APPENDIX A

Geotechnical Report

DRAFT GEOTECHNICAL REPORT

COLUMBINE STATION
IMPROVEMENT PROJECT
San Jose Water Company
City of San Jose, California

Submitted To:

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VERTICAL SCIENCES, INC.

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COLUMBINE STATION IMPROVEMENT PROJECT
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CITY OF SAN JOSE, CALIFORNIA

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1 GENERAL

This report presents the results of our geotechnical study for San Jose Water Company's (SJWC) Columbine Station Improvement Project (CSIP), located in the City of San Jose, California. Vertical Sciences, Inc. (VSI), has prepared this report at the request of Water Works Engineers, LLC (WWE). The project location is shown on Plate 1 – Site Location Map. The following sections present our understanding of the project, the purpose of our study, and the geotechnical findings, conclusions, and recommendations for the project. Our services were performed in general accordance with our proposals dated July 8, 2016.

1.1 PROJECT LOCATION

CSIP is located at Assessor Parcel Number 654-21-002 within the eastern margin of the City of San Jose. The physical address of the site is 3650 Clayton Road, San Jose, California. Latitude and longitude for the approximate center of the existing reservoir are as follows:

- Latitude: 37° 21' 13.9" (37.353858°)
- Longitude: -121° 47' 32.7" (-121.792414°)

1.2 PROJECT UNDERSTANDING

The Columbine Station site is in an eastern portion of the City of San Jose (City) at the western margin and base of the Los Buellis Hills of the Diablo Range. The site currently has one lined and covered earthen water storage reservoir along with a smaller uncovered sump, all of which were constructed in 1963. We understand that the existing reservoir was inspected and found to have significant structural deterioration, necessitating replacement of the reservoir. SJWC (2016) performed an evaluation and found that replacing the 19.6-million-gallon covered earthen reservoir with two pre-stressed concrete tanks was the preferred alternative for this project.

Those tanks are proposed to each have storage capacities of about 6.6-million gallons and will be 205 feet in diameter. Both tanks will be 33.5 feet tall and will have a finished floor elevations of about 321 feet. We understand that high water level elevation within the tanks is 348 feet. The proposed tank configurations are shown on Plate 2 – Project Elements.

The proposed tanks will have a 9-inch thick slab and floor, and columns spaced at about 18- to 21-foot grid intervals. Anticipated loads for the tank wall and columns is 8,000 pounds per lineal foot and 110 kips, respectively. The tanks, as identified by SJWC, will be partially buried.

1.3 STUDY PURPOSE

The purpose of our geotechnical study was to explore and evaluate selected site surface and subsurface conditions to provide geotechnical engineering recommendations related to the design and construction of the proposed improvements, and to identify potential geologic hazards that could impact the project. Those tasks had a three-fold purpose:

- To characterize geologic hazards that pose an adverse effect on the performance of the proposed tank(s);
- To estimate settlement and allowable bearing values for proposed subgrade soils for use in designing the proposed tank foundation and tank slab; and
- To develop geotechnical recommendations for the design and construction of the proposed project.

1.4 PREVIOUS WORK PERFORMED & REFERENCES REVIEWED

As part of this study, we attempted to obtain pertinent historical geotechnical data for the Columbine Station site through information requests from SJWC and a file review at the California Department of Water Resources', Division of Safety of Dams' (DSOD) offices. In addition, we reviewed site specific piezometric information and pertinent regional geological data, aerial photographs, topographic maps, and groundwater data. The following discusses the results of our attempts to gather historical data for the site.

Four site-specific geotechnical studies have been reported for the existing Columbine Station:

1. Test holes advanced by San Jose Water Works (1961) for the construction of the existing Columbine Station;
2. A soils investigation for the Columbine Drive Reservoir by Gribaldo, Jacobs, Jones and Associates (1968);
3. Evaluations of the static and seismic stability of the Columbine Station by Wahler (1985); and
4. A planning-level study performed by AMEC (2013) that included the Columbine Station site.

Of those previous studies, we were able to obtain and review San Jose Water Works (1961), Wahler (1985), and AMEC (2013). Gribaldo, Jacobs, Jones and Associates (1968) study was not available from SJWC nor DSOD, except for one drill hole (Boring 3) obtained from DSOD files. Thus, that study was not available for review for this project.

DSOD performed several reviews on site conditions, regional geologic conditions, and information previously reported by other consultants. Those studies (Thronson, 1966 & 1968; Viarnes, 1966 & 1968; Wong, 1985; Enzler, 1988; Burns, 2004; Lam, 2004) were obtained and reviewed as part of our services.

Several piezometers have been constructed and monitored at the project site. We obtained monitoring results from between 1994 and 2007 for those piezometers as part of this study (Earth Tech, 2000a, 2000b, 2001, 2002, 2003, 2004, 2005, 2006, 2007, and 2008; Aecom, 2009).

Previous regional geologic studies and maps have been prepared for the project area, which are relevant to the proposed project. Those studies that we obtained and reviewed consisted of:

REGIONAL GEOLOGIC & GEOTECHNICAL REFERENCES		
Source	**Date**	**General Title**
Helley & Wesling	1990	Quaternary geologic map of the San Jose East quadrangle
Wentworth et al.	1999	Preliminary geologic map of San Jose 30x60-minute quadrangle
Dibblee & Minch	2005	Geologic map of the San Jose East quadrangle
Wiegers	2011	Landslide inventory map of San Jose East quadrangle
AMEC	2013	Planning-level geotechnical study of the Columbine Station

Those references noted above are fully cited in Section 10.0 of this report.

In addition to the aforementioned geotechnical and geological references reviewed, VSI reviewed historical aerial photographs of the project region. That review was to observe potential geomorphic indicators present on those aerial photographs that would assist us in our evaluation of potential landslide and fault hazards. Aerial photographs were reviewed from 1948, 1956, 1960, 1968, 1980, 1987, 1993, 1998, 2002, 2005, 2009, and 2012.

Additional documents were referred to during this study and are referenced in the text and cited in Section 10.0 of this report.

1.5 SCOPE OF SERVICES

Services performed for this study are in general conformance with the proposed scope of services presented in our July 8, 2016 proposal. Our scope of services included:

- Reconnaissance of the site surface conditions, topography, and existing drainage features;
- Attempted acquisition of existing, available geotechnical data relevant to the project site;
- Review of pertinent, selected regional geological data;
- Acquisition and review of selected historical aerial photographs of the project region;
- Acquisition of drilling permits from Santa Clara Valley Water District;

- Advancement of 10 drill holes at selected locations shown on Plate 3 – Geotechnical Map. Exploration procedures and Logs of Drill Holes are presented in Appendix A – Subsurface Exploration;
- Performance of laboratory testing on selected samples obtained during our field investigation. Laboratory test procedures and results of those tests are presented in Appendix B – Laboratory Testing;
- Estimated of settlement for the proposed project.
- Preparation of this report, which includes:
  - A description of the proposed project;
  - A summary of our field exploration and laboratory testing programs;
  - A description of site surface and subsurface conditions encountered during our field investigation;
  - Site-specific seismic response spectra along with 2013 California Building Code (CBC) seismic design criteria;
  - A geotechnical map showing approximate field exploration locations, presented as Plate 3;
  - Geotechnical recommendations for:
    - Construction of proposed slopes at the project site;
    - Site preparation, engineered fill, site drainage, and subgrades;
    - Suitability of on-site materials for use as engineered fill;
    - Foundation and slab-on-grade design;
    - Temporary excavations, shoring, and trench backfill;
    - Trench backfill and compaction recommendations;
    - Lateral earth pressures for retaining wall design and construction; and
    - Preliminary structural pavement design.
  - Appendices that present a summary of our field investigation procedures and laboratory testing programs.

2 FINDINGS

2.1 SITE HISTORY

Based on review of aerial photographs and historical topographic maps it appears that the following historical development has occurred at the tank sites.

SITE HISTORY	
Year	**Development**
1948, 1953, 1960	Aerial photographs from 1948 through 1960 show the existing reservoir area developed with row-based agricultural crops that appear to be a relatively young orchard. Clayton Road is present and has the approximately same alignment as its current path.
1968	The reservoir is constructed and site improvements appear largely the same as at the present except the olive trees surrounding the property are immature. In addition, residential development along Columbine Drive has been constructed.
1980	No significant changes from 1968 except the multi-unit development located south of the site has been constructed.
2012	No apparent changes from 1980.

2.2 FIELD INVESTIGATION

VSI conducted a geotechnical field investigation to:

- Evaluate subsurface soil and rock conditions at selected locations;
- Evaluate the approximate location of faulting relative to the proposed tank locations; and
- Provide subsurface data for evaluation of slope stability, settlement, and the proposed tank improvements.

Our field geotechnical investigation consisted of reconnaissance-level geologic mapping of the project site and adjacent areas, and subsurface exploration through advancement of ten drill holes. The drill holes were advanced on July 25 through July 27, 2016 and August 8 and 9, 2016. The exploration locations are shown on Plate 3. Descriptions of soils and rocks encountered are presented on the drill hole logs included in Appendix A.

2.3 SITE CONDITIONS

2.3.1 Surface Conditions

The site is currently developed at a covered, lined reservoir, with improvements that include a sump, valve vaults, electrical equipment, a generator, a cellular communications station and tower, and various other improvements. The existing reservoir was created by excavating soils from the reservoir basin and placing those soils as embankments to increase the volume of the impoundment. The embankments are predominately present on the southern and western margins of the reservoir with thinner embankments along the north portion of the site. Those embankments are inclined at about 2:1 (horizontal to vertical) to 1.5:1 for interior and exterior reservoir slopes, respectively. We understand that the embankments retain sufficient volumes of water such that the entire reservoir is under jurisdictional review by the DSOD. The entire impoundment area is covered with a wood-supported, corrugated roof that locally supports solar electric panels.

The reservoir is bounded by Clayton Road to the north, a cellular communications station and tower to the east, private residences to the south, and the sump, vaults, and electrical equipment to the west. Slopes descend to the reservoir from the north and east and range in height up to about 16 feet. Numerous mature olive trees are present along the northern, eastern, and southern margins of the reservoir. A stairway descends westerly from the reservoir to the sump.

Drainage from the site occurs as sheet flow from the north and east captured by gunite-lined "V" ditches along the eastern margin of the site. Elsewhere, sheet flow runs along paved surfaces to drop inlets that direct flow to drainage ditches that are present on site or out to municipal storm drains located in Columbine Drive.

The existing reservoir bottom has an elevation of 321 to 325 feet (datum NAVD 88; Sharah Dunlap Sawyer [SDS], 2016). The top of the paved embankment surrounding the reservoir is at an elevation of about 345 feet. Elevations on the site range from about 305 to about 369 feet.

2.3.2 Subsurface Conditions

Subsurface conditions were explored at selected locations at the site during this study. As noted in Section 2.2, 10 drill holes were advanced for this study.

Based on the subsurface information collected during this study, the subsurface conditions consist of artificial fill materials, landslide deposits, older alluvium, and the Panoche Formation. Cross sections were prepared at selected locations for this site, which are shown on Plate 3. The cross sections are presented as Plates 4.1 through 4.3 – Geotechnical Cross Section A-A' through C-C', respectively. The following sections discuss these materials in greater detail

2.3.2.1 Artificial Fill (af)

Artificial fill materials were encountered in drill holes DH-3 through DH-9. Those materials were used to construct the berms surrounding the existing covered reservoir and range in thickness up to about 24 feet. No artificial fill materials were identified as present at or below the target elevation of the tank pad.

Artificial fill materials consisted of clayey sand, silty clay, and clay with local trace to moderate amounts of fine to medium subangular to subrounded gravel. The materials were generally medium stiff to hard, with most samples being very stiff to hard. The soils were damp, nonplastic to plastic.

2.3.2.2 Landslide Deposits (Qls)

Landslide deposits are present at the northeastern margin of the existing reservoir. The landslide was identified by Wahler (1985) and discussed in greater detail by Dudley (1988a, 1988b). The landslide appears to be up to about 15 feet thick, based on mapping by Dudley (1988b) and drill holes advanced during this study. As such, the landslide daylights within the flank of the existing reservoir and does not extend down to or below the proposed tank slab elevation.

Drill holes DH-2 and DH-3 advanced for this study penetrate the landslide deposits. Soils encountered within the landslide consist of clayey sand, silty clay, and clay that range in consistency from stiff to very stiff. Those soils were moist, plastic, and locally contained fine to medium grained sand, and trace to minor subangular to subrounded fine to medium gravel.

2.3.2.3 Older Alluvium (Qoal)/Panoche Formation (Kpc)

The site is mapped as being underlain by older alluvium (uplifted fluvial terraces) and Panoche Formation (Dibblee & Minch, 2005). Sediments encountered beneath artificial fill and landslide deposits were dense to very dense/stiff to hard, with some materials appearing saprolitic in appearance. There is a potential that the older alluvial soil mapped in the area might be the weathered regolith of the underlying Panoche Formation based on the relic rock texture observed in some samples. Because of this, we have grouped the older alluvium and Panoche Formation into one mapped unit on Plate 3 and Plates 4.1 through 4.3. It should be noted that there is a potential that the older alluvium is a separate unit compared to the Panoche Formation; however, this distinction is moot from a geotechnical standpoint based on the consistency of materials beneath the proposed tanks.

The Panoche Formation is the underlying bedrock beneath the existing reservoir (Dibblee & Minch, 2005). The site has also been mapped as being underlain by conglomerates of the Berryessa Formation (Wentworth et al., 1999) and the Oakland Conglomerate (Dudley, 1988a; Wahler, 1985). Dibblee (1972) had originally mapped the site as being underlain by the Oakland Conglomerate; however, more recent mapping by others, including Dibblee & Minch (2005), have not recognized the Oakland Conglomerate and Dibblee & Minch (2005) have mapped the underlying geologic unit as the Panoche Formation. For purposes of this report, we have elected to describe the underlying bedrock as the Panoche Formation.

Older Alluvium/Panoche Formation was encountered in each of the drill holes advanced for this study. The Panoche Formation is thought to be up to 9,500 feet thick (Anderson & Pack, 1915) and was not fully penetrated by any of the drill holes. At the project site, it consisted of sandy clay with gravel, clayey sand, sand, and sandy gravel. As noted on Plates

4.1 through 4.3, most materials encountered beneath the proposed tank sites consisted of dense to very dense sand and gravel. In local areas, weak rock was encountered (SJWW, 1961) and much of the Older Alluvium/Panoche Formation encountered in our drill holes likely represents saprolitic soils of underlying sandstone and conglomerate bedrock materials. The soils were generally moist, locally slightly plastic, fine to medium grained with some to abundant subangular to subrounded fine to coarse gravel.

2.4 GEOLOGIC CONDITIONS

2.4.1 Regional Geology

The project site is located on the eastern margin of the Coast Ranges Geomorphic/Geologic province of California. The Coast Ranges province is a northwest-trending mountain range that is about 50 miles wide and extends about 400 miles from its southern terminus in Santa Barbara County north into Shasta County and southern Oregon. It is bordered to the west by the Pacific Ocean, to the south by Transverse Ranges Province, to the east by the Great Valley province, and to the north by the Klamath Mountains Province. The province is separated into the Northern and Southern Coast Ranges at the Golden Gate (Hinds, 1952).

The Coast Ranges province is composed predominately of Cenozoic- and Mesozoic-age sedimentary rocks. Lesser amount of Pleistocene-aged volcanic rocks occurs locally within this province (such as at Clear Lake) as do granitic rocks of the Salinian Block, located west of the San Andreas fault.

The project site is in the Los Buellis Hills of the Diablo Range within the Southern Coast Ranges. The Diablo Range are composed of Plio-Pleistocene-age sedimentary deposits to Jurassic-age sedimentary and metavolcanic rocks. In the project area, partially consolidated sedimentary deposits of the Panoche Formation are exposed (Dibblee & Minch, 2005), as shown on Plate 5 – Regional Geologic Map. Some small- to moderate-scale landslides have been mapped locally throughout the project area (Weigers, 2011).

2.4.2 Local Geologic Setting

As noted in Section 2.3.2.3, the Panoche Formation is the underlying bedrock beneath the existing reservoir (Dibblee & Minch, 2005). It has also been mapped as being underlain by conglomerates of the Berryessa Formation (Wentworth et al., 1999) and the Oakland Conglomerate (Dudley, 1988a). For purposes of this report, we've elected to describe the underlying bedrock as the Panoche Formation.

The Panoche Formation is a Cretaceous-age marine sedimentary deposit that consists of intercalated sandstone and conglomerate rock. In the project area, those materials are moderately lithified, thinly to thickly bedded, poorly indurated, and weak (grade R2 from ISRM, 1981). With depth, it is anticipated that weathering will decrease and that rock strength will increase.

2.4.3 Groundwater

Groundwater was not encountered in drill holes advanced for this study or by Wahler (1985). Piezometers were constructed by Wahler (1985) and supplemented and/or replaced by Earth Tech (2005). Data from monitoring of the seventeen piezometers from between 1994 and 2008 was obtained and reviewed for this study. Of the seventeen piezometers, 9 were constructed solely within the foundation soils beneath the reservoir embankments, and 8 were installed solely within the embankment soils.

Per AECOM (2009) and Earth Tech (2000 through 2008), piezometers within the foundation soils have historically been dry, apart from one recorded groundwater reading in piezometer 11B in 2005 that was 0.5 feet above the piezometer bottom at an elevation of 316.5 feet. However, this was a one-time occurrence and could be indicative of infiltration from the ground surface or other flaws in the piezometer.

In an attempt to estimate the depth of groundwater beneath the site, we performed searches on the California State Water Resources Geotracker database and the Department of Toxic Substances Envirostor database. Only one site with pertinent geological information was available from Geotracker and none from Envirostor. The identified site is located at 1510 Mt. Pleasant Drive, which is about 2,400 feet west of the project site. At that location, drill holes were advanced and no groundwater was encountered within 50 feet of the ground surface (Exceltech, 1990).

A search for groundwater information was also made through the Department of Water Resources' Water Library website. The closest relatively deep well information that we could find was for Well Number 07S01E01G001M, located near the intersection of Mt Vista and Lochner Drives in San Jose (DWR, 2016). That well is located about 1.1 miles west of the project site in older alluvial oils. Groundwater recorded in that well ranged in depth from 39 to 45 feet (DWR, 2016).

Groundwater elevations will fluctuate over time. The depth to groundwater can vary throughout the year and from year to year. Intense and long duration precipitation, modification of topography, and cultural land use changes can contribute to fluctuations in groundwater levels. Localized saturated conditions or perched groundwater conditions near the ground surface could be present during and following periods of heavy precipitation or if on-site sources contribute water. If groundwater is encountered during construction, it is the Contractor's responsibility to install mitigation measures for adverse impacts caused by groundwater encountered in excavations.

3 GEOLOGICAL HAZARDS

3.1 FAULTING & SEISMICITY

3.1.1 Regulatory Seismic Setting

The State of California designates faults as active, potentially active, and inactive depending on the recency of movement that can be substantiated for a fault. Fault activity is rated as follows:

FAULT ACTIVITY RATINGS		
Fault Activity Rating	**Geologic Period of Last Rupture**	**Time Interval (Years)**
Active	Holocene	Within last 11,000 Years
Potentially Active	Quaternary	>11,000 to 1.6 Million Years
Inactive	Pre-Quaternary	Greater than 1.6 Million Years

The California Geologic Survey (CGS) evaluates the activity rating of a fault in fault evaluation reports (FER). FERs compile available geologic and seismologic data and evaluate if a fault should be zoned as active, potentially active, or inactive. If an FER evaluates a fault as active, then it is typically incorporated into a Special Studies Zone in accordance with the Alquist-Priolo Earthquake Fault Zoning Act (AP Act). AP Act Special Studies Zones require site-specific evaluation of fault location and require a structure setback if the fault is found traversing a project site.

Several potentially active and active faults have been mapped proximal to the project site, as shown on Plate 6 – Regional Fault Map. The site is not located within an AP Act special study zone but is relatively close to two special studies zones, as shown on Plate 7 -Special Studies Zones. Furthermore, it is located within a zone designated by Santa Clara County as having a potential for fault rupture hazard, as shown on Plate 7.

3.1.2 Project Site Fault Hazards

The project site is in a zone of stress transfer between the San Andreas and Hayward/Calaveras fault systems. This stress transfer and associated bends in the fault systems have created active seismogenic fault sources in addition to sympathetic faults that are likely not to be seismogenic but to move co-seismically with ruptures along the near-by Hayward and/or Calaveras faults. Thus, the project site is situated, as shown on Plates 6 and 7, proximal to several relatively close active faults.

The most significant major fault relative to the site is the Hayward fault, located about 3,300 feet east of the project. The Hayward fault is delineated by an apparent pattern of drainage deflections in a right-lateral sense, truncated drainages, side-hill terraces, tonal changes across the fault, and displacement of Holocene-age alluvium (Burns, 2004; Wahler, 1985). It is about 87 miles long and has the highest probability of generating an earthquake with a magnitude (M) 6.7 or greater between 2002 and 2031 (30-year exposure period) as compared to the other regional faults (Working Group of California Earthquake Probabilities, 2003). The Hayward fault is thought responsible for the October 21, 1868 M7+ (estimated) earthquake that caused over 3 feet of horizontal offset with a rupture length of 20 miles (Wahler, 1985).

The Calaveras fault is a north-northwest trending fault that is about 90 miles long and has been historically active. It is located about 5.5 miles northeast of the site and is thought capable of generating earthquakes with M>7. Historically, the Calaveras fault is considered the source of the M6.5 July 3, 1861 earthquake and is known to be the source of the April 25, 1981 M6.2 Morgan Hill earthquake (Wahler, 1985).

In addition to the Hayward and Calaveras faults, the Evergreen and Quimby faults are located close to the project site. The Evergreen and Quimby faults are located about 1,000 feet west and 650 feet east of the reservoir, respectively. Per Hitchcock and Brankman (2002) both faults are east dipping reverse/thrust faults that likely intersect the Calaveras fault at a depth of 12,000 to 15,000 feet. Those depths are thought to be too shallow to make these faults capable of being primary sources for large earthquakes. Instead, they likely experience displacement during earthquakes and deformation occurring along the Calaveras fault (Hitchcock and Brandman, 2002). However, both faults have experienced Holocene displacement and have been zoned as active by the state, as noted on Plate 7.

Site observations and review of historical aerial photographs of the site did not identify any indications of faulting projecting through the project site. We observed no deflected drainages, tonal variations, or geomorphic indicators of faulting in the pre-development aerial photographs. Furthermore, explorations performed by SJWW (1961), Wahler (1985), and this study did not identify gouge zones, displaced soils/formation materials, aquitards or aquicludes, or other indications that faulting is present across the site. Finally, work by DSOD (Burns, 2004) did not identify any faults extending through the project site.

3.1.3 Historical Seismicity

Northern California is a seismically active area that has been subjected to numerous historical earthquakes. A search of the California Historical Earthquake Database (CGS, 2015) found a total of 424 moment magnitude (M_W) 4.0 or greater earthquakes have occurred within a 50-mile radius of the project site. Of those earthquakes, 114, 20, and 4, and 1 had a M_W of >5.0, >6.0, >7.0, and >8.0, respectively, between 1769 and 2000. Of those earthquakes, the largest was a M_W 8.3 that occurred on April 18, 1906 along the San Andreas fault (the Great San Francisco Earthquake).

3.1.4 CBC Seismic Design Recommendations

We understand that the proposed tank will be designed and constructed under the 2013 California Building Code (CBC) criteria. At a minimum, structures should be designed in accordance with the following seismic design criteria:

CBC SEISMIC DESIGN PARAMETERS

California Building Code	Parameter	CBC Designation
Site Coordinates	Latitude	37.228403°
	Longitude	-121.917961°
Section 1613.5.3 Table 1613.5.3(1)	Site Coefficient, F_a	1.0
Section 1613.5.3 Table 1613.5.3(2)	Site Coefficient, F_v	1.3
Section 1613.5.1 Figure 1613.5	Site Class Designation	C
	Seismic Factor, Site Class B at 0.2 Seconds, S_s	1.702g
	Seismic Factor, Site Class B at 1.0 Seconds, S_1	0.629g
Section 1613.5.3	Site Specific Response Parameter for Site Class C at 0.2 Seconds, S_{MS}	1.702g
	Site Specific Response Parameter for Site Class C at 1.0 Seconds, S_{M1}	0.818g
Section 1613.5.4	$S_{DS}=2/3S_{MS}$	1.135g
	$S_{D1}=2/3S_{M1}$	0.545g

Per the 2013 CBC

3.1.5 Probabilistic Estimates of Strong Ground Motion

Probabilistic evaluations of horizontal strong ground motion that could affect the site were performed using attenuation evaluation methods provided by the U.S. Geological Survey (USGS, 2016). The evaluations were performed using an estimated shear wave velocity in the upper 100 feet of the profile of 400 meters per second. Evaluations were performed for upper-bound (UBE) and design-basis (DBE) probabilistic exposures. The UBE corresponds to horizontal ground accelerations having a 10 percent probability of exceedance in a 100-year exposure period, with a statistical return period of 949 years. The DBE corresponds to horizontal ground accelerations having a 10 percent probability of exceedance in a 50-year, exposure period, with a statistical return period of 475 years. The results of these evaluations are presented in the following table:

PROBABILISTIC GROUND MOTION DATA				
Earthquake Level	Probabilistic Estimate Exposure Period (years)	Probability of Exceedance (%)	Return Period (years)	Estimated Peak Horizontal Ground Acceleration (g)
Upper-Bound Ground-Motion	100	10	949	0.75
Design-Basis Ground-Motion	50	10	475	0.61

It should be noted that although the seismic hazard models used for this study predict the probability of exceedance for various levels of acceleration in a given exposure period, the models are not able to account for the effect that the passage of time since past earthquakes has on future earthquake probability. Thus, while time may affect the incipient risk of earthquakes occurring, the UBE and DBE values are based on any 100-year and 50-year exposure period, respectively, regardless of how recently earthquakes have occurred.

3.1.6 Site-Specific Response Spectra

The project site is located within 10 kilometers of an active fault zone (the Evergreen, Quimby, and Hayward faults). As required, a site-specific seismic hazard analysis was performed in accordance with Chapter 11 and Chapter 21 of ASCE 7-10 and the American Water Works Association (AWWA) Standard for welded steel tanks for water storage. Seismic design values for the project were estimated using code guidelines, the most recent version of the Seismic Hazard Curves, Response Parameters and Design Parameters application from the U.S. Geological Survey (USGS, 2016), EZ-FRISK (2016), and subsurface information obtained from explorations performed on site. Latitude and longitude for the site used in the software are described in Sections 1.1 and 3.1.4. Attenuation curves of Campbell & Bozorgnia (2007), Boore-Atkinson (2007), and Chiou Youngs (2006), all for soil conditions, were used for deterministic and probabilistic spectral ground motion estimates.

Design response spectra are presented as Plate 8 – Response Spectra Analysis and Plate 9 – Design Response Spectra. The estimated values of spectral acceleration (Sa) for the Design Response Spectra at damping ratios of 5-percent were adjusted to 0.5-percent using the procedure recommended by AWWA standards, are presented on Plate 8.

Empirical attenuation relationships allow for the estimation of response spectral ordinates for periods up to 5 seconds. For tank design, spectral ordinates are extrapolated to higher sloshing periods of up to 15 seconds. The spectral values beyond a 5-second period were extrapolated assuming constant spectral displacement.

Based on the subsurface conditions encountered, a shallow soil profile was used for both probabilistic and deterministic seismic hazards evaluations for the site. The MCE used to generate the spectra is defined both probabilistically and deterministically. The recommended design spectrum shown on Plate 9 is estimated from the following comparisons of probabilistic MCE and deterministic MCE: the lesser of the probabilistic ground motion having a 2-percent probability of exceedance in 50 years (2,475-year return period MCE) calculated with 5-percent damping, the greater of 150-percent of the median deterministic ground motion calculated for 5-percent damping, and a deterministic lower-bound spectrum calculated according to ASCE 7-10 Section 21.2. Additionally, the recommended design response spectrum presented on Plate 9 is defined by ASCE 7-10 as the greater of the site-specific MCE calculated above, or 80-percent of the general response spectrum calculated per ASCE 7-10 Section 11.4. The deterministic, probabilistic, and general spectra used for comparison are shown on Plate 8.

3.1.7 Site Specific Ground Motion Analyses

Section 3.1.4 above provides CBC codified ground motion values. Site specific ground motion values were also estimated for the project site using EZ-FRISK (2016) software with the same site latitude and longitude values noted in Section 3.1.4. The following site-specific ground motion values have been estimated for this project:

SITE-SPECIFIC GROUND MOTION VALUES	
Parameter	**Value**
Seismic Factor, Site Class C at 0.2 Seconds, S_{DS}	1.043g
Seismic Factor, Site Class C at 1.0 Seconds, S_{D1}	0.538g
Site Specific Response Parameter for Site Class C at 0.2 Seconds, S_{MS}	1.565g
Site Specific Response Parameter for Site Class C at 1.0 Seconds, S_{M1}	0.806g

3.2 LANDSLIDES

As noted on Plate 3 and Plate 10 – Existing Landslide, a landslide has been mapped at the northeastern margin of the facility. That landslide is relatively shallow, daylights in the existing reservoir slope, and does not project beneath the proposed tank. The landslide appears to be confined in the clay-rich regolith soils of the Panoche Formation (Dudley, 1988a), and there have been no reported adverse effects to the existing reservoir from that landslide since the reservoir was constructed in 1963. Once the proposed western and eastern tanks are constructed they will have a separation of about 40 and 65 feet from the landslide, to those tanks, respectively. However, both tanks will be partially buried and the engineered fill placed around the tank will buttress that landslide, thus, improving its stability as compared to its current condition, which is unbuttressed and unsupported. It is anticipated that if movement of the landslide occurs during construction, there will be sufficient separation between landslide deposits and the tanks such that those materials should not impact the integrity of the tank.

Additional landslides have been mapped in the project region and the northern and eastern margins of the site are mapped with Santa Clara County's Landslide Hazard Zone and the state's landslide hazard zone (CGS, 2000), as shown on Plate 11 – Landslide Hazards. We have reviewed aerial photographs of the project region and conducted site observations and in our opinion, mapping by Weigers (2011) and CGS (2000) appears to depict the regional landslide conditions. The landslide located at the northeastern margin of the existing reservoir is not depicted on those maps but is shown on Plate 10, which is based on our mapping and that of Dudley (1988b).

Based on surface mapping of the area the landslides discussed above were not observed to have indications of recent, incipient, or on-going landslide movement.

3.3 LIQUEFACTION AND LATERAL SPREADING

Liquefaction is described as the sudden loss of soil shear strength due to a rapid increase of soil pore water pressures caused by cyclic loading from a seismic event. In simple terms, it means that a liquefied soil acts more like a fluid than a solid when shaken during an earthquake. For liquefaction to occur, the following are needed:

- Granular soils (sand, silty sand, sandy silt, and some gravels);
- A high groundwater table; and
- A low density in the granular soils underlying the site.

If those criteria are present, then there is a potential that the soils could liquefy during a seismic event.

The adverse effects of liquefaction include local and regional ground settlement, ground cracking and expulsion of water and sand, the partial or complete loss of bearing and confining forces used to support loads, amplification of seismic shaking, and lateral spreading. In general, the effects of liquefaction on the proposed project could include:

- Lateral spreading;
- Vertical settlement; and/or
- The soils surrounding lifelines can lose their strength and those lifelines can become damaged or severed.

Lateral spreading is defined as lateral earth movement of liquefied soils, or soil riding on a liquefied soil layer, down slope toward an unsupported slope face, such as a creek bank, or an inclined slope face. In general, lateral spreading has been observed on low to moderate gradient slopes, but has been noted on slopes inclined as flat as one degree.

Medium dense to dense and stiff to hard soils and rock materials were encountered in explorations advanced at the site. Groundwater was not encountered at depths shallower than 65 feet. It is our opinion that the presence of stiff to hard fine-grained soils and formational materials beneath the proposed tank bottom precludes the potential that liquefaction will occur. Based on those observations, it is our opinion that liquefaction potential poses a low risk to development of the proposed project at the site. This opinion is supported by CGS (2000) and by the County, which does not include the project area within a potentially liquefiable zone.

3.4 EXPANSION POTENTIAL & SLOPE CREEP

There is a direct relationship between plasticity of a soil and the potential for expansive behavior, with expansive soil generally having a high plasticity. Thus, granular soils typically have a low potential to be expansive, whereas, clay-rich soils can have a low to high potential to be expansive. Wahler (1985) performed five Plasticity Index (PI) tests and reported PI values ranging from 16 to 35, with an average PI of 26. All of Wahler's PIs, except at WA-3, were obtained from embankment soils and may not be representative of foundation soil plasticity. At WA-3, a PI was obtained for foundation soils and found to have a value of 16.

Atterberg limit testing was performed on select samples during this study to estimate the plasticity of foundation soils. The results of that testing found that on-site soils have a PI of between 6 and 22, with an average PI of 13.

PIs of 6 to 22 are correlated to soils having a low to medium potential for expansion (Day, 1999), as noted in the following table:

EXPANSION POTENTIAL – PLASTICITY INDEX CORRELATION

Plasticity Index	Correlated Expansion Potential
0 – 10	Very Low
10 – 15	Low
15 – 25	Medium
25 – 35	High
35+	Very High

Taken from Day (1999)

3.5 SOIL CHEMISTRY

One selected sample of near-surface soils encountered at the site was subjected to chemical analysis for assessment of corrosion and reactivity with concrete. The sample was tested for soluble sulfates and chlorides. Testing was conducted by Sunland Analytical of Rancho Cordova and results are presented below, as well as included in the appendix of laboratory results.

SOIL CHEMISTRY RESULTS					
Sample Location	**Sample Depth**	**Sulfates (ppm)**	**Chlorides (ppm)**	**pH**	**Resistivity (ohms-cm)**
DH-7	25'	71.7	25.2	8.0	1,530

According to the ACI-318, a sulfate concentration below 0.10 percent by weight (1,000 ppm) is negligible. A chloride content of less than 500 ppm is generally considered non-corrosive to reinforced concrete.

Minimum resistivity testing performed on the soil sample indicated the soils are considered to be corrosive to buried metal objects. A commonly accepted correlation between soil resistivity and corrosivity towards ferrous metals (NACE Corrosion Basics, 1984) is provided below:

RESISTIVITY & CORROSION CORRELATION	
Minimum Resistivity (ohm-cm)	**Corrosion Potential**
0 to 1000	Severely Corrosive
1,000 to 2,000	Corrosive
2,000 to 10,000	Moderately Corrosive
Over 10,000	Mildly Corrosive

Thus, according to the table above, the soils are estimated to be corrosive based upon the soil resistivity.

Because engineered fill materials could be placed during construction, we recommend that verification samples be tested to confirm that soils in contact with concrete and steel have similar, or lower, corrosion potential characteristics as the sample tested for this study.

4 ENGINEERING PROPERTIES OF SELECTED ON-SITE SOILS

4.1 GENERAL

The purpose of the laboratory testing program was to help classify soils and rock materials, and provide relevant physical indices and engineering properties of the subsurface materials. The primary objectives of the program were to:

- Classify and characterize selected sampled subsurface materials;
- Evaluate existing selected in-situ conditions; and
- Develop relevant consolidation, strength, and permeability estimates of selected subsurface materials.

To meet these objectives, various tests were performed on selected samples. Test types are generally grouped into the following categories: classification/index tests, moisture content/density evaluations, consolidation tests, permeability tests, relevant strength tests, and subgrade characterization tests

The numbers of the various tests performed for the project are noted below:

SUMMARY OF LABORATORY TESTS PERFORMED		
Laboratory Test	**Number of Tests**	**Standard Designation[1]**
Moisture/Density & Moisture Content	61	ASTM D2216
Sieve Analysis with #200 Wash	7	ASTM D422
Atterberg Limits	3	ASTM D4318
Modified Proctor	2	ASTM D1557
Consolidation	5	ASTM D2435
Direct Shear	1	ASTM D3080
Torsional Ring Shear	1	ASTM D7608
Unconsolidated-Undrained Triaxial Shear	1	ASTM D2850
Unconfined Compression	4	ASTM D2166
R-Value	1	Caltrans 301
Soil Chemistry	1	ASTM G51 & G57 Caltrans 417 & 422
1 – ASTM International (2007), Caltrans (2000)		

Results of those tests are presented on the Logs of Drill Holes located in Appendix A and/or in Appendix B.

4.2 CLASSIFICATION/INDEX TESTING

The purpose of the laboratory testing program was to supplement field classification of soils.

4.2.1 In-Situ Moisture & Density Content

In-situ moisture values obtained from this study are noted on the project drill hole logs presented in Appendix A. Moisture content values obtained during this study ranged from 2.4 to 22.5 percent. The average moisture content obtained during this study is 10.5 percent.

In-situ dry densities from this study are also noted on the drill hole logs presented in Appendix A. Dry density values obtained during this study ranged from 85.5 to 116.9 pounds per cubic foot (pcf). The average dry density for all samples tested was 108.3 pcf.

Wahler (1985) tested 17 samples and found moisture contents ranging from 12.2 to 22.9 percent with dry densities ranging from 102.5 to 118.1 pcf.

4.2.2 Grain-Size Distribution

Grain-size distributions were performed on seven selected samples during this study. The samples tested had a range of about 16 to 43 percent passing the No. 200 sieve indicating that the material was predominately granular. The average amount passing the No. 200 sieve was 25.2 percent.

Wahler (1985) performed six grain-size distribution tests. The results of their tests found that 22 to 78 percent of the soils were retained on the No. 200 sieve, with an average of 52 percent retained.

4.2.3 Plasticity

Plasticity of three selected samples was tested during this study. The samples tested were lean clay (USCS symbol CL) with a maximum liquid limit of 39 and PI of 22. The following table presents the results of VSI's plasticity testing

VSI PLASTICITY TEST RESULTS			
Exploration	Depth (ft)	Liquid Limit	Plasticity Index
DH-3	45	28	6
DH-4	30	39	22
DH-5	35	26	10

In addition, Wahler (1985) performed five plasticity tests on samples obtained from their drill holes. The results of those tests are as follows:

WAHLER PLASTICITY TEST RESULTS			
Exploration	Depth (ft)	Liquid Limit	Plasticity Index
WA-1	7.5’-10’	36	21
WA-1	22.5’-25’	47	31
WA-2	2.5’-5’	43	26
WA-2	12.5’-15’	53	35
WA-3	2.5’-5’	31	16

4.2.4 Maximum Density/Optimum Moisture Content

Maximum density and optimum moisture content tests were performed on two selected sample during this study. The maximum densities obtained from these tests were 122.6 and 127.5 pcf and optimum moisture contents of 10.8 and 11.4 percent.

Wahler (1985) also performed two maximum density and optimum moisture content tests and obtained values of 112.5 and 114.0 for maximum dry density and 15 and 16 percent optimum moisture contents.

4.3 STRENGTH & VOLUMETRIC TESTING

4.3.1 Direct Shear Tests

One consolidated, drained, direct shear test (ASTM D3080) was performed on a selected sample collected during this study. The results indicate that the sample had a cohesion intercept (C) of 100 psf with an angle of internal friction (Ø) of 37.5 degrees.

4.3.2 Torsional Ring Shear Tests

One torsional shear strength test was performed on a selected sample of soil obtained from the landslide deposits located at the northeastern margin of the reservoir. Drained peak and residual torsional shear strength tests (ASTM D6467) were performed on that sample. The results had a cohesion intercepts (C) of 0 pounds per square foot (psf) with angles of internal friction (Ø) of 23 and 26 degrees for residual and peak tests, respectively.

4.3.3 Triaxial Shear Tests

Unconsolidated-undrained triaxial shear tests were performed on one selected sample obtained during this study. The test was performed in accordance with standard test ASTM D2850. In addition, consolidated-undrained triaxial shear tests were performed on two samples by Wahler (1985).

4.3.4 Unconfined Compression

Four uniaxial unconfined compression tests were performed on soil and rock materials obtained from this study. The tests were performed in accordance with standard test method ASTM D2166. Results of those tests are as follows:

UNCONFINED COMPRESSION TEST RESULTS		
Exploration	Depth (ft)	Unconfined Compressive Strength (psi)
DH-4	35	1,774
DH-6	31	8,369
DH-6	35	14,996
DH-9	15	2,552

4.3.5 Consolidation Tests

The consolidation characteristics of selected foundation soils were estimated by performing one-dimensional consolidation on five samples in general accordance with ASTM test method D2435. The consolidation data provides evaluation of the soil pre-consolidation pressure and compression indices for evaluating post-construction settlements.

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

Based on the results of our investigation, it is our opinion that the site is suitable for the proposed improvements provided recommendations presented, herein, are utilized during design and construction of the project. Specific comments and recommendations regarding the geotechnical aspects of project design and construction are presented in the following sections of this report and are intended to be refined, where needed, as the project moves from predesign to design stages.

Recommendations presented, herein, are based upon the preliminary site plans provided by SJWC and WWE along with stated assumptions. Changes in the configuration from those studied during this investigation may require supplemental recommendations.

