GEOTECHNICAL REPORT

Reservoir B Replacement Study
Paradise Irrigation District
Town of Paradise & Butte County, California

Submitted To:

Mr. Sami Kader P.E.
WATER WORKS ENGINEERS
1405 Victor Avenue
Suite A
Redding, CA 96003

Prepared by:
Vertical Sciences, Inc.

July 31, 2018
Project No. 170025

VERTICAL SCIENCES, INC.

July 31, 2017
Project: 170025

Mr. Sami Kader, P.E.
WATERWORKS ENGINEERS, LLC
1405 Victor Avenue., Suite A
Redding, California 96003

Subject: **Preliminary (Desktop) Geotechnical Report**
Reservoir B Replacement – Planning & Design
Paradise Irrigation District
Town of Paradise & Unincorporated Butte County, California

Dear Mr. Kader:

Vertical Sciences, Inc. (VSI), is pleased to submit this preliminary (desktop) geotechnical report to Water Works Engineers. LLC, for Paradise Irrigation District's Reservoir B Replacement Planning & Design study. The project is located within the Town of Paradise and unincorporated Butte County, California. This report is being submitted in accordance with our proposal dated April 6, 2017.

We appreciate the opportunity to perform this preliminary study. If you have any questions pertaining to this preliminary report, or if we may be of further service, please contact us at (530) 638-5263 at your earliest convenience.

Sincerely,

VERTICAL SCIENCES, INC.

REGISTERED GEOLOGIST
JAMES A. BIANCHIN
EG NO. 1644
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
JON M. EVERETT
No. 2531
Exp. 6-30-19
GEOTECHNICAL
STATE OF CALIFORNIA

James A. Bianchin, C.E.G.
Principal Engineering Geologist

Jon M. Everett, P.E., G.E.
Principal Geotechnical Engineer

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1 GENERAL

This report presents the results of our geotechnical study for Paradise Irrigation District's (PID) Reservoir B Replacement study located in the Town of Paradise and in unincorporated Butte County, 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 proposal dated April 6, 2017.

1.1 PROJECT LOCATION

The proposed project is composed of three elements that are located at three different locations. The following table presents the locations of each of those project elements:

PROJECT LOCATION				
Project Element	**APN**	**Address**	**Latitude[1]**	**Longitude[1]**
Pump Station	065-260-011	13888 Pine Needle Dr., Magalia	39.814538°	-121.581817°
Pipeline	065-260-010 066-470-025 066-470-020 066-450-011 066-460-005 066-460-014	Various addresses along Pine Needle Drive, Depot Lane, and Skyway in Magalia and Paradise	39.807192°	-121.577372°
Reservoir B Tanks	050-070-075 050-070-077	8770 Skyway, Paradise	39.784963°	-121.595223°
[1] – Approximate center point of the proposed pipeline alignment.				

A map showing the locations of the project elements presented above are shown on Plate 2 – Project Elements.

1.2 PROJECT UNDERSTANDING

We understand that the PID has an existing 3-million-gallon reservoir within its B pressure zone called the B Reservoir. Water stored in that reservoir is pumped to Reservoir A, which services a separate section of PID's service area. We understand that Reservoir B has insufficient capacity for its service area and for wildland fire flow requirements. To improve the resiliency and reliability of current conditions, we understand that the project consists of the following elements:

1. Construction of a new pump station adjacent to the Treated Water Storage Tank (TWST) at the PID Water Treatment Plant (WTP);
2. Installation of a new pipeline that will connect the proposed pump station with existing Reservoir A; and

3. Construction of two new treated water storage tanks at the existing Reservoir B site.

The following paragraphs describe those project elements in greater detail.

1.2.1 Pump Station

The proposed pump station will be situated as shown on Plate 3.1 – Geotechnical Map, Pump Station. The pump station is proposed to have plan dimensions of about 23 feet wide by 40 feet long with a finish floor elevation of 2,209 feet. The pumps are proposed to consist of vertical turbines with a pump-can invert elevation of about 2,191, making the can invert depth about 21 feet below grade. It is anticipated that the pump station structure will be constructed using steel shell or concrete masonry unit (CMU) materials. Foundation loads are unknown at this time.

