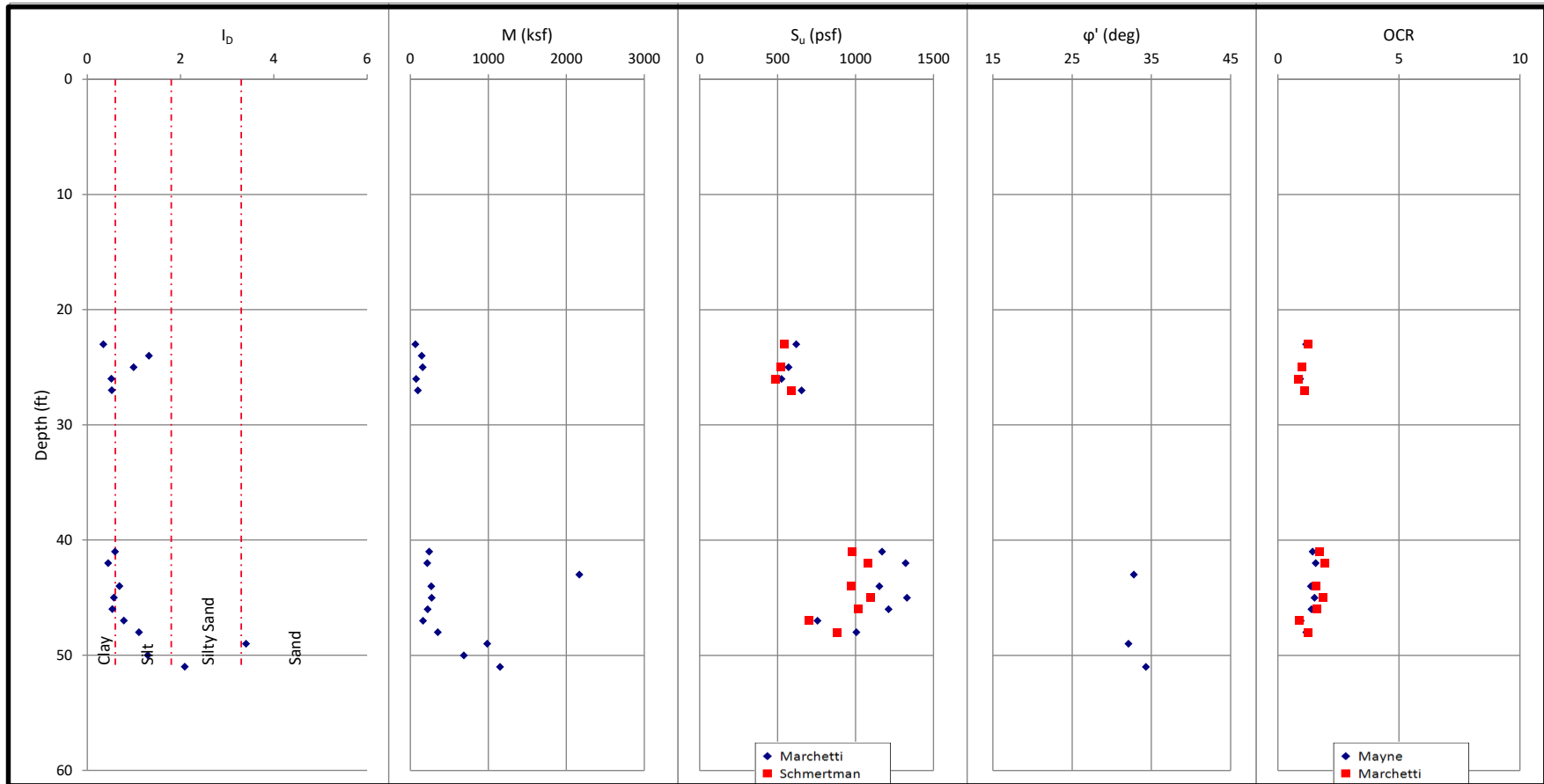


CALIFORNIA PUSH  
TECHNOLOGIES  
INCORPORATED

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Client ENGEO Inc.  
Sounding ID 1-DMT03  
Date Mar 29, 2018