5.2 GEOLOGIC HAZARDS

5.2.1 Faulting

As previously noted, the project site is located relatively close to the Quimby and Evergreen faults, both of which are splays of the southeast segment of the Hayward fault. Review of aerial photographs, field mapping, and subsurface exploration did not identify any faulting at the project site. In addition, prior work by Wahler (1985) and review by DSOD (Burns, 2004) did not identify faulting across the site. Based on those factors, it is our opinion that the risks associated with faulting across the project site is low.

5.2.2 Landslides

A landslide is present at the northeast corner of the site, as shown on Plates 3 and 8. We did not observe indications that those landslide materials are currently moving or had moved relatively recently. Furthermore, we did not observe the presence of other landslides in the project area that might impact the project. It is our opinion that there is little risk of damage to the project improvement due to landsliding.

5.2.3 Liquefaction

Based on our observations and material exposed during the investigation, it our opinion that liquefaction and lateral spreading pose a low risk of adversely affecting the project site or proposed improvements.

5.2.4 Expansive Soils

Soils with a low to moderate expansion potential are present at the project site. See Section 3.4 of this report for a description of those soils. The foundation materials on which the tank will be situated should consist of moderately to slightly weathered rock that should have a low to moderate potential for expansion. However, as recommended in subsequent sections of this report, it is anticipated that the proposed tanks will rest on a five-foot thick layer of compacted aggregate base material, which will also serve to reduce the potential for adverse effects caused by moderately expansive soils on site. Because of this, it is our opinion that expansive soils should have relatively little adverse effect on the project design, construction, or performance, and no additional mitigations are needed to address this issue.

5.3 SITE PREPARATION AND GRADING

5.3.1 Stripping

Prior to general site grading and/or construction of planned improvements, existing vegetation, trees, organic topsoil, debris, and deleterious materials should be stripped and disposed of off-site or outside the construction limits. Stripping depths of about 2 to 4 inches should be anticipated for portions of the project area that have vegetation and trees. Where trees and large shrubs are currently present, or have fallen or been removed within the last seven years, deeper stripping to remove root balls will be needed. Such deeper stripping could exceed three or more feet in depth. In areas developed by the existing reservoir or void of vegetation, stripping depths should be anticipated to be less than an inch unless organic or deleterious materials are encountered.

5.3.2 Existing Utilities, Wells, and/or Foundations

It is anticipated that existing pipelines and/or subsurface improvements are located within the area of the proposed tanks. When buried improvements are encountered during construction, they should be removed and/or rerouted beyond construction limits. Buried tanks or wells, if present, should be removed in compliance with applicable regulatory agency requirements. Existing, below-grade utility pipelines that extend beyond the limits of the proposed construction and that will be abandoned in-place should be plugged with lean concrete or grout to prevent migration of soil and/or water. All excavations resulting from removal and demolition activities should be cleaned of loose or disturbed material prior to placing any fill or backfill.

5.3.3 Keying and Benching

The proposed tank pads will involve cutting a slope to create sufficient area for construction of the tank foundations and tanks. No keyways are anticipated to be required as part of this construction.

As engineered fill materials are placed against existing slopes or temporary cut slopes created during tank construction, benches should be graded into those slopes to tie the engineered fill and competent intact soil materials together. Benches should be a minimum of 6 feet wide and have vertical backcuts at least 4 feet tall. The benches should be inclined into the slope a minimum of 2-percent, as shown on Plate 12 – Keying and Benching Details.

5.3.4 Scarification and Compaction

Following site stripping and overexcavation, areas to receive engineered fill should be scarified to a minimum depth of 8 inches, uniformly moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent of the maximum dry density as determined using standard test method ASTM D1557[1]. If competent rock is exposed in subgrade to receive engineered fill materials, scarification does not need to be performed. If such rock is exposed, we recommend that an experienced, California-licensed geotechnical engineer or engineering geologist observe the subgrade prior to fill placement to confirm that scarification is not needed.

5.3.5 Wet/Unstable Soil Conditions

Following periods of precipitation, following the rainy season, or where the existing reservoir liner has been leaking, near-surface on-site soils may be significantly over optimum moisture content. These conditions could hinder equipment access as well as efforts to compact site soils to a specified level of compaction. If over optimum soil moisture content conditions are encountered during construction, disking to aerate, replacement with imported material, chemical treatment, stabilization with a geotextile fabric or grid, and/or other methods will likely be required to facilitate earthwork operations. The applicable method of stabilization is the Contractor's responsibility and will depend on the contractor's capabilities and experience, as well as other project-related factors beyond the scope of this investigation. Therefore, if over-optimum moisture within the soil is encountered during construction, VSI should review these conditions (as well as the contractor's capabilities) and, if requested, provide recommendations for their treatment.

5.3.6 Site Drainage

Grading should be performed in such a manner that provides positive surface gradient away from all structures. The ponding of water should not be allowed adjacent to structures, retaining walls, or the top of cut or fill sections. Surface runoff should be directed toward engineered collection systems or suitable discharge areas and not allowed to flow over slopes. Discharge from structures should also be collected, conveyed, and discharged away into engineered systems, such as storm drains. Landscape plantings around structures should be avoided or be dry climate tolerant and require minimal irrigation.

5.3.7 Excavation Characteristics & Bulking

Exploration at the site occurred using CME-75 drill rigs utilizing 8.25-inch diameter hollow-stem augers. Penetration of underlying soil and rock materials was performed with little difficulty within the existing engineered fill materials and moderate to high difficulty in the underlying Panoche Formation, especially with depth. The dense consistency and presence of aggregate resulting in practical refusal of the hollow-stem auger within the Panoche

[1] This test procedure applies wherever relative compaction, maximum dry density, or optimum moisture content is referenced within this report.

Formation for most drill holes. Because those drill holes have a relatively small diameter, aggregate can make penetration of the auger difficult where heavy grading equipment is typically not limited to the confined excavation area. Because of this, it is our opinion that the underlying Panoche Formation should be excavatable with heavy grading equipment with moderate to high difficulty. Blasting and other relatively unconventional excavation methods are not anticipated as necessary for this site.

It should be noted that the ability to excavate underlying soil and rock materials does not imply that the excavated materials will be of small enough dimension to be used within engineered fill, as discussed in Section 5.3.11, without further mechanical breaking or crushing of those materials.

Bulking or shrinkage of excavated materials at the project site can be estimated using the following information:

SHRINKAGE & BULKING FACTORS		
Material	**Shrinkage**	**Bulking**
Landslide Deposits & Artificial Fill Materials	3% to 5%	
Upper 10 feet of Older Alluvium/Panoche Formation	1% to 3%	
Materials below upper 10 feet of the Older Alluvium/Panoche Formation		3% to 5%

The shrinkage and bulking factors do not include the shrinkage due to segregation of oversized rock materials or zones of highly organic soils from engineered fill materials being placed. Based on our observations, we estimate that less than 5 percent should consist of oversize materials. This number could locally be larger. These factors should be included in volume calculations for on-site soils that are excavated then compacted per recommendations within this report.

5.3.8 Temporary Slopes

This section explicitly excludes trench slopes for buried utilities. Temporary trench excavations are discussed in Section 5.6.4 of this report.

Construction of the proposed project is anticipated to require temporary slopes to facilitate construction of below-ground improvements. All temporary excavations must comply with applicable local, state, and federal safety regulations, including the current OSHA Excavation and Trench Safety Standards. Construction site safety is the responsibility of the Contractor, who should be solely responsible for the means, methods, and sequencing of construction operations so that a safe working environment is maintained.

Temporary construction slopes can be constructed at varying inclinations depending on slope height. We recommend referring to Plate 13 – Factor of Safety vs. Slope Inclination and utilizing a Factor of Safety (FOS) of 1.2 to evaluate temporary slope inclination versus height. The following table provides maximum slope inclination versus height for temporary slopes having a static FOS of 1.2 or higher.

TEMPORARY SLOPE INCLINATIONS	
Slope Height (feet)	**Maximum Inclination (degrees)**
10	50°
20	42°
30	39°
40	37°

We recommend that efforts be made during construction to limit exposure of temporary slopes more than 20 feet in height to seasonal dry times of year. Temporary cut slopes exposed between November and March have an increased risk of failure.

Heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a 1:1 (horizontal to vertical) projection from the toe of the excavation to the ground surface, unless shoring is being used and has specifically been designed for those surcharge loads. Where the stability of adjoining improvements, walls, utility poles, or other structures is endangered by excavation operations, support systems such as shoring, bracing, or underpinning may be required to provide structural stability and to protect personnel working within the excavation.

During wet weather, earthen berms or other methods should be used to prevent runoff water from entering excavations. All runoff water entering the excavation(s) should be collected and disposed of outside the construction limits.

5.3.9 Permanent Slopes

Permanent slopes should be constructed at inclinations of 2:1 or flatter. If proposed unsupported cut slopes cannot be excavated at 2:1 or flatter, then additional slope stability analyses will need to be performed to confirm the maximum slope inclination pertinent to the slope height and location. If a minimum FOS of 1.1 and 1.5 for pseudostatic and static conditions, respectively, cannot be obtained for slopes steeper than 2:1 than additional slope reinforcements or retaining structures will be necessary to support some or the entire proposed slope. Slope reinforcement can include construction of retaining walls, installation of soil nails, construction of soldier pile or tieback walls, etc. Retaining walls/retention systems should be of sufficient height to allow construction of permanent cut slopes above the walls that meet the inclination recommendations made herein.

5.3.10 Overexcavation

As discussed in Section 5.4.3 of this report, a 5-foot thick engineered and reinforced fill pad (mechanically stabilized earth composite raft or MSE raft) is recommended beneath the proposed tanks, which will require overexcavation and removal of an equal volume of existing soils. Otherwise, overexcavation of soils and/or rock materials at the site is not anticipated to be needed unless a transition lot is created, as discussed in Section 5.4.2 of this report.

5.3.11 On-Site Soil Materials

It is our opinion that most of the near-surface soils encountered at the site can be used for general engineered fill provided they are free of organics, debris, oversized particles (>3") and deleterious materials. If highly plastic clayey materials (materials having a plasticity index exceeding 30 and a liquid limit more than 50) are encountered during grading, those materials should be segregated and excluded from engineered fill, where possible. If potentially unsuitable soil is considered for use as engineered fill, VSI should observe, test, and provide recommendations as to the suitability of the material prior to placement as engineered fill.

5.3.12 Imported Fill Materials - General

If imported fill materials are used for this project, they should consist of soil and/or soil-aggregate mixtures generally less than 3 inches in maximum dimension, nearly free of organic or other deleterious debris, and essentially non-plastic. Typically, well-graded mixtures of gravel, sand, non-plastic silt, and small quantities of clay are acceptable for use as imported engineered fill within foundation areas. Imported fill materials should be sampled and tested prior to importation to the project site to verify that those materials meet recommended material criteria noted below. Specific requirements for imported fill materials, as well as applicable test procedures to verify material suitability are as follows:

IMPORTED FILL RECOMMENDATIONS				
GRADATION				
Sieve Size	**General Fill**	**Granular Fill**	**Test Procedures**	
	Percent Passing		**ASTM**	**AASHTO**
3-inch	100	100	D422	T88
¾-inch	70 – 100	70 – 100	D422	T88
No. 200	0 - 30	<5	D422	T88
PLASTICITY				
Liquid Limit	<30	NA	D4318	T89
Plasticity Index	<12	Nonplastic	D4318	T90
ORGANIC CONTENT	<3%	<3%	D2974	NA

5.3.13 Materials - Granular

All granular fill should consist of imported soil mixtures generally less than 3 inches in maximum dimension, nearly free of organic or other deleterious debris, and essentially non-plastic. Specific requirements for granular fill, as well as applicable test procedures to verify material suitability are presented in Section 5.3.12 of this report.

5.3.14 Controlled Low Strength Material

Controlled low strength material (CLSM) can be used to backfill excavated areas or as engineered fill materials. CLSM consists of a fluid, workable mixture of aggregate, cement, and water that is of limited strength as to allow future excavation and maintenance of buried improvements yet capable of supporting the proposed improvements. If CLSM is used as engineered fill materials, we recommend that those materials conform and be placed per specifications presented in Section 19-3.062 of the Caltrans Standard Specifications (most current edition).

5.3.15 Placement & Compaction

In general, soil and/or soil-aggregate mixtures used for engineered fill should be uniformly moisture-conditioned to within 3-percent of optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to at least 90 percent relative compaction in accordance with standard test method ASTM D1557[2]. As discussed in in Section 5.4.3 of this report, the materials in the MSE raft under the tanks should be compacted to at least 95-percent relative compaction per standard test method ASTM D1557. It is recommended that fill materials be placed and compacted uniformly in elevation around buried structures and that the vertical elevation differential of contiguous lifts diverge no more than three feet around the structure during compaction. Testing should be performed to verify that the relative compactions are being obtained as recommended herein. Compaction testing, at a minimum, should consist of one test per every 500 cubic yards of soil being placed or at every 1.5-foot vertical fill interval, whichever comes first.

In general, a "sheep's foot" or "wedge foot" compactor should be used to compact fine-grained fill materials. A vibrating smooth drum roller could be used to compact granular fill materials and final fill surfaces.

5.4 FOUNDATIONS & SLABS

5.4.1 Summary of Tank Foundation Design Recommendations

The following table provides a summary of foundation design recommendations made in Section 5.4.

[2] This test procedure applies wherever relative compaction, maximum dry density, or optimum moisture content is referenced within this report.

SUMMARY OF TANK FOUNDATION DESIGN RECOMMENDATIONS	
Foundation Element	**Recommended Value**
Shallow Foundation System	
Anticipated foundation materials:	MSE Raft/Panoche Deposits
Minimum embedment depth:	18 inches
Allowable tank bearing pressure:	3,500 psf
Passive Pressure & Coefficient of Friction	
Passive pressure:	350 pcf
Base coefficient of friction:	0.30 to 0.40
Estimated Settlement	
Total:	<3.0”
Differential:	<0.3” in 25 ft
Slab-on-Grade	
Modulus of subgrade reaction (K_{1s})	250 pci

We recommend that the following section be consulted for more details regarding the above recommendations.

5.4.2 Transition Lots

We recommend that the tank and other structure foundations be founded entirely within native, undisturbed soil/rock. Structure foundations should not be founded on a combination of undisturbed Panoche Formation and engineered fill materials (transition lot). For the proposed tank sites, transition lots are not anticipated, as shown on Plates 4.1 through 4.3. However, if a transition lot is estimated to be present beneath other structures proposed for this project, we recommend that **one** of the following mitigation options be incorporated into the proposed grading scheme for the project design:

- The area of cuts supporting the proposed foundations should be overexcavated below the planned bottom of footings to a depth of at least 3 times the width of the foundation. VSI should observe and approve the overexcavated area once exposed. Overexcavation limits should extend throughout the cut area and to a minimum of five horizontal feet past the perimeter foundations of the structure. The overexcavated area should then be backfilled in accordance with recommendations presented in Section 5.3.15 of this report;

OR

- Proposed foundations should be deepened to extend through engineered fill materials to be supported on competent undisturbed native soils, so that the entire foundation system for the structure rests on undisturbed native soils. If this depth is less than 5 feet below the planned bottom of the foundation, then a two-sack sand-cement slurry can be used as backfill in lieu of structural concrete, from the excavation bottom up to the planned bottom of the proposed foundation. VSI should observe and approve the deepened foundation excavation prior to placement of slurry or structural concrete.

5.4.3 MSE Raft

VSI recommends the construction of a mechanically stabilized earth composite raft foundation (MSE raft) beneath the tanks to provide a more uniform bearing layer immediately under the tanks and to help reduce differential settlement to within the threshold values specified by WWE. The MSE raft should consist of a five-foot thick granular blanket reinforced with geogrid. The granular materials should consist of aggregate base material, approved by VSI prior to import to the site, compacted to a minimum of 95-percent relative compaction in accordance with Section 5.3.15 of this report. Aggregate base should conform with the requirements specified for Class 2 Aggregate Base in Section 26-1.02B of the Caltrans Standard Specifications (latest edition). The geogrid should consist of a minimum of three layers of Tensar Geogrid TX5 (or equivalent), equispaced vertically within the aggregate materials. The MSE raft should extend entirely beneath the tanks and a minimum of 5 feet horizontally beyond the outside edge of the tank foundations. Plate 14 – Geosynthetic Composite Raft Foundation Illustration, provides details of the MSE raft.

5.4.4 Shallow Foundations

Foundations must be sized, embedded, and reinforced in accordance with recommendation made by the project structural engineer. All foundation excavations should be made level, except for vertical steps. The allowable bearing pressures provided below are based on a recommended minimum embedment depth of 18 inches below the graded engineered fill surface and a minimum width of 12 inches. Footing thicknesses should be determined by the Structural Engineer. Deep foundation systems, such as CIDH or driven piles, are not anticipated for this project.

5.4.5 Allowable Bearing Pressures

It is assumed that all foundations for the proposed structures will rest entirely on either engineered fill or undisturbed Panoche Formation materials as discussed above. For non-tank structure foundations, isolated and continuous footing elements should be proportioned for dead loads plus probable maximum live load, and a maximum allowable bearing pressure of 2,500 pounds per square foot (psf). More specific bearing pressure recommendations can be provided, if desired, once further details of the structures are known.

The proposed tanks should be designed so that they apply a relatively uniform pressure of 3,500 psf across the entire base when fully loaded.

An increase of allowable bearing pressure by one-third for short-term loading due to wind or seismic forces should NOT be incorporated unless an alternative load combination, as described in Section 1605.3.2 of the 2013 CBC, is applied. The allowable bearing value is for vertical loads only; eccentric loads may require adjustment to the values recommended above. We recommend that VSI be allowed to observe foundation excavations to confirm projected site conditions.

5.4.6 Estimated Tank Settlements

Potential settlement was evaluated using SETTLE3D, developed by Rocscience (2012). Results of settlement analysis are presented on Plate 15 – Estimated Settlement.

The anticipated total settlement for both tank foundations, if construction occurs as recommended within this report, is estimated to be less than three inches. Differential settlement between the center of the tanks and the outside edges is estimated to be approximately one inch or less over a horizontal distance of about 100 feet.

Differential settlement has a greater potential to adversely affect the performance of the proposed tanks. We understand from WWE that differential settlement values should not exceed ½-inch vertical settlement over a horizontal distance of 50 feet.

5.4.7 Frost Penetration

It should be noted that frost heave is not typically a hazard in the San Jose area and is generally not considered in design of foundation systems. Therefore, no recommendations for frost protection have been provided herein.

5.4.8 Slab-on-Grade Design

All ground-supported slabs should be designed to support the anticipated loading conditions. Reinforcement for slabs should be designed to maintain structural integrity, and should not be less than that required to meet pertinent code, shrinkage, and temperature requirements. Reinforcement should be placed at mid-thickness in the slab with provisions to ensure it stays in that position during construction and concrete placement.

A modulus of subgrade reaction (k_{s1}) of 250 pounds per cubic inch (pci) is recommended for design of mat-type foundations. That modulus of subgrade reaction value represents a presumptive value based on soil classification. No plate-load tests were performed as part of this study. The modulus value is for a 1-foot-square plate and must be corrected for mat size and shape, assuming a cohesionless subgrade.

Subgrade soils supporting interior concrete floor slabs should be scarified to a minimum depth of 8 inches, uniformly moisture-conditioned to near the optimum moisture content, and compacted to at least 90-percent relative compaction.

It is unlikely that shallow groundwater will be present beneath the tank sites based on groundwater data available for the project region. Regardless, we understand that a three-foot thick layer of aggregate base materials with an underdrain system will be placed beneath the tank slabs at the site. If seasonal groundwater elevations approach the elevation of the tank slab, the pervious aggregate base materials and underdrain system increase drainage and reduce the potential for groundwater to rise to the slab level. Provisions should be made to outlet (daylight) and drain any accumulated water within the aggregate base. In our opinion, no additional drainage measures need to be designed and constructed for the project.

5.4.9 Lateral Earth Pressures

It is our understanding that buried structures and retaining walls (heretofore referred to as retaining walls) might be utilized in this project. Retaining walls, including buried concrete tank walls, should be designed to resist earth pressures exerted by the retained, compacted backfill plus any additional lateral force that will be applied to the wall due to surface loads placed at or near the wall. The recommended equivalent fluid weights presented below are for static (non-earthquake) conditions.

LATERAL EARTH PRESSURES UNDER STATIC CONDITIONS		
Lateral Earth Pressure Condition	**Slope Inclination Above Structure**	**Equivalent Fluid Weight (pcf)**
		Drained
At-Rest	Flat	55
Active	Flat	40
At-Rest	2:1	70
Active	2:1	55

The resultant force of the static lateral force prism should be applied at a distance of 33 percent of the wall height above the soil elevation on the toe side of the wall.

The tabulated values are based on a soil unit weight of 125 pounds per cubic foot (pcf), and do not provide for surcharge conditions resulting from construction materials, equipment, or vehicle traffic. Loads not considered as surcharges should bear behind a 1:1 (horizontal to vertical) line projected upward from the base of the shoring. If surcharges are expected, VSI should be advised so that we can provide additional recommendations as needed. Surcharge loads induce additional pressures on earth retaining structures. An additional lateral load on non-yielding walls equal to 0.5 times the applied surcharge pressure should be included in the design for uniform area surcharge pressures. Lateral pressures for other surcharge loading conditions can be provided, if required.

Sliding resistance, passive pressures, and safety factors are discussed below in Section 5.5.5, 5.5.6, and 5.5.7, respectively.

5.5 RETAINING WALLS

5.5.1 Summary of Retaining Wall Recommendations

The following table provides a summary of retaining wall and retainage improvement recommendations.

SUMMARY OF RETAINING WALL DESIGN RECOMMENDATIONS	
Foundation Element	**Recommended Value**
Shallow Foundation System (Spread Footings)	
Anticipated foundation materials:	Panoche Formation
Minimum embedment depth:	18 inches
Allowable bearing pressure:	3,000 psf
Allowable bearing pressure increase with depth:	250 psf/ft
Maximum allowable bearing pressure:	3,000 to 4,500 psf
Lateral Earth Pressures (Drained Conditions)	
At-rest (level ground above):	55 pcf
Active (level ground above):	40 pcf
At-rest (2:1 slope above):	70 pcf
Active (2:1 slope above):	55 pcf
Dynamic (seismic) forces:	See Section 5.5.4

We recommend that the above recommendations be utilized following review of Section 5.4 and 5.5 of this report, which contain additional details.

5.5.2 Lateral Earth Pressures

If retaining walls are utilized in this project, they should be designed to resist earth pressures exerted by the retained, compacted backfill plus any additional lateral force that will be applied to the wall due to surface loads placed at or near the wall. The recommended equivalent fluid weights are presented in section 5.4.9 of this report.

The resultant force of the static lateral force prism should be applied at a distance of 33 percent of the wall height above the bottom of the foundation on the back of the wall.

The tabulated values are based on a soil unit weight of 125 pounds per cubic foot (pcf), and do not provide for surcharge conditions resulting from foundations, vehicle traffic, or compaction equipment. The drained values do not provide for hydrostatic forces (for example, standing water in the backfill materials). Foundation loads not considered as surcharges should bear behind a 1:1 (horizontal to vertical) line projected upward from the base of the wall. If conditions such as surcharge resulting from footings or hydrostatic forces are expected, VSI should be advised so that we can provide additional recommendations as needed.

Surcharge loads induce additional pressures on earth retaining structures. An additional lateral load on non-yielding walls equal to 0.5 times the applied surcharge pressure should be included in the design for uniform area surcharge pressures. Lateral pressures for other surcharge loading conditions can be provided, if required.

5.5.3 Drainage Measures

Drainage measures should be constructed behind the proposed retaining walls to reduce the potential for groundwater accumulation. To help reduce the potential for the buildup of hydrostatic forces behind walls, a granular free-draining backfill, at least 2 feet thick, should be placed behind the wall, as shown on Plate 16 – Retaining Wall Details. The two-foot thick layer can be decreased to one foot in thickness if wrapped with a geosynthetic filter fabric; however, the structural engineer should be consulted to confirm that the retaining wall is design to withstand potential increased stresses due to compaction closer to the wall. The free-draining backfill should consist of clean, coarse-grained material with no more than 5 percent passing the No. 200 sieve. Acceptable backfill would be:

- Pervious Backfill conforming to Item 300-3.5.2 of the *Standard Specifications for Public Works Construction* (Greenbook), most current edition;
- Permeable Material (Class 2) conforming to Item 68-1.025 if the *Caltrans Standard Specifications*, most current edition;
- Pea gravel having a nominal diameter or ¼-inch; or
- Crushed stone sized between ¼-inch and ½-inch.

In lieu of free-draining backfill materials of the types suggested above, manufactured (geosynthetic) drainage systems (for example MiraDrain manufactured by TC Mirafi, Inc., or equivalent) can be used against retaining or below-grade walls. Manufacturer recommendations for the installation and maintenance of these products should generally be followed, although they should be reviewed by VSI for approval. In addition, manufactured drainage systems should be attached to the retaining wall face as opposed to the excavated slope face. This implies that provisions to protect the integrity of the drainage panels will need to be made while fill materials are placed behind the walls.

A perforated drainpipe system should be installed at the base of the wall to collect water from the free-draining material and/or geosynthetic drainage system. The drainpipe system should allow gravity drainage of the collected water away from the buried wall or, as a less preferred option, should be tied into a sump and pump system to remove the water to an acceptable outlet facility.

Finish surface grades should be sloped away from the retaining walls and designed to channel water to an acceptable collection and offsite disposal system. Provisions should be included for removal of surface runoff that may tend to collect behind the backs of walls and for drainage of water away from the fronts of walls. Also, provisions should be included to mitigate the infiltration of surface water into the below-ground, free-draining backfill/geosynthetic drainage system by placing a minimum of 18-inches of low permeability compacted soil over the top of those materials.

5.5.4 Dynamic Earth Pressures

For unrestrained walls, the increase in lateral earth pressure acting on the wall resulting from earthquake loading can be estimated using the approach of Seed and Whitman (1970). That theory assumes that sufficient wall movement occurs during seismic shaking to allow active earth pressure conditions to develop. For restrained walls, the increase in lateral earth pressure resulting from earthquake loading also can be estimated using these relations. Because that theory assumes that sufficient movement occurs so that active earth pressure conditions develop during seismic shaking, the applicability of the theory to restrained or basement walls is not direct; however, there have been studies (Nadim and Whitman, 1992) that suggest the theory can be used for such walls.

In the Seed and Whitman (1970) approach, the total dynamic pressure can be divided into static and dynamic components. The estimated dynamic lateral force increase (based on seismic loading conditions) for either unrestrained or restrained walls, could be taken as the following:

$$P_E = 3/8 * pga * \gamma_t * H^2$$

Where:

P_E	=	Seismically-induced horizontal force (lbs per lineal foot of wall)
Pga	=	Peak Ground Acceleration (g)
γ_t	=	Total unit weight of backfill (pcf)
H	=	Height of the wall below the ground surface (ft)

Peak ground acceleration (pga) values for the site are provided in Section 3.1 of this report. The centroid of the dynamic lateral force increment should be applied at a distance of 0.6*H above the base of the wall.

To estimate the total dynamic lateral force, the dynamic lateral force increase should be added to the static earth pressure force computed using recommendations for active lateral earth pressures presented above. That recommendation is based on the concept that during shaking, earth pressures recommended for permanent conditions will be reduced to those more closely approximating active conditions.

5.5.5 Sliding Resistance

Ultimate sliding resistance generated through a compacted soil/concrete interface can be computed by multiplying the total dead weight structural loads by the friction coefficient of 0.30 and 0.35 for native soils and imported granular engineered fill, respectively. A coefficient of friction of 0.40 can be used between aggregate base and the concrete tank foundations and slab on grade. If a membrane, such as polysheeting or PVC, is utilized between the tank foundations and/or slab, then the coefficient of friction between the foundations and/or slab and that sheeting should be established through consultation with the membrane manufacturer.

5.5.6 Passive Resistance

Ultimate passive resistance developed from lateral bearing of shallow foundation elements bearing against compacted soil surfaces for that portion of the foundation element extending below a depth of 1 foot below the lowest adjacent grade can be estimated using an equivalent fluid weight if 350 pcf.

5.5.7 Safety Factors

Sliding resistance and passive pressure may be used together without reduction in conjunction with recommended safety factors outlined below. A minimum factor of safety of 1.5 is recommended for foundation sliding, where sliding resistance and passive pressure are used together

5.5.8 Construction Considerations

Prior to placing steel or concrete, foundation excavations should be cleaned of all debris, loose or disturbed soil, and any water. A representative of VSI should observe all foundation excavations prior to concrete placement.

5.6 PIPELINES & TRENCH BACKFILL

5.6.1 External Loads on Buried Pipelines

External loads on buried pipes will consist of loads due to the overlying earth materials, loads due to construction activities, loads due to traffic, and other post construction land uses. It is recommended that the pipe be designed to resist the imposed loads with a factor of safety and an amount of deflection, as recommended by the pipeline manufacturer.

Loads on the pipe due to the overlying soil will be dependent upon the depth of placement, type and method of backfill, the configuration of the trench, the depth of ground water, and whether any additional fill will be placed above the pipeline, on the ground surface. The earth loads on the pipe can be estimated using formulas developed by Marston (1930) and Spangler (1982).

The following Marston formula can be used to estimate vertical soil loads on rigid pipeline placed in backfilled trenches or tunneled in place (American Concrete Pipe Association [ACPA], 2011):

$$W_d = C_d \gamma B_d^2$$
$$W_t = C_t \gamma B_t^2 - 2cC_tB_t$$

Where:

W_d/W_t	=	Vertical soil load on rigid pipe due to trench backfill/overlying soils, respectively (pounds per foot [lb./ft])
γ	=	145 pounds per cubic foot (pcf) for imported granular trench backfill; and 125 pcf for native soil trench backfill
B_d/B_t	=	Trench width/width of tunnel bore (feet)
C_d/C_t	=	See below
c	=	Coefficient of Cohesion

Plate 17 – Marston's Load Coefficients, can be used to estimate C_d and C_t. The parameters C_d and C_t will depend on: 1) the backfill type; 2) the trench or tunnel width; and 3) the installation depth. For a trench installation with a ratio of backfill depth to trench width at the top of pipe (H/B_d) of at least 1 and for a trench width at top of pipe no greater than 3 times the pipe diameter, the value of C_d and C_t may be calculated using the following equation (ACPA, 2011):

$$C_{d/t} = \frac{1 - e^{-2K\mu' \frac{H}{B_d \text{ or } B_t}}}{2K\mu'}$$

Where:

K	=	Rankine's lateral earth pressure coefficient
μ'	=	Friction coefficient between fill material and sides of trench
H	=	Backfill height above pipe crown

The value Kμ' is dependent on the backfill type, degree of compaction, and moisture content. Where backfill materials are compacted as recommended in Section 5.6.6, the following estimated Kμ' values are applicable for various types of soil and rock encountered during this study and anticipated to be used within the trench zone:

ESTIMATED Kμ' VALUES FOR PIPE DESIGN

Soil Type	Kμ'
Clay (CL,CH)	0.120
Silt (ML)	0.130
Clayey Sand (SC) and Weathered Bedrock	0.150
Estimated from ASCE (1982)	

For flexible pipelines, the prism method (Moser & Folkman, 2008) can be used to estimate the vertical soil loads imposed on pipelines in new trenches. That formula is as follows:

$$W = B\gamma H$$

Where:

W	=	Vertical soil load (lb./ft)
B	=	Outside diameter of the pipeline (ft)
γ	=	145 pounds per cubic foot (pcf) for imported granular trench backfill; and 125 pcf for native soil trench backfill
H	=	Depth of backfill (ft)

In addition to the dead loads noted above, the proposed pipeline will be subjected to vertical live loads within roadways and driveways. Vertical soil pressures due to live vehicular loads can be estimated using the graph presented on Plate 18 – Vertical Soil Pressures Induced by Live Loads.

5.6.2 Modulus of Soil Reaction (E')

Flexible and semi-rigid pipes are typically designed to withstand a certain amount of deflection from applied earth loads. Those deflections can be estimated with the equations developed by Spangler (1982). The modulus of soil reaction (E') values for the project were estimated using relations of Howard (1996). The table below presents E'_b values, which are recommended E' values for pipe zone backfill materials (pipe zone backfill). The recommended E'_b values presented in the table below apply to the initial backfill materials along the sides of the pipe at the recommended level of compaction.

MODULUS OF SOIL REACTION FOR PIPE ZONE BACKFILL MATERIALS (E'_B)

Soil Type	Depth of Burial	Recommended E'_b (psi)
Pipe Bedding and Pipe Embedment (clean crushed rock or sand)	5'	1,000
	10'	1,500
	15'	1,600
	15'+	1,700
Soil-Cement Slurry (backfilled within 2 days of placement)	Not Applicable	3,000

Where the zone of backfill beside the pipe is less than five times the pipeline diameter, the E'_b values above may not be applicable and the constrained soil modulus E'_n will affect flexible pipe design. E'_n corresponds to the E' value for the natural trench wall soils. The actual lateral soil modulus at the pipe depth will lie somewhere in between E'_b and E'_n depending on the trench width. The following E'_n values are recommended for varying earth materials based on data obtained in our field and laboratory investigations.

E'$_N$ VALUES FOR ON-SITE MATERIALS	
Earth Material	**E'$_n$ Value (psi)**
Intact Landslide Deposits	400
Existing Artificial Fill	1,000
Undisturbed Panoche Formation	2,500
Engineered Fill	1,200

Intact Panoche Formation can be anticipated beneath the entire graded pad for the proposed tanks. Plate 3 and Plates 4.1 through 4.3 provide maps and cross sections showing the anticipated distribution of geologic materials across the site.

For trench widths of less than five times the diameter of the pipe, the composite design E_c' (E'_b and E'_n) may be calculated using the Soil Support Combining Factors (S_c) presented in the table below, where B_d is the trench width at pipe springline and D is the diameter of the pipe.

SOIL SUPPORT COMBINING FACTORS (S_C)						
E'_n/E'_b	**B_d/D=1.5**	**B_d/D=2.0**	**B_d/D=2.5**	**B_d/D=3.0**	**B_d/D=4.0**	**B_d/D=5.0**
0.1	0.15	0.30	0.60	0.80	0.90	1.00
0.2	0.30	0.45	0.70	0.85	0.92	1.00
0.4	0.50	0.60	0.80	0.90	0.95	1.00
0.6	0.70	0.80	0.90	0.95	1.00	1.00
0.8	0.85	0.90	0.95	0.98	1.00	1.00
1.0	1.00	1.00	1.00	1.00	1.00	1.00
1.5	1.30	1.15	1.10	1.05	1.00	1.00
2.0	1.50	1.30	1.15	1.10	1.05	1.00
3.0	1.75	1.45	1.30	1.20	1.08	1.00
>5.0	2.00	1.60	1.40	1.25	1.10	1.00
Source: "Pipeline Installation," A. Howard, 1996						

The corresponding composite design E_c' can be calculated by selecting the appropriate S_c value from the table above and multiplying the appropriate E'_b value by S_c, as noted below:

$$E_c' = E'_b(S_c)$$

5.6.3 *Thrust Resistance*

Where the proposed pipelines change direction abruptly, resistance to thrust, if needed, can be provided by mobilizing frictional resistance between pipe and the surrounding soil, by use of a thrust block, by use of restrained pipe joints, or by a combination of the above.

To design thrust resistance by mobilizing frictional resistance, we recommend that a coefficient of friction of 0.20 for PVC or HDPE pipelines be used. The coefficient of friction value includes a factor of safety of 1.5 and assumes that a sand with a sand equivalent (SE) of 30 or greater will be placed within the pipe zone in accordance with recommendations presented in Section 5.6.5.1. For design of thrust block resistance, a passive lateral earth pressure of 350 psf/ft of depth may be used.

5.6.4 *Excavations, Trenches, Dewatering, & Shoring*

5.6.4.1 Excavation and Trench Slopes

Construction of the proposed project will require temporary excavations and trenching to facilitate construction of earthwork, pipelines, manholes, vaults, and other below ground improvements. All temporary excavations and slope inclinations must comply with applicable local, state, and federal safety regulations, including the current OSHA Excavation and Trench Safety Standards. Construction site safety is the responsibility of the Contractor, who should be solely responsible for the means, methods, and sequencing of construction operations so that a safe working environment is maintained.

Subsurface soil conditions encountered in project excavations are to be monitored and evaluated by the Contractor in accordance with OHSA guidelines. OSHA soil classification typing includes the following:

OSHA SOIL TYPE DETERMINATIONS	
Stable Rock	Natural solid mineral matter that can be excavated with vertical sides and remain intact while exposed. It is usually identified by a rock name such as granite or sandstone. Determining whether a deposit is of this type may be difficult unless it is known whether cracks exist and whether or not the cracks run into or away from the excavation.
Type A Soils	Cohesive soils with an unconfined compressive strength of 1.5 tons per square foot (tsf) (144 kPa) or greater. Examples of Type A cohesive soils are often: clay, silty clay, sandy clay, clay loam and, in some cases, silty clay loam and sandy clay loam. (No soil is Type A if it is fissured, is subject to vibration of any type, has previously been disturbed, is part of a sloped, layered system where the layers dip into the excavation on a slope of 4 horizontal to 1 vertical (4H:1V) or greater, or has seeping water.
Type B Soils	Cohesive soils with an unconfined compressive strength greater than 0.5 tsf (48 kPa) but less than 1.5 tsf (144 kPa). Examples of other Type B soils are: angular gravel; silt; silt loam; previously disturbed soils unless otherwise classified as Type C; soils that meet the unconfined compressive strength or cementation requirements of Type A soils but are fissured or subject to vibration; dry unstable rock; and layered systems sloping into the trench at a slope less than 4H:1V (only if the material would be classified as a Type B soil).
Type C Soils	Cohesive soils with an unconfined compressive strength of 0.5 tsf (48 kPa) or less. Other Type C soils include granular soils such as gravel, sand and loamy sand, submerged soil, soil from which water is freely seeping, and submerged rock that is not stable. Also included in this classification is material in a sloped, layered system where the layers dip into the excavation or have a slope of four horizontal to one vertical (4H:1V) or greater.
Layered Geological Strata	Where soils are configured in layers, i.e., where a layered geologic structure exists, the soil must be classified on the basis of the soil classification of the weakest soil layer. Each layer may be classified individually if a more stable layer lies below a less stable layer, i.e., where a Type C soil rests on top of stable rock.

Preliminary OSHA Soil Types of A/B are anticipated at the project site. Actual OSHA Soil Types at the site should be determined during construction by the Contractor's Competent Person or by a registered design professional retained by the Contractor as soils are exposed within the excavations. OSHA allows designation of slope inclinations based on soil types without the support of a registered design professional if those slopes are less than 20 feet high. To do so, the Contractor is required to designate a "Competent Person" that takes the ultimate responsibility for soil type classification.

The following maximum slope inclinations are allowed based upon OSHA soil types:

OSHA MAXIMUM ALLOWABLE SLOPES	
Soil Type	**Slope Ratio[1]**
Stable Rock	Vertical
Type A	¾:1
Type B	1:1
Type C	1½:1
[1] – horizontal:vertical	

Based on the soils observed at the project site during this investigation, it is not anticipated that loose, running, raveling, and/or flowing conditions will be encountered in excavations or trenches. However, if such conditions are encountered during construction, inclinations of unshored slope excavations may not stand exposed at the slope ratios noted above for OSHA Soil Types. In such situations, proposed excavations in those areas could fail and expand in an area much larger than the proposed width unless the excavation and/or trench is shored and adequately supported.

Heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a 1:1 (horizontal to vertical) projection from the toe of an unsupported trench or other excavation to the ground surface. Where the stability of project improvements is endangered by excavation operations, support systems such as shoring, bracing, or underpinning may be required to provide structural stability and to protect personnel working within the excavation.

5.6.4.2 Dewatering

Groundwater was not encountered within explorations advanced for this study. In addition, piezometers constructed around the project site have generally been dry, as reported by other and as discussed in Section 2.4. It should be noted that this study was performed during a significant drought in the project region and groundwater could be higher than encountered if normal or greater than normal precipitation levels occur between when the study was performed and when construction commences. In addition, if construction is performed during winter or early spring or following a wet weather season then shallower groundwater could be encountered in areas not observed in our explorations. In addition, as previously noted, there is a potential for local perched water conditions to be present and/or for existing trenches and underground utilities to store and transport groundwater that could impact construction.

It is the Contractor's responsibility for developing and implementing the means and measures for capturing and removing or diverting groundwater during construction of the proposed pipeline. If groundwater is encountered during construction, it is recommended that the contractor install measures to capture and/or divert groundwater from entering the excavations. If this is not possible, then the contractor should channel groundwater to flow towards collection points to be removed from the excavations and disposed of at an approved area.