Retaining walls and engineered fill are proposed to construct the pump house and slab. We understand that the proposed retaining walls will be located along portions northwest, southwest, and southeast of the proposed structure and will be up to about 10 feet tall. We understand that engineered fill thicknesses will be up to approximately 10 feet thick.

1.2.2 Pipeline

The proposed pipeline will extend between the proposed pump station at the WTP, as described in Section 1.2.1, and the existing Reservoir A. This approximately 1.4-mile-long pipeline will consist of 16-inch diameter ductile iron pipe (DIP) or polyvinyl chloride (PVC) piping materials. It is anticipated that the invert depth of the proposed pipeline will be about 5 feet below existing grades along the alignment.

The pipeline alignment will extend from the WTP south along Pine Needle Drive to New Skyway, south along New Skyway towards Pentz Road, continue south on New Skyway for about 850 feet, then west to tie into Zone A existing piping. This alignment is shown on Plates 3.2.1 and 3.2.2 – Geotechnical Map, Pipeline Alignment.

1.2.3 Reservoir B Tanks

We understand that existing Reservoir B is lined with a Hypalon liner and covered with a floating Hypalon material. We understand that the reservoir is 16 feet deep and was constructed around 1985 by excavating materials from within and along the northeastern margin of the basin and placing those materials as embankments around the southwestern, southeastern, and northwestern margin. Minor embankments are also present along the basin's northeastern margin.

For this site, the existing reservoir is to be demolished and replaced with two 2.5-million-gallon steel water storage tanks. The locations of the proposed tanks are shown on Plate 3.3 – Geotechnical Map, Reservoir B Tanks. Both tanks will have sidewalls that are about 24 feet tall, 145-feet in diameter, and will have finished floor elevations of about 2,168 feet. Each tank will have a dome that will be up to about 3 feet taller than the tank sidewalls.

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 improvements;
- To estimate settlement and allowable bearing values for proposed subgrade soils for use in designing the proposed structure foundations and slabs; and
- To develop geotechnical recommendations for the design and construction of the proposed project.

1.4 SCOPE OF SERVICES

Services performed for this study are in general conformance with the proposed scope of services presented in our April 6, 2017 proposal. Our scope of services included:

- Reconnaissance of the site surface conditions, topography, and existing drainage features;
- Acquisition of two drilling permits from Butte County Environmental Health Department;
- Acquisition of one encroachment permit from Butte County Department of Public Works for exploration within the County's easement along Skyway;
- Advancement of 12 drill holes at selected locations shown on Plates 3.1 through 3.3. 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;
- Performance of geophysical refraction surveys at the Reservoir B site. The results of those surveys are presented in Appendix C – Geophysical Surveys;
- Evaluation of selected geotechnical criteria for the design and construction of 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;
- 2016 California Building Code (CBC) seismic design criteria;
- Geotechnical maps showing approximate field exploration locations, presented as Plates 3.1 – 3.3;
- Geotechnical recommendations for:
  - Construction of proposed slopes at the project sites;
  - Site preparation, engineered fill, site drainage, and subgrades;
  - Suitability of on-site materials for use as engineered fill;
  - Foundation and slab-on-grade design;
  - Lateral earth pressures for retaining wall design and construction;
  - Modulus of soil reaction (E') for flexible pipeline design;
  - Temporary excavations and shoring;
  - Trench backfill and compaction; and
  - Preliminary structural pavement design.
- Appendices that present a summary of our field investigation procedures and laboratory testing programs.

1.5 PREVIOUS WORK PERFORMED & REFERENCES REVIEWED

A preliminary (desktop) geotechnical study (VSI, 2017) was performed for the proposed project and is included as Appendix D – Preliminary (Desktop) Geotechnical Report. That report compiled selected, existing, available geotechnical and geological information for the proposed project. Those references may be discussed herein; however, more detailed information may be present in the desktop study, so we recommend that the information in Appendix D be utilized along with this report. If there is a conflict between information presented within this report and that within Appendix D, then information presented within this report should take preference.