Coordinate System WGS 84 / UTM zone 10N  
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### DILATOMETER TEST RESULTS



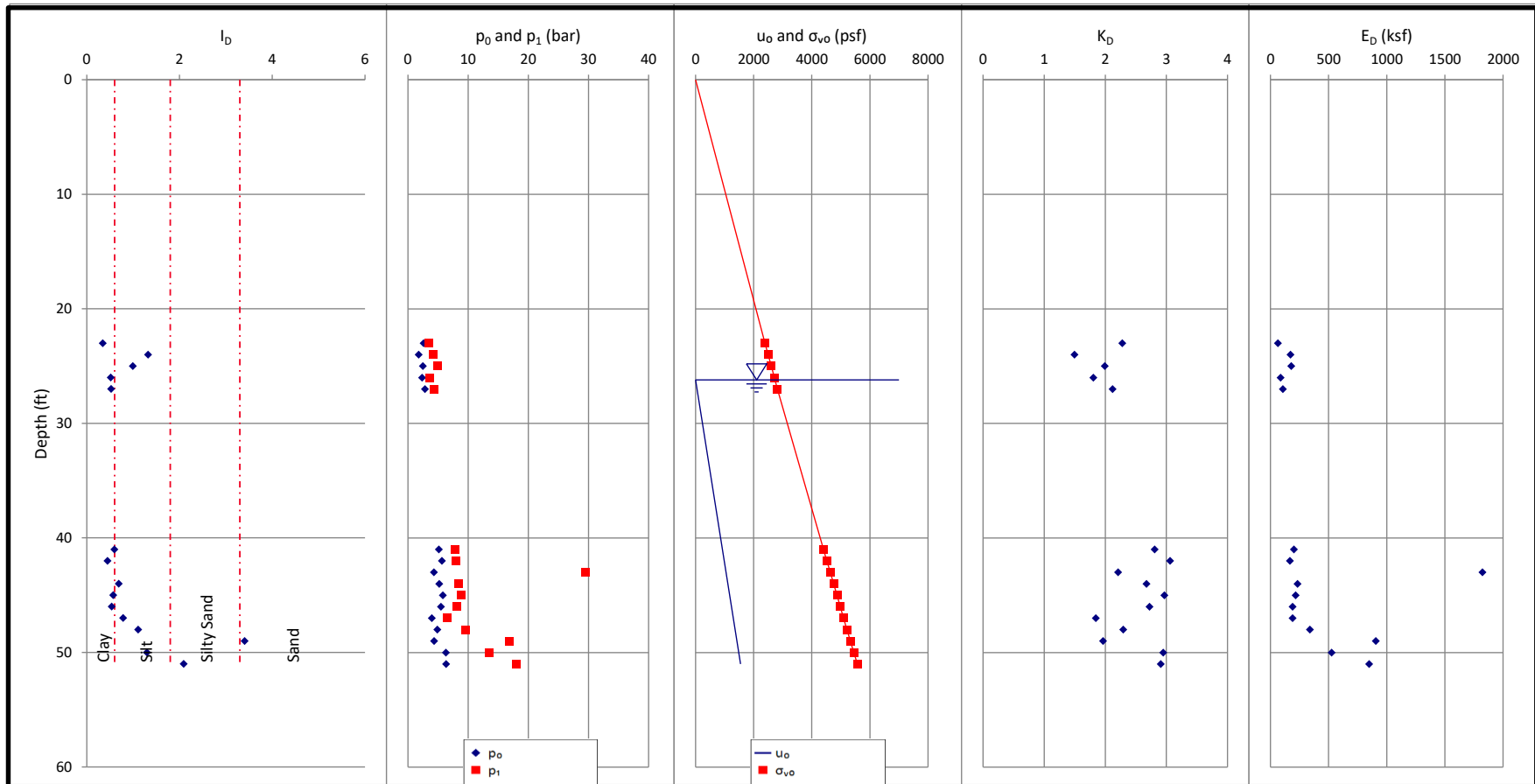


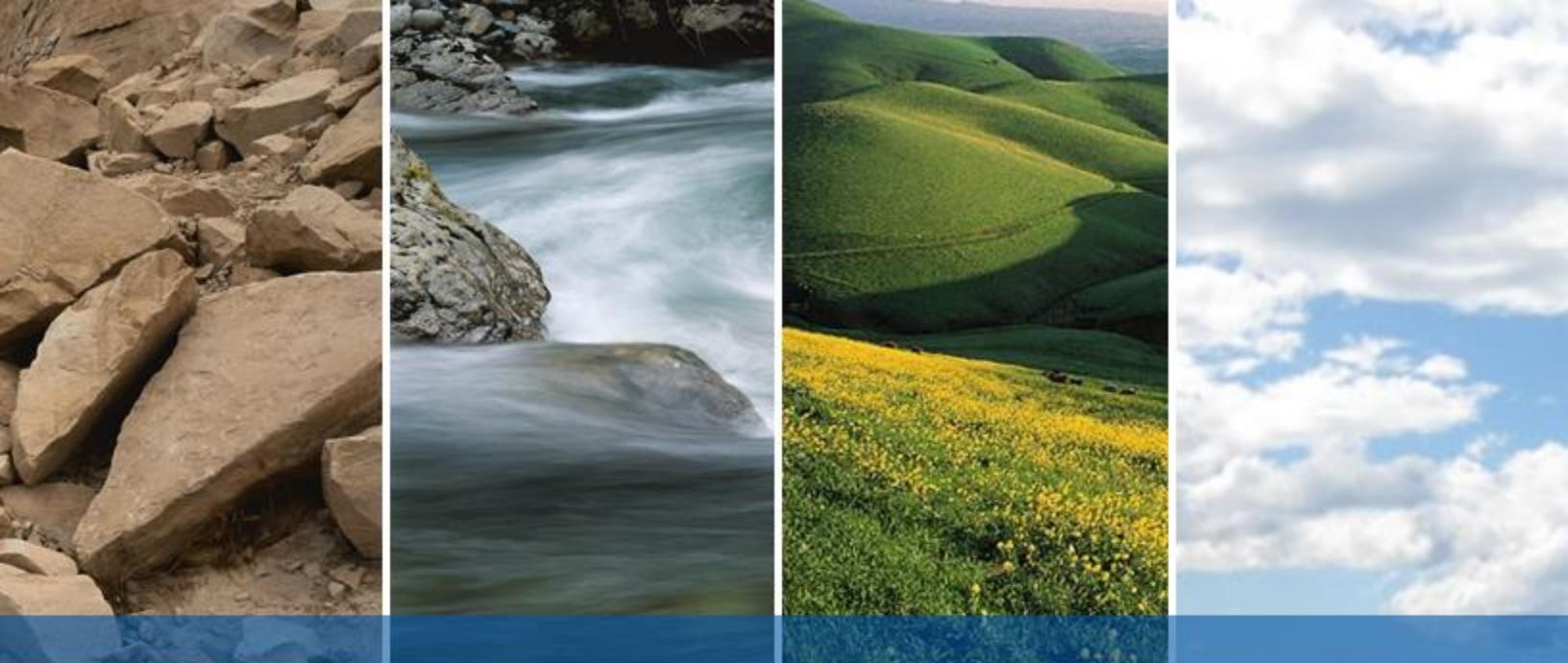
CALIFORNIA PUSH  
TECHNOLOGIES  
INCORPORATED

Job No. 18-56026  
Job Name Google Spartan  
Client ENGEO Inc.  
Sounding ID 1-DMT03  
Date Mar 29, 2018

Coordinate System WGS 84 / UTM zone 10N  
Northing (m) 4132465  
Easting (m) 597260  
Ground Water Depth (ft) 26.2  
Gauge Zero Offset (bar) 0

### DILATOMETER TEST RESULTS





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