5.6.4.3 Shoring

Preliminary design of braced shoring for trenches may be based on the preliminary shoring pressure diagrams provide on Plate 19 - Preliminary Shoring Pressure Diagrams. The preliminary shoring pressure diagrams provided on Plate 19 represent typical soil conditions encountered during this study. Final earth pressures and pressure diagrams for the design and implementation of individual shoring systems will be dependent upon the following:

- The actual subsurface conditions encountered during construction;
- The shoring type, design, and installation method; and
- Surcharge pressures from traffic, equipment, stockpiles, etc.

Few noncohesive sandy materials were encountered within explorations advanced for this study. If thick layers of cohesionless materials are encountered then those materials could flow or ravel, if in a wet or saturated condition, or ravel or run when dry (Federal Highways Administration, 2014). Flowing soils act like a viscous fluid and can enter a trench from the sidewalls and can flow for relatively long distances. Raveling soils have chunks or flakes of material falling or toppling from trench sidewalls into the trench. Running soils are unstable at angles greater than their angle of repose and will run like pea gravel, granulated sugar or dune sand from a trench side wall into the trench until the slope flattens to that angle of repose.

Hydraulic speed shores and trench box shoring in flowing, running, or raveling ground conditions should not be allowed. Furthermore, soils subject to running, flowing, or raveling will have insufficient strength and stand-up time to safely hold full-depth vertical excavations long enough for complete trench box or speed-shore installations. Vertical excavations in such soils will most likely experience excavation wall loss and related undermining of adjacent pavements, utilities, structures, and improvements. Therefore, as a precautionary measure, shoring with trench boxes in flowing, running, or raveling soils will require very careful interior excavation through the trench box so that there are no unsupported vertical excavation faces as the trench box is incrementally lowered into place.

Additionally, pre-advancing/driving steel backer plates in soil around the exterior perimeter of the trench box and ahead of excavations within the trench box may be necessary to maintain stable sidewalls and protect adjacent pavement, utilities, and structures. Shoring with speed shores in running or fast raveling ground will require solid sheet backing to provide full face support.

In localized cases near critical structures or utilities, special shoring or ground improvement (such as grout stabilization) prior to excavation may be needed to reduce consequential damage. The Contractor should be required to provide any special shoring designs for engineering review. Areas requiring special shoring design should receive preconstruction condition surveys and video/photo documentation of conditions.

Shoring systems that do not provide positive support of excavation walls may allow surface settlement and related damage to existing roadways, utilities, structures, and improvements. A summary of the potential surface settlement of passively-shored excavations is provided in the following table:

POTENTIAL SURFACE SETTLEMENT OF PASSIVELY-SHORED EXCAVATIONS		
Soil Type	**Surface Settlement (% of Excavation Depth)**	**Lateral Zone of Disturbance (Multiples of Excavation Depth)**
Sand	0.5%H	H
Soft to medium stiff clay	1%-2%H	3-4H
Stiff clay	<1%H	2H
Suprenant and Basham (1993)		

5.6.5 *Pipe Zone & Trench Zone Materials*

The use of appropriate pipe zone and trench zone backfill materials is critical for the long-term performance of a buried, flexible pipeline. Pipe zone and trench zone backfill materials are discussed below. Plate 20 - Trench Nomenclature, graphically illustrates the locations of pipe zone and trench zone backfill areas.

5.6.5.1 Pipe Zone Backfill

The pipe zone, as discussed herein, is that cross-sectional area that extends from the bottom of the trench to 6 inches over the crown of the pipeline, and from trench wall to trench wall, as shown on Plate 20. Pipe zone backfill materials should consist of imported soil having an SE of no less than 30 and having a particle size no greater than ½-inch in maximum dimension, per Section 306-1.2.1 of the Greenbook. On-site soils will likely not meet these recommendations.

5.6.5.2 Trench Zone Backfill

Trench zone backfill (i.e., material placed between the top of pipe zone backfill and finished subgrade) may consist of on-site soils or imported materials. If on-site soils are used, then those materials should be screened of deleterious materials, organic debris, highly plastic clay, and oversized materials having dimensions of greater than 3 inches in any direction prior to placement within the trench.

Alternatively, imported soils can be used as trench zone backfill. We recommend that imported trench zone materials conform to recommendations presented for imported general engineered fill materials presented in Section 5.3.12 of this report. Those imported materials should be free of deleterious materials, organic debris, or clasts exceeding 3 inches in diameter in any direction.

5.6.5.3 Controlled Low Strength Backfill

An alternative to the use of pipe zone and trench zone backfill materials noted above is the use of controlled low strength material (CLSM) as pipe and/or trench zone backfill. CLSM consists of a fluid, workable mixture of aggregate, cement, and water that is of limited strength as to allow future excavation and maintenance of buried improvements yet capable of supporting the proposed pipeline and backfill. If CLSM is used in the pipe zone or trench zone, we recommend that those materials conform and be placed according to specifications presented in Section 19-3.062 of the Caltrans Standard Specifications (most current edition). Care should be taken during placement of CLSM materials to prevent the pipeline from floating.

5.6.6 *Placement & Compaction*

Trench backfill should be placed and compacted in accordance with recommendations previously provided for engineered fill. Mechanical compaction should be the means in which compaction is achieved. Jetting should not be allowed as a means of compaction. Per Section 306-1.3.3 of the Greenbook, jetting is not allowed if the trench sidewalls have an SE of less than 15.

Special care should be given to ensuring that adequate compaction is made beneath the haunches of the pipeline (that area from the pipe springline to the pipe invert, as shown on Plate 20) and that no voids remain in this space. Compaction tests of pipe zone backfill should be performed at horizontal intervals of no more than 300 feet and vertical intervals of no more than 18 inches. Within the pipe zone, compaction tests should be performed near springline and near the top of the pipe zone backfill. Assessment of the potential presence of voids within the haunch area should be performed following completion of those compaction tests. If voids are observed, then the contractor should be required to rework the pipe zone materials to eliminate the presence of voids in the pipeline haunches. Retesting of the pipe zone materials should then be performed. All areas of failing compaction tests should be reworked and retested until the specified relative compaction is achieved. Compaction of trench zone backfill should be performed at horizontal intervals of no more than 300 feet and vertical intervals of no more than 18 inches.

Placement of CLSM materials should be performed in accordance with specifications presented in Caltrans Standard Specification 19-3.062. If CLSM is used, then compaction tests are not required; however, a minimum of four hours should be allowed between placement of CLSM and placement of engineered fill materials above the CLSM, as noted in Caltrans Standard Specification 19-3.062.

5.6.7 *Trench Subgrade Stabilization*

Soft and yielding trench subgrade is unlikely to be encountered along the bottom of trench excavations made within the Panoche Formation. However, if yielding subgrade is observed, it is recommended that the bottom of trenches be stabilized prior to placement of the pipeline bedding so that, in the judgment of the geotechnical engineer, the trench subgrade is firm and unyielding. The Contractor should have the sole responsibility for design and implementation of trench subgrade stabilization techniques. Some methods that we have observed used to stabilize trench subgrades include the following:

- Use of ¾–inch to 1½-inch floatrock worked into the trench bottom and covered with a geotextile fabric such as Mirafi 500X;
- Placement of a geotextile fabric, such as Mirafi 500X, on the trench bottom and covered with at least one foot of compacted processed miscellaneous base (PMB) conforming to the requirements of Section 200-2.5 of the Greenbook, latest edition;
- Overexcavation of trench subgrade and placement of two-sack sand-cement slurry; and
- In extreme conditions, injection grouting along the trench alignment.

If floatrock is used, typically sand with an SE of 50 or more should be used to fill the voids in the rock prior to placement of pipe bedding materials.

6 PAVEMENT DESIGN

6.1 R-Values

An R-value test was performed on a selected sample of on-site soils obtained during subsurface exploration at the site. The R-value test was performed in accordance with Caltrans test method CT-301 and is presented in Appendix B. A laboratory R-value of 21 was obtained from the testing. Because the actual subgrade materials that will be present at finish subgrade are unknown now, we recommend that confirmatory R-value tests be obtained during construction. If construction R-values are significantly different than the R-value reported above, then we can modify the pavement design at that time to reflect the constructed conditions.

6.2 Subgrade Preparation

All subgrade soils should be scarified to a minimum depth of 1-foot, moisture conditioned as necessary to near optimum moisture conditions and compacted to a minimum of 95 percent of the maximum dry density as determined by AASHTO (American Association of

State Highway and Transportation Officials) Test Method T-180. The subgrade should be smooth and unyielding prior to the placement of aggregate base rock. Density testing and proof rolling of the subgrade using a loaded water truck should be performed with satisfactory results prior to placement of the aggregate base rock. Concrete curbs and landscape planters that border pavement sections should be embedded into the subgrade soils a minimum of 2 inches to reduce the migration of meteoric and irrigation water into the pavement section.

If soft and yielding areas are found during construction, VSI should review these conditions (as well as the contractor's capabilities) and, if requested, provide recommendations for their treatment.

6.3 Aggregate Base

The aggregate base (AB) should be of such quality as to meet or exceed Caltrans specifications for Class 2 AB and should have a minimum R-value of 78. The AB should be spread in thin lifts restricted to 8 inches in loose thickness or less, moisture conditioned as necessary to near optimum moisture content and compacted to a minimum of 95 percent of the maximum dry density as determined by AASHTO T-180. Density testing and/or proof rolling should be performed prior to placement of the asphalt paving.

6.4 Asphalt Concrete Paving

An R-value obtained for this study had a value of 21. Traffic indices (TI) for proposed project access roads were not available to us at the preparation time of this report. To provide recommendations for structural pavement sections, we evaluated design criteria for five TIs ranging from 5.5 to 10. Using those criteria, we have prepared AC structural pavement section recommendations based on Caltrans' pavement standards. Recommendations for full depth AC, and AC and AB sections are provided in the following table:

PRELIMINARY STRUCTURAL PAVEMENT SECTIONS			
Scenario	Traffic Index	Type "B" Asphaltic Concrete Thickness (ft)	Class 2 Aggregate Base Thickness (ft)
Full Depth AC	5	0.55	NA
	6	0.65	NA
	7	0.75	NA
	8	0.85	NA
	9	0.95	NA
	10	1.00	NA
AC & AB	5	0.20	0.70
	6	0.25	0.85
	7	0.35	0.95
	8	0.40	1.15
	9	0.45	1.35
	10	0.50	1.55

Asphalt paving materials and equipment should meet or exceed current Caltrans specifications.

7 REVIEW OF PLANS AND SPECIFICATIONS

We recommend VSI conduct a general review of final plans and specifications to evaluate whether recommendations contained herein have been properly interpreted and implemented during design. If VSI is not retained to perform this recommended review, we will assume no responsibility for misinterpretation of our recommendations.

8 ADDITIONAL SERVICES

This report and its associated recommendations were intended to assist WWE during predesign stages of the project. We recommend that as the project continues that VSI be given the opportunity to collaborate on the project refinements so that: 1) we can confirm that project design conforms with recommendations made, herein; and 2) preliminary recommendations made within this report can be refined, where necessary, based on the design elements of the project. VSI should be provided the opportunity to review and comment on project plans and specifications prior to bid advertisement for the project.

9 LIMITATIONS

This report has been prepared in substantial accordance with the generally accepted geotechnical engineering practice, as it existed in the site area at the time our services were rendered. No other warranty, either express or implied, is made.

Conclusions and recommendations contained in this report were based on the conditions encountered during our field investigation and are applicable only to those project features described herein (see Section 1.2 – Project Understanding). Soil and rock deposits can vary in type, strength, and other geotechnical properties between points of observation and exploration. Additionally, groundwater and soil moisture conditions can also vary seasonally and for other reasons. Therefore, we do not and cannot have a complete knowledge of the subsurface conditions underlying the project site. The conclusions and recommendations presented in this report are based upon the findings at the points of exploration, and interpolation and extrapolation of information between and beyond the points of observation, and are subject to confirmation based on the conditions revealed by construction. If conditions encountered during construction differ from those described in this report, or if the scope or nature of the proposed construction changes, we should be notified immediately to review and, if deemed necessary, conduct additional studies and/or provide supplemental recommendations. When final site design plans (grading, foundation, retaining walls, etc.) become available, VSI should have the opportunity to review the plans to ensure the recommendations presented in this report remain valid and applicable to the proposed project.

Recommendations provided in this report assume that an experienced, properly licensed geotechnical engineering company will conduct an adequate program of testing and observation during the construction phase to evaluate compliance with our recommendations.

The scope of services provided by VSI for this project did not include the investigation and/or evaluation of toxic substances, or soil or groundwater contamination of any type. If such conditions are encountered during site development, additional studies may be required. Further, services provided by VSI for this project did not include the evaluation of the presence of critical environmental habitats or culturally sensitive areas.

This report may be used only by our client and their agents and only for the purposes stated herein, within a reasonable time from its issuance. Land use, site conditions, and other factors may change over time that may require additional studies. In the event significant time elapses between the issuance date of this report and construction, VSI shall be notified of such occurrence to review current conditions. Depending on that review, VSI may require that additional studies be conducted and that an updated or revised report is issued.

Any party other than our client who wishes to use all or any portion of this report shall notify VSI of such intended use. Based on the intended use as well as other site-related factors, VSI may require that additional studies be conducted and that an updated or revised report be issued. Failure to comply with any of the requirements outlined above by the client or any other party shall release VSI from any liability arising from the unauthorized use of this report.

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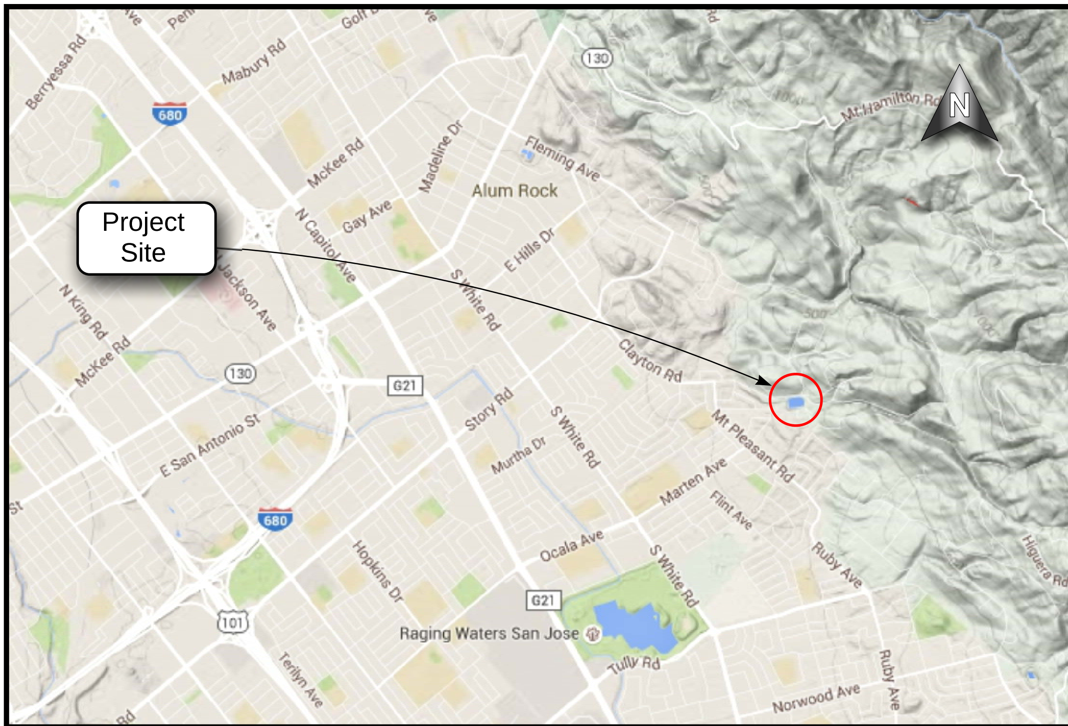
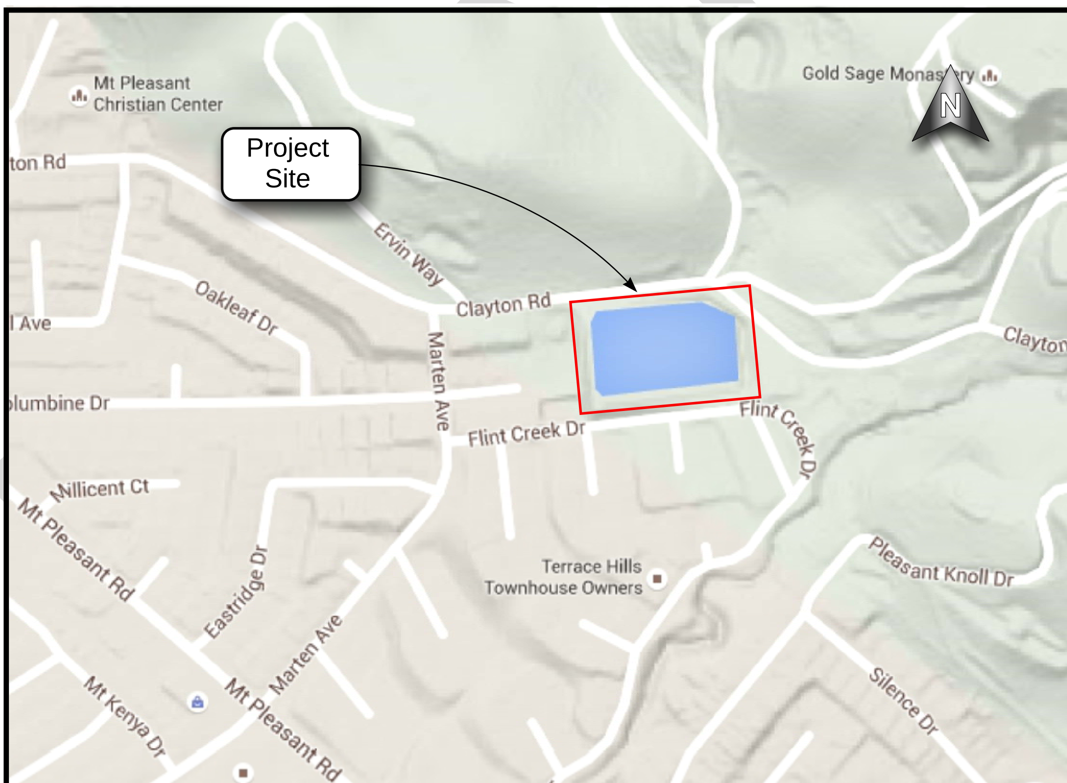
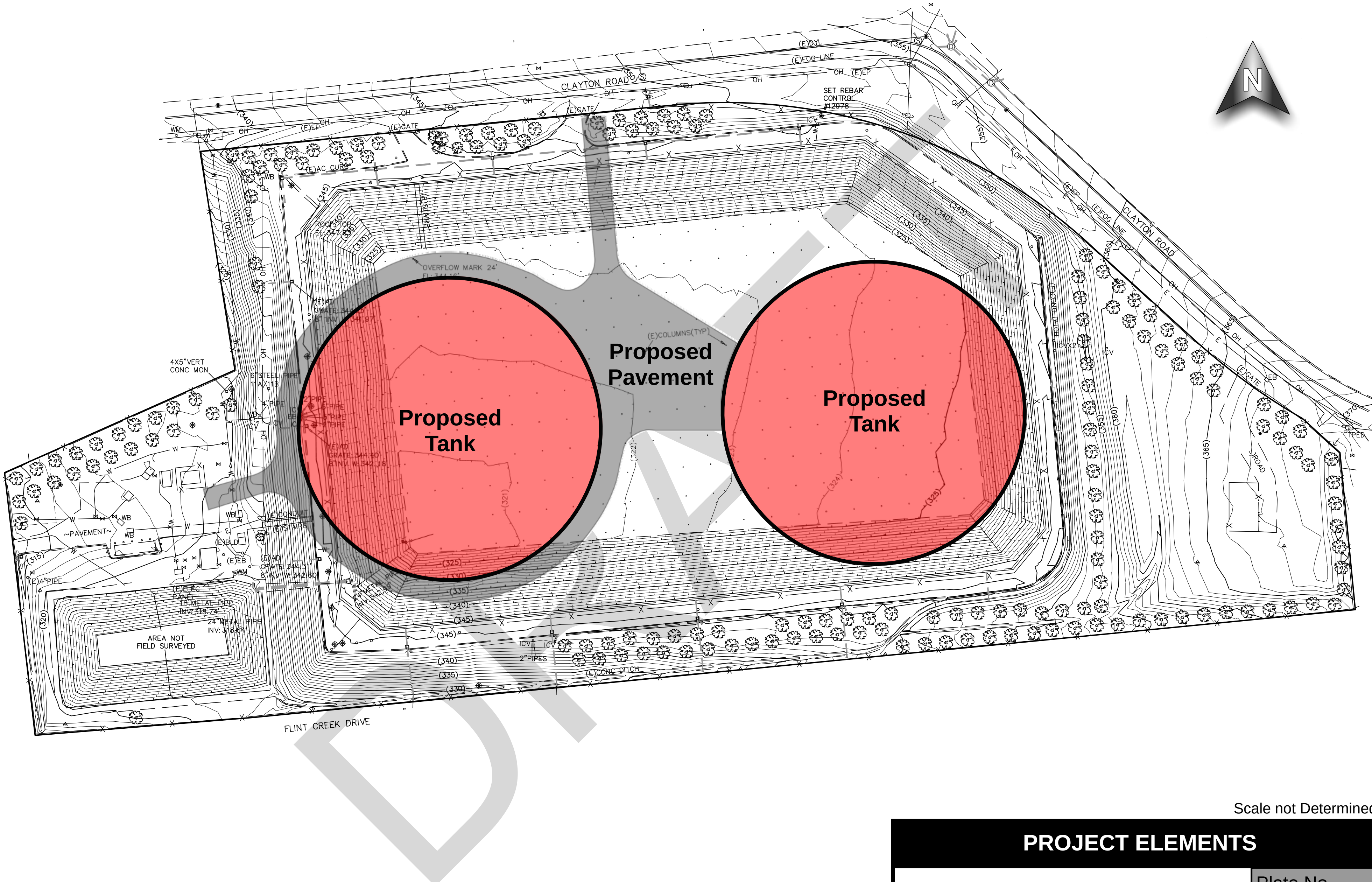




Scale undetermined
Base map derived from Google Maps

SITE LOCATION MAP	
Columbine Station San Jose Water Company Water Works Engineers San Jose, California	Plate No. 1
VERTICAL SCIENCES, INC.	Project no. 160026



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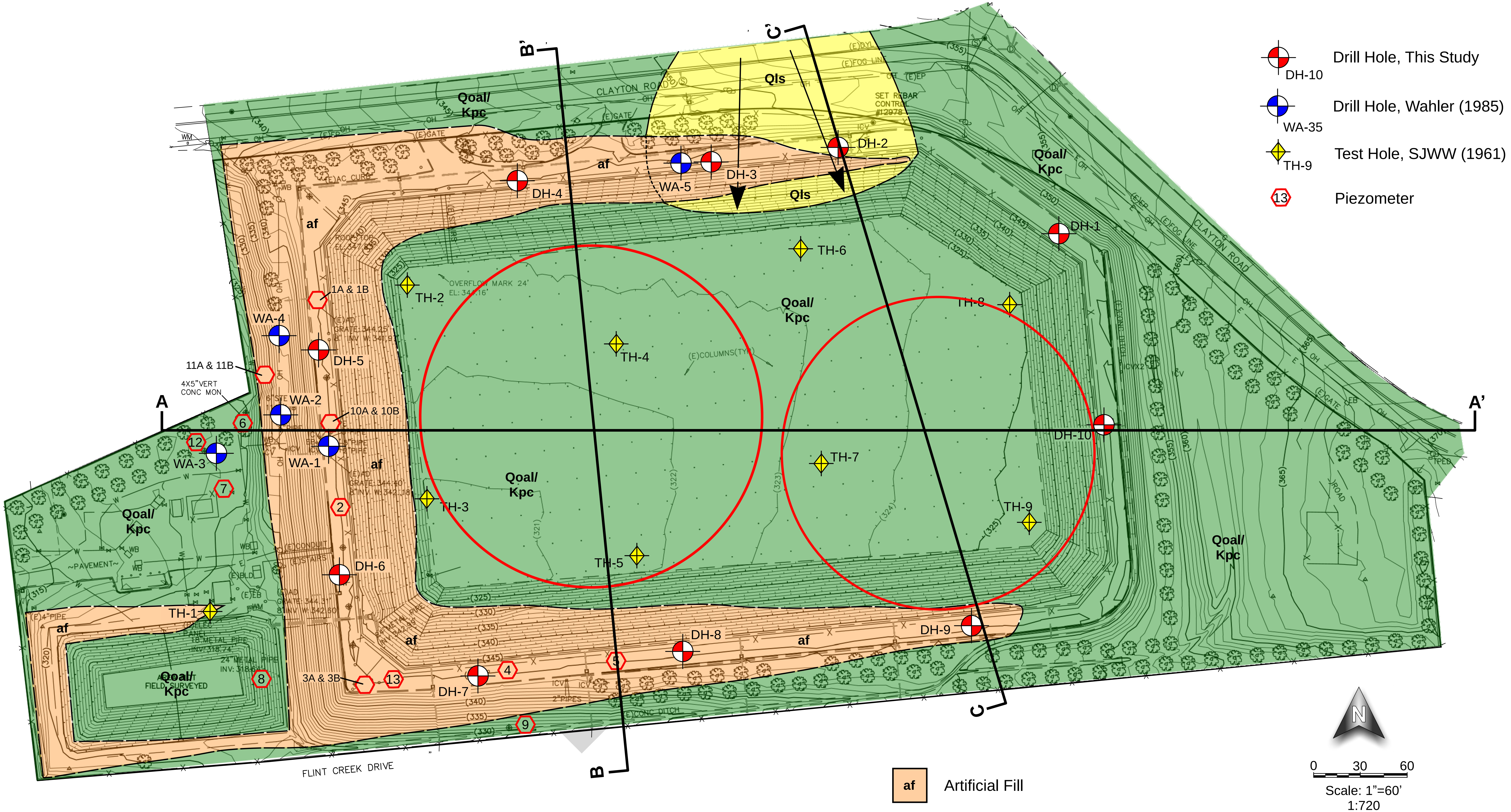
PROJECT ELEMENTS

Columbine Station San Jose Water Company Water Works Engineers San Jose, California	Plate No. 2
VERTICAL SCIENCES, INC.	Project no. 160026

Base map & improvement maps from WWE 2017



Proposed Tank Location

A A'

Geotechnical Cross Section
See Plated 6.1 through 6.3 for Sections

Formation Contact
Dashed where inferred &
short dashed where concealed

af	Artificial Fill
Qls	Landslide Deposits
Qoal/ Kpc	Older Alluvium/ Panoche Formation

GEOTECHNICAL MAP

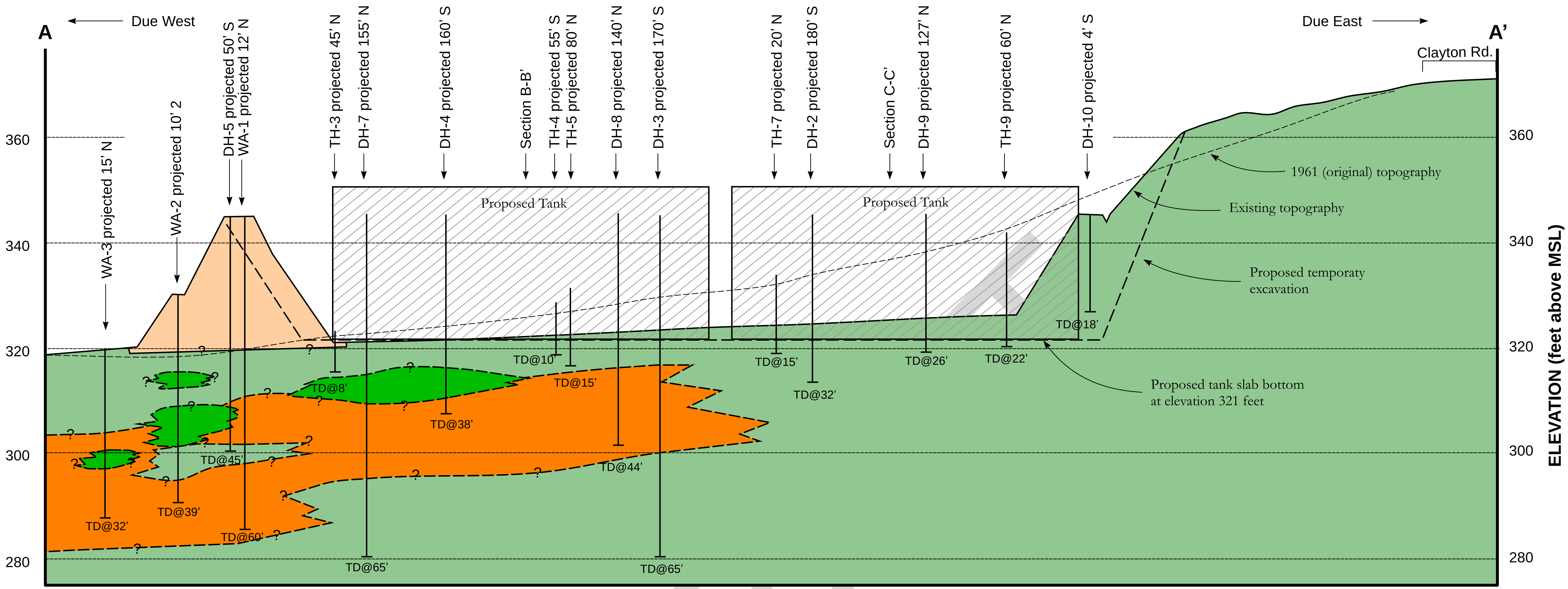
Columbine Station
San Jose Water Company
Water Works Engineers
San Jose, California

Plate No. 3

VERTICAL SCIENCES, INC.

Project no. 160026

Base map derived from Sharrah Dunlap Sawyer (2016)



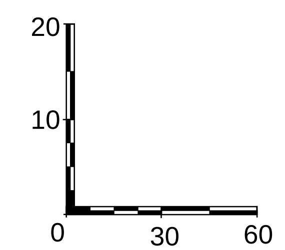
Artificial Fill
- Predominately clay-rich soils (USCS CL/CH)

Older Alluvium/ Panoche Formation
- Predominately clay-rich soils (USCS CL/CH)
- Predominately granular soils (USCS SC/SM/SP) with varying amounts of gravel
- Predominately gravel-rich soils (USCS GM/GP)

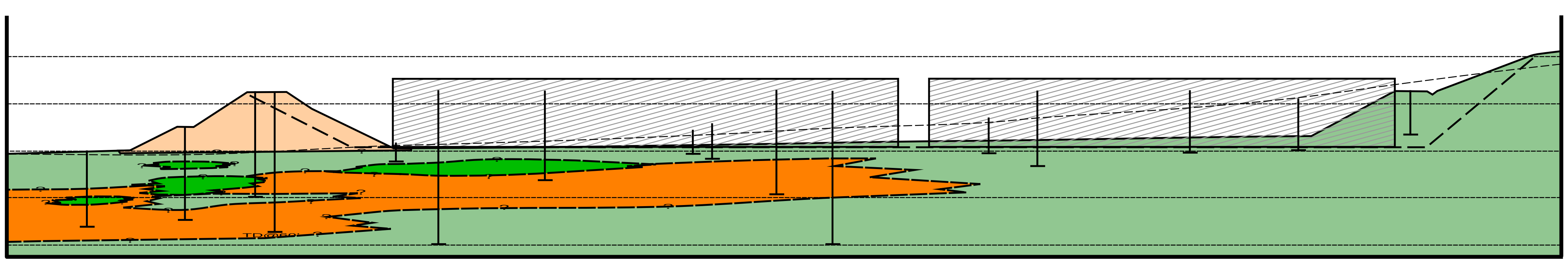
DH-10 — Drill Hole Number & Projected Distance from Cross Section Location

Drill Hole Location

TD@65' — Total Drill Hole Depth (ft)

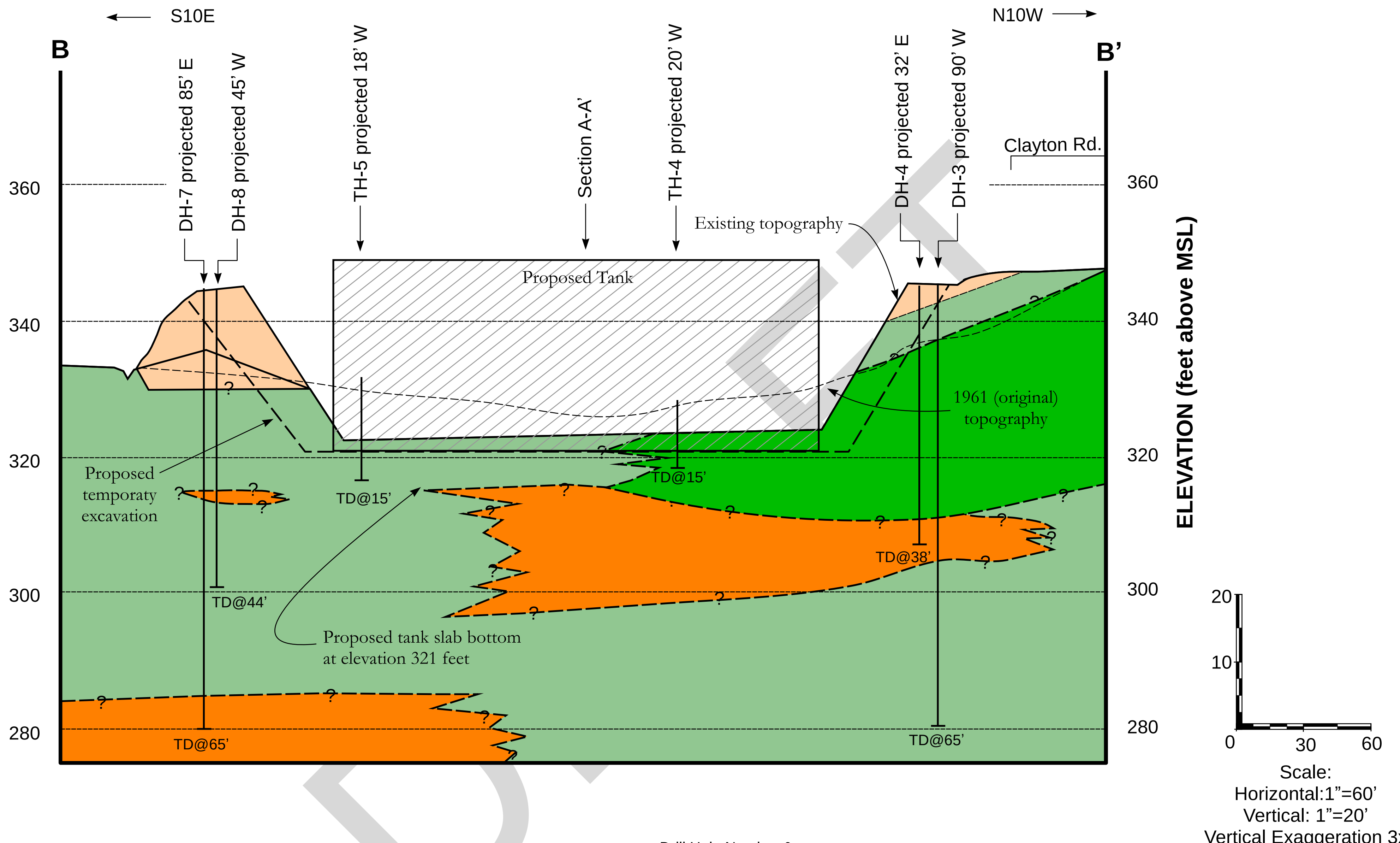


Scale:
Horizontal:1"=60'
Vertical: 1"=20'
Vertical Exaggeration 3x



NO VERTICAL EXAGGERATION

GEOTECHNICAL SECTION A-A'	
Columbine Station San Jose Water Company Water Works Engineers San Jose, California	Plate No. 4.1
VERTICAL SCIENCES, INC.	Project no. 160026



Artificial Fill		Predominately clay-rich soils (USCS CL/CH)
Older Alluvium/ Panoche Formation		Predominately clay-rich soils (USCS CL/CH)
		Predominately granular soils (USCS SC/SM/SP) with varying amounts of gravel
		Predominately gravel-rich soils (USCS GM/GP)

DH-10 — Drill Hole Number & Projected Distance from Cross Section Location

Drill Hole Location

TD@65’ — Total Drill Hole Depth (ft)

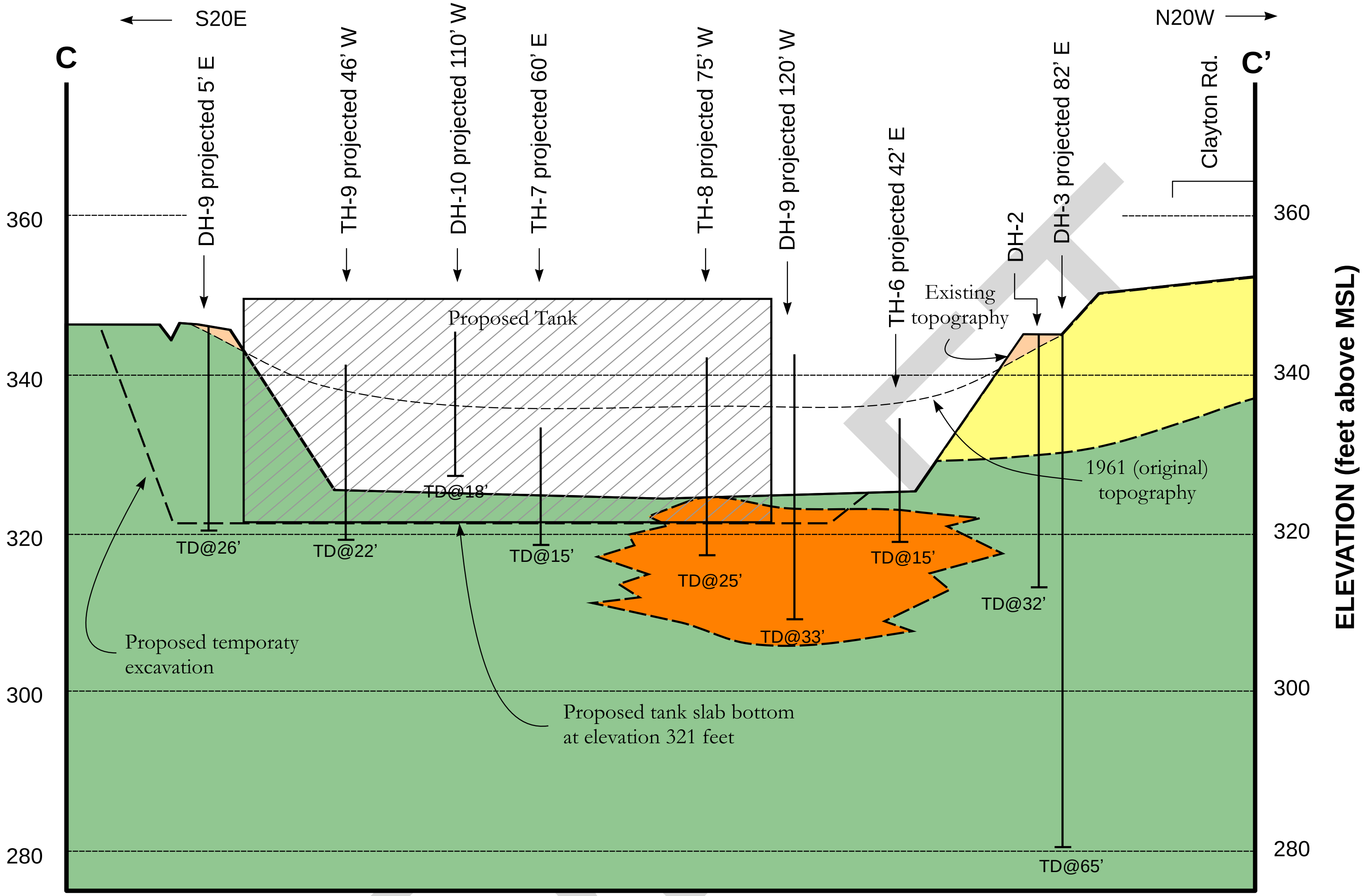
GEOTECHNICAL SECTION B-B’

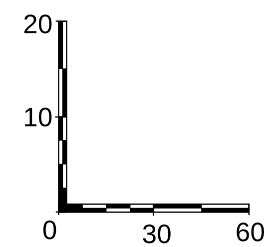
Columbine Station
San Jose Water Company
Water Works Engineers
San Jose, California

Plate No.
4.2

VERTICAL SCIENCES, INC.