At the proposed pump station location, site-specific geotechnical evaluations have been performed by Moore & Taber (1971), Kleinfelder (1992), and Taber (2015). Moore & Taber (1971) performed exploration and geophysical refraction surveys in the area of the existing WTP as part of modifications to Magalia Dam improvements and modifications. Kleinfelder (1992) performed subsurface exploration and refraction surveys across the WTP site to provide recommendations for design and construction of the existing WTP facilities in use today. Taber (2015) performed coring of asphaltic concrete and subsurface exploration to help assess causes and mitigations for distress observed in site paving and structures.

Prior to construction of Reservoir B, a series of exploratory backhoe test pits and trenches were excavated within the Reservoir B footprint (Clendenen Engineers, 1984 & 1985). Those test pits and trenches were excavated beneath the site.

Additional documents were referred to during this study and are referenced in the text and cited in Section 9.0 of this report.

2 FINDINGS

2.1 SITE HISTORY OF WATER TREATMENT PLANT AND RESERVOIR B

Based on review of aerial photographs and historical topographic maps it appears that the following historical development has occurred at the Reservoir B, Pump Station, and Pipeline Alignment sites.

SITE HISTORY	
IMAGE/ MAP YEAR	**OBSERVED DEVELOPMENT**
Pump Station	
1951	Magalia Reservoir in place along with limited treatment facilities located near Little Butte Creek.
1993	Limited development of existing WTP. Treatment facilities still adjacent to Little Butte Creek.
1998	WTP substantially built out to current facility, including the TWST.
2005-2012	Substantially similar buildout as observed today.
Pipeline Alignment	
1951	Pine Cone Road and New Skyway alignment are present. Reservoir A not constructed.
1993	Reservoir A constructed
Reservoir B	
1951	Reservoir B not constructed. Site largely fallow with oak trees and grasses. Minor possible agricultural/garden plot at northwest corner. Possible structure at northeast corner.
1993	Reservoir B constructed. No floating cover is present.
2009	Floating cover present.

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 sites and adjacent areas, and subsurface exploration through advancement of twelve drill holes. The Reservoir B drill holes were advanced on November 6, 8, and 9, 2017; the Pump Station drill holes were advanced on December 6 and 10, 2017; and the Pipeline drill holes were advanced on December 10, 2017. The exploration locations are shown on Plates 31 through 3.3. Descriptions of soils and rocks encountered are presented on the drill hole logs included in Appendix A.

In addition, geophysical refraction surveys were performed at the Reservoir B site to try to identify where and how deep volcanic rock might be located. The geophysical surveys were performed on November 29, 2017. Results of the surveys are discussed within this report and presented in Appendix C.

2.3 SITE CONDITIONS

2.3.1 Surface Conditions

2.3.1.1 Pump Station

The proposed pump station site is located on a landscaped and undeveloped slope located between the treated water storage tank (TWST) and a paved access road located along the southwest margin of the Operations Building and Chemical Storage Area. That slope is inclined at about 3:1 (horizontal to vertical) and covered with low grasses and shrubs, and numerous trees. Elevations at the site range from about 2,198 to 2,209 feet (WWE, 2017) and drainage occurs as sheetflow toward the southwest.

2.3.1.2 Pipeline Alignment

The proposed pipeline extends beneath existing paved roadways, unpaved roadway shoulders, or unpaved lands leading from Pine Needle Drive to Reservoir A. Asphaltic concrete (AC) paved surfaces of the WTP, Pine Needle Drive, Skyway, and New Skyway will be disturbed during construction of the pipeline. Unpaved shoulder areas along New Skyway, which are covered with gravel and/or soils, are present above the proposed pipeline in select locations.

Elevations along the pipeline alignment range from about 2,190 to 2,316 feet (WWE, 2017). Generally, drainage occurs across the proposed pipeline alignment as sheetflow that extends over slopes or is captured and diverted to storm drain improvements.

2.3.1.3 Reservoir B Tanks

The Reservoir B site is a developed facility that includes:

- The existing lined and covered reservoir;
- A 36-inch diameter water transmission pipeline;
- An existing structure;
- A sump and AC-paved area adjacent to the sump;
- Numerous fences; and
- An unpaved, gravel-lined access road.

The existing reservoir is about 320 feet long and 235 feet wide, pentagonal in shape, and was created by grading embankments around the reservoir perimeter. The embankments ascend from outside the reservoir to a perimeter access road surrounding the reservoir then descend into the reservoir basin. Those embankments range in height from about 4 to 13 feet and are inclined at about 4:1 to 2:1 on the exterior embankments and 1:1 within the reservoir basin. The top of the reservoir embankment is at about elevation 2,185 feet and the bottom of the reservoir is at an elevation of about 2,168 feet (WWE, 2017). The access road around the reservoir perimeter ranges in width from about 6 to 10 feet. The reservoir is lined and covered by a geosynthetic material and metal posts are present around the perimeter that support the cover materials. Motion detectors and light standards are also present at locations around the reservoir perimeter road. A concrete vault and pump control box are located at the southeastern end of the reservoir.

A CMU structure is present at the northern portion of the site near the entrance gate. This structure is single story, about 20 feet long and wide, and is surrounded by sidewalks and a low CMU retaining wall. A metal 40-foot long shipping container is present adjacent to the structure and another is present about 80 feet south of the structure.

A sump is present at the southwest margin of the facility. The sump is about 6 feet deep and accessed by an unpaved ramp. Adjacent to the sump is an irregularly-shaped AC-paved area.

The areas outside of the existing reservoir and structure are unpaved and generally fallow, except southwest of the reservoir where construction materials are stockpiled for use by PID. Piles of boulder-size columnar basalt are present at the southeastern portion of the facility and visible within the reservoir and sump embankment fills. Numerous trees are present within and surrounding the facility. Shrubs and hedges are present along the northeastern margin of the reservoir and locally in other fallow areas.

A chain-link fence surrounds the reservoir and an additional fence surrounds the entire facility.

2.3.2 Subsurface Conditions

2.3.2.1 Pump Station

The pump station site is underlain by artificial fill and ultramafic rocks (serpentinite), as shown on Plates 4.1 and 4.2 – Cross Sections A-A' and B-B', respectively. The artificial fill encountered in our explorations consists of clayey sand, sandy clay, and silty clay with varying amounts of gravel, cobbles, and boulders. It ranges in thickness from about 10 feet near the operations building to 18 feet thick or more at drill hole PS-2. It is anticipated to have a highly variable thickness across the proposed pump station site as evidenced in the photograph labeled "2/28/94 West Side Ops" within Taber (2015). That photo shows the undulatory and sloping serpentinite surface between the Operations Building and the TWST.

Underlying the artificial fill are ultramafic rock materials consisting of serpentinite. Those materials are buff to greenish grey, moderately weathered, moderately hard, poorly to moderately indurated, and slightly to highly fractured. Four feet of the serpentinite was penetrated using solid-stem auger drilling methods prior to experiencing practical refusal. It is unlikely that the full depth of the serpentinite will be penetrated during construction at this site.

2.3.2.2 Pipeline Alignment

The proposed pipeline alignment is underlain by a variety of subsurface conditions, as noted on Plates 3.2.1 through 3.2.2 – Geotechnical Map – Pipeline Alignment. The following table discusses anticipated subsurface conditions along the proposed pipeline alignment.

ANTICIPATED SUBSURFACE CONDITIONS - PIPELINE		
Approximate Stations[1]		Anticipated Subsurface Conditions
From	To	
10+00	14+00	Artificial fill containing coarse- and fine- grained sediments with varying amounts of gravel, cobbles, and boulders. Some serpentinite rock might be locally encountered.
14+00	24+00	Predominantly serpentinite. Locally, some difficult excavation conditions might be present. Some perched groundwater might also be present.
24+00	73+00	Alternating intervals of intact Tuscan Formation and artificial fill. Materials are anticipated to consist of coarse- and fine-grained sediments with localized areas of gravel and cobbles. Some cemented Tuscan Formation conglomerate and tuffaceous materials might pose moderate to difficult excavation characteristics.
73+00	82+33	Colluvial soils and Tuscan Formation. Some volcanic rock might be present locally but should not be ubiquitous. Some local perched water might be present.