Project no.
160026





Scale:
Horizontal:1"=60'
Vertical: 1"=20'
Vertical Exaggeration 3x

Artificial Fill — Predominately clay-rich soils (USCS CL/CH)

Landslide Deposits — Predominately clay-rich soils (USCS CL/CH)

Older Alluvium/ Panoche Formation
- Predominately clay-rich soils (USCS CL/CH)
- Predominately granular soils (USCS SC/SM/SP) with varying amounts of gravel
- Predominately gravel-rich soils (USCS GM/GP) & Sandstone/Conglomerate

DH-10 — Drill Hole Number & Projected Distance from Cross Section Location

Drill Hole Location

TD@65' — Total Drill Hole Depth (ft)

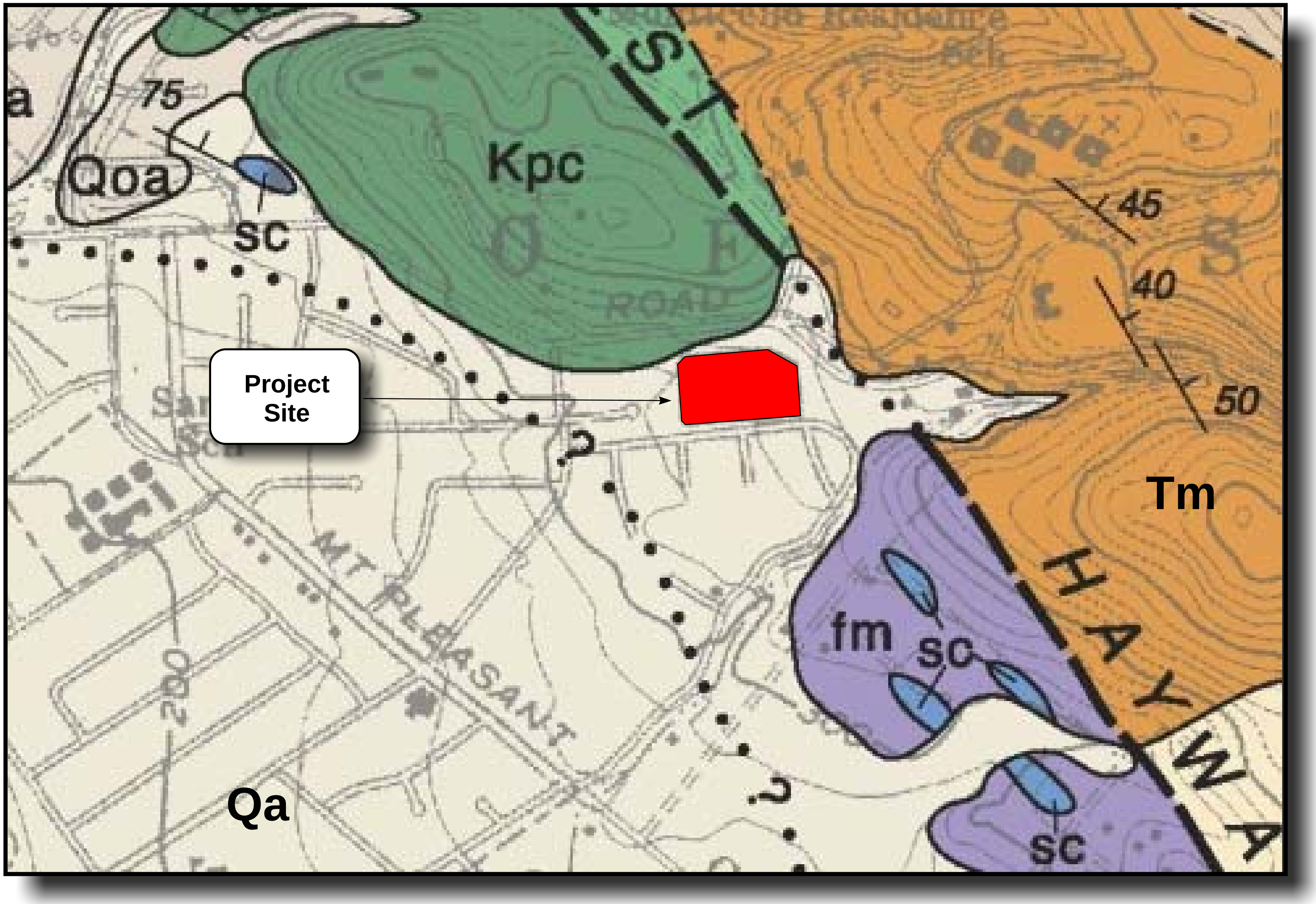
GEOTECHNICAL SECTION C-C'

Columbine Station
San Jose Water Company
Water Works Engineers
San Jose, California

Plate No.
4.3

VERTICAL SCIENCES, INC.

Project no.
160026



Qa	Alluvium
Qoa	Older Alluvium
Tm	Monterey Formation
Kpc	Panoche Formation
fm	Franciscan Melange

Geologic Contact: dashed where approximate, dotted where covered, queried where uncertain

63 84 ?

Fault: showing dip of fault and and trend of striae on fault surface (arrow); bar and ball on downthrown side; dashed where approximate, dotted where concealed; queried where uncertain

Base map from Dibblee & Minch (2005)

Scale not Determined

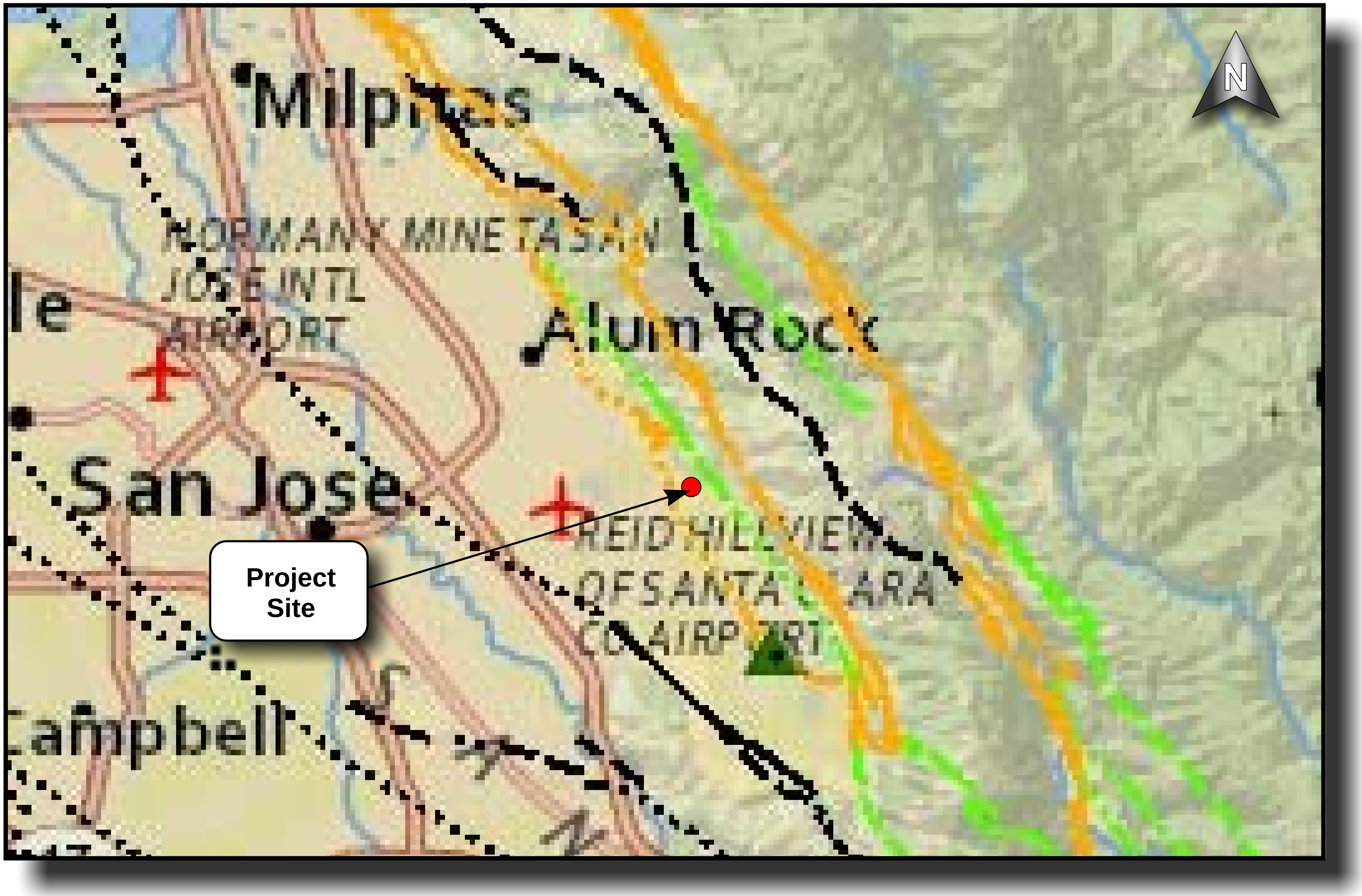
REGIONAL GEOLOGIC MAP

Columbine Station
San Jose Water Company
Water Works Engineers
San Jose, California

Plate No. 5

VERTICAL SCIENCES, INC.

Project no. 160026



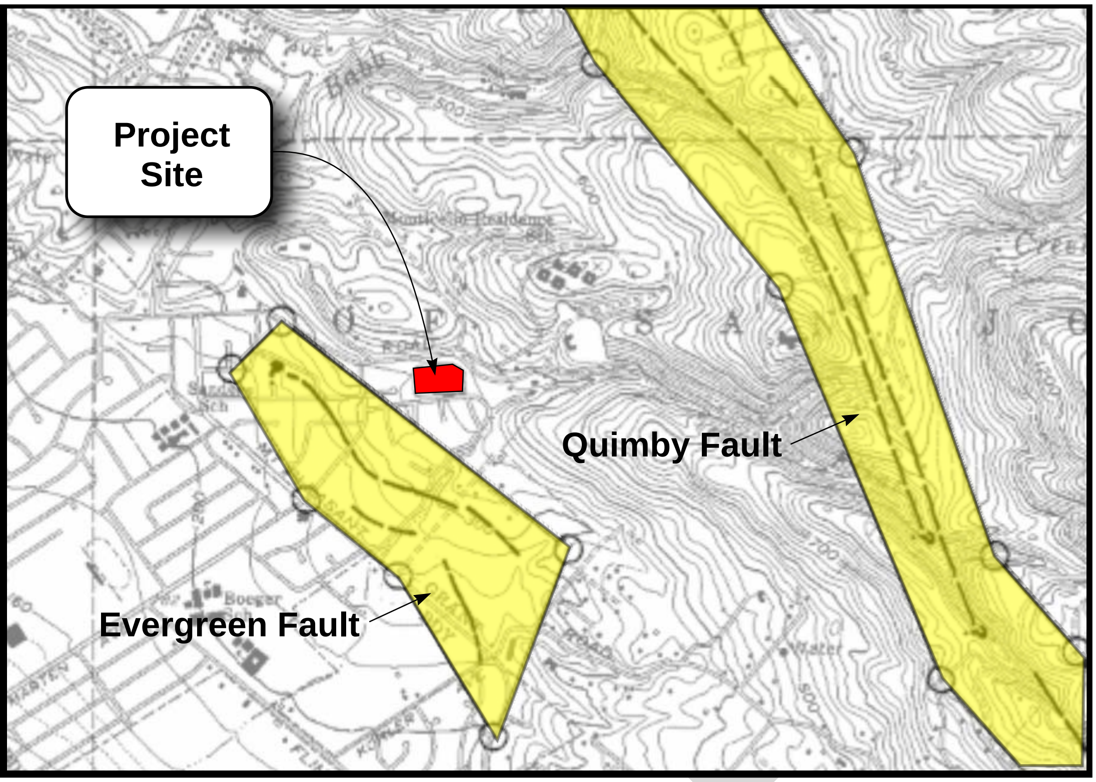
Active	
Historic Displacement (last 200 years)	Holocene Displacement (last 11,700 years)
Potentially Active	**Inactive**
Late Quaternary Displacement (last 700,000 years)	Quaternary Fault (last 1.6 million years)

Scale not Determined

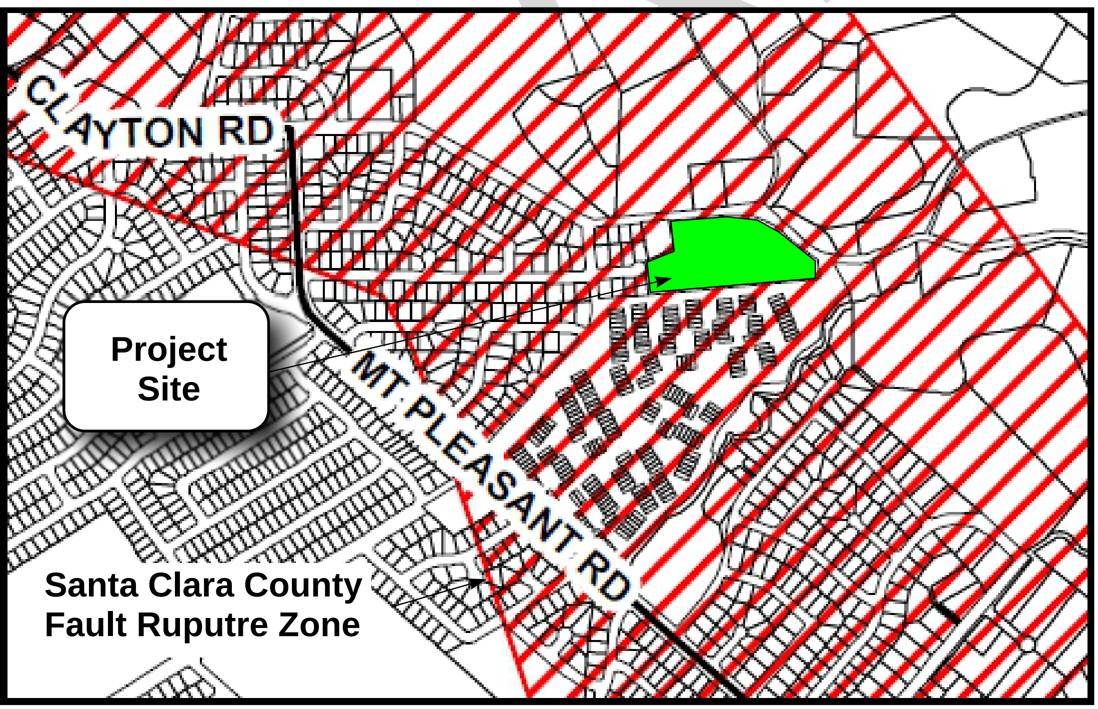
REGIONAL FAULT MAP	
Columbine Station San Jose Water Company Water Works Engineers San Jose, California	Plate No. 6
VERTICAL SCIENCES, INC.	Project no. 160026

Map from USGS Interactive Fault Map

California Special Studies Zones

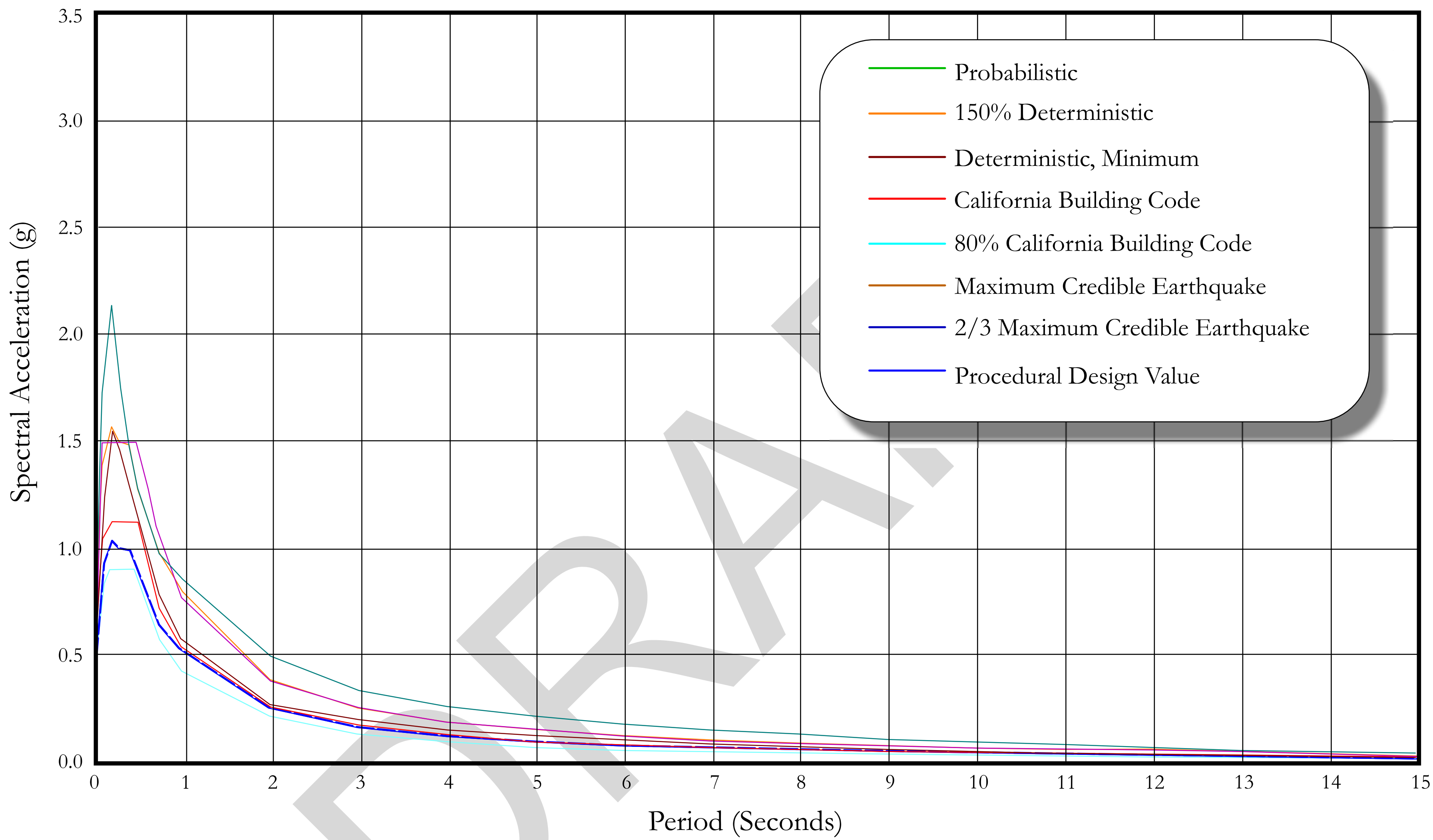


County Special Studies Zones



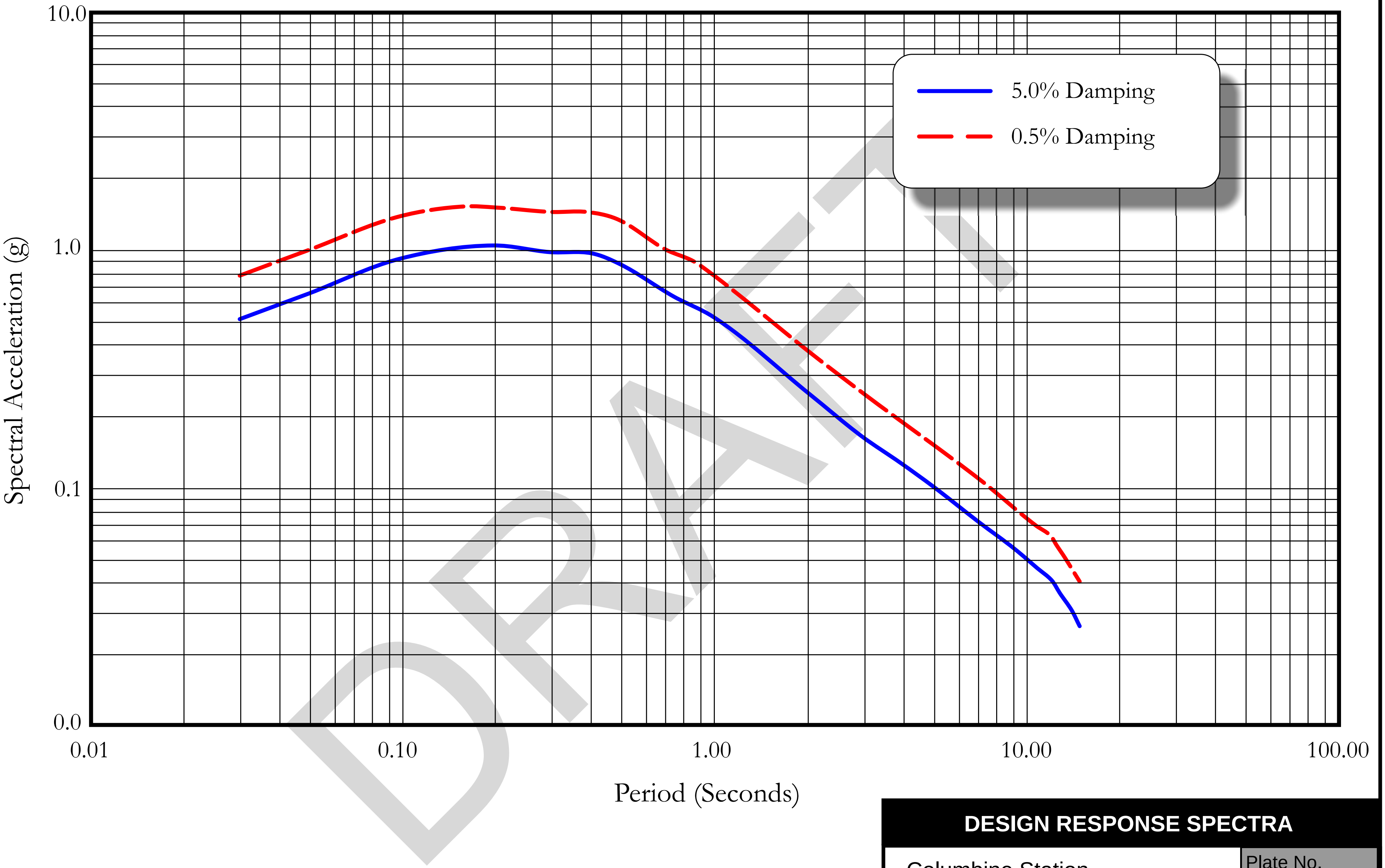
Scale not Determined

SPECIAL STUDIES ZONES	
Columbine Station San Jose Water Company Water Works Engineers San Jose, California	Plate No. 7
VERTICAL SCIENCES, INC.	Project no. 160026



RESPONSE SPECTRA ANALYSIS

Columbine Station San Jose Water Company Water Works Engineers San Jose, California	Plate No. 8
VERTICAL SCIENCES, INC.	Project no. 160026



DESIGN RESPONSE SPECTRA

Columbine Station
San Jose Water Company
Water Works Engineers
San Jose, California

Plate No.
9

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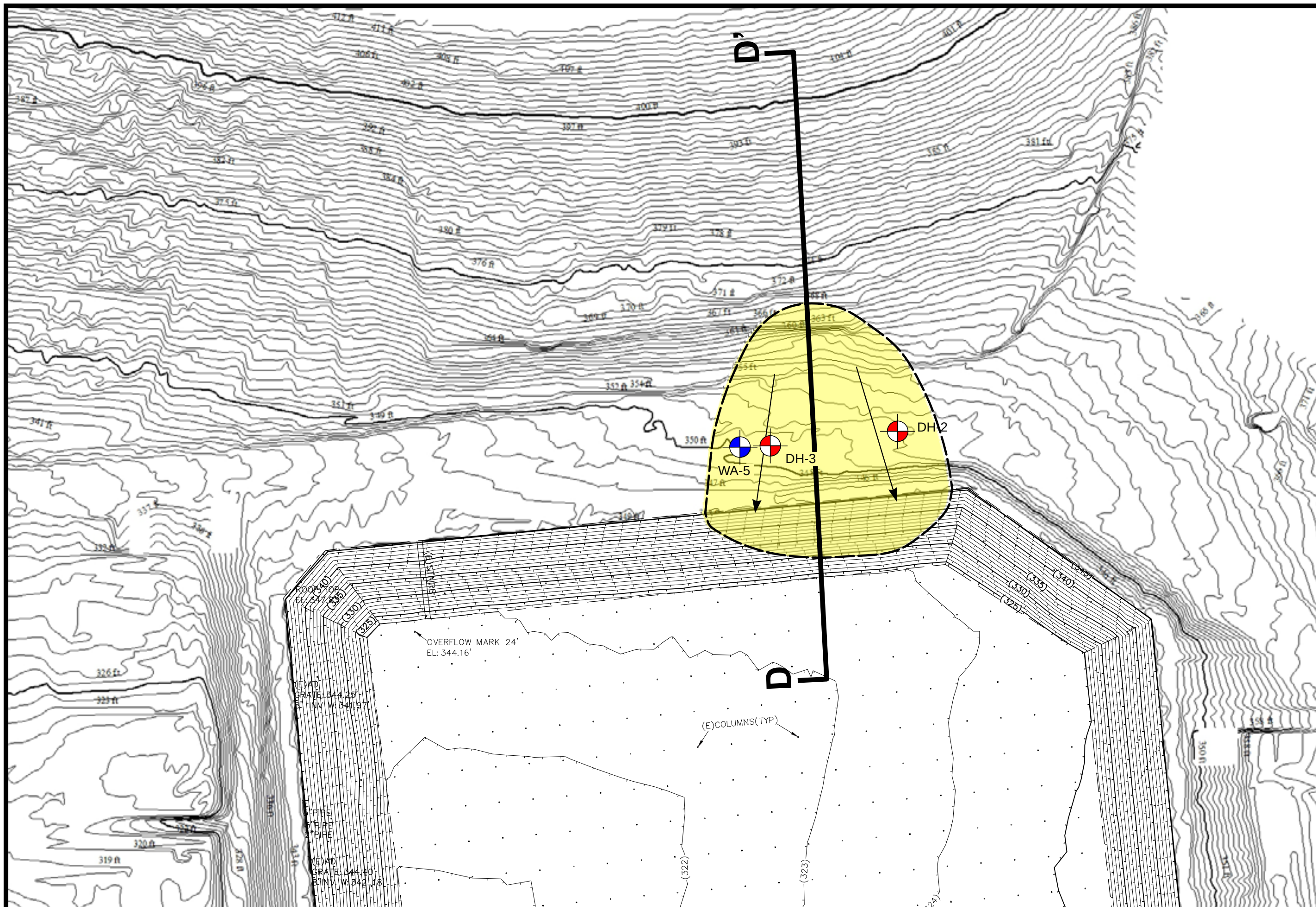
Project no.
160026



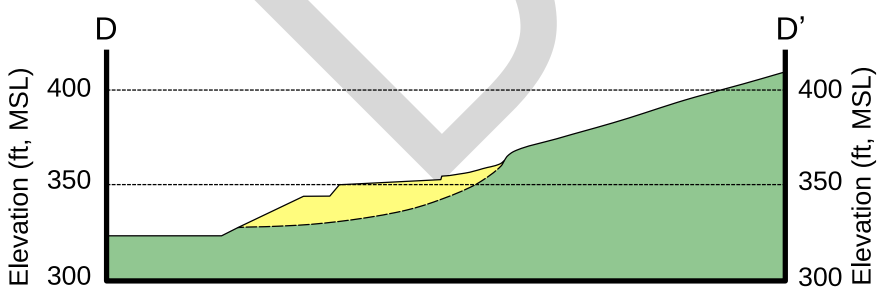
Drill Hole, This Study (DH-10)

Drill Hole, Wahler (1985) (WA-35)

Landslide deposits
Arrows note direction of movement

20
10
0 30 60

Scale:
Horizontal:1"=100'
Vertical: 1"=100'
Vertical Exaggeration 1x



Base map from SDS (2016) and VSI (2016).

EXISTING LANDSLIDE

Columbine Station
San Jose Water Company
Water Works Engineers
San Jose, California

Plate No. 10

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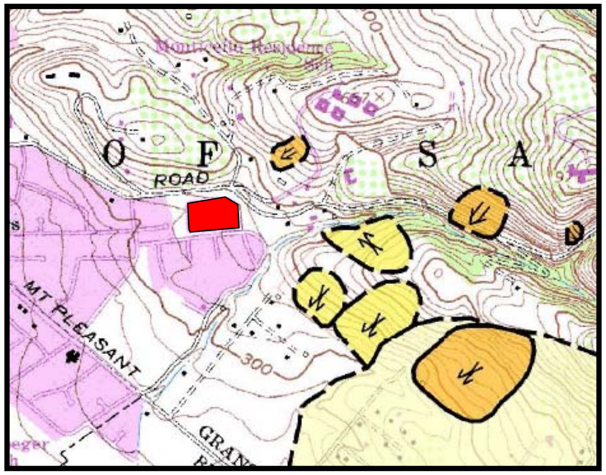
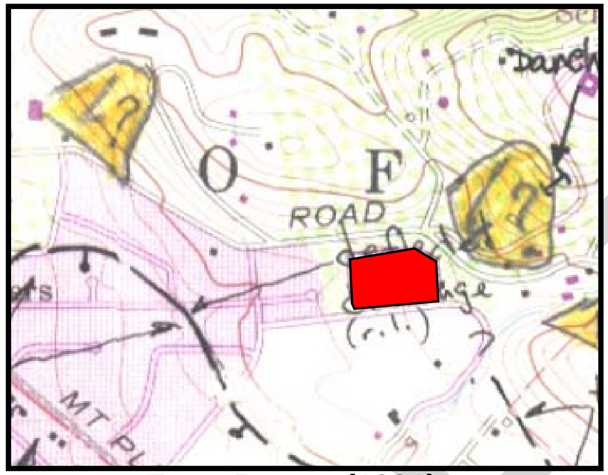
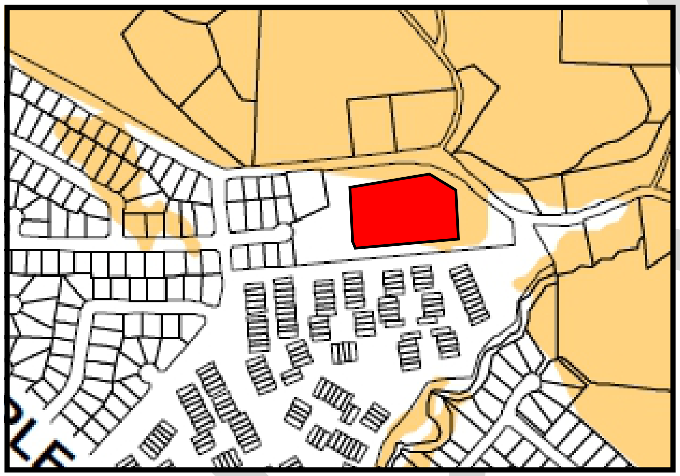


From Wiegers (2011)

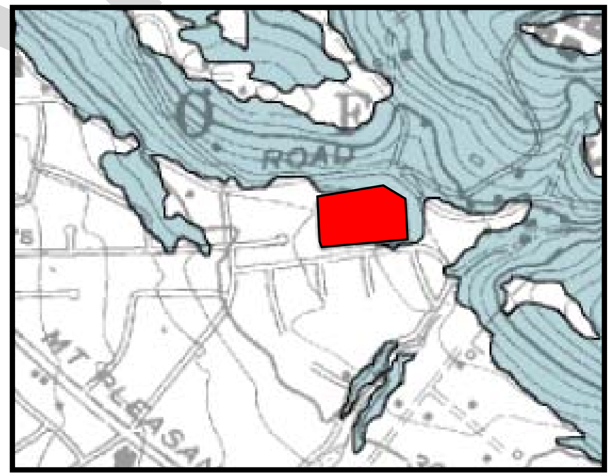


From Bryant (1981)

From CGS (2000)



From Santa Clara County



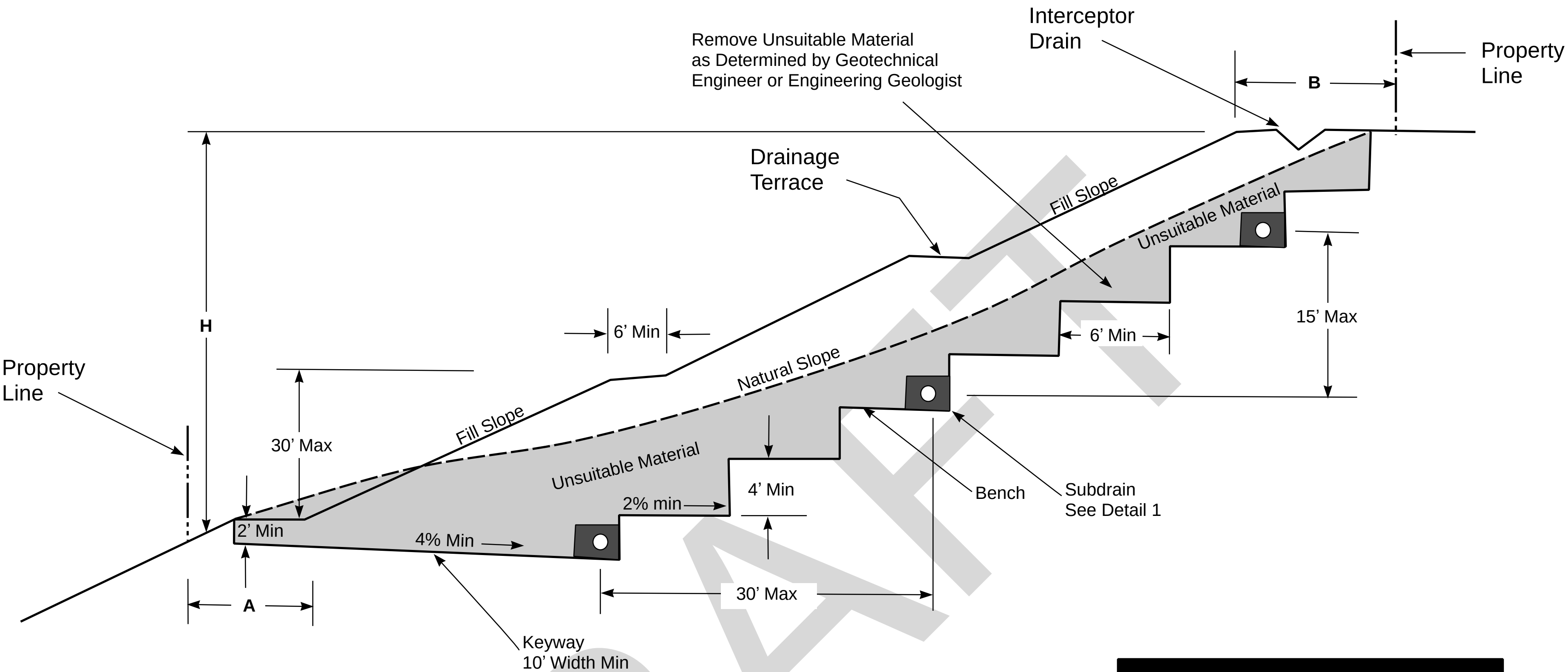
From CGS (2000)

N

Existing Reservoir Site

Scale not Determined

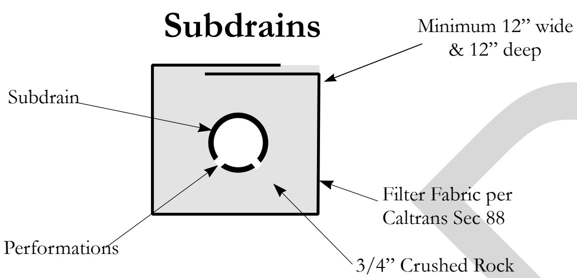
LANDSLIDE HAZARDS	
Columbine Station San Jose Water Company Water Works Engineers San Jose, California	Plate No. 11
VERTICAL SCIENCES, INC.	Project no. 160026



H	A	B
Height, if <5' no Keyway Required	H/5 2' Min 20' Max	H/5 2' Min 10' Max

Per Appendix J of 2010 California Building Code

Detail 1 Subdrains



Subdrain should be minimum 4" diameter, Schedule 40 PVC pipe or corregated HDPE pipe. If fill thickness exceeds 15', then only Schedule 40 PVC pipe should be allowed. PVC pipe should have a minimum of eight 1/2" diameter holes per lineal foot of pipeline along at least two rows separated by 90 radial degrees. The two rows of holes should be installed in the lower portion of the trench as noted in Detail 1. If corregated HDPE is used, perforations should be spaced at min. 3-inch intervals along the length of the pipeline and four, equally spaced rows of performations should be present along the pipeline axis. Subdrain outlets should be to an approved drainage facility. Inlet pipelines should be capped.