[1] - From WWE (2018)

2.3.2.3 Reservoir B Tanks

The proposed tank site is underlain by silty clay, clayey silt, clayey sand, and silty sand with varying amounts of gravel, cobbles, and boulders, as shown on Plates 4.3 through 4.5 – Cross Section C-C' through E-E', respectively. Those materials range from dry to wet, soft to hard (if plastic) and medium dense to very dense (if granular). Fine-grained soils were slightly plastic to plastic. Granular soils have sand ranging from fine to coarse with most being fine to medium grained. Saprolitic fine to coarse gravels and cobbles were encountered locally as was woody organic debris.

2.4 GEOLOGIC CONDITIONS

2.4.1 Regional Geology

Regional geologic conditions are presented within the preliminary (desktop) geotechnical report (VSI, 2017) presented in Appendix B and will not be repeated, herein. Plate 5 – Regional Geologic Map, shows the mapped geologic conditions presented by Saucedo & Wagner (1992).

2.4.2 Local Geologic Setting

2.4.2.1 Pump Station

The proposed pump station site is underlain by artificial fill and ultramafic rock materials, as shown on Plate 3.1 and Plate 5. The artificial fill materials are present on the descending slope located east of the Operations Building/Chemical Storage area and the TWST. The artificial fill material sits unconformably on ultramafic rocks, which consist of serpentinite. The proposed pump station is anticipated to rest on the serpentinite and should not fully penetrate those materials.

2.4.2.2 Pipeline Alignment

The proposed pipeline alignment extends across artificial fill, ultramafic rocks, Tuscan Formation, and possibly the Olivine basalt of Paradise, as shown on Plate 5 (regional) and Plates 3.2.1 and 3.2.2 (pipeline). Artificial fill materials are present within the WTP and at various locations along New Skyway. Ultramafic rock materials are present at the WTP and along Pine Needle Drive. The Tuscan Formation is present along New Skyway and as the pipeline extends towards Reservoir A. Locally, intact volcanic flow units of the Olivine Basalt of Paradise might be encountered along the alignment, especially south of the southern intersection of Skyway and New Skyway.

2.4.2.3 Reservoir B Tanks

The proposed tank sites are mapped as being underlain by Pliocene-age basaltic rocks (Saucedo & Wagner, 1992), as shown on Plate 5. Those materials are reported to consist of olivine basalt that is grey and vesicular, with a glomeroporphyritic texture (plagioclase crystals are clustered into phenocryst groupings; Helley & Harwood, 1985). Those basaltic rocks have been mapped across the entire project region surrounding the proposed tanks site.

Exploration performed by Clendenen (1985), as shown on Plate 3.3, reported columnar basalt along the entire northeastern margin of the reservoir. As-built documents (Clendenen, 1985) also show columnar basalt beneath the northeastern and southwestern berms forming the existing reservoir. Clendenen (1984) performed test pits within the reservoir footprint but the logs of those test pits were not available for this report.

Despite the findings reported by Clendenen (1985) and Saucedo & Wagener (1992), no intact volcanic flow deposits were encountered during this study at the site, as shown on Plates 3.3, and 4.3 through 4.5. Only sedimentary deposits were encountered in explorations advanced for this study. It is our opinion that these sedimentary deposits are part of the Tuscan Formation (Saucedo & Wagner, 1992). Those Cenozoic-age deposits consist of interbedded conglomerates, breccia, sandstone, tuff, and siltstone deposited by lahars and debris flows (Helley & Harwood, 1985; Smith et al., 2007; Staton et al., 2014) that exceed 1,700 feet in thickness. Woody debris and detritus are locally present within these sediments (Helley & Harwood, 1985) and were encountered in our samples.

As noted above, no intact volcanic rocks were encountered in our subsurface exploration or inferred from the geophysical surveys, all of which were performed around the site perimeter due to the presence of the existing reservoir. The geophysical surveys estimated that seismic velocities of underlying earth materials to a depth of up to about 60 feet were less than 3,800 feet per second (f/s); however, basaltic flow deposits would be anticipated to have a seismic velocity in excess of 8,000 f/s (Redpath, 2017). Thus, we did not observe the presence of intact volcanic rock in our subsurface explorations.

That being said, there is evidence that intact volcanic flow deposits consisting of columnar basalt was, and possibly still is, locally present within the footprint of the existing facility. That evidence includes:

- Abundant columnar basalt boulders exposed within the existing reservoir embankments and present locally across the site. This implies that rock was encountered during construction of the reservoir;
- PID indicated that the 36-inch water transmission main pipeline extending across the site has an unusual arcuate alignment because the contractor constructing the pipeline wanted to avoid basaltic rock materials (Neil Essila, email correspondence, 2017);
- A discussion with the neighbor located south of the existing reservoir indicated that the volume of basaltic boulders stockpiled at the site was at one time much greater but much of those materials had been removed from the site. In addition, the neighbor indicated that a PID employee present during construction of the existing reservoir reported to him that intact volcanic flow deposits had been encountered during construction of the reservoir and were the source of the boulders at the site.

Thus, it is likely that volcanic flow deposits were, and possibly are, present within the footprint of the existing reservoir. Because the existing reservoir is in use and is lined with a geosynthetic liner, it was not possible to explore within the footprint of the reservoir during this study to confirm the presence or absence of intact volcanic flow deposits.

If intact volcanic flows were present at the site, then those rocks may have been partially or fully removed during construction. Alternatively, they could still be present within the footprint of the reservoir basin in areas inaccessible for exploration. Plate 6 – Volcanic Rock Scenarios, provides a few illustrations regarding how intact volcanic rock may have been dealt with during construction of the existing reservoir. Of those illustrations, Scenario 1 seems the most likely since no indications of intact volcanic rock were observed in any of our explorations or geophysical surveys. Volcanic rock materials that might still be present under Scenarios 2 and 3 should have been observed in some of our subsurface explorations and/or with the geophysical refraction surveys. The greatest uncertainty is in regard to whether intact volcanic rock is still present within the basin of the existing reservoir, which cannot be resolved at this time. Following the removal of the existing reservoir and prior to the start of construction, additional exploration will be required to determine the presence and extent of any intact volcanic rock under the footprint of the proposed Reservoir B Tanks.

2.4.3 Groundwater

2.4.3.1 General

Groundwater elevations at project improvement locations 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 any necessary measures to mitigate adverse effects of groundwater on proposed construction activities.

The following sections discuss groundwater encountered within explorations advanced for this study, and by Taber (2015) and Kleinfelder (1992).

2.4.3.2 Pump Station

Groundwater was not encountered in explorations advanced for this study at the proposed pump station site. No groundwater was encountered in explorations advanced by Taber (2015) at the WTP site. Kleinfelder (1992) reported minor seepage at a depth of about 5.5 feet at their test pit TP-3, which was located at the southwest side of the TWST. No other observations of groundwater were reported during the Kleinfelder study.

2.4.3.3 Pipeline Alignment

Groundwater was not encountered during explorations advanced along the pipeline alignment for this study. No other subsurface data along the proposed alignment was available during this study.

2.4.3.4 Reservoir B Tanks

Groundwater was observed in drill holes T2 and T3 at the tank site during this study. In T2, water was measured at a depth of 7 feet approximately 24 hours following completion of drilling that hole. This reading may not be accurate because heavy rains occurred during that time period and surface water may have entered the drill hole prior to measurement. In drill hole T3, groundwater was encountered at a depth of about 20 feet.

There is a potential that both of these groundwater zones represent perched water conditions and not a continuous groundwater table. This is suggested by the geophysical refraction surveys which did not identify a subsurface zone having a seismic velocity of about 5,000 feet, which would correspond to a groundwater table (see Appendix C).

3 GEOLOGICAL HAZARDS

Geologic hazards are discussed in VSI (2017), presented in Appendix D, and will not be discussed, herein, unless this study has modified findings presented within the preliminary (desktop) geotechnical report.