Drainage Terraces	
H (ft)	Width (min)
0-60	6'
60-120	6' with one 12' wide at midslope
>120	Designed by Project Engineer

Per Appendix J of 2010 California Building Code

KEYING & BENCHING DETAILS

Columbine Station
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San Jose, California

Plate No.
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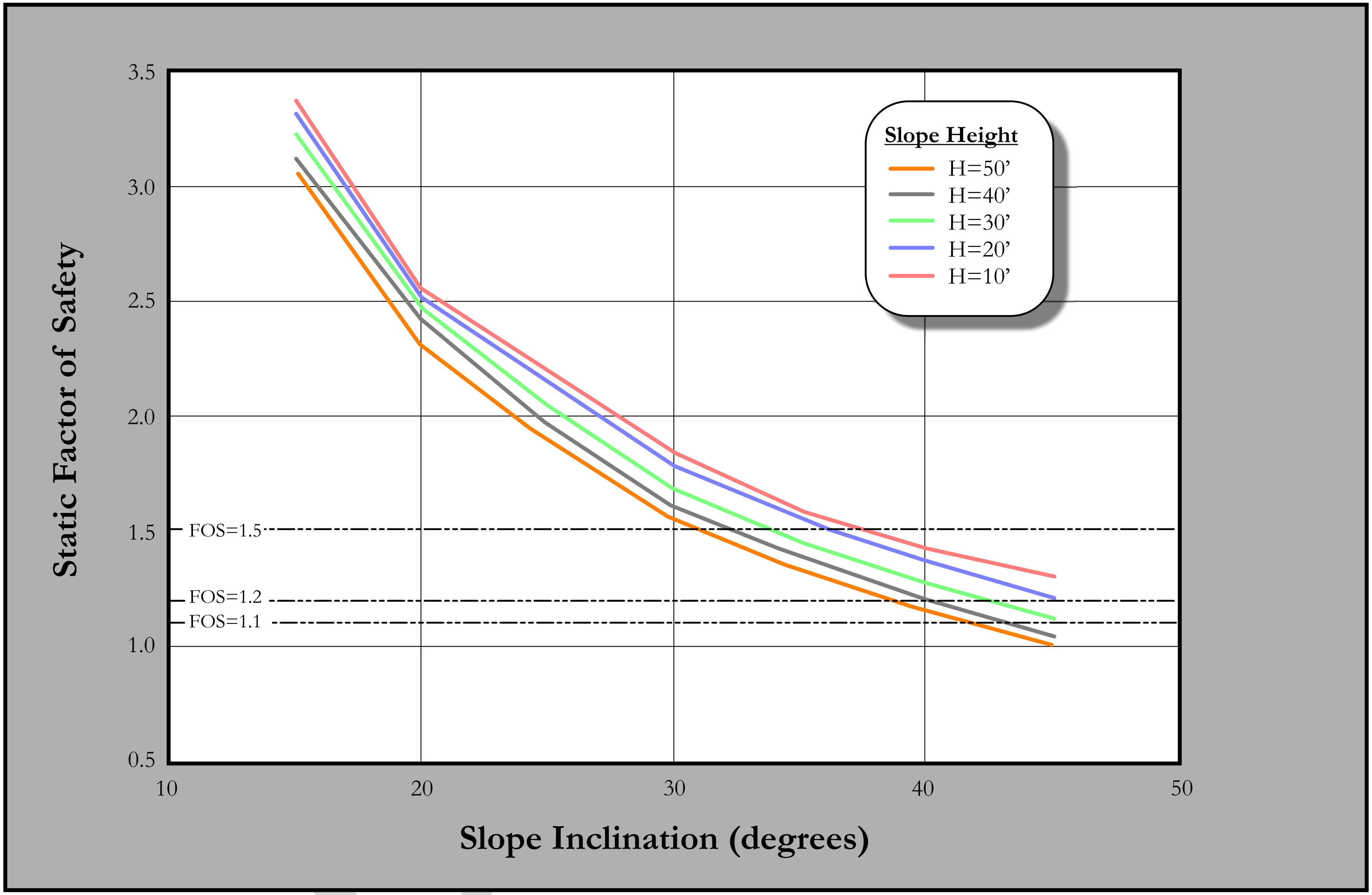
Project no.
160026



**FACTOR OF SAFETY VS.
TEMPORARY SLOPE INCLINATION**

Columbine Station
San Jose Water Company
Water Works Engineers
San Jose, California

Plate No. 13

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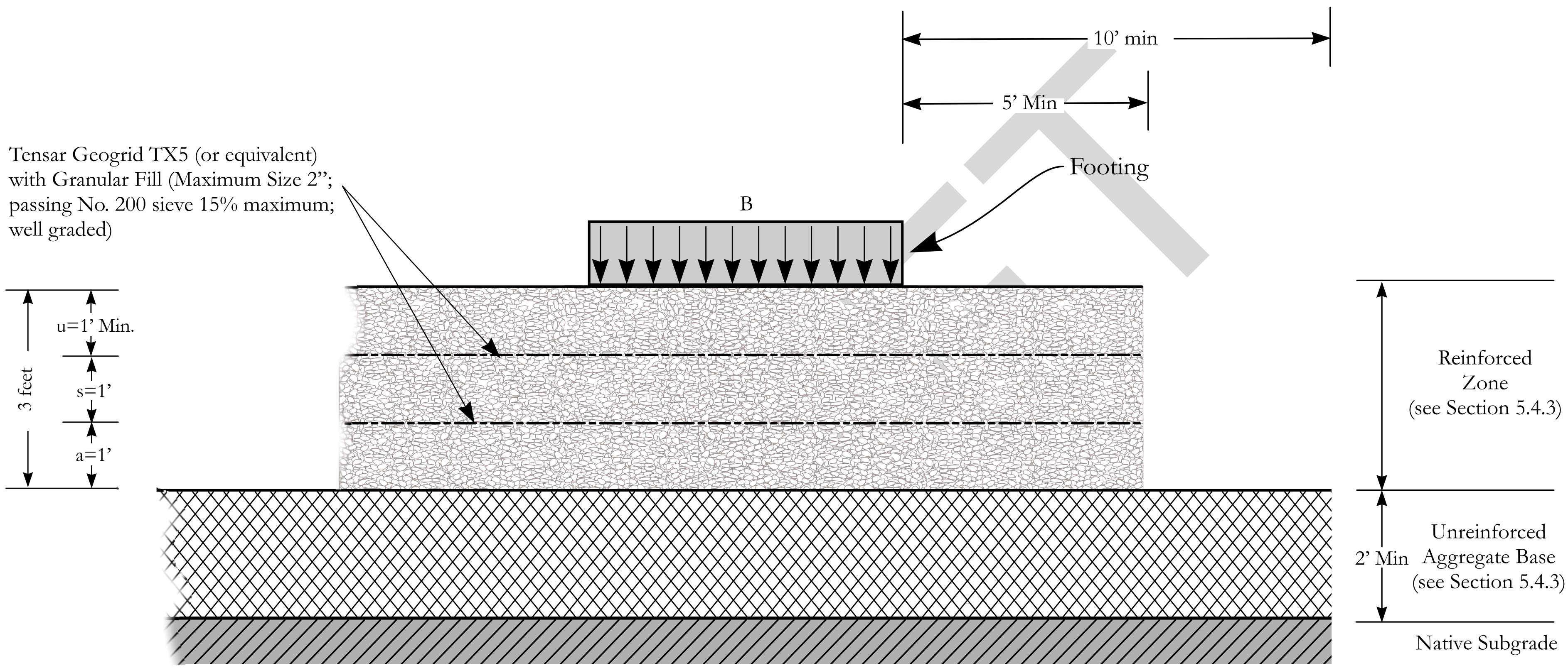
Project no. 160026



Not to Scale

GEOSYNTHETIC RAFT FOUNDATION

Columbine Station
San Jose Water Company
Water Works Engineers
San Jose, California

Plate No. 14

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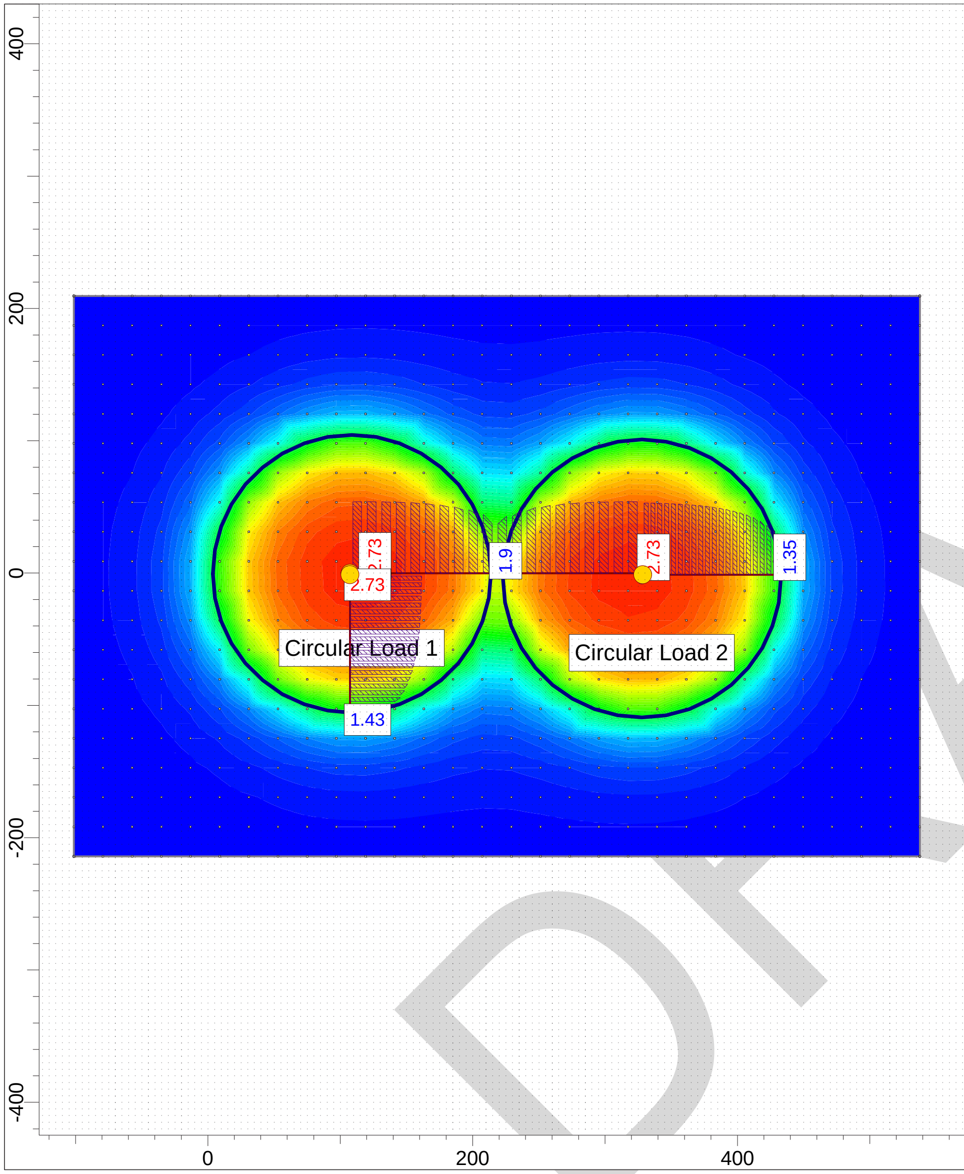

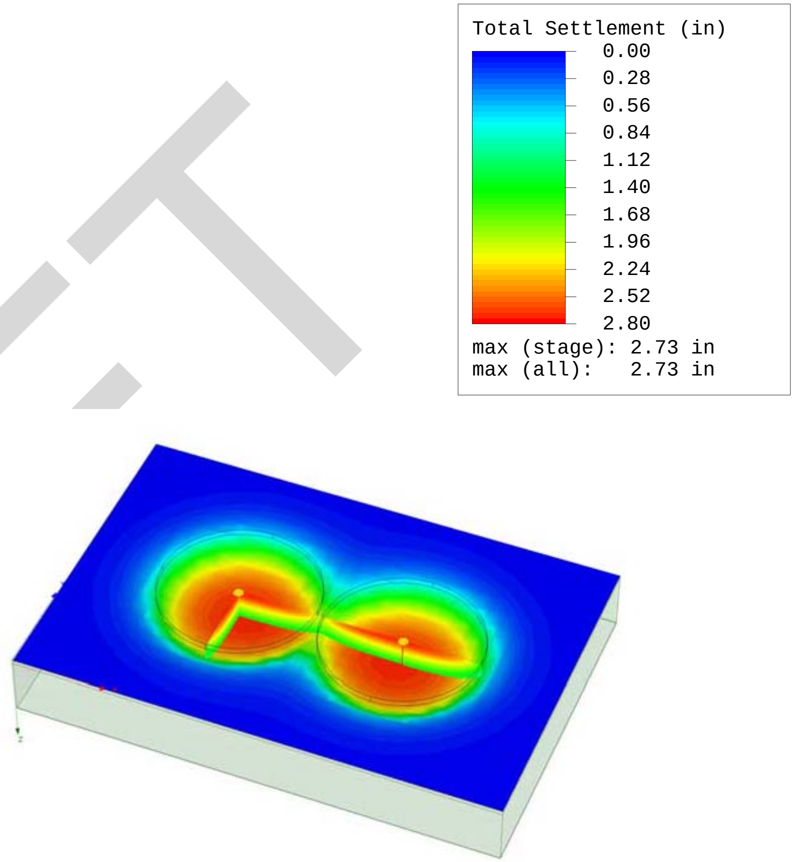

ESTIMATED SETTLEMENT

Columbine Station
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San Jose, California

Plate No.
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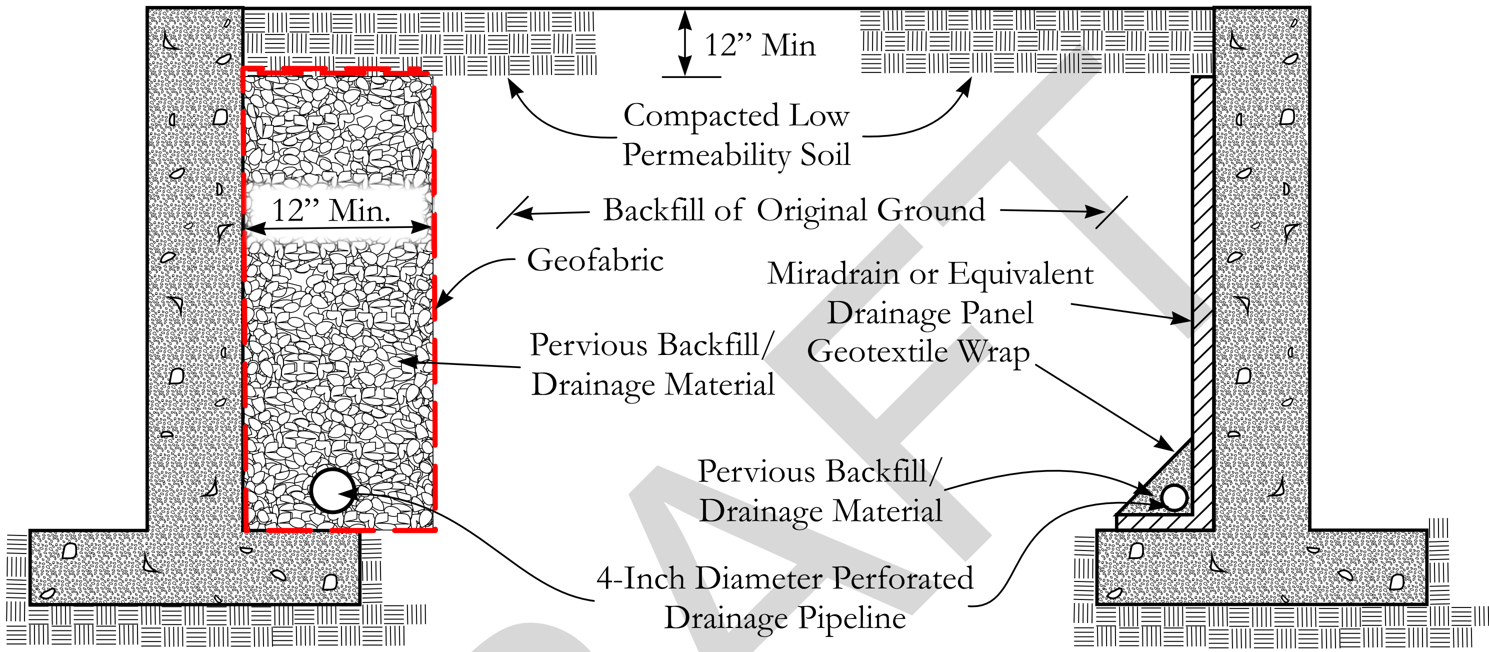
Project no.
160026

Conventional Retaining Wall Drainage Blanket

Geosynthetic Retaining Wall Drainage Panel



General Notes

Pervious backfill/drainage material should conform to Pervious Backfill per Greenbook specifications, Class 2 Permeable
Material per Caltrans Standard Specifications, pea gravel having a nominal 1/4-inch diameter, or crushed stone sized
between 1/4-inch and 1/2-inch.

Geosynthetic wrapping material should conform to Caltrans Standard Specifications Section 88, placed per manufacturer's
specifications.

Performated drain pipe should ocnsist of 4-inch diameter Schedule 40 PVC, with two sets of 1/4-inch (maximum) diameter
performations drilled axially at 90 degrees to each other, with at least one perforation per line spaced at 12 inches, and the
perforations facing downward.

Drainage should be collected in a solid conduit and diverted to a proper, approved drainage facility.

RETAINING WALL DETAILS

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San Jose, California

Plate No.
16

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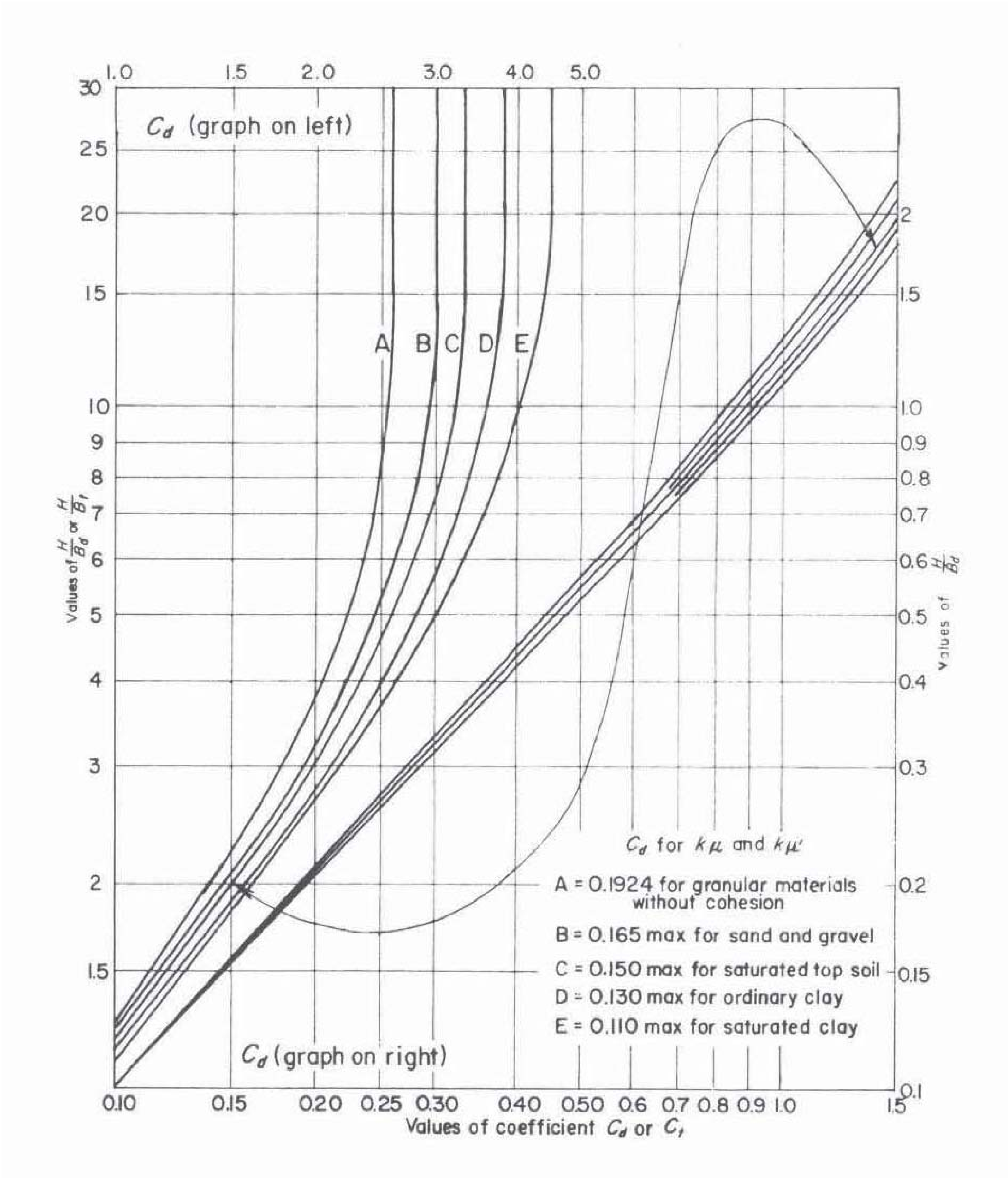
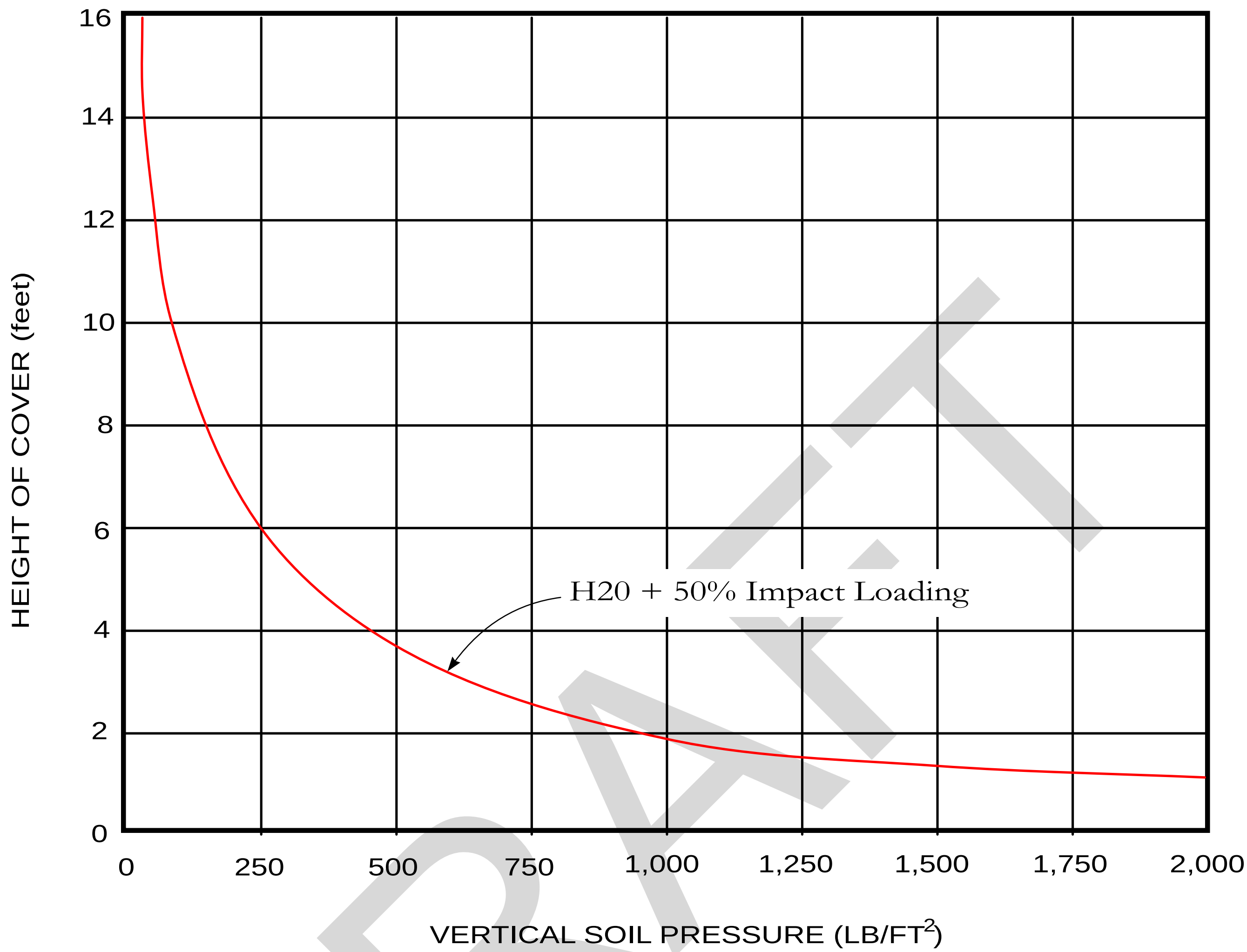


Curve from ASCE (1982)

MARSTON'S LOAD COEFFICIENTS

Columbine Station San Jose Water Company Water Works Engineers San Jose, California	Plate No. 17
VERTICAL SCIENCES, INC.	Project no. 160026



Apply vertical soil pressure to diameter of pipeline (horizontal projection to calculate vertical load

H20 +50% Impact Loading: Simulates a highway load of a 20-ton truck with a 50% impact factor to account for the dynamic effects of traffic

Derived from Moser & Feldman (2008)

VERTICAL SOIL PRESSURES INDUCED BY LIVE LOADS

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San Jose Water Company
Water Works Engineers
San Jose, California

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18

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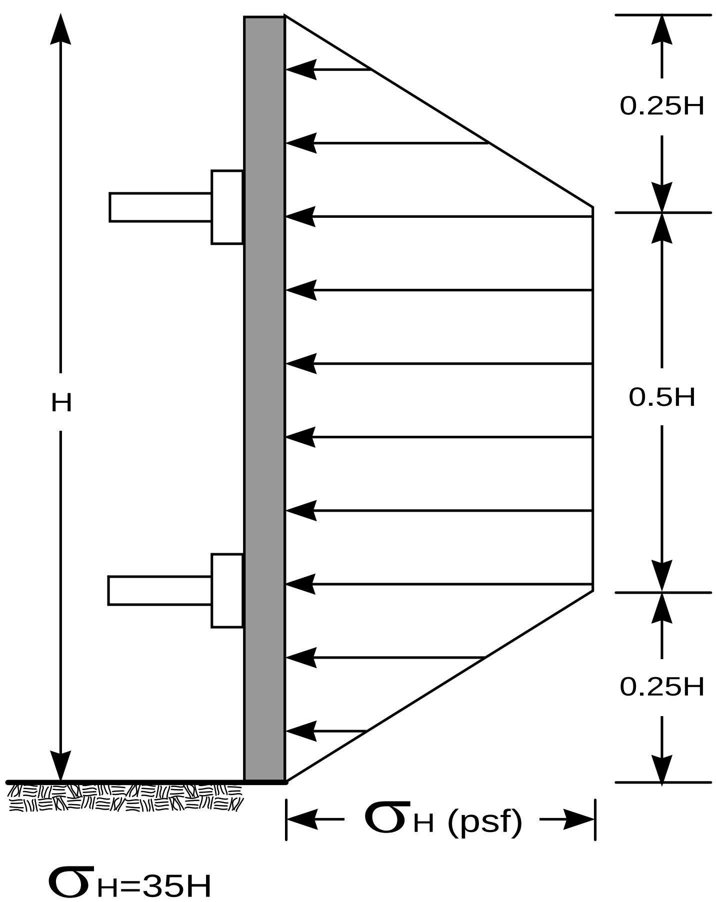
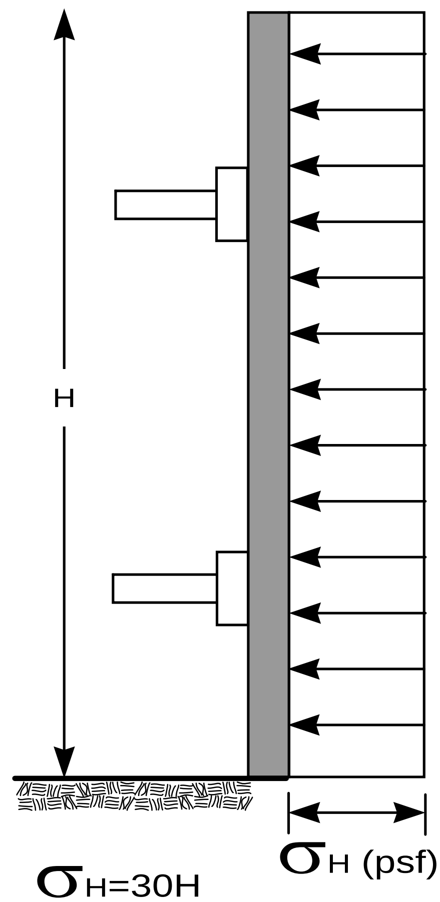
Project no.
160026

CLAY PRESSURE DIAGRAM



SAND PRESSURE DIAGRAM



Preliminary shoring pressure diagrams are for excavations in unsaturated soils only.

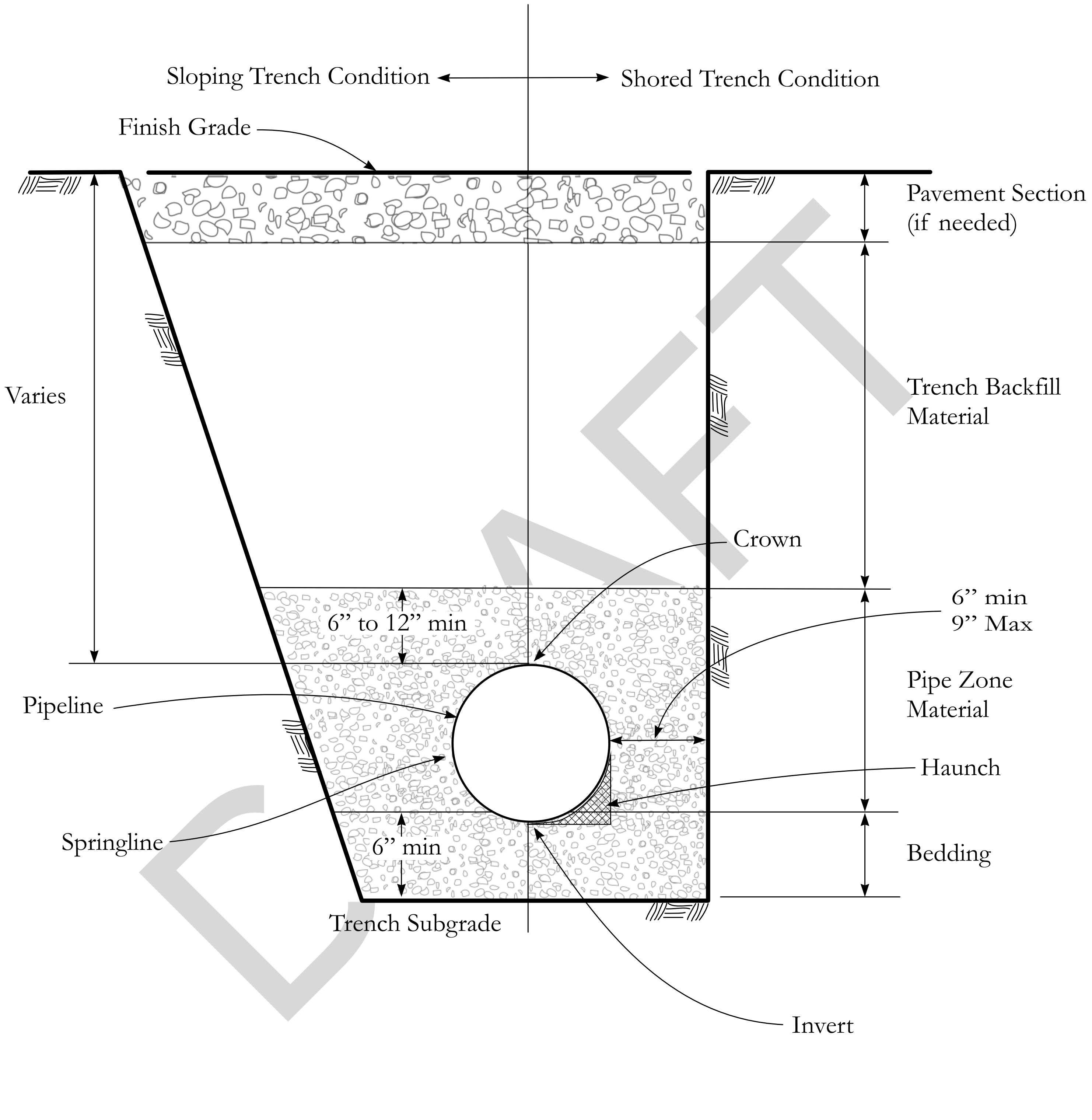
These preliminary shoring pressure diagrams do not take into account hydrostatic pressures nor surcharge pressures. The effects of these conditions must be added to these pressure diagrams where applicable.

Excavation base stability should be analyzed after base width has been selected.

Final design shoring pressure diagrams will need to be developed by the Contractor based on selection of a shoring system and the actual soil, groundwater, and surcharge conditions encountered during construction.

PRELIMINARY SHORING PRESSURE DIAGRAMS

Columbine Station San Jose Water Company Water Works Engineers San Jose, California	Plate No. 19
VERTICAL SCIENCES, INC.	Project no. 160026



Not to Scale

TRENCH NOMENCLATURE

Columbine Station
San Jose Water Company
Water Works Engineers
San Jose, California

Plate No. 20

VERTICAL SCIENCES, INC.

Project no. 160026

APPENDIX A
SUBSURFACE EXPLORATION

The subsurface exploration program for this study consisted of the advancement of ten exploratory drill holes at selected locations at the project site, as on Plate 3. Prior to exploration, drilling permits were obtained, where required, from Santa Clara Valley Water District. The drill holes were advanced on February 18 through February 24, 2016 using a Mobile Drill B-59 drill rig provided by HEW Drilling Company of Palo Alto, California, and April 8 and 9, 2016 using a track-mounted CME-75 drill rig provided by Britton Exploration of Los Gatos, California. The drill holes were advanced using 8.25-inch diameter flight augers.

Select samples of soils were collected from selected depth increments in each drill hole using California modified split-spoon and/or Standard Penetration Test (SPT) samplers. Samplers were driven by a 140-pound hammer situated on the drill rig, in accordance with standard test method ASTM D1586-11 Bulk samples were also obtained at selected depth intervals. Sample types and depths are presented on Plates A-2.1 through A-2.10. All samples were returned to VSI's Redding, California office for assignment of laboratory testing. The results of the testing procedures are attached within Appendix B.

The exploration logs describe the earth materials encountered. The logs also show the location, exploration number, date of exploration, and the names of the logger and equipment used. A VSI geologist, using ASTM 2488 for visual soil classification, logged the explorations. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual and may change with time. The drill holes were backfilled with cement grout. Where asphaltic concrete was disturbed, the holes were patched using quick-set concrete dyed black to match the existing pavement. Soils generated by drilling operations were spoiled at each drill hole location.

The drill hole logs are presented as Plates A-2.1 through A-2.10. A legend to the drill hole logs is presented as Plate A-1.1.

LOG OF EXPLORATION: Expl. No.

PROJECT:	VSI's Project Name	**EXPL. VENDOR:**	Expl. Subcontractor	**SURFACE ELEVATION:**	Expl. Elevation
PROJECT NO.:	VSI's Project No.	**EXPL. METHOD:**	Method of Expl.	**TOTAL DEPTH OF HOLE:**	Total Depth of Expl.
LOCATION:	General Location	**LOGGED BY:**	VSI's Logger	**DEPTH TO WATER:**	Depth to Water
START DATE:	Date Started	**CHECKED BY:**	VSI's Reviewer	**BACKFILLED WITH:**	Backfill Materials
END DATE:	Date Finished	**HAMMER TYPE:**	Type of Sample Hammer		

Depth (ft)	**Material Symbol**	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0			1			SAMPLES/BLOW COUNT SYMBOLS KEY Bulk Soils Sample							
			2	(24)		California modified split spoon sampler (CMSS) Brackets on blow counts indicates CMSS sample							CMSS: 2-3/8" ID, 3" OD, Driven
5			3	50:5"		Standard penetration test (SPT) sample and blow count							SPT: 1-3/8" ID, 2" OD, Driven
						No sample recovery							Blow counts are recorded as the number of blows required for one foot of sampler penetration using a 140-lb hammer falling 30 inches. Typically, sampler is driven 18" and the initial 6" discarded.
						LITHOLOGIC GRAPHICS DESCRIPTIONS FOR SOILS MATERIALS (per ASTM D2487 & D2488)							
10					GW	well graded GRAVEL							
					GP	poorly graded GRAVEL							
					GM	silty GRAVEL							
					GC	clayey GRAVEL							
15					SW	well graded SAND							Initial water level measurement
					SP	poorly graded SAND							Water level after initial measurement (may not represent stabilized water levels)
					SM	silty SAND							
					SC	clayey SAND							
20					ML	low plasticity SILT							
					MH	high plasticity SILT							
					CL	lean CLAY							Lab Abbreviations DS-direct shear; C-consolidation; GS-sieve; EI-Expansion Index; PI-Plasticity; UC-Unconfined; SC-soil chem.; SE-sand equiv.; R-R value; P-curve; PP-pocket penetrometer.
					CH	fat CLAY							
25					PT	organic soils or peat							
					OL	organic SILTS or CLAYS with low plasticity							
					OH	organic SILTS or CLAYS with high plasticity							
30					RX	ROCK							

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

VERTICAL SCIENCES, INC.

PLATE NO.: A-1.1

LOG OF EXPLORATION: DH-1

PROJECT:	Columbine Station	**EXPL. VENDOR:**	HEW Drilling	**SURFACE ELEVATION:**	345 Feet
PROJECT NO.:	160026	**EXPL. METHOD:**	8.25" HSA	**DEPTH OF HOLE:**	33.5 feet
LOCATION:	San Jose, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	July 26, 2016	**CHECKED BY:**	D.Ryan	**BACKFILLED WITH:**	Cement Grout
END DATE:	July 26, 2016	**HAMMER TYPE:**	140-Lb		

Depth (ft)	**Material Symbol**	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0						ASPHALTIC CONCRETE (AC) 4.0" thick.							
					CL/ CH	OLDER ALLUVIUM (Qoal)/ PANOCHE FORMATION (Kpc) Silty CLAY to CLAY, moderate yellowish brown, moist, plastic, with fine to medium sand and subrounded to subangular fine to medium gravel.							
5			1	(50:4")	CL	Sandy CLAY, moderate yellowish brown, moist, hard, slightly plastic, with fine to coarse sand, subrounded fine gravel, and calcium carbonate precipitates.		114.8	14.5				
10			2	(42)	SC	Clayey SAND, moderate yellowish brown, moist and locally wet, dense, slightly plastic, fine to coarse grained, with subangular fine to coarse gravel and possibly cobbles.		112.8	11.3				
15			3	(50:5")	SC	Silty SAND, moderate yellowish brown, moist, very dense, fine to medium grained with subangular fine to coarse gravel.		113.3	6.7				
20			3	(95)									
						At 22 feet: with abundant gravel.							

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

VERTICAL SCIENCES, INC.

PLATE NO.: A-2.1

LOG OF EXPLORATION: DH-1

PROJECT:	Columbine Station	**EXPL. VENDOR:**	HEW Drilling	**SURFACE ELEVATION:**	344 Feet
PROJECT NO.:	160026	**EXPL. METHOD:**	8.25" HSA	**DEPTH OF HOLE:**	33.5 feet
LOCATION:	San Jose, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	July 26, 2016	**CHECKED BY:**	D.Ryan	**BACKFILLED WITH:**	Cement Grout
END DATE:	July 26, 2016	**HAMMER TYPE:**	140-Lb		

Depth (ft)	**Material Symbol**	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
25			4	(50:3")	SP/ GP	Gravelly SAND to Sandy GRAVEL, moderate yellowish brown with some dark brown gravel, dry, very dense, fine to medium grained with subangular to subrounded fine to coarse gravel and possible cobbles. Appears to be saprolitic bedrock.		114.1	8.2				
30			5	61					8.6	20			GS
						At 33 feet: hard drilling conditions.							
			6	50:5"					6.7	16			GS

Bottom of Drill Hole at a Depth of 34 Feet
Practical Refusal Encountered at a Depth of 33.5 Feet

LOG OF EXPLORATION: DH-2

PROJECT: Columbine Station
PROJECT NO.: 160026
LOCATION: San Jose, CA
START DATE: July 25, 2016
END DATE: July 25, 2016

EXPL. VENDOR: HEW Drilling
EXPL. METHOD: 8.25" HSA
LOGGED BY: J.Bianchin
CHECKED BY: D.Ryan
HAMMER TYPE: 140-Lb

SURFACE ELEVATION: 345 Feet
DEPTH OF HOLE: 32 feet
DEPTH TO WATER: Not Encountered
BACKFILLED WITH: Cement Grout

Depth (ft)	**Material Symbol**	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0						ASPHALTIC CONCRETE (AC) 3.0” thick.							
						AGGREGATE BASE (AB), 3.0” thick.							
					CL/ CH	OLDER ALLUVIUM (Qoal)/ PANOCHE FORMATION (Kpc) Silty CLAY to CLAY, moderate to dark brown, moist, plastic, with trace angular sand and minor subrounded to subangular fine to medium gravel. (Saprolitic bedrock)							
5			1	(23)		At 5 feet: stiff		99.6	21.6				
10			2	(42)		At 10 feet: moderate yellowish brown, no sand, trace subrounded fine to medium gravel.			6.4				Torsion ring shear
15			3	(55)	SC	Clayey SAND with Gravel, moderate yellowish brown, moist, very dense, slightly plastic, fine to medium grained, with subangular to subrounded coarse gravel and cobbles.			7.4				
20			4	(50:4”)	SM	Silty SAND with Gravel, moderate yellowish brown, damp, very dense, fine to medium grained with subangular fine to coarse gravel and cobbles.		114.0	6.6				

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

VERTICAL SCIENCES, INC.

PLATE NO.: A-2.2

LOG OF EXPLORATION: DH-2

PROJECT:	Columbine Station	**EXPL. VENDOR:**	HEW Drilling	**SURFACE ELEVATION:**	345 Feet
PROJECT NO.:	160026	**EXPL. METHOD:**	8.25" HSA	**DEPTH OF HOLE:**	32 feet
LOCATION:	San Jose, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	July 25, 2016	**CHECKED BY:**	D.Ryan	**BACKFILLED WITH:**	Cement Grout
END DATE:	July 25, 2016	**HAMMER TYPE:**	140-Lb		

Depth (ft)	**Material Symbol**	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
25			5	54	SM	At 25 feet: dry. At 27 feet: possible clay interbed for 2 feet.							
30			6	50:4"	SM	Silty SAND with Gravel, moderate yellowish brown, damp, very dense, slightly plastic, fine to medium grained with subangular fine to coarse gravel and cobbles, and trace to moderate clay.							
			7	50:0.5"									

Bottom of Drill Hole at a Depth of 32 Feet
Practical Refusal Encountered at a Depth of 32 Feet

LOG OF EXPLORATION: DH-3

PROJECT:	Columbine Station	**EXPL. VENDOR:**	Britton Exploration	**SURFACE ELEVATION:**	345 Feet
PROJECT NO.:	160026	**EXPL. METHOD:**	Dry Coring & HSA	**DEPTH OF HOLE:**	65 feet
LOCATION:	San Jose, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	Aug. 8, 2016	**CHECKED BY:**	D.Ryan	**BACKFILLED WITH:**	Cement Grout
END DATE:	Aug. 8, 2016	**HAMMER TYPE:**	140-Lb		

Depth (ft)	**Material Symbol**	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0						ASPHALTIC CONCRETE (AC) 3.0" thick.							
						AGGREGATE BASE (AB), 3.0" thick.							
		Dry core run 1			SC	LANDSLIDE DEPOSITS (Qls) Clayey SAND, moderate yellowish brown, dry, slightly plastic, fine to medium grained, with fine to coarse subangular gravel and cobbles and few fine roots.							Dry coring
5		Dry core run 2			CL	Silty CLAY, dark brown, dry, hard.							
10		Dry core run 3			CL/ CH	CLAY, moderate brown, moist, hard, slightly plastic.							
					CL	Sandy CLAY, moderate brown mottled tan and white, dry, hard, fine to medium grained with abundant subangular fine to medium gravel.							
15		Dry core run 4			SC	OLDER ALLUVIUM (Qoal)/ PANOCHE FORMATION (Kpc) Clayey SAND, moderate brown, moist, medium stiff, plastic, with fine to medium subangular gravel.							
20					SC	Clayey SAND with Gravel, moderate brown, moist, very dense, fine to medium grained with fine to medium subrounded gravel.							Hollow-stem Auger

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

VERTICAL SCIENCES, INC.

PLATE NO.: A-2.3

LOG OF EXPLORATION: DH-3

PROJECT: Columbine Station
PROJECT NO.: 160026
LOCATION: San Jose, CA
START DATE: Aug. 8, 2016
END DATE: Aug. 8, 2016

EXPL. VENDOR: Britton Exploration
EXPL. METHOD: Dry Coring & HSA
LOGGED BY: J.Bianchin
CHECKED BY: D.Ryan
HAMMER TYPE: 140-Lb

SURFACE ELEVATION: 345 Feet
DEPTH OF HOLE: 65 feet
DEPTH TO WATER: Not Encountered
BACKFILLED WITH: Cement Grout

Depth (ft)	Material Symbol	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
25			1	(56)	SM	Silty SAND with Gravel, moderate yellowish brown, moist, very dense, fine to coarse grained with subangular to subrounded fine to medium gravel.		113.3	11.6				
30			2	(54)					7.2				
35			3	50:6"		At 35 feet: with fine to coarse gravel and possible cobbles.			5.6				
40			4	59					7.1	18			GS
						From 42 to 44 feet: decrease in gravels and cobbles.							
45			5	50:2"	SC	Clayey SAND to Sandy CLAY, moderate brown, moist, very dense/hard, slightly plastic, fine to medium grained. At 46 feet: abundant gravels and cobbles.			9.5		28	6	PI

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

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PLATE NO.: A-2.3

LOG OF EXPLORATION: DH-3

PROJECT: Columbine Station
PROJECT NO.: 160026
LOCATION: San Jose, CA
START DATE: Aug. 8, 2016
END DATE: Aug. 8, 2016

EXPL. VENDOR: Britton Exploration
EXPL. METHOD: Dry Coring & HSA
LOGGED BY: J.Bianchin
CHECKED BY: D.Ryan
HAMMER TYPE: 140-Lb

SURFACE ELEVATION: 345 Feet
DEPTH OF HOLE: 65 feet
DEPTH TO WATER: Not Encountered
BACKFILLED WITH: Cement Grout

Depth (ft)	Material Symbol	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
50			6	50:4"	SM	Silty SAND, moderate brown, dry, very dense, fine to medium grained with trace coarse sand and subangular fine to medium gravel.			2.4	24			GS
55			7	50:2"		At 55 feet: with moderate subangular fine to medium gravel.			5.8				
60			8	50:2"	SC	Clayey SAND, moderate brown mottled white and tan, moist, very dense, slightly plastic, fine to medium grained with subangular fine to medium gravel.			9.2				
65			9	50:0"									

Bottom of Drill Hole at a Depth of 65.2 Feet
Practical Refusal Encountered at a Depth of 65 Feet

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

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LOG OF EXPLORATION: DH-4

PROJECT:	Columbine Station	**EXPL. VENDOR:**	HEW Drilling	**SURFACE ELEVATION:**	345 Feet
PROJECT NO.:	160026	**EXPL. METHOD:**	8.25" HSA	**DEPTH OF HOLE:**	38 feet
LOCATION:	San Jose, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	July 25, 2016	**CHECKED BY:**	D.Ryan	**BACKFILLED WITH:**	Cement Grout
END DATE:	July 25, 2016	**HAMMER TYPE:**	140-Lb		

Depth (ft)	**Material Symbol**	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0						ASPHALTIC CONCRETE (AC) 3.0" thick.							
						AGGREGATE BASE (AB), 3.0" thick.							
			B1		CL	ARTIFICIAL FILL (af) Silty CLAY with Gravel, moderate brown, dry, slightly plastic with subangular to subround fine to medium gravel. At 3 to 4 feet: with cobbles and/or boulders.							Curve, R-Value
5			1	(50:4")									
					CL/ CH	OLDER ALLUVIUM (Qoal)/ PANOCHE FORMATION (Kpc) Silty CLAY to CLAY, moderate brown to reddish brown, moist, stiff to hard, plastic, with trace to moderate subrounded fine to medium gravel.							
10				(50:5.5")									No recovery
15				(50:5")									No recovery. Stiff drilling conditions
20			2	75	CL	CLAY, moderate brown, dry, hard, nonplastic, with trace coarse sand and subrounded fine gravel and trace subrounded coarse gravel.			10.7				

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

VERTICAL SCIENCES, INC.

PLATE NO.: A-2.4

LOG OF EXPLORATION: DH-4

PROJECT:	Columbine Station	**EXPL. VENDOR:**	HEW Drilling	**SURFACE ELEVATION:**	345 Feet
PROJECT NO.:	160026	**EXPL. METHOD:**	8.25" HSA	**DEPTH OF HOLE:**	38 feet
LOCATION:	San Jose, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	July 25, 2016	**CHECKED BY:**	D.Ryan	**BACKFILLED WITH:**	Cement Grout
END DATE:	July 25, 2016	**HAMMER TYPE:**	140-Lb		

Depth (ft)	Material Symbol	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
25			3	57	CL	Sandy CLAY, moderate brown, moist, very stiff, slightly plastic, with fine to coarse sand and trace subrounded fine to medium gravel.			15.2				
30			4	56		At 30 feet: with calcium carbonate precipitation and veining.			11.3		39	22	PI
35			5	(50:3")	GP/ SP	Sandy GRAVEL to Gravelly SAND, moderate brown, dry, very dense, fine to medium grained with abundant subangular fine to medium gravel.							UC=1,774 psf
			6	50:5"		At 38 feet: with sandstone, gravel, cobbles, boulders, and/or bedrock.			11.6				

Bottom of Drill Hole at a Depth of 38.4 Feet
Practical Refusal Encountered at a Depth of 38 Feet

LOG OF EXPLORATION: DH-5

PROJECT: Columbine Station
PROJECT NO.: 160026
LOCATION: San Jose, CA
START DATE: July 27, 2016
END DATE: July 27, 2016

EXPL. VENDOR: HEW Drilling
EXPL. METHOD: 8.25" HSA
LOGGED BY: J.Bianchin
CHECKED BY: D.Ryan
HAMMER TYPE: 140-Lb

SURFACE ELEVATION: 344.5 Feet
DEPTH OF HOLE: 43 feet
DEPTH TO WATER: Not Encountered
BACKFILLED WITH: Cement Grout

Depth (ft)	Material Symbol	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0						ASPHALTIC CONCRETE (AC) 3.0" thick.							
						AGGREGATE BASE (AB), 3.0" thick.							
					CL	ARTIFICIAL FILL (af) Silty CLAY with Gravel, moderate brown, moist, plastic, with subangular fine gravel and trace fine to medium sand.							
5			1	(24)		At 5 feet: medium stiff.		112.8	15.1				
10													
15			2	(20)	CH	CLAY, dark brown to black, moist, medium stiff, plastic, with subangular coarse gravel and cobbles.		103.3	22.5				Appears to contain organic debris.
20			3	(21)	CL/SC	Sandy CLAY to Clayey SAND, moderate brown, moist, medium stiff/dense, fine grained with subangular coarse gravel composed of sandstone.		102.8	19.4				
					CH	CLAY, dark brown, moist, slightly plastic.							

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

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PLATE NO.: A-2.5

LOG OF EXPLORATION: DH-5

PROJECT: Columbine Station
PROJECT NO.: 160026
LOCATION: San Jose, CA
START DATE: July 27, 2016
END DATE: July 27, 2016

EXPL. VENDOR: HEW Drilling
EXPL. METHOD: 8.25" HSA
LOGGED BY: J.Bianchin
CHECKED BY: D.Ryan
HAMMER TYPE: 140-Lb

SURFACE ELEVATION: 344.5 Feet
DEPTH OF HOLE: 43 feet
DEPTH TO WATER: Not Encountered
BACKFILLED WITH: Cement Grout

Depth (ft)	Material Symbol	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
25			4	(45)				100.8	18.6				
					CL	OLDER ALLUVIUM (Qoal)/ PANOCHE FORMATION (Kpc) SANDSTONE (possible cobble), moderate yellowish brown, dry, very dense, fine to medium grained.							
30			5	(50:3")	SM	Silty SAND with Gravel, moderate yellowish brown, dry, very dense, fine to medium grained with abundant subangular to angular fine to medium gravel (possible saprolitic bedrock).							
35			6	52					7.1		26	10	PI
					SC	Sandy CLAY, moderate yellowish brown, moist, very stiff, plastic.							
40			7	81	SM/ SC	Silty SAND, moderate yellowish brown, dry, very dense, fine grained with subangular fine gravel, interbedded with Clayey SAND, moderate brown, moist, very dense, fine to medium grained, slightly plastic, with subangular fine to medium gravel.			8.7				Difficult drilling conditions
			8	50:3"					4.9				

Bottom of Drill Hole at a Depth of 43.25 Feet
Practical Refusal Encountered at a Depth of 43 Feet

LOG OF EXPLORATION: DH-6

PROJECT:	Columbine Station	**EXPL. VENDOR:**	HEW Drilling	**SURFACE ELEVATION:**	344.5 Feet
PROJECT NO.:	160026	**EXPL. METHOD:**	8.25" HSA	**DEPTH OF HOLE:**	44 feet
LOCATION:	San Jose, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	July 27, 2016	**CHECKED BY:**	D.Ryan	**BACKFILLED WITH:**	Cement Grout
END DATE:	July 27, 2016	**HAMMER TYPE:**	140-Lb		

Depth (ft)	**Material Symbol**	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0						ASPHALTIC CONCRETE (AC) 4.0" thick.							
					CL	ARTIFICIAL FILL (af) Silty CLAY with Sand, moderate brown to moderate yellowish brown, moist, plastic, fine to medium grained.							
5													
10			1	(21)		At 10 feet: stiff, with fine to coarse sand and trace to moderate subrounded fine to coarse gravel.							
15													
20			2	(22)									Consol.
					CL	CLAY to Sandy CLAY, dark brown to black, moist, medium stiff, slighly plastic, fine grained with roots and woody debris.							DS

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LOG OF EXPLORATION: DH-6

PROJECT:	Columbine Station	**EXPL. VENDOR:**	HEW Drilling	**SURFACE ELEVATION:**	344.5 Feet
PROJECT NO.:	160026	**EXPL. METHOD:**	8.25" HSA	**DEPTH OF HOLE:**	44 feet
LOCATION:	San Jose, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	July 27, 2016	**CHECKED BY:**	D.Ryan	**BACKFILLED WITH:**	Cement Grout
END DATE:	July 27, 2016	**HAMMER TYPE:**	140-Lb		

Depth (ft)	**Material Symbol**	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
25			3	(44)	CL/ SC	OLDER ALLUVIUM (Qoal)/ PANOCHE FORMATION (Kpc) Sandy CLAY to Clayey SAND, moderate brown, moist, stiff/dense, plastic, fine to medium grained with trace to moderate subrounded to subangular fine to medium gravel.							
30			4	(51)	CL	Silty to Sandy CLAY with Gravel, moderate yellowish brown, damp, hard, slightly plastic, fine grained with subangular to angular fine gravel.							Consol UC=8,369 psf
35			5	(50:4”)	GP/ SP	Sandy GRAVEL to Gravelly SAND, moderate yellowish brown, dry, very dense, fine to medium grained with abundant subangular to subrounded fine to coarse gravel and cobbles.		103.1	16.3				UC=14,996 psf
40			6	(50:3”)									Consol
													Difficult drilling conditions
45			7	33									

Bottom of Drill Hole at a Depth of 45.5 Feet
Practical Refusal Encountered at a Depth of 44 Feet

LOG OF EXPLORATION: DH-7

PROJECT:	Columbine Station	**EXPL. VENDOR:**	Britton Exploration	**SURFACE ELEVATION:**	345 Feet
PROJECT NO.:	160026	**EXPL. METHOD:**	Dry Coring & HSA	**DEPTH OF HOLE:**	65 feet
LOCATION:	San Jose, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	Aug. 9, 2016	**CHECKED BY:**	D.Ryan	**BACKFILLED WITH:**	Cement Grout
END DATE:	Aug. 9, 2016	**HAMMER TYPE:**	140-Lb		

Depth (ft)	**Material Symbol**	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0						ASPHALTIC CONCRETE (AC) 3.0" thick.							
			B1			AGGREGATE BASE (AB), 3.0" thick.							Curve
					CL	ARTIFICIAL FILL (af) Sandy CLAY, moderate brown, moist, plastic, fine to medium grained with trace coarse sand, and local subangular fine to medium gravel.							
5													
10													
15													
20													
					SC	Clayey SAND, moderate yellowish brown, moist, dense, slightly plastic, fine to medium grained, with subangular fine to medium gravel.							

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

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PLATE NO.: A-2.7

LOG OF EXPLORATION: DH-7

PROJECT:	Columbine Station	**EXPL. VENDOR:**	Britton Exploration	**SURFACE ELEVATION:**	345 Feet
PROJECT NO.:	160026	**EXPL. METHOD:**	Dry Coring & HSA	**DEPTH OF HOLE:**	65 feet
LOCATION:	San Jose, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	Aug. 9, 2016	**CHECKED BY:**	D.Ryan	**BACKFILLED WITH:**	Cement Grout
END DATE:	Aug. 9, 2016	**HAMMER TYPE:**	140-Lb		

Depth (ft)	Material Symbol	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
25			1a 1b	(27)	SC	Clayey SAND, moderate yellowish brown, moist, dense, slightly plastic, fine to medium grained, with subangular fine to medium gravel.		108.1	16.0				Soil Chem
30			2	(49)	CL	OLDER ALLUVIUM (Qoal)/ PANOCHE FORMATION (Kpc) Sandy CLAY, moderate brown mottled moderate yellowish brown, moist, stiff to hard, slightly plastic, fine grained with subrounded medium to coarse gravel and local calcium carbonate precipitates and veining.							
35			3	(50:5")		At 35 feet: increasing clay content.							Consol
					SM	Silty SAND with Gravel, moderate brown to moderate yellowish brown, moist, very dense, fine to medium grained, with subrounded coarse gravel. At 37.5 feet: abundant gravel and cobbles.							Hard drilling conditions
40			4	34					10.5				
					SM/ ML	Silty SAND to Sandy SILT with Gravel, moderate yellowish brown, moist, dense, fine grained with subangular fine gravel.							
45			5	21		At 45 feet: abundant gravel and cobbles. .			5.2				Gravel/cobble in sampler nose cone.

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

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PLATE NO.: A-2.7

LOG OF EXPLORATION: DH-7

PROJECT:	Columbine Station	**EXPL. VENDOR:**	Britton Exploration	**SURFACE ELEVATION:**	345 Feet
PROJECT NO.:	160026	**EXPL. METHOD:**	Dry Coring & HSA	**DEPTH OF HOLE:**	65 feet
LOCATION:	San Jose, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	Aug. 9, 2016	**CHECKED BY:**	D.Ryan	**BACKFILLED WITH:**	Cement Grout
END DATE:	Aug. 9, 2016	**HAMMER TYPE:**	140-Lb		

Depth (ft)	**Material Symbol**	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
50			6	36	SM	Silty SAND with Gravel, moderate yellowish brown, dry, dense, fine grained, with subrounded fine to medium gravel. At 52 feet: decrease in gravel content.			5.1				
55			7	50	SC	Clayey SAND, moderate brown, moist, very dense, slightly plastic, fine grained with angular fine gravel.			16.7				
60			8	50:3"	SP/ GP	Gravelly SAND to Sandy GRAVEL, moderate yellowish brown, dry, very dense, fine to medium grained with abundant subangular to subround fine to medium gravel.			6.9				Hard drilling conditions
65			9	50:5"					16.2				

Bottom of Drill Hole at a Depth of 66.5

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

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PLATE NO.: A-2.7

LOG OF EXPLORATION: DH-8

PROJECT:	Columbine Station	**EXPL. VENDOR:**	HEW Drilling	**SURFACE ELEVATION:**	344.5 Feet
PROJECT NO.:	160026	**EXPL. METHOD:**	8.25" HSA	**DEPTH OF HOLE:**	44 feet
LOCATION:	San Jose, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	July 27, 2016	**CHECKED BY:**	D.Ryan	**BACKFILLED WITH:**	Cement Grout
END DATE:	July 27, 2016	**HAMMER TYPE:**	140-Lb		

Depth (ft)	Material Symbol	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0						ASPHALTIC CONCRETE (AC) 4.0" thick.							
					CL	ARTIFICIAL FILL (af) Silty CLAY with Gravel, moderate brown, dry to moist, slightly plastic, with trace fine sand and trace to moderate subangular fine to medium gravel.							
5			1	(14)		At 5 feet: moist, stiff, plastic.		104.5	19.3				
10			2 2a	(10)	CL	CLAY, moderate brown, moist, soft to slightly stiff, plastic.							
15			3	(28)				113.3	14.5				
					CL	Silty CLAY, moderate yellowish brown, moist, dense, slightly plastic, fine to medium grained with local trace subangular medium gravel.							
					CH	CLAY, moderate brown, dry, stiff, nonplastic.							
20			4	(32)				111.8	14.6				
					CL	OLDER ALLUVIUM (Qoal)/ PANOCHE FORMATION (Kpc) Silty CLAY, moderate brown, moist, medium dense, slightly plastic, fine to medium grained with calcium carbonate veining.							

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

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PLATE NO.: A-2.8

LOG OF EXPLORATION: DH-8

PROJECT:	Columbine Station	**EXPL. VENDOR:**	HEW Drilling	**SURFACE ELEVATION:**	344.5 Feet
PROJECT NO.:	160026	**EXPL. METHOD:**	8.25" HSA	**DEPTH OF HOLE:**	44 feet
LOCATION:	San Jose, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	July 27, 2016	**CHECKED BY:**	D.Ryan	**BACKFILLED WITH:**	Cement Grout
END DATE:	July 27, 2016	**HAMMER TYPE:**	140-Lb		

Depth (ft)	**Material Symbol**	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
25			5	(92)	CL/SC	Sandy CLAY to Clayey SAND, moderate brown, moist, very dense/hard, slightly plastic, fine grained with calcium carbonate precipitate veining.		108.9	10.2	43			GS
30			6	(50:5")	CL	Silty to Sandy CLAY with Gravel, moderate yellowish brown, damp, hard, slightly plastic, fine grained with subangular to angular fine gravel.							Consol
35			7	(50:5")	GP/SP	Sandy GRAVEL to Gravelly SAND, moderate yellowish brown with dark brown gravel, dry, very dense, fine to medium grained, with subangular to subrounded fine to coarse gravel and cobbles, and local calcium carbonate precipitates.		106.8	7.5				Triaxial shear Difficult drilling conditions
40			8	43		At 40 feet: with trace to moderate clay, slightly plastic.			16.2				
45			9	83		At 44 feet: with little to no clay.			7.7	17			GS

Bottom of Drill Hole at a Depth of 45.5 Feet
Practical Refusal Encountered at a Depth of 44 Feet

LOG OF EXPLORATION: DH-9

PROJECT: Columbine Station
PROJECT NO.: 160026
LOCATION: San Jose, CA
START DATE: July 26, 2016
END DATE: July 26, 2016

EXPL. VENDOR: HEW Drilling
EXPL. METHOD: 8.25" HSA
LOGGED BY: J.Bianchin
CHECKED BY: D.Ryan
HAMMER TYPE: 140-Lb

SURFACE ELEVATION: 345 Feet
DEPTH OF HOLE: 26 feet
DEPTH TO WATER: Not Encountered
BACKFILLED WITH: Cement Grout

Depth (ft)	Material Symbol	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0						ASPHALTIC CONCRETE (AC) 2.0” thick.							
						AGGREGATE BASE (AB) 3.0” thick							
					CL	ARTIFICIAL FILL (af) Silty CLAY with Gravel, moderate brown to dark brown, moist, slightly plastic, with trace to moderate subangular fine to coarse gravel.							
5			1	(63)	CL	Sandy CLAY, moderate brown, moist, very stiff, slightly plastic, fine to medium grained with trace to few subangular fine gravel and trace coarse sand. At 8 feet: increased gravel.		117.7	12.3				
10			2	(44)	SM	Silty SAND with Gravel, moderate brown, dry, medium dense to dense, fine to medium grained with subangular fine to medium gravel and trace local clay (saprolitic bedrock?).		103.2	15.8				
15			3	(34)	SM	OLDER ALLUVIUM (Qoal)/ PANOCHE FORMATION (Kpc) Silty SAND, moderate brown, moist, medium dense, fine to medium grained with local interbedded subangular fine to medium gravel with calcium carbonate precipitates.							
20			4	(50:4”)	SP	Gravelly SAND, moderate yellowish brown, dry to moist, very dense, fine to medium grained, with subrounded fine to coarse gravel. From 21 to 23 feet: abundant gravel, cobbles, and possibly boulders.		116.9	6.2	39			GS UC=2,522 psf

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

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PLATE NO.: A-2.9

LOG OF EXPLORATION: DH-9

PROJECT:	Columbine Station	**EXPL. VENDOR:**	HEW Drilling	**SURFACE ELEVATION:**	344.5 Feet
PROJECT NO.:	160026	**EXPL. METHOD:**	8.25" HSA	**DEPTH OF HOLE:**	26 feet
LOCATION:	San Jose, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	July 26, 2016	**CHECKED BY:**	D.Ryan	**BACKFILLED WITH:**	Cement Grout
END DATE:	July 26, 2016	**HAMMER TYPE:**	140-Lb		

Depth (ft)	**Material Symbol**	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
25			5	94					6.0				

Bottom of Drill Hole at a Depth of 26.5 Feet
Practical Refusal Encountered at a Depth of 25 Feet

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

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PLATE NO.: A-2.9

LOG OF EXPLORATION: DH-10

PROJECT:	Columbine Station	**EXPL. VENDOR:**	HEW Drilling	**SURFACE ELEVATION:**	345 Feet
PROJECT NO.:	160026	**EXPL. METHOD:**	8.25" HSA	**DEPTH OF HOLE:**	18 feet
LOCATION:	San Jose, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	July 26, 2016	**CHECKED BY:**	D.Ryan	**BACKFILLED WITH:**	Cement Grout
END DATE:	July 26, 2016	**HAMMER TYPE:**	140-Lb		

Depth (ft)	Material Symbol	Sample	Sample No.	Blow Count (blows/ft)	USCS Symbol	Material Description	Water Table	Unit Dry Weight, pcf	Water Content, %	% Passing No. 200	Liquid Limit	Plasticity Index	Notes & Assigned Laboratory
0						ASPHALTIC CONCRETE (AC) 4.0" thick.							
					CL	OLDER ALLUVIUM (Qoal)/ PINOCHE FORMATION (Kpc) Silty CLAY to CLAY, moderate brown, moist, slightly plastic, with trace fine to medium sand and trace to few subangular fine gravel.							Difficult drilling conditions
5			1	42	CL	Clayey SAND to Sandy CLAY with Gravel, moderate yellowish brown to moderate brown, dry to moist, dense/stiff, fine to medium grained with subangular fine to medium gravel.			9.0				
10			2	79	SM	Silty SAND with Gravel, moderate yellowish brown, dry, very dense, fine to medium grained with abundant subangular fine to coarse gravel (saprolitic bedrock?).			5.2				
15			3	50:5"					5.6				Very hard drilling
			4	50:5"					3.3				

Bottom of Drill Hole at a Depth of 18.4 Feet
Practical Refusal Encountered at a Depth of 18 Feet

The log and data presented are a simplification of actual conditions encountered at the given location and time of exploration. Subsurface conditions may differ at other locations and with the passage of time.

VERTICAL SCIENCES, INC.

PLATE NO.: A-2.10

APPENDIX B
LABORATORY TESTING

Laboratory Analyses

Laboratory tests were performed on selected bulk soil samples to estimate engineering characteristics of the various earth materials encountered. Testing was performed under procedures described in one of the following references:

- ASTM Standards for Soil Testing, latest revision;
- Lambe, T. William, Soil Testing for Engineers, Wiley, New York, 1951;
- Laboratory Soils Testing, U.S. Army, Office of the Chief of Engineers, Engineering Manual No. 1110-2-1906, November 30, 1970.

In Situ Moisture Density Relations

Dry density estimates and/or moisture content evaluations were performed on selected soil samples collected during this study. Tests were performed using standard test methods ASTM D2216 for moisture content or ASTM D2937 for dry unit weights. The results are presented on the respective Log of Drill holes.

Grain Size Distribution

Grain size distribution was determined for seven selected soil samples in accordance with standard test method ASTM D422. The grain size distribution data are shown on the attached plate labeled *Particle Size Distribution.*

Plasticity Index Tests

Atterberg Limits (plastic limit, liquid limit, and plasticity index) tests were performed on six selected samples in accordance with standard test method ASTM D4318. The results of the tests are presented on the drill hole logs and on attached plates labeled *Plasticity Chart and Data*.

Consolidation

Five consolidation tests were performed on selected relatively undisturbed samples using standard test method ASTM D2435. The results of the tests are presented on attached plate labeled *Consolidation Test*.

Direct Shear Tests

Consolidated-drained direct shear testing was performed on one selected sample obtained during this study. The testing was performed in accordance with standard test method ASTM D3080. The results of the tests are presented on the attached plate labeled *Consolidated Drained Direct Shear Test.*

Torsion Ring Shear tests

Peak and residual torsion ring shear testing was performed on one selected sample obtained during this study. The testing was performed in accordance with standard test method ASTM D6467 and 7608. The results of the tests are presented on the attached plates labeled *Drained Residual Torsional Shear Test* and

Drained Fully Softened Peak Torsional Shear Test.

Triaxial Shear Tests

Unconsolidated-Undrained triaxial testing was performed on one selected soil sample in accordance with standard test method ASTM D2850. Results of the test are presented on the attached plate labeled *Unconsolidated-Undrained Triaxial Test.*

Maximum Density/Optimum Moisture

Maximum density and optimum moisture testing was performed on two selected samples. Testing was performed in accordance with standard test method ASTM D1556. Results of the testing are presented on the attached plates labeled *Compaction Test Report*.

R-Value

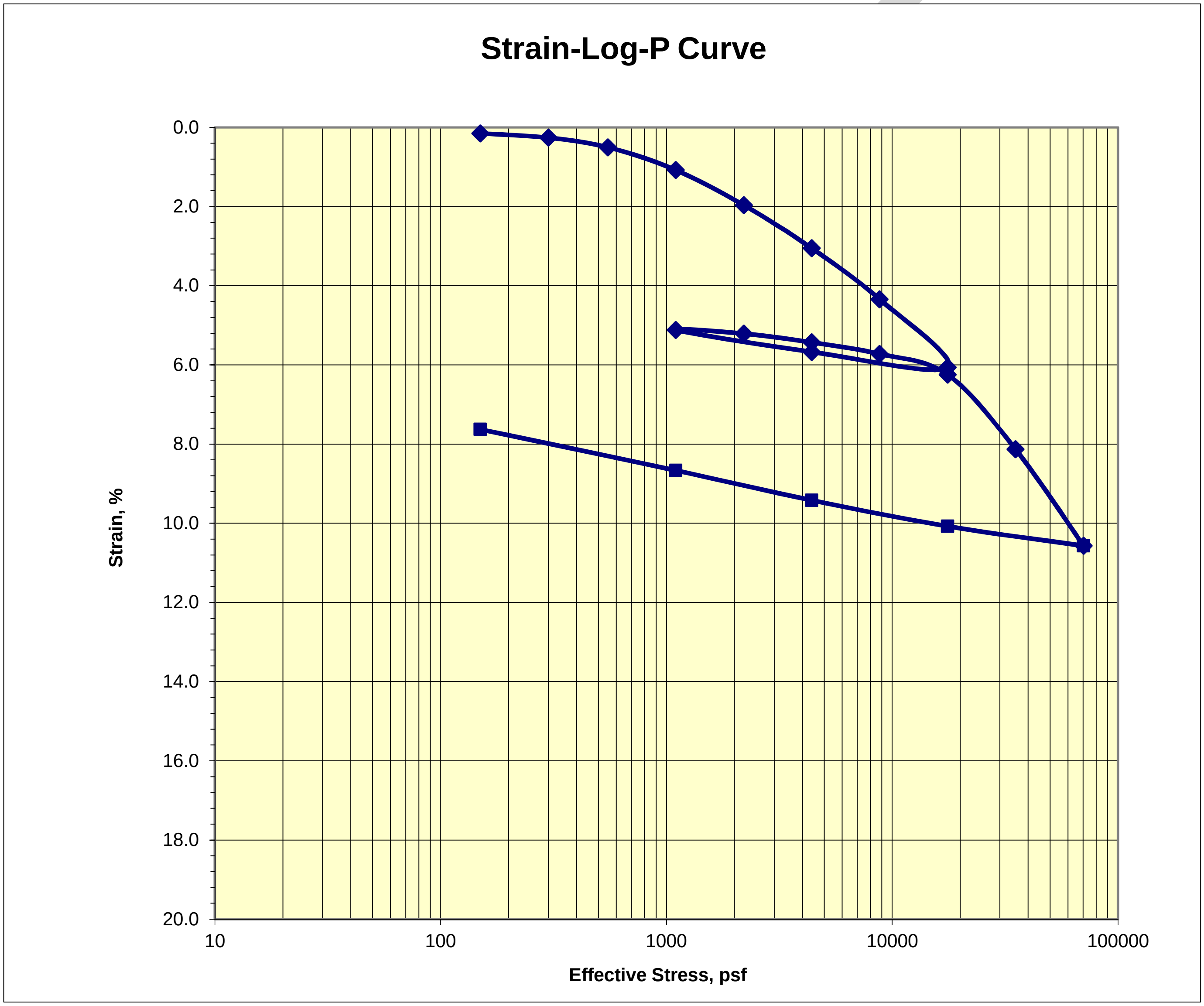
R-value testing was performed on one selected sample in accordance with standard test method ASTM D2844. Results of the testing are presented on the attached Plate labeled *"R" Value Test Report.*

Soil-Chemistry for Corrosion

One test was performed on a selected soil samples to evaluate pH, resistivity, chloride and sulfate contents, along with other cations and anions. The results of the tests are presented on the attached *Soil Chemistry* sheets.

Consolidation Test
ASTM D2435

Job No.:	968-001	**Boring:**	DH-6	**Run By:**	MD
Client:	Vertical Sciences, Inc.	**Sample:**		**Reduced:**	PJ
Project:	Columbine Station - 160026	**Depth, ft.:**	30.5	**Checked:**	PJ/DC
Soil Type:	Yellowish Bown Clayey SAND			**Date:**	9/6/2016

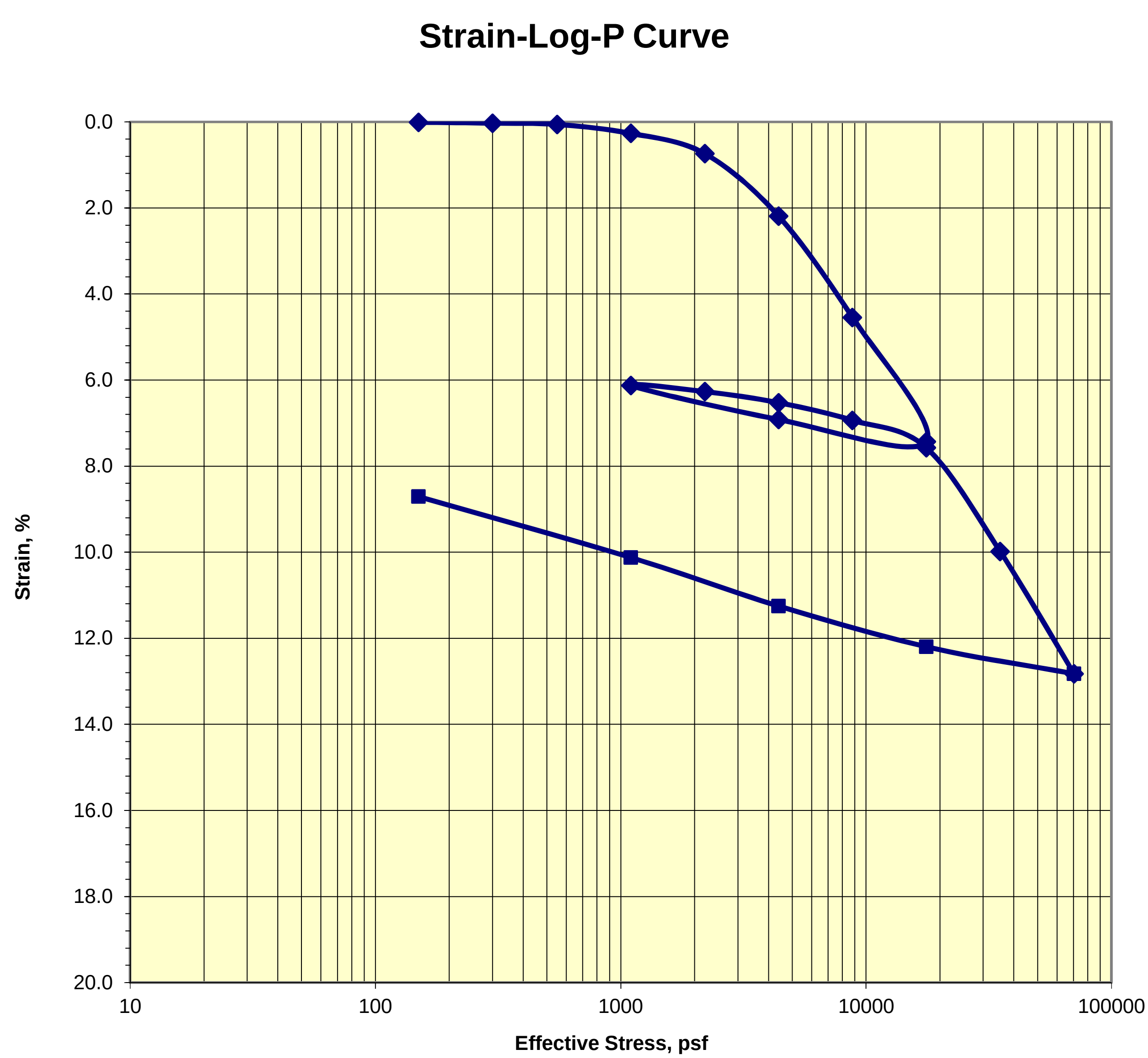


Assumed Gs 2.7	Initial	Final
Moisture %:	14.8	12.8
Dry Density, pcf:	116.1	125.2
Void Ratio:	0.451	0.347
% Saturation:	88.5	100.0

Remarks: Rebound-reload loop performed with 1 point on the virgin curve per clients instructions.

Consolidation Test
ASTM D2435

Job No.:	968-001	**Boring:**	DH-6	**Run By:**	MD
Client:	Vertical Sciences, Inc.	**Sample:**		**Reduced:**	PJ
Project:	Columbine Station - 160026	**Depth, ft.:**	40	**Checked:**	PJ/DC
Soil Type:	Olive Brown Sandy CLAY/ Clayey SAND, trace Gravel			**Date:**	9/2/2016

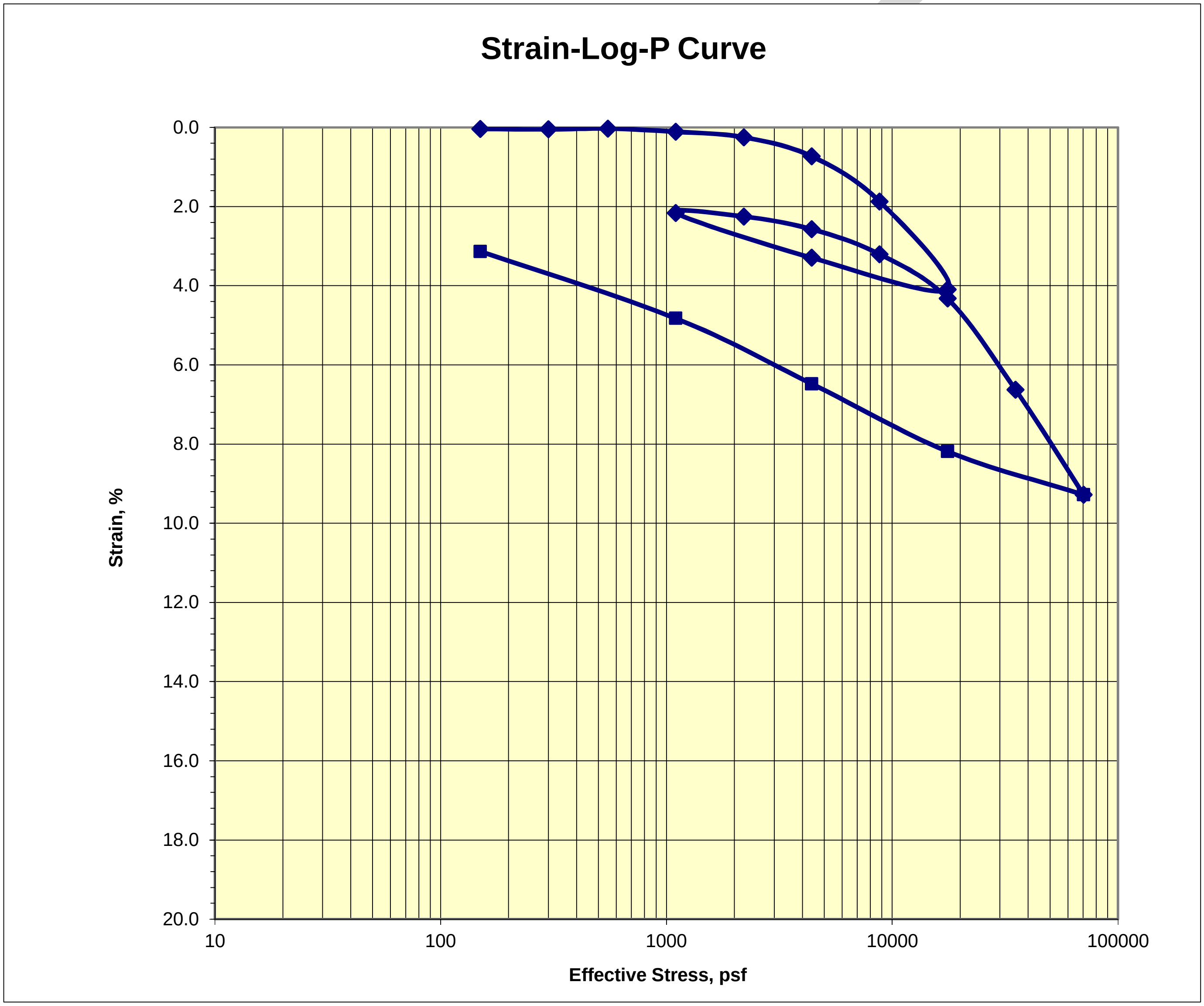


Assumed Gs 2.75	Initial	Final
Moisture %:	13.3	13.6
Dry Density, pcf:	113.8	124.8
Void Ratio:	0.508	0.375
% Saturation:	71.8	100.0

Remarks: Rebound-reload loop performed with 1 point on the virgin curve per clients instructions.

Consolidation Test
ASTM D2435

Job No.:	968-001	**Boring:**	DH-7	**Run By:**	MD
Client:	Vertical Sciences, Inc.	**Sample:**		**Reduced:**	PJ
Project:	Columbine Station - 160026	**Depth, ft.:**	30	**Checked:**	PJ/DC
Soil Type:	Olive Brown Sandy CLAY w/ Gravel			**Date:**	9/1/2016



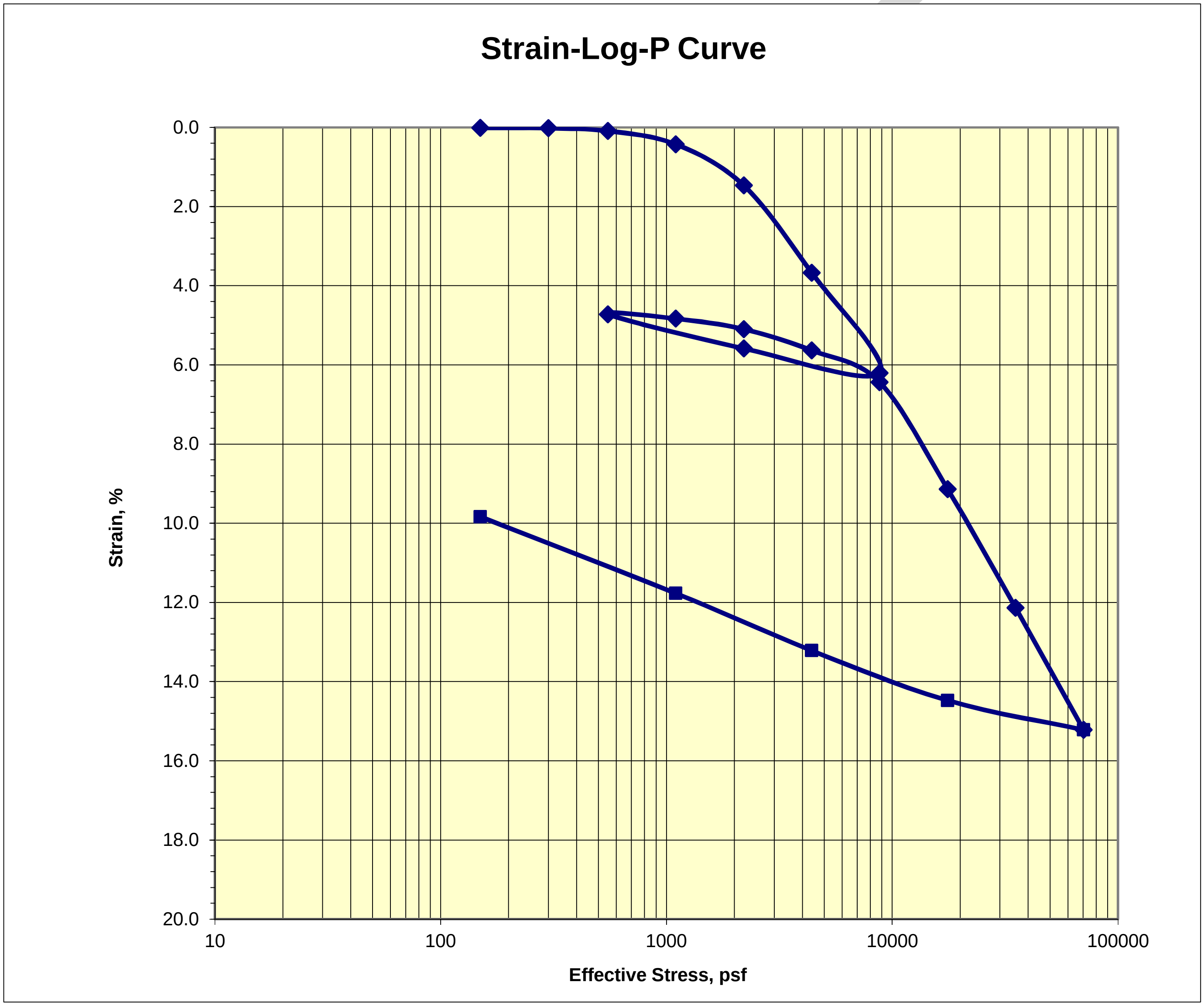
Assumed Gs 2.7	Initial	Final
Moisture %:	14.4	15.7
Dry Density, pcf:	115.1	118.4
Void Ratio:	0.465	0.424
% Saturation:	83.6	100.0

Remarks: Rebound-reload loop performed with 1 point on the virgin curve per clients instructions.