3.1 CBC SEISMIC DESIGN RECOMMENDATIONS

We understand that the proposed tank will be designed and constructed under the 2016 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**	
		Pump Station	**Tank Site**
Site Coordinates	Latitude	39.814538°	39.784963°
	Longitude	-121.581817°	-121.595223°
Section 1613.5.3 Table 1613.5.3(1)	Site Coefficient, F_a	1.243	1.223
Section 1613.5.3 Table 1613.5.3(2)	Site Coefficient, F_v	2.022	1.500
Section 1613.5.1 Figure 1613.5	Site Class Designation	D	C
	Seismic Factor, Site Class B at 0.2 Seconds, S_s	0.652g	0.643g
	Seismic Factor, Site Class B at 1.0 Seconds, S_1	0.263g	0.265g
Section 1613.5.3	Site Specific Response Parameter for Site Class C at 0.2 Seconds, S_{MS}	0.833g	0.735g
	Site Specific Response Parameter for Site Class C at 1.0 Seconds, S_{M1}	0.493g	0.407g
Section 1613.5.4	$S_{DS}=2/3S_{MS}$	0.555g	0.490g
	$S_{D1}=2/3S_{M1}$	0.328g	0.272g
Per the 2016 CBC			

3.2 SITE-SPECIFIC RESPONSE SPECTRA

The project site is not located within 10 kilometers of an active fault zone. The closest active fault (Indian Valley fault) is located about 40 miles northeast of the site near Crescent Mills, California. Because of this, a site-specific seismic hazard analysis is not required, per Chapter 11 and Chapter 21 of ASCE 7-10 and the American Water Works Association (AWWA) Standard for welded steel tanks for water storage.

3.3 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. Plasticity Index (PI) tests performed for this study resulted in the following PI values:

PLASTICITY INDEX TEST RESULTS			
Location	**Exploration Location**	**Depth (ft)**	**Plasticity Index**
Reservoir B	T1	5	25
Reservoir B	T4	5	37
Kleinfelder?	TP-1	11	34

In addition, Kleinfelder (1992) performed one PI test and reported a value of 34. That value was obtained at the WTP from a location about 200 feet northwest of the pump station site.

The PI values performed for this study and by Kleinfelder (1992) indicate that soils in the Reservoir B area have medium to very high expansion potentials, as noted in the following table (Day, 1999):

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)	

3.4 SOIL CHEMISTRY

Selected samples of near-surface soils encountered at the pump station and tank sites were subjected to chemical analysis for assessment of corrosion and reactivity with concrete. The samples were 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						
Site	**Exploration & Depth (ft)**	**Soil Type**	**Sulfates (ppm)**	**Chlorides (ppm)**	**pH**	**Resistivity (ohms-cm)**
Tanks	T-1, 3'-5'	Silty Clay	16	<2	6.1	23,653
Pump Station	PS-1, 1'-5'	Sandy Clay	96	<2	6.6	5,299

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.

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 tested for this study are estimated to be mildly to moderately 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 than the samples tested for this study.

3.5 NATURALLY OCCURRING ASBESTOS

Ultramafic rock, such as serpentinite, amphibolite, peridotite, dunite, pyroxenite, hornblendite, etc., can contain asbestiform minerals, which are fibrous, silica-rich crystals that can cause lung cancer, mesothelioma, asbestosis, and other health-related issues, if present. Typically, six minerals within ultramafic rocks are responsible for the primary, naturally occurring asbestiform concerns for health-related issues: chrysotile, tremolite, actinolite, anthophyllite, crocidolite, and amosite. These minerals may or may not be present in ultramafic rocks; thus, the presence of ultramafic rock does not automatically indicate that there is a health hazard. The presence of asbestiform minerals can sometimes be discerned in the field based on visual examination of rock exposures but, most often, must be confirmed using laboratory testing.