COOPER TESTING LABORATORY

Consolidation Test
ASTM D2435

Job No.:	968-001	**Boring:**	DH-7	**Run By:**	MD
Client:	Vertical Sciences, Inc.	**Sample:**		**Reduced:**	PJ
Project:	Columbine Station - 160026	**Depth, ft.:**	35	**Checked:**	PJ/DC
Soil Type:	Yellowish Brown Clayey SAND			**Date:**	9/6/2016



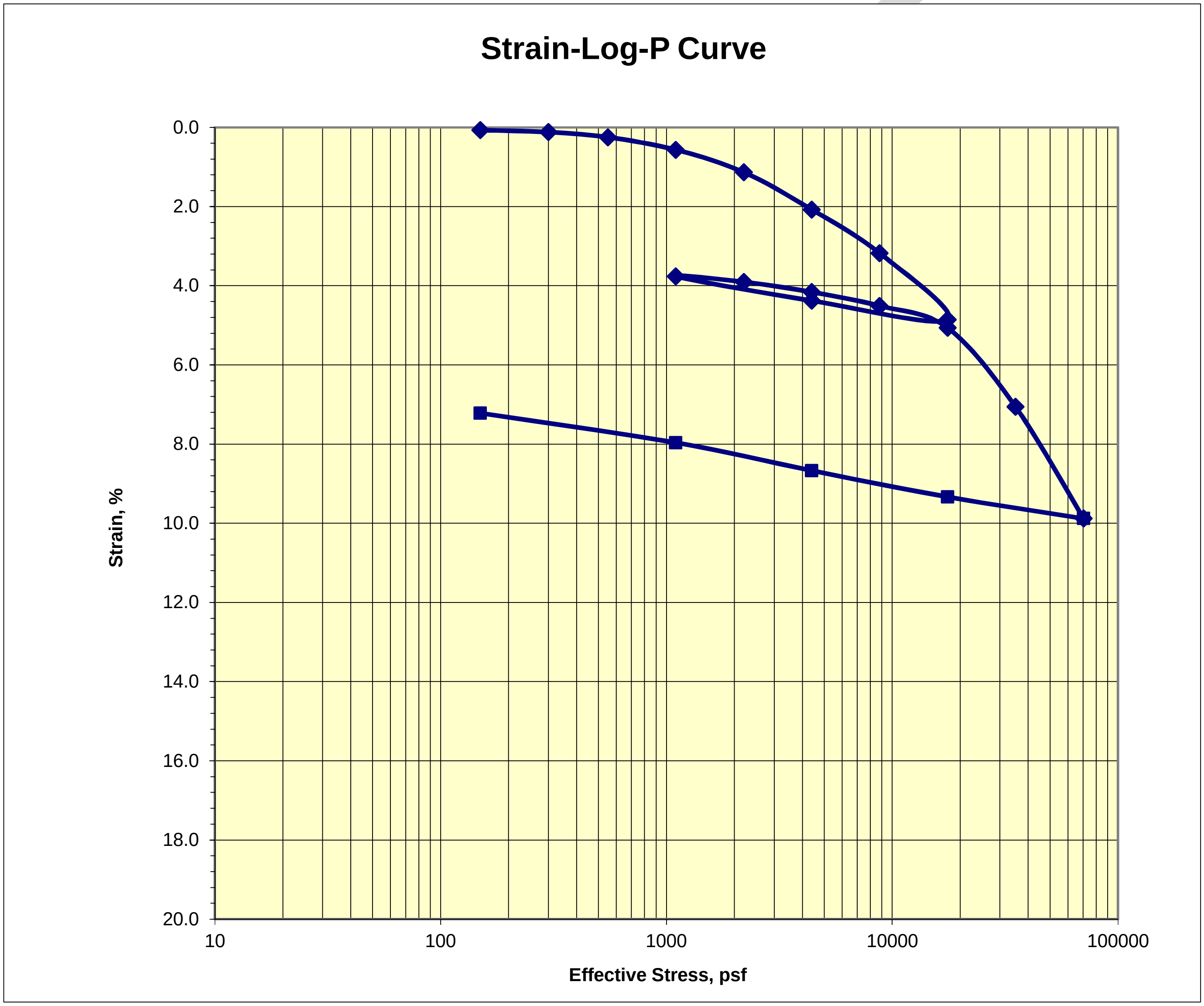
Assumed Gs 2.7	**Initial**	**Final**
Moisture %:	18.8	17.5
Dry Density, pcf:	103.7	114.6
Void Ratio:	0.625	0.471
% Saturation:	81.0	100.0

Remarks: Rebound-reload loop performed with 1 point on the virgin curve per clients instructions.

Consolidation Test

ASTM D2435

Job No.:	968-001	**Boring:**	DH-8	**Run By:**	MD
Client:	Vertical Sciences, Inc.	**Sample:**		**Reduced:**	PJ
Project:	Columbine Station - 160026	**Depth, ft.:**	30	**Checked:**	PJ/DC
Soil Type:	Olive Brown Silty SAND (slightly plastic)			**Date:**	9/6/2016



Assumed Gs 2.65	Initial	Final
Moisture %:	13.5	14.4
Dry Density, pcf:	110.4	119.8
Void Ratio:	0.498	0.381
% Saturation:	72.0	100.0

Remarks: Rebound-reload loop performed with 1 point on the virgin curve per clients instructions.

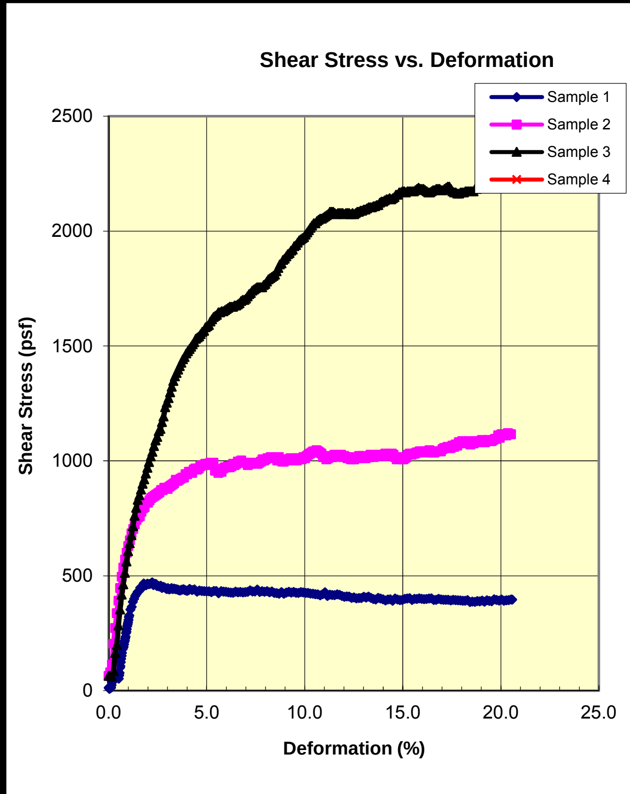
Consolidated Drained Direct Shear
(ASTM D3080)

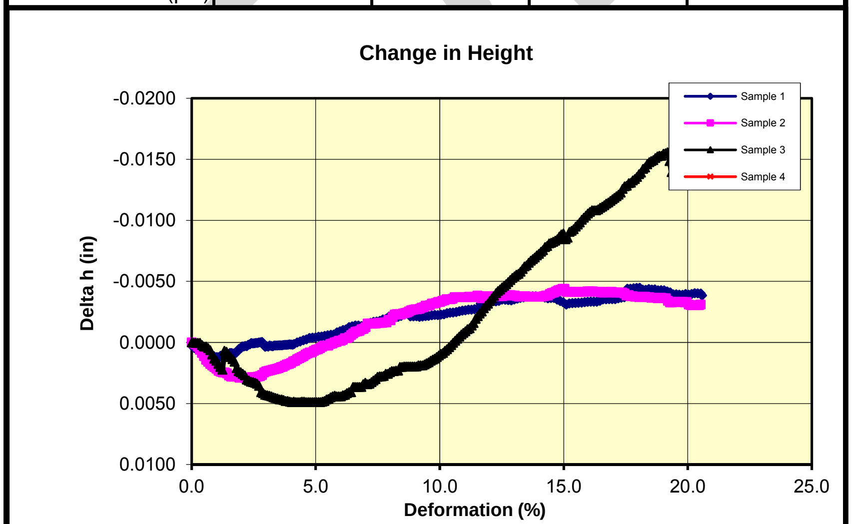
CTL Job #:	968-001	Project #:	160026	By:	MD
Client:	Vertical Sciences, Inc.	Date:	8/23/2016	Checked:	PJ
Project Name:	Columbine Station	Remolding Info:			

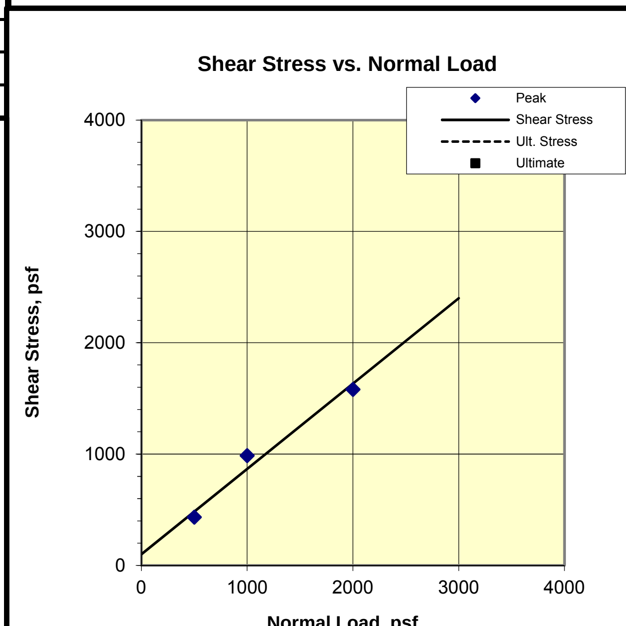
Phi (deg)	37.5	Ult. Phi (deg)	
Cohesion (psf)	100	Ult. Cohesion (psf)	

Specimen Data

	1	2	3	4
Boring:	DH-6	DH-6	DH-6	
Sample:	2	2	2	
Depth (ft):	20	20	20	
Visual Description:	Yellowish Brown Mottled Dark Gray Sandy CLAY w/ Gravel	Yellowish Brown Mottled Dark Gray Sandy CLAY w/ Gravel	Yellowish Brown Mottled Dark Gray Sandy CLAY w/ Gravel	
Normal Load (psf)	500	1000	2000	
Dry Mass of Specimen (g)	123.5	125.3	127.6	
Initial Height (in)	1.01	1.01	1.01	
Initial Diameter (in)	2.43	2.43	2.43	
Initial Void Ratio	0.739	0.711	0.691	
Initial Moisture (%)	24.5	21.3	22.3	
Initial Wet Density (pcf)	125.1	123.9	126.5	
Initial Dry Density (pcf)	100.5	102.1	103.4	
Initial Saturation (%)	92.7	84.0	90.5	
ΔHeight Consol (in)	0.0053	0.0183	0.0286	
At Test Void Ratio	0.730	0.680	0.643	
At Test Moisture (%)	25.3	21.7	22.2	
At Test Wet Density (pcf)	126.7	126.7	130.1	
At Test Dry Density (pcf)	101.1	104.1	106.5	
At Test Saturation (%)	97.0	89.2	96.8	
Strain Rate (%/min)	0.004	0.004	0.004	
Strengths Picked at	5%	5%	5%	
Shear Stress (psf)	433	987	1581	
ΔHeight (in) at 5%	-0.0004	0.0007	0.0049	
Ultimate Stress (psf)				







Remarks: Gravel in or near shear plane on all 3 samples may influence results. Major patching due to Gravel on all 3 points.

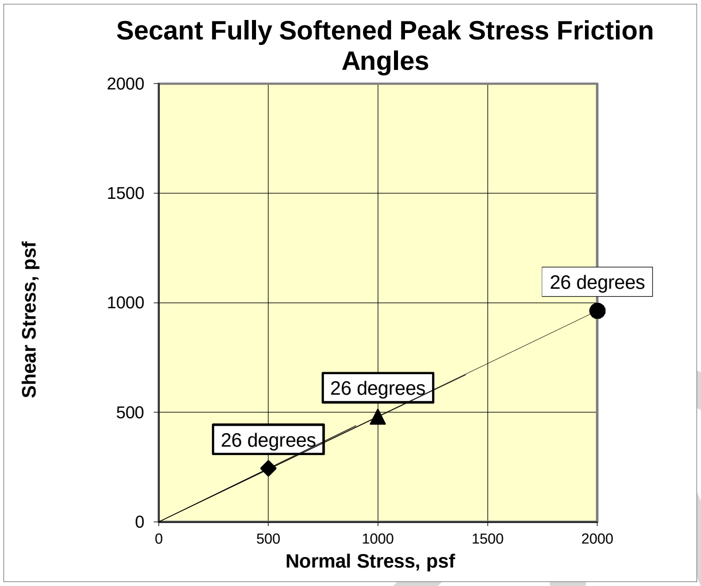
Drained Fully Softened Peak Torsional Shear Strength (ASTM D7608)

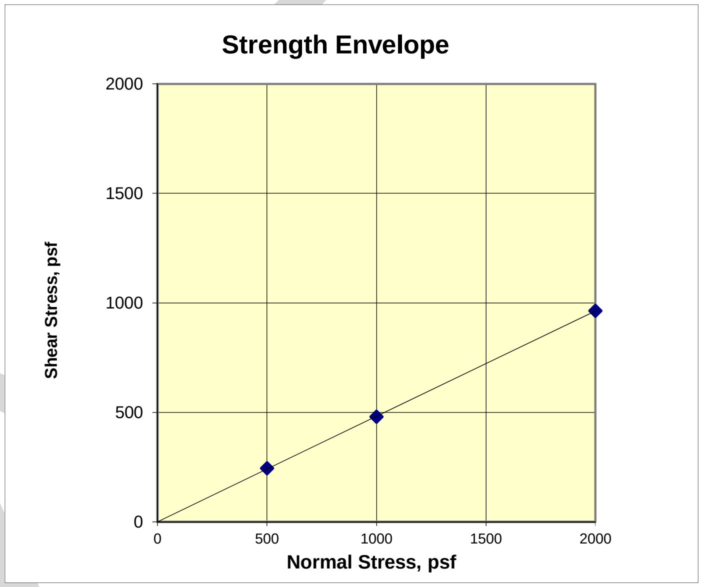
CTL Job No.:	968-001	**Boring:**	DH-2	**Date:**	8/29/2016	**Clay, %:**	
Client:	Vertical Sciences, Inc.	**Sample:**		**By:**	PJ	**LL:**	
Project Name:	Columbine Station	**Depth (ft):**	10	**Checked:**	DC	**PL:**	
Project Number:	160026	**Test Type:**	Fully Softened Peak				

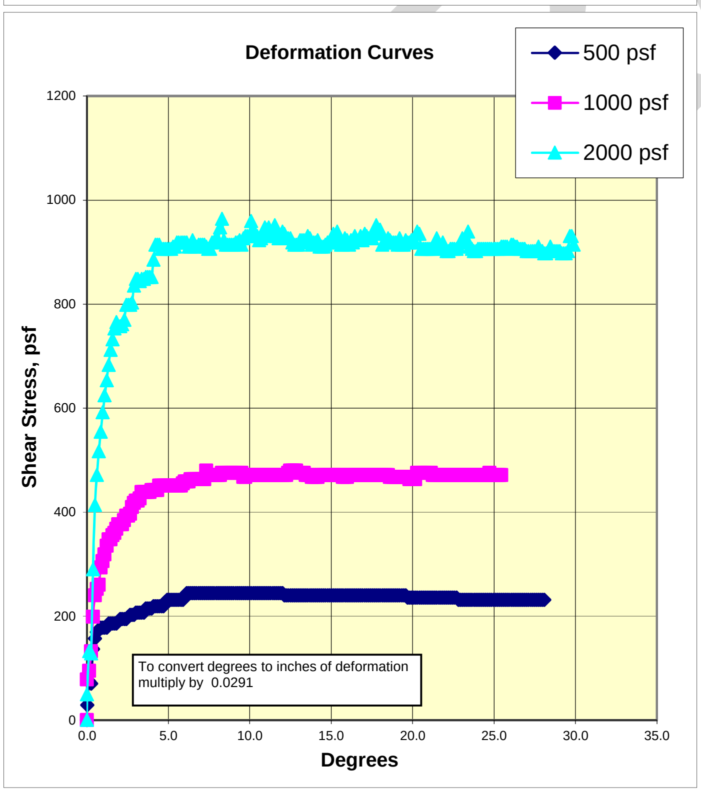
Soil Type: Olive Brown Clayey SAND

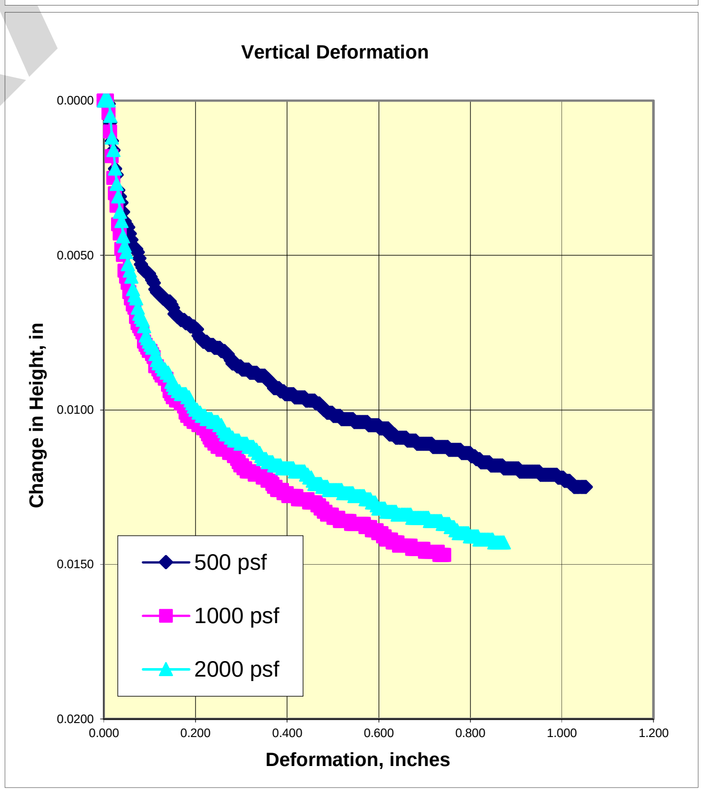
Remarks:

Normal Stress, psf:	500	1000	2000
Secant Phi, deg.:	26	26	26









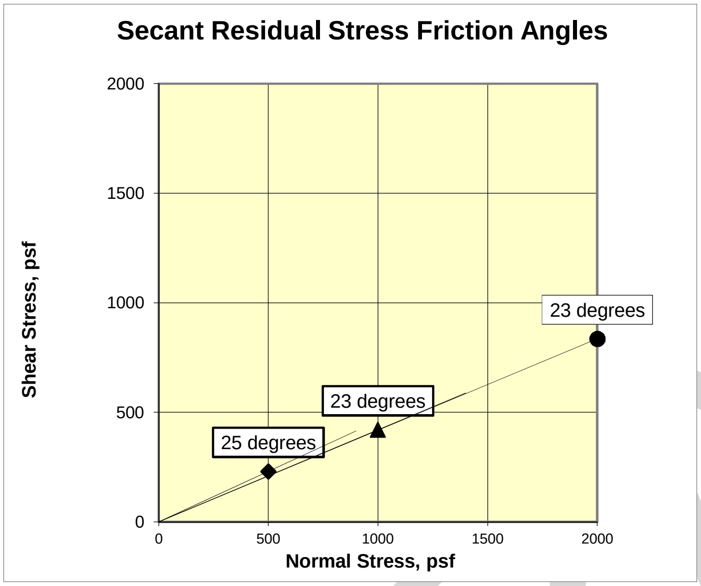
Drained Residual Torsional Shear Strength (ASTM D6467)

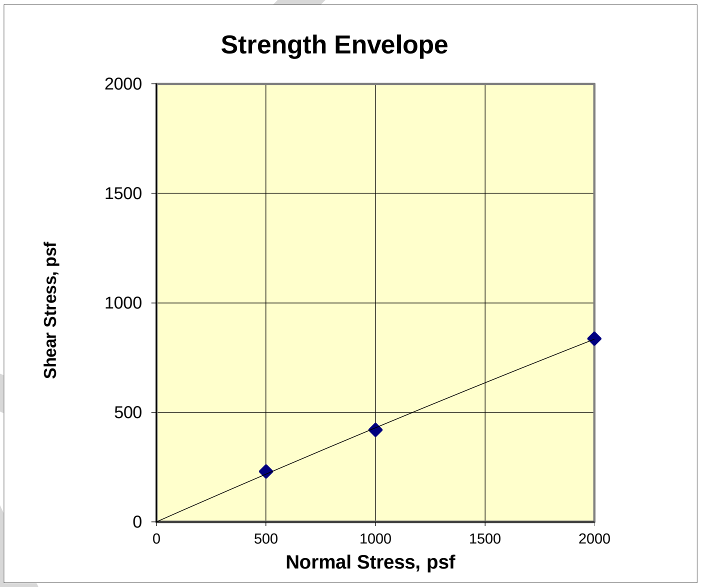
CTL Job No.:	968-001	Boring:	DH-2	Date:	8/29/2016	Clay, %:	
Client:	Vertical Sciences, Inc.	Sample:		By:	PJ	LL:	
Project Name:	Columbine Station	Depth (ft):	10	Checked:	DC	PL:	
Project Number:	160026	Test Type:	Fully Softened Residual				

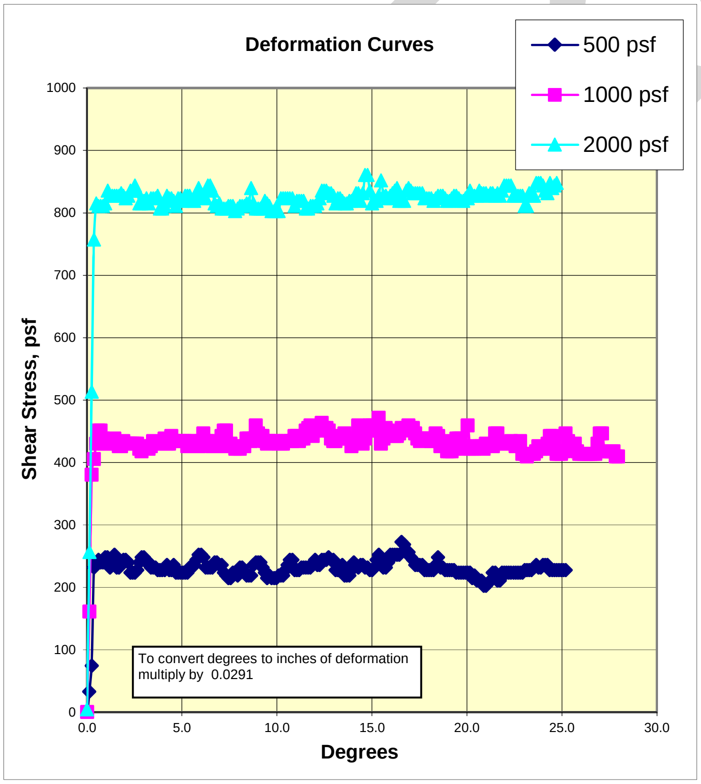
Soil Type: Olive Brown Clayey SAND

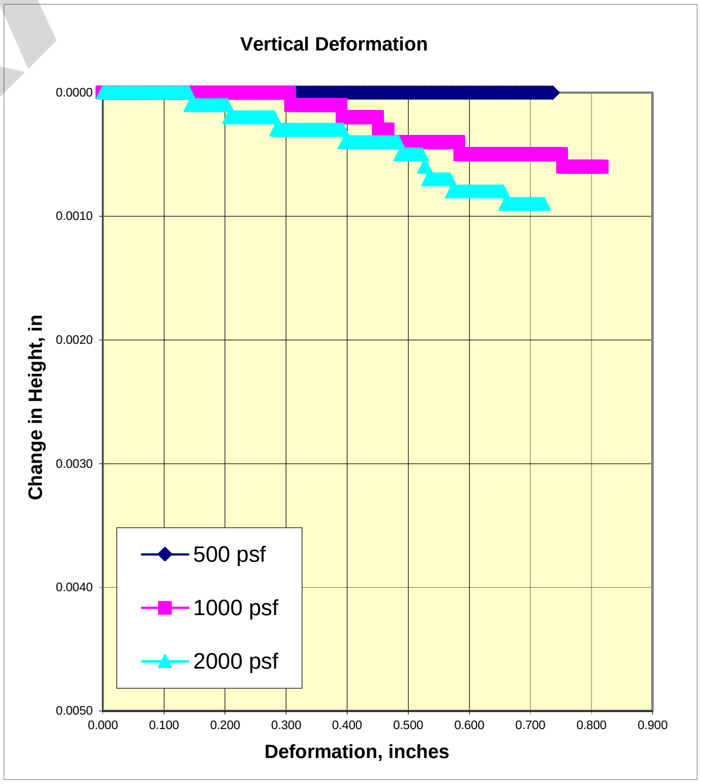
Normal Stress, psf:	500	1000	2000
Secant Phi, deg.:	25	23	23

Remarks: A small friction correction was applied to each point.



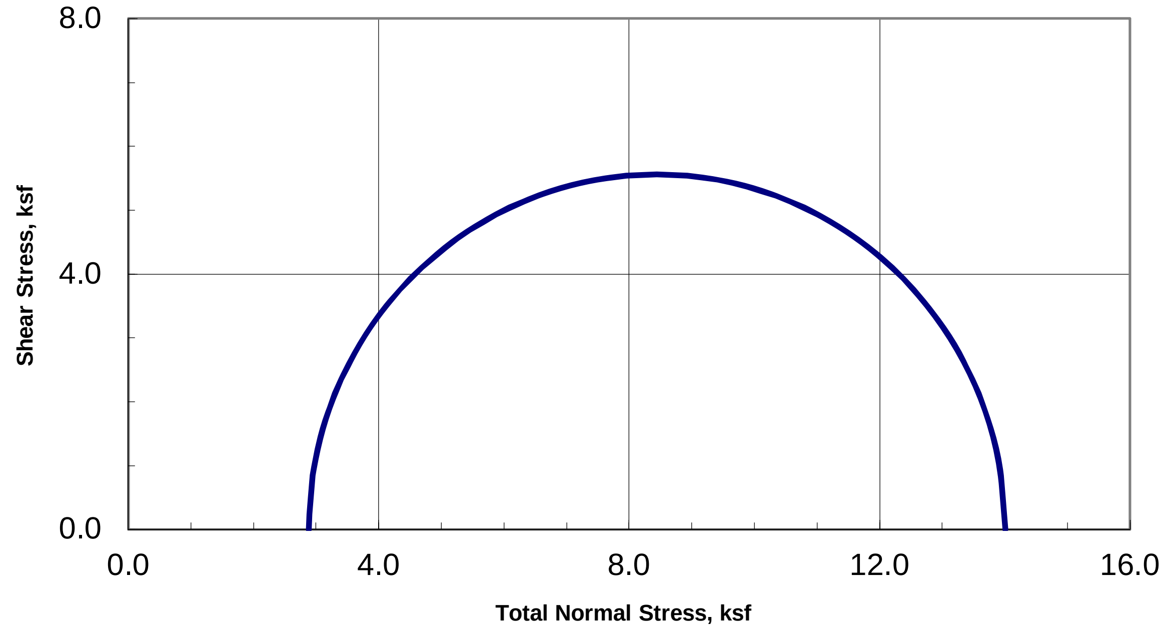


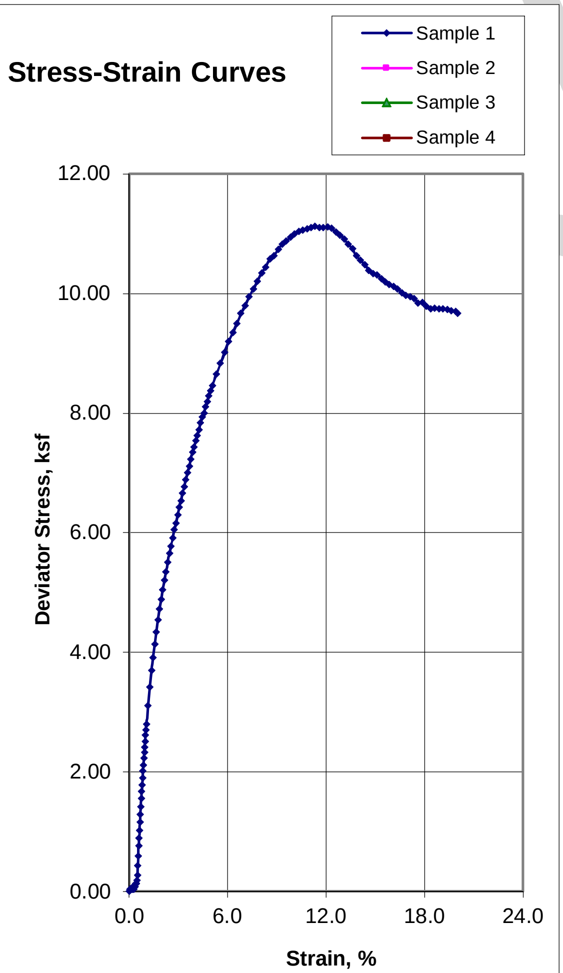




Cooper Testing Labs, Inc.
937 Commercial Street
Palo Alto, CA 94303

Unconsolidated-Undrained Triaxial Test
ASTM D2850





Sample Data				
	1	2	3	4
Moisture %	7.5			
Dry Den,pcf	106.8			
Void Ratio	0.578			
Saturation %	34.8			
Height in	4.99			
Diameter in	2.42			
Cell psi	20.0			
Strain %	11.33			
Deviator, ksf	11.123			
Rate %/min	1.00			
in/min	0.050			
Job No.:	968-001			
Client:	Vertical Sciences, Inc.			
Project:	Columbine Station - 160026			
Boring:	DH-8			
Sample:	7			
Depth ft:	35			

Visual Soil Description

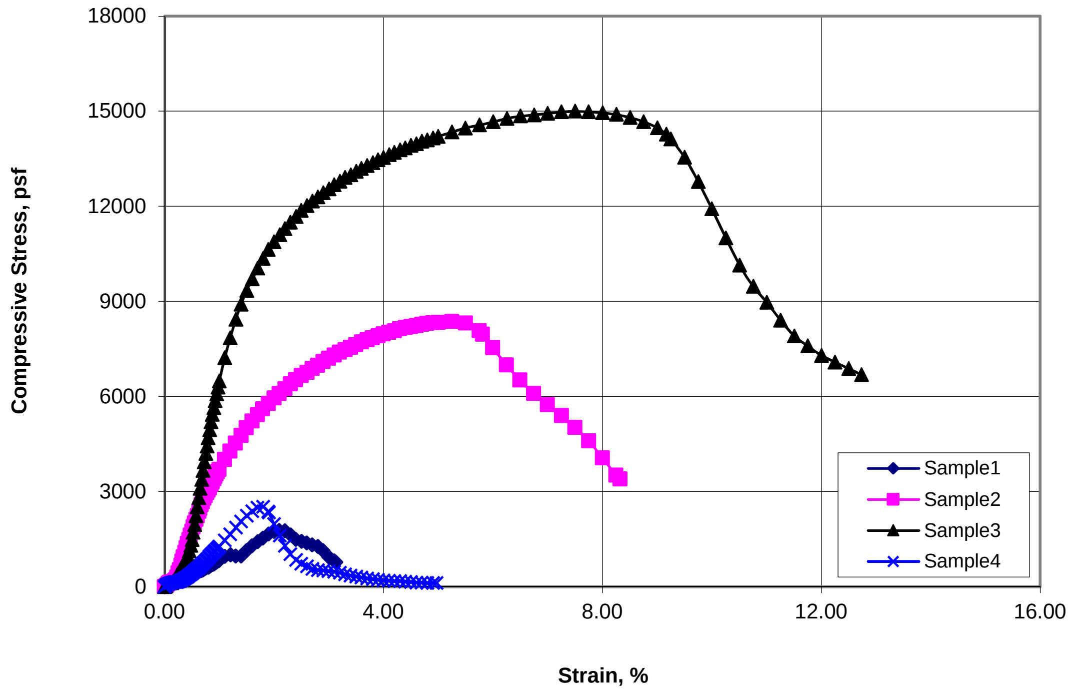
Sample #	
1	Yellowish Brown Silty SAND w/ Gravel (slightly plastic)
2	
3	
4	

Remarks:

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.

Unconfined Compressive Strength

ASTM D2166



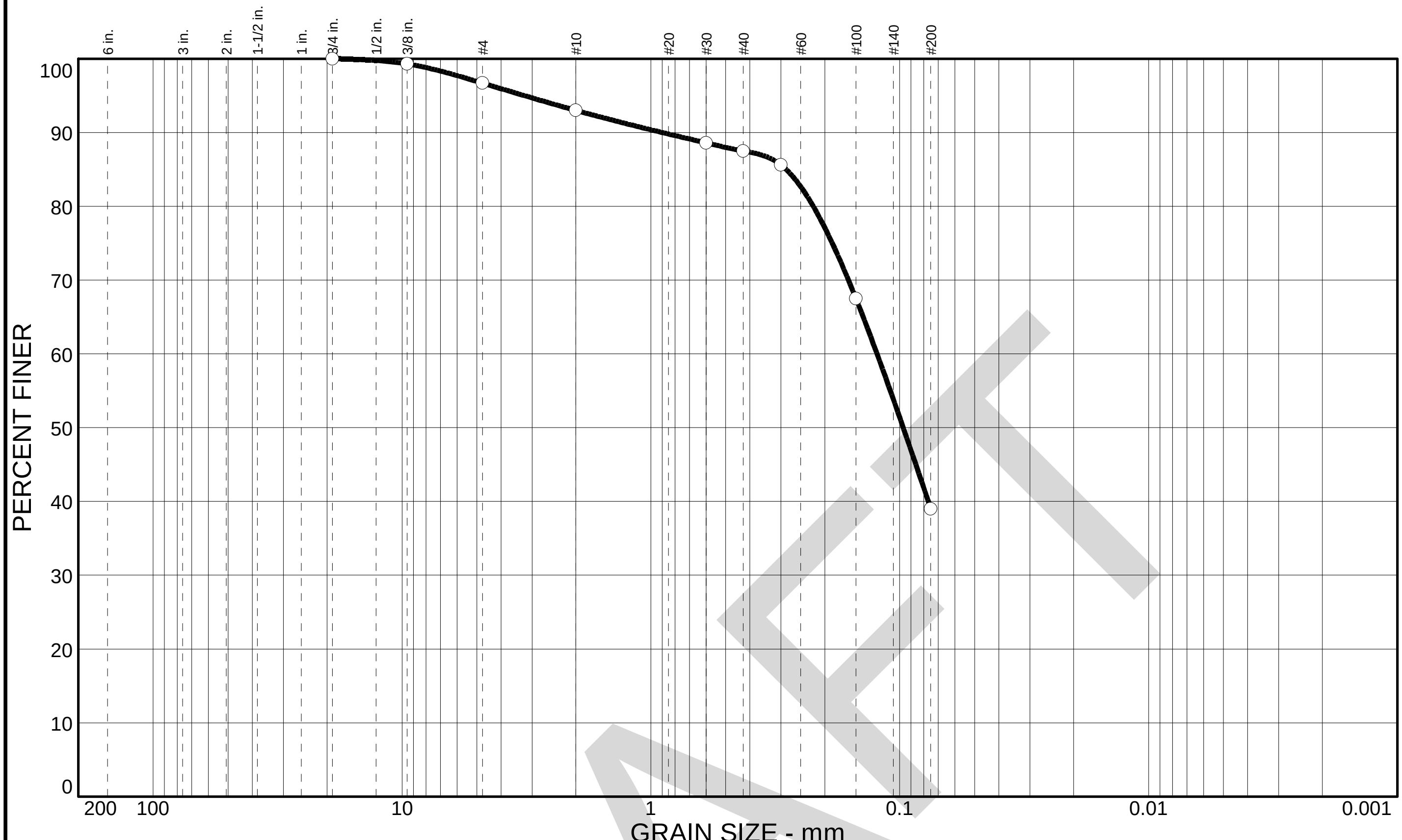
Sample No.:	1	2	3	4	
Unconfined Compressive Strength, psf	1774	8369	14996	2522	
Unconfined Compressive Strength, psi	12.3	58.1	104.1	17.5	
Undrained Shear Strength, psf	887	4184	7498	1261	
Failure Strain, %	2.1	5.3	7.5	1.8	
Strain Rate, % per minute	1.0	1.0	1.0	1.0	
Strain Rate, inches/minute	0.05	0.05	0.05	0.05	
Moisture Content, %	5.5	14.1	15.8	10.7	
Dry Density, pcf	121.4	121.2	116.3	105.6	
Saturation, %	38.3	97.2	95.1	48.6	
Void Ratio	0.388	0.390	0.449	0.597	
Specimen Diameter, inches	2.410	2.400	2.400	2.400	
Specimen Height, inches	4.99	5.00	4.99	5.00	
Height to Diameter Ratio	2.1	2.1	2.1	2.1	
Assumed Specific Gravity	2.70	2.70	2.70	2.70	

Sample Location				
	Boring	Sample	Depth, ft.	Soil Description
1	DH-4	5C	35	Olive Brown Clayey GRAVEL w/ Sand
2	DH-6	4B	31	Yellowish Brown Clayey SAND w/ Gravel
3	DH-6	5	35	Olive Brown Sandy CLAY w/ Gravel
4	DH-9	3	15	Light Olive Brown Silty SAND

Job No.:	968-001	Type of Sample	Undisturbed
Client:	Vertical Sciences, Inc.		
Project:	Columbine Station - 160026	Remarks:	
Date:	9/8/2016 By: MD/RU		

COOPER TESTING LABORATORY

Particle Size Distribution Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		3.3	57.7	39.0					

SIEVE inches size	PERCENT FINER ○		
3/4"	100.0		
3/8"	99.3		
	GRAIN SIZE		
D_{60}	0.123		
D_{30}			
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER ○		
#4	96.7		
#10	93.0		
#30	88.6		
#40	87.5		
#50	85.6		
#100	67.5		
#200	39.0		

SOIL DESCRIPTION

○ Light Olive Brown Silty SAND

REMARKS:

○

○ Source: DH-9 Sample No.: 3 Elev./Depth: 15'

COOPER TESTING LABORATORY

Client: Vertical Sciences, Inc.

Project: Columbine Station - 160026

Project No.: 968-001 Figure

M T I
Materials Testing, Inc.
8798 Airport Road
Redding, California 96002
(530) 222-1116, fax 222-1611
865 Cotting Lane, Suite A
Vacaville, California 95688
(707) 447-4025, fax 447-4143

Client: Vertical Sciences, Inc.
P.O. Box 491535
Redding, CA 96049

Client No.: 3195-001
Report No.: 0300-001
Date: 09/20/16

Project: Columbine Station – Job No. 160026
San Jose, California

Submitted by: Client
Submitted Date: 09/08/16

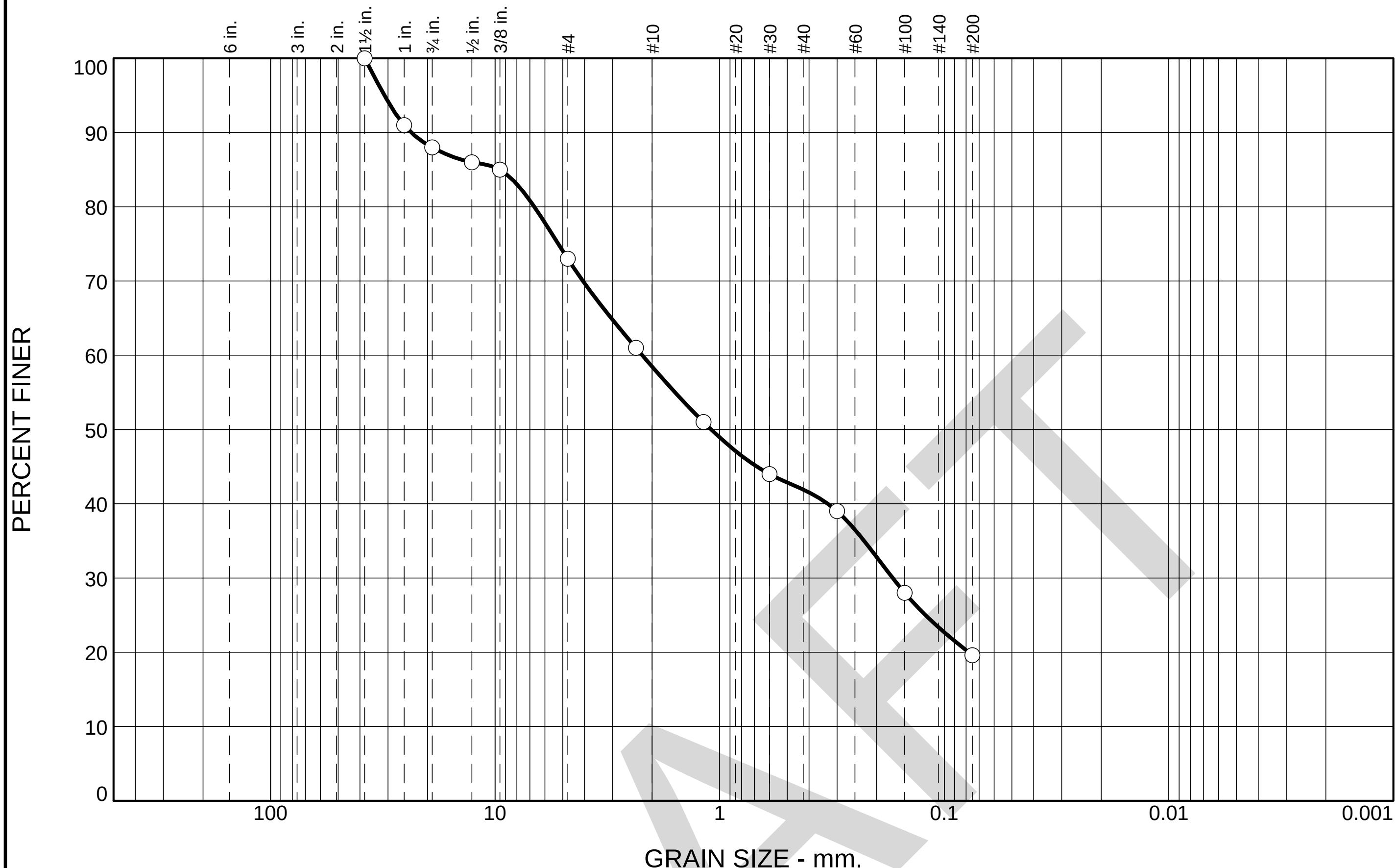
Density of Soil in Place by the Drive-Cylinder Method (ASTM D2937), Liquid Limit, Plastic Limit & Plasticity Index of Soils (ASTM D4318) And Moisture Content of Soil (ASTM D2216)

Sample #	Description	Dry Density p.c.f.	Moisture Content %	Liquid Limit	Plastic Limit	Plastic Index
DH-1 – 1a @ 5.5’	Light Brown Sandy Clay with Gravel (visual)	114.8	14.5	---	---	---
DH-1 – 2 @ 10’	Light Brown Clayey Sand with Gravel (visual)	112.8	11.3	---	---	---
DH-1 – 3 @ 15’	Light Brown Clayey Sand with Gravel (visual)	113.3	6.7	---	---	---
DH-1 – 4 @ 25’	Light Brown Clayey Sand with Gravel (visual)	114.1	8.2	---	---	---
DH-1 – 5 @ 30’	Light Brown Clayey Sand with Gravel (visual)	---	8.6	---	---	---
DH-1 – 6 @ 33.5’	Light Brown Clayey Gravel with Sand (visual)	---	6.7	---	---	---
DH-2 – 1 @ 5’	Strong Brown Sandy Clay (visual)	99.6	21.6	---	---	---
DH-2 – 4 @ 20’	Light Brown Clayey Sand with Gravel (visual)	114.0	6.6	---	---	---
DH-2 – 5 @ 25’	Light Brown Clayey Sand with Gravel (visual)	---	6.4	---	---	---
DH-2 – 6 @ 30’	Light Brown Clayey Sand with Gravel (visual)	---	7.4	---	---	---
DH-3 – 1 @ 25’	Light Brown Sandy Clay with Gravel (visual)	113.3	11.6	---	---	---

Sample #	Description	Dry Density p.c.f.	Moisture Content %	Liquid Limit	Plastic Limit	Plastic Index
DH-3 – 2 @ 30’	Light Brown Clayey Sand with Gravel (visual)	---	7.2	---	---	---
DH-3 – 3 @ 35’	Light Brown Clayey Sand with Gravel (visual)	---	5.6	---	---	---
DH-3 – 4 @ 40’	Light Brown Clayey Sand with Gravel (visual)	---	7.1	---	---	---
DH-3 – 5 @ 45’	Light Brown Clayey Sand with Gravel (visual)	---	9.5	28	22	6
DH-3 – 6 @ 50’	Light Brown Clayey Sand with Gravel (visual)	---	2.4	---	---	---
DH-3 – 7 @ 55’	Light Brown Clayey Sand (visual)	---	5.8	---	---	---
DH-3 – 8 @ 60’	Light Brown Clayey Sand (visual)	---	9.2	---	---	---
DH-4 – 1 @ 5’	Light Brown Sandy Clay with Gravel (visual)	85.5	10.0	---	---	---
DH-4 – 2 @ 20’	Light Brown Sandy Clay (visual)	---	10.7	---	---	---
DH-4 – 3 @ 25’	Strong Brown Sandy Clay (visual)	---	15.2	---	---	---
DH-4 – 4 @ 30’	Light Brown Sandy Clay with Gravel (visual)	---	11.3	39	17	22
DH-4 – 6 @ 38’	Light Brown Clayey Sand with Gravel (visual)	---	5.4	---	---	---
DH-5 – 1 @ 5’	Strong Brown Clay with Sand & Gravel (visual)	112.8	15.1	---	---	---
DH-5 – 2 @ 15’	Dark Brown Sandy Clay with Gravel (visual)	103.3	22.5	---	---	---
DH-5 – 3 @ 20’	Strong Brown Sandy Clay (visual)	102.8	19.4	---	---	---
DH-5 – 4 @ 25’	Dark Brown Sandy Clay (visual)	100.8	18.6	---	---	---
DH-5 – 6 @ 35’	Light Brown Clayey Sand with Gravel (visual)	---	7.1	26	16	10
DH-5 – 7 @ 40’	Light Brown Clayey Sand with Gravel (visual)	---	8.7	---	---	---
DH-5 – 8 @ 43’	Light Brown Clayey Sand with Gravel (visual)	---	4.9	---	---	---
DH-6 – 1A @ 10.5’	Brown Sandy Clay with Gravel (visual)	103.1	16.3	---	---	---
DH-7 – 1B @ 26’	Strong Brown Sandy Clay with Gravel (visual)	108.1	16.0	---	---	---

Sample #	Description	Dry Density p.c.f.	Moisture Content %	Liquid Limit	Plastic Limit	Plastic Index
DH-7 – 4 @ 40’	Light Brown Clayey Sand with Gravel (visual)	---	10.5	---	---	---
DH-7 – 5 @ 45’	Light Brown Clayey Sand with Gravel (visual)	---	5.2	---	---	---
DH-7 – 6 @ 50’	Light Brown Clayey Sand with Gravel (visual)	---	5.1	---	---	---
DH-7 – 7 @ 55’	Strong Brown Sandy Clay (visual)	---	16.7	---	---	---
DH-7 – 8 @ 60’	Brown Sandy Clay (visual)	---	6.9	---	---	---
DH-7 – 9 @ 65’	Light Brown Clayey Sand with Gravel (visual)	---	16.2	---	---	---
DH-8 – 1 @ 5’	Strong Brown Sandy Clay with Gravel (visual)	104.5	19.3	---	---	---
DH-8 – 3 @ 15’	Strong Brown Sandy Clay with Gravel (visual)	113.3	14.5	---	---	---
DH-8 – 4 @ 20’	Strong Brown Sandy Clay with Gravel (visual)	111.8	14.6	---	---	---
DH-8 – 5 @ 25’	Strong Brown Clayey Sand (visual)	108.9	10.2	---	---	---
DH-8 – 8 @ 40’	Light Brown Clayey Sand with Gravel (visual)	---	16.2	---	---	---
DH-8 – 9 @ 44’	Light Brown Clayey Sand with Gravel (visual)	---	7.7	---	---	---
DH-9 – 1 @ 5’	Strong Brown Sandy Clay with Gravel (visual)	117.7	12.3	---	---	---
DH-9 – 2 @ 10’	Light Brown Sandy Clay with Gravel (visual)	103.2	15.8	---	---	---
DH-9 – 4 @ 20’	Light Brown Sandy Clay with Gravel (visual)	116.9	6.2	---	---	---
DH-9 – 5 @ 25’	Light Brown Clayey Sand with Gravel (visual)	---	6.0	---	---	---
DH-10 – 1 @ 5’	Light Brown Clayey Sand with Gravel (visual)	---	9.0	---	---	---
DH-10 – 2 @ 10’	Light Brown Clayey Sand with Gravel (visual)	---	5.2	---	---	---
DH-10 – 3 @ 15’	Light Brown Clayey Sand with Gravel (visual)	---	5.6	---	---	---
DH-10 – 4 @ 18’	Light Brown Clayey Sand with Gravel (visual)	---	3.3	---	---	---

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	12	15	15	16	22	20	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2"	100		
1"	91		
3/4"	88		
1/2"	86		
3/8"	85		
#4	73		
#8	61		
#16	51		
#30	44		
#50	39		
#100	28		
#200	20		

* (no specification provided)

Material Description

Light Brown Clayey Sand with Gravel (visual)

Atterberg Limits

PL= --- LL= --- PI= ---

Coefficients

D_{90}= 23.6426 D_{85}= 9.5250 D_{60}= 2.2114
D_{50}= 1.0908 D_{30}= 0.1697 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

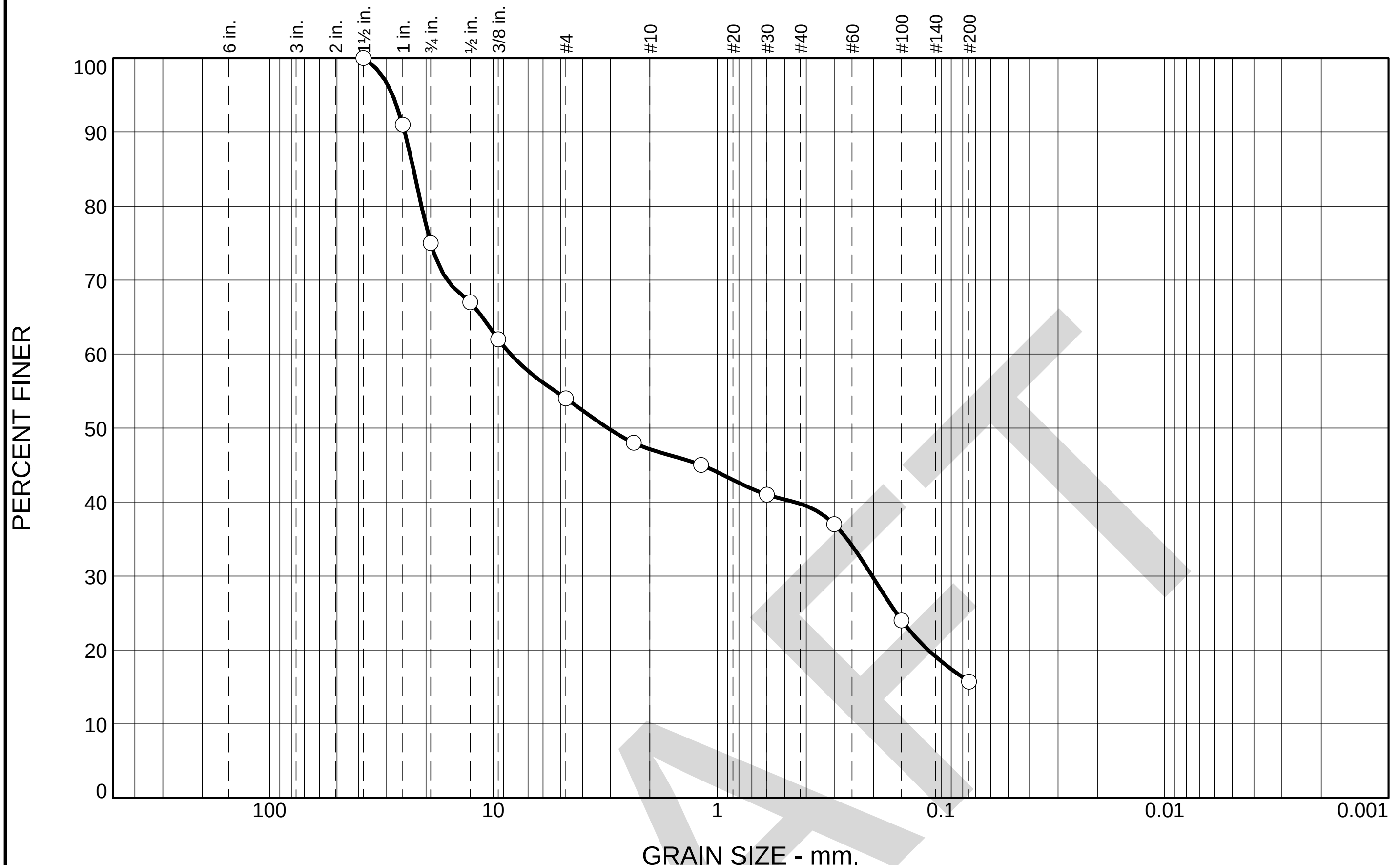
Material tested in accordance with ASTM D6913.

Location: DH-1 - 5
Sample Number: 5 **Depth:** 30' **Date:** 09/20/16

M T I Materials Testing, Inc.

Client: Vertical Sciences, Inc.
Project: Columbine Station - Job No. 160026
San Jose, CA
Project No: 3195-001 **Figure** 0300-002

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	25	21	7	7	24	16	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2"	100		
1"	91		
3/4"	75		
1/2"	67		
3/8"	62		
#4	54		
#8	48		
#16	45		
#30	41		
#50	37		
#100	24		
#200	16		

* (no specification provided)

Material Description

Light Brown Clayey Gravel with Sand (visual)

Atterberg Limits

PL= --- LL= --- PI= ---

Coefficients

D_{90}= 24.8918 D_{85}= 22.7830 D_{60}= 8.3960
D_{50}= 3.0819 D_{30}= 0.2031 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= GC AASHTO=

Remarks

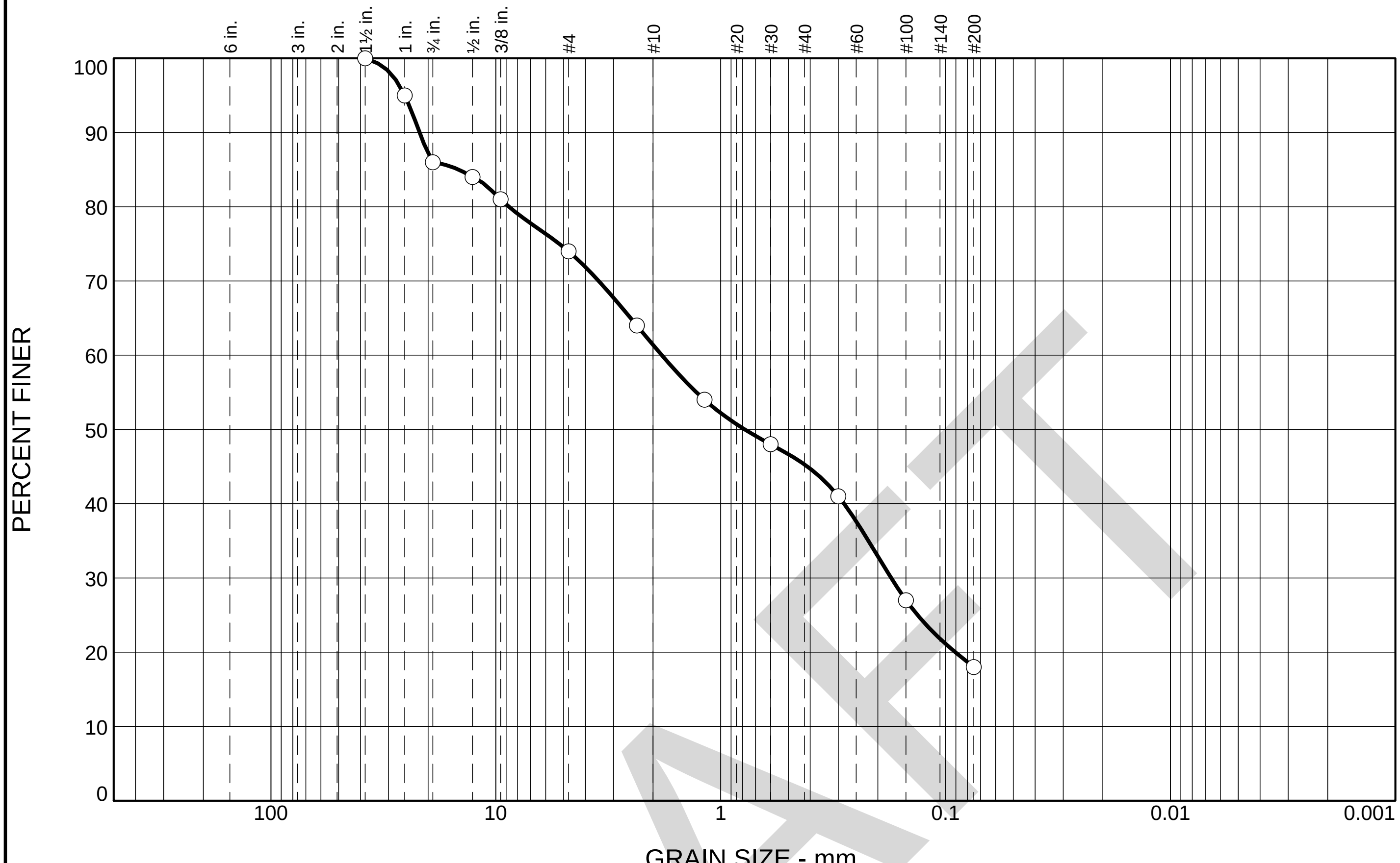
Material tested in accordance with ASTM D6913.

Location: DH-1 - 6
Sample Number: 6 **Depth:** 33.5' **Date:** 09/20/16

M T I Materials Testing, Inc.

Client: Vertical Sciences, Inc.
Project: Columbine Station - Job No. 160026
San Jose, CA
Project No: 3195-001 **Figure** 0300-003

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	14	12	13	16	27	18	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2"	100		
1"	95		
3/4"	86		
1/2"	84		
3/8"	81		
#4	74		
#8	64		
#16	54		
#30	48		
#50	41		
#100	27		
#200	18		

* (no specification provided)

Material Description

Light Brown Clayey Sand with Gravel (visual)

Atterberg Limits

PL= --- LL= --- PI= ---

Coefficients

D_{90}= 21.8253 D_{85}= 14.6451 D_{60}= 1.8267
D_{50}= 0.7810 D_{30}= 0.1748 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO= ---

Remarks

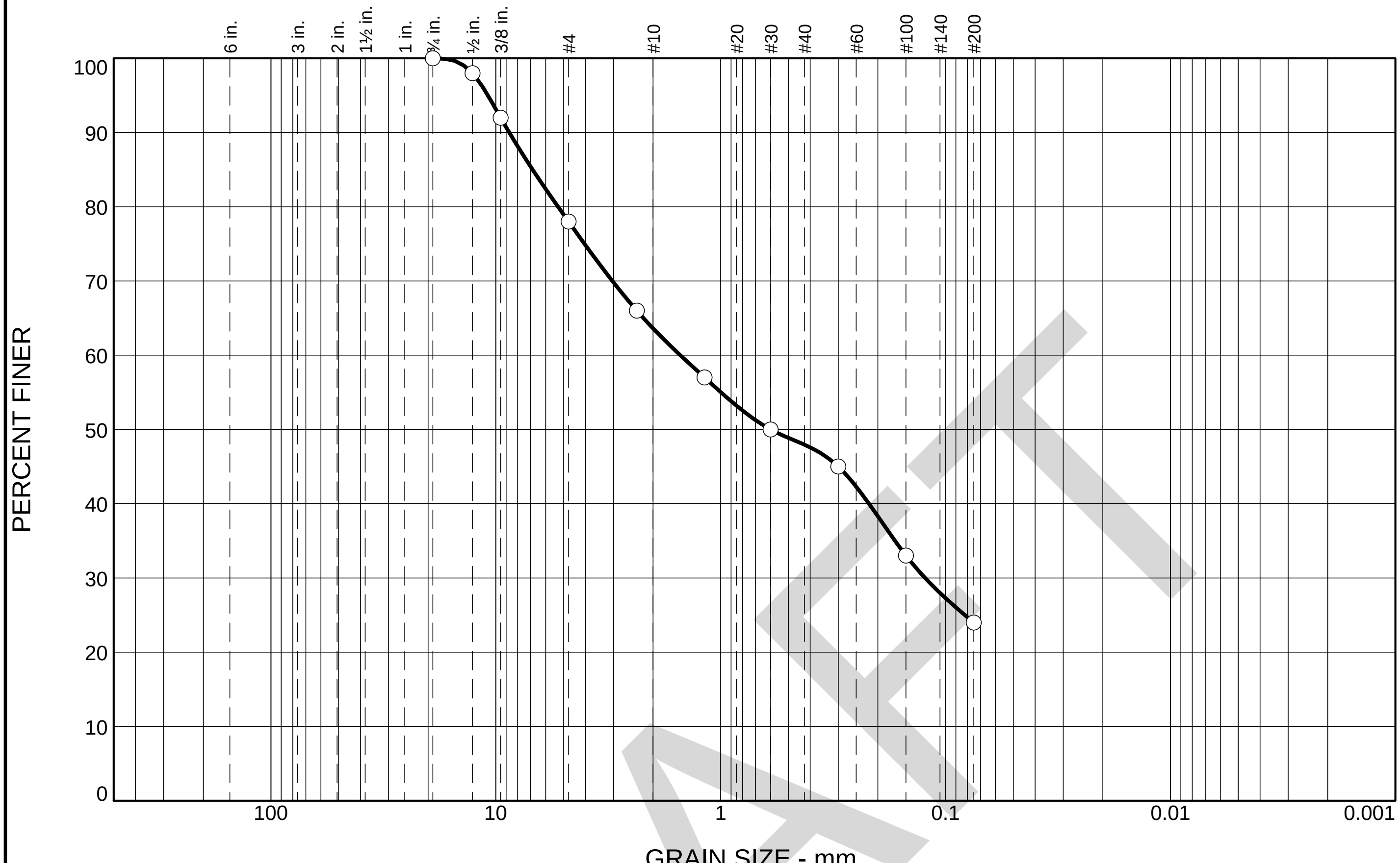
Material tested in accordance with ASTM D6913.

Location: DH-3 - 4
Sample Number: 16 **Depth:** 40' **Date:** 09/20/2016

Materials Testing, Inc.

Client: Vertical Sciences, Inc.
Project: Columbine Station - Job No. 160026
San Jose, CA
Project No: 3195-001 **Figure** 0300-004

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	22	14	16	24	24	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100		
1/2"	98		
3/8"	92		
#4	78		
#8	66		
#16	57		
#30	50		
#50	45		
#100	33		
#200	24		

* (no specification provided)

Material Description

Light Brown Clayey Sand with Gravel (visual)

Atterberg Limits

PL= --- LL= --- PI= ---

Coefficients

D_{90}= 8.7176 D_{85}= 6.8653 D_{60}= 1.5119
D_{50}= 0.6000 D_{30}= 0.1234 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

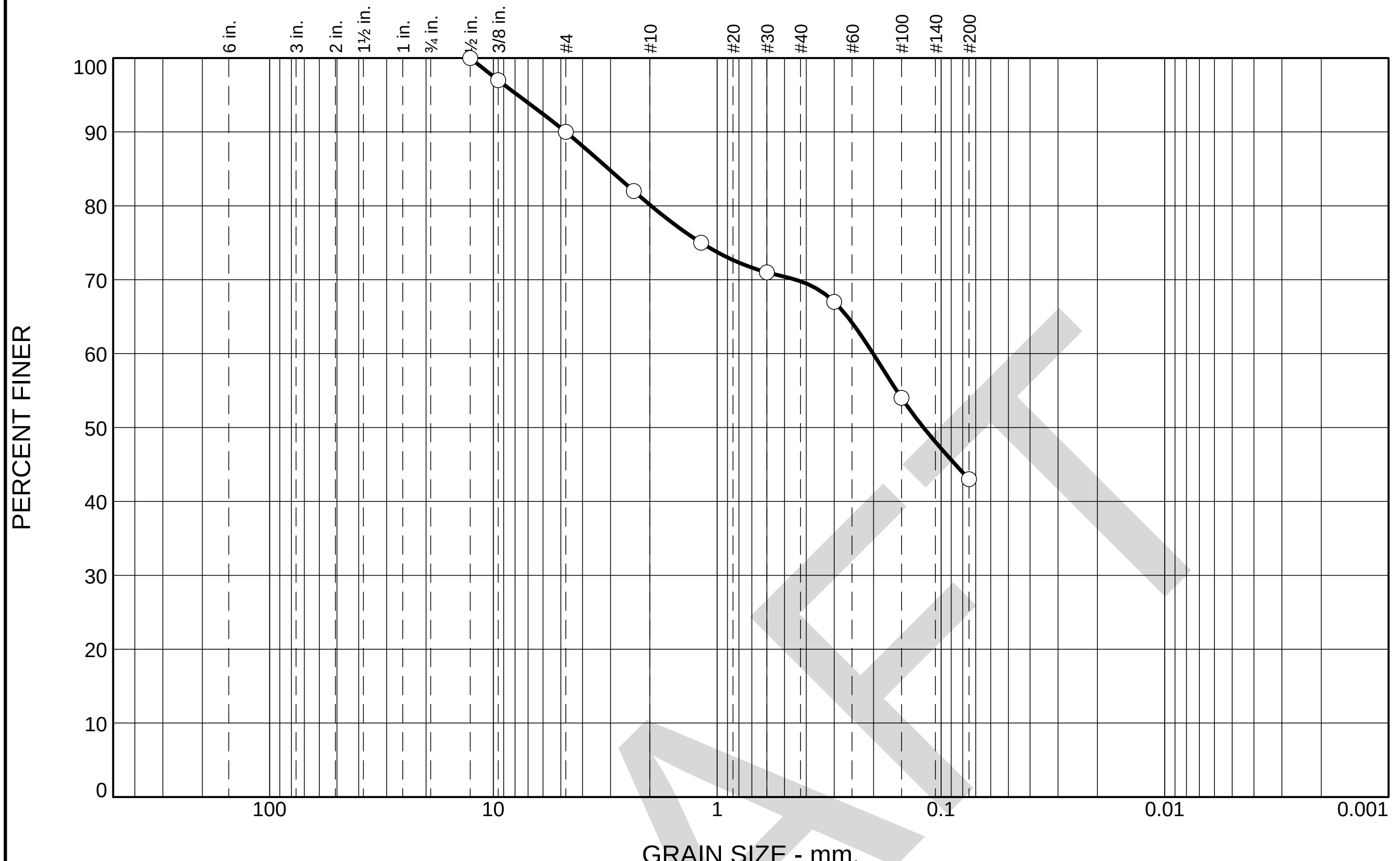
Material tested in accordance with ASTM D6913.

Location: DH-3 - 6
Sample Number: 18 **Depth:** 50' **Date:** 09/20/16

Materials Testing, Inc.

Client: Vertical Sciences, Inc.
Project: Columbine Station - Job No. 160026
San Jose, CA
Project No: 3195-001 **Figure** 0300-005

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	10	10	10	27	43	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2"	100		
3/8"	97		
#4	90		
#8	82		
#16	75		
#30	71		
#50	67		
#100	54		
#200	43		

* (no specification provided)

Material Description

Strong Brown Clayey Sand (visual)

Atterberg Limits

PL= --- LL= --- PI= ---

Coefficients

D_{90}= 4.7500 D_{85}= 3.0572 D_{60}= 0.2013
D_{50}= 0.1198 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

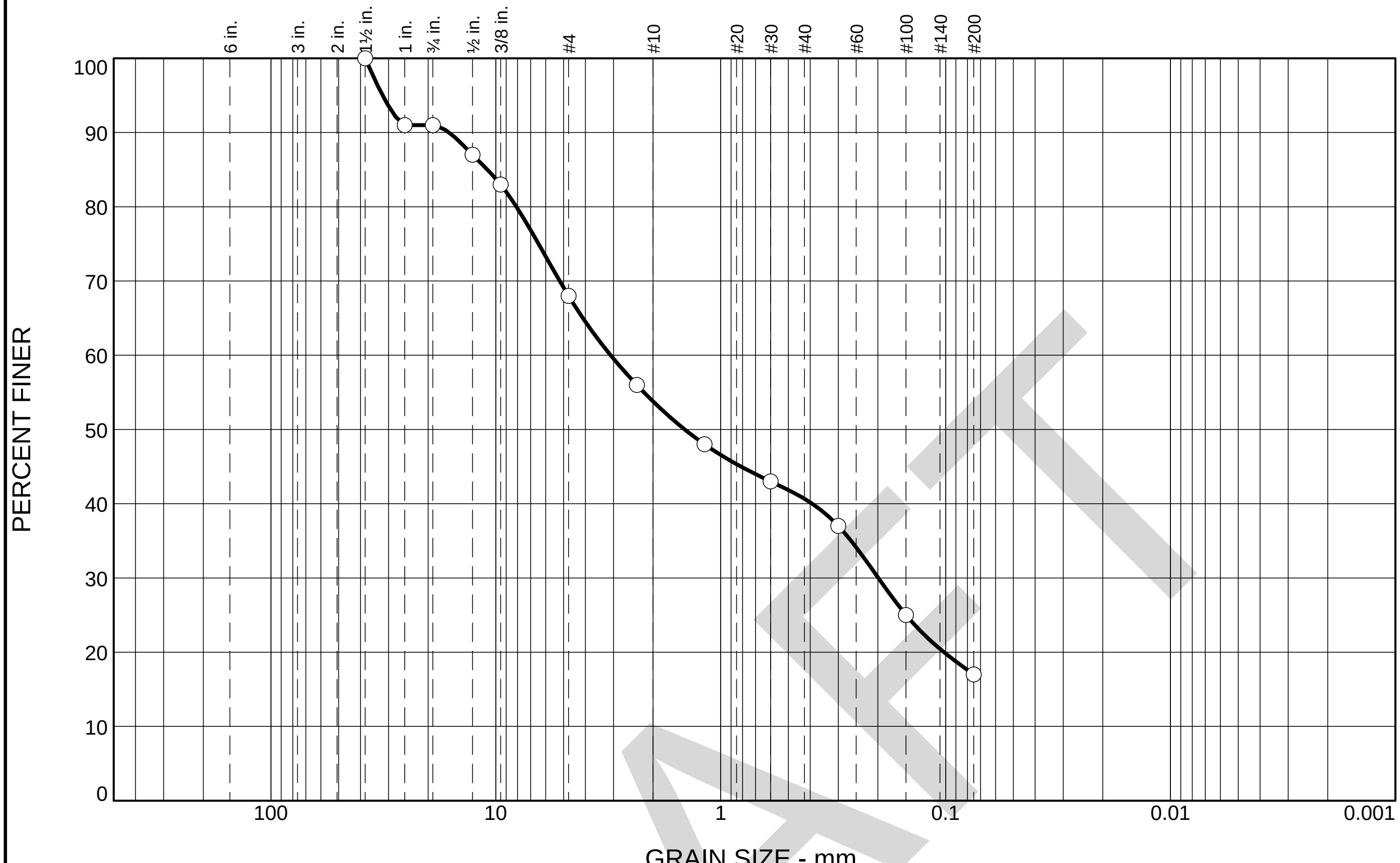
Material tested in accordance with ASTM D6913.

Location: DH-8 - 5
Sample Number: 51 **Depth:** 25' **Date:** 09/20/16

Materials Testing, Inc.

Client: Vertical Sciences, Inc.
Project: Columbine Station - Job No. 160026
San Jose, CA
Project No: 3195-001 **Figure** 0300-006

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	9	23	14	13	24	17	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2"	100		
1"	91		
3/4"	91		
1/2"	87		
3/8"	83		
#4	68		
#8	56		
#16	48		
#30	43		
#50	37		
#100	25		
#200	17		

* (no specification provided)

Material Description

Light Brown Clayey Sand with Gravel (visual)

Atterberg Limits

PL= --- LL= --- PI= ---

Coefficients

D_{90}= 16.1377 D_{85}= 10.9087 D_{60}= 3.0878
D_{50}= 1.4451 D_{30}= 0.1993 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

Material tested in accordance with ASTM D6913.

Location: DH-8 - 9
Sample Number: 55 **Depth:** 44' **Date:** 09/20/16

Materials Testing, Inc.

Client: Vertical Sciences, Inc.
Project: Columbine Station - Job No. 160026
San Jose, CA
Project No: 3195-001 **Figure** 0300-007

COMPACTION TEST REPORT

Test specification: ASTM D1557-12 Method B Modified
ASTM D 4718-87 Oversize Corr. Applied to Each Test Point

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/8 in.	% < No.200
	USCS	AASHTO						
1'- 5'	CL			2.60			12.9	

ROCK CORRECTED TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 127.5 pcf Optimum moisture = 11.4 %	Light Brown Sandy Clay with Gravel & RAP

Project No. 3195-001 **Client:** Vertical Sciences, Inc.
Project: Columbine Station - Job No. 160026
San Jose, CA
○ **Location:** DH-4 - B1 **Sample Number:** 26

Materials Testing, Inc.

Remarks:
Curve #1
(09/20/16)

Figure 0300-008

COMPACTION TEST REPORT

Test specification: ASTM D1557-12 Method B Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/8 in.	% < No.200
	USCS	AASHTO						
1'- 5'	CL							

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 122.6 pcf Optimum moisture = 10.8 %	Brown Sandy Clay

Project No. 3195-001 **Client:** Vertical Sciences, Inc.

Project: Columbine Station - Job No. 160026

San Jose, CA

○ **Location:** DH-7 - B1 **Sample Number:** 46

Materials Testing, Inc.

Remarks:

Curve #2
(09/20/16)

Figure 0300-009

M T I
Materials Testing, Inc.
8798 Airport Road
Redding, California 96002
(530) 222-1116, fax 222-1611
865 Cotting Lane, Suite A
Vacaville, California 95688
(707) 447-4025, fax 447-4143

Client:	Vertical Sciences, Inc. P.O. Box 491535 Redding, CA 96049	**Client No:** **Report No:** **Date:**	3195-001 0300-010 09/21/16
Subject:	Columbine Station – Job No. 160026 San Jose, California	**Submitted by:** **Submitted Date:**	Client 09/08/16

"R" VALUE TEST REPORT
(ASTM D2844)

Sample:	26
Description:	Light Brown Sandy Clay with Gravel & Rap
Location:	DH-4 – B1 @ 1-5'

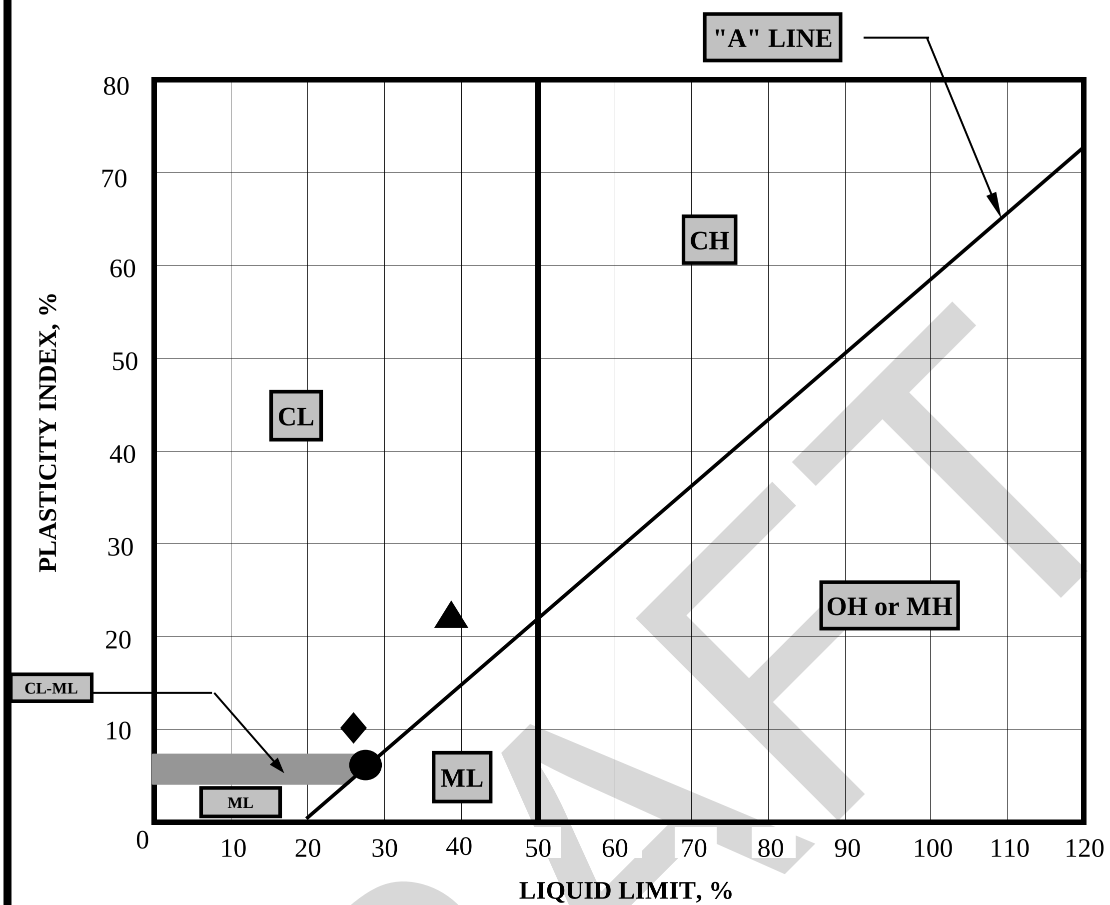
SIEVE ANALYSIS

Sieve Size	1"	3/4"	1/2"	3/8"	#4
"As Received" (Percent Pass)	100	98	94	84	73
"As Used" (Percent Pass)		100	96	91	74

RESISTANCE VALUE

Specimen Number	Dry Unit Weight, PCF	Moisture (%)	Exudation Pressure (PSI)	Expansion Pressure Dial Reading & PSF		R-Value
1	120.3	13.0	453	7	30	33
2	115.8	14.0	290	4	17	20
3	112.4	16.5	218	1	4	16

R-Value @ 300 PSI Exudation Pressure = 21



KEY SYMBOL	SAMPLE NUMBER	DEPTH	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, LL, %	PLASTIC LIMIT, PL, %	PLASTICITY INDEX, PI, %	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
●	17	45.0'	N/A	28	22	6	N/A	CL
▲	24	30.0'	N/A	39	17	22	N/A	CL
◆	32	35.0'	N/A	26	16	10	N/A	CH

Note: Atterberg Limits tested in accordance with ASTM D4318.

Materials Testing, Inc.

PLASTICITY CHART AND DATA

Columbine Station – Job No. 160026
San Jose, California

Client No:	Date:	Report No:
3195-001	9/21/2016	0300-011

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 10/19/2016
Date Submitted 10/12/2016

To: Jim Bianchin
Vertical Sciences, Inc.
P.O. Box 491535
Redding, CA 96049

From: Gene Oliphant, Ph.D. \ Randy Horney RA
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : COLUMBINE TANK Site ID : COLUMBINE DH11A.
Thank you for your business.

* For future reference to this analysis please use SUN # 73008-152389.

EVALUATION FOR SOIL CORROSION

Soil pH	8.04		
Minimum Resistivity	1.53	ohm-cm (x1000)	
Chloride	25.2 ppm	00.00252	%
Sulfate	71.7 ppm	00.00717	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422