Naturally occurring asbestos can be hazardous to human health if it is disturbed, becomes airborne and is inhaled. If NOA is not disturbed and fibers are not released into the air, then it is typically not considered a health hazard. Inhalation is the primary exposure route of concern, because breathing asbestos fibers may cause them to become trapped in the lungs. Ingestion is another, albeit less common, pathway of concern, because swallowing asbestos fibers may also cause the fibers to be trapped in body tissues. Asbestos is not absorbed through the skin, so merely touching it does not pose a significant risk to human health. Asbestos fibers are not water soluble and do not move through groundwater to any appreciable extent. Based on studies of other insoluble particles of similar size, the expected migration rate of an asbestos fiber through soils by the forces of groundwater is approximately 1 to 10 centimeters (0.4 to 4 inches) per 3,000 to 40,000 years (New Hampshire DES, 2010). Thus, asbestos is not considered a groundwater contaminant.

In California, NOA is considered a concern if it exceeds a concentration of more than 0.25-percent (CGS, 2002). If NOA concentrations exceed that threshold, then mitigation measures are typically required to reduce the potential of inducing NOA to become aerosol.

As discussed in Section 2.4.2.1 and 2.4.2.2, ultramafic rocks in the form of serpentinite and chloritized serpentinite were observed at the pump station and a portion of the pipeline alignment. Ultramafic rocks were not observed at the Reservoir B site nor along the majority of the pipeline alignment.

One soil sample each was obtained from the pump station site and from about Station 18+00 along the proposed pipeline alignment. Those samples were transmitted from our Redding office to Asbestos TEM Laboratories, Inc., to perform testing for the potential presence of NOA. The Chain of Custody form used to transmit samples is included in Appendix B – Laboratory Testing. Testing was performed on each sample using a polarized light microscope with a point count of 400 in conformance with standard test method CARB 435. Results of the laboratory testing found that NOA was not present in the sample obtained from soils at the proposed pump station site but that NOA exceeded 6% within the sample obtained from the proposed pipeline alignment. That value exceeds the concentration of 0.25%, noted above and is considered a risk to health unless measures are taken to keep asbestos from becoming airborne. This is discussed in greater detail in Section 5.2, below.

Results of the testing are included in Appendix B.

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	34	ASTM D2216
Sieve Analysis with #200 Wash	4	ASTM D422
Atterberg Limits	2	ASTM D4318
Modified Proctor	4	ASTM D1557
Consolidation	4	ASTM D2435
Direct Shear	2	ASTM D3080
Naturally Occurring Asbestos	2	CARB 435
Soil Chemistry	2	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 4.1 to 57.1 with an average of 33.4. The average moisture content for the tank, pump station, and pipeline alignment was 39.1, 12.1, and 16 percent, respectively.

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 61.7 to 118.6 with an average of 83.9 pounds per cubic foot (pcf). The average dry density for the tank, pump station, and pipeline alignment was 75.4, 104.8, and 98.2 pcf, respectively.

4.2.2 Grain-Size Distribution

Grain-size distributions were performed on three selected samples during this study. The samples tested had a range of about 6 to 68 percent passing the No. 200 sieve. The average amount passing the No. 200 sieve was 43.3 percent.

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
T1	5	50	25
T4	5	58	37

4.2.4 Maximum Density/Optimum Moisture Content

Maximum density and optimum moisture content tests were performed on four selected bulk samples obtained during this study. The maximum densities obtained from these tests ranged from 94.1 to 138.1 pcf and optimum moisture contents of 9.0 to 26.1 percent.

4.3 STRENGTH & VOLUMETRIC TESTING

4.3.1 Direct Shear Tests

Two consolidated, drained, direct shear tests (ASTM D3080) was performed on a selected sample collected during this study. The results indicate that the samples had a cohesion intercept (C) range of 150 to 400 psf with an angle of internal friction (Ø) value range of 37.8 to greater than 45 degrees.

4.3.2 Consolidation Tests

The consolidation characteristics of selected foundation soils were estimated by performing one-dimensional consolidation on four 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 sites explored are suitable for the proposed improvements at those sites, as discussed in Section 1.2 of this report, 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 WWE along with stated assumptions. Changes in the configurations from those studied during this investigation may require supplemental recommendations.

5.2 GEOLOGIC HAZARDS

With the exception of expansive soils and naturally occurring asbestos, geologic hazards appear to pose a relatively low risk to the proposed project elements. Expansive soils and NOA are discussed in greater detail, below.

5.2.1 Expansive Soils

Soils with a medium to very high expansion potential are present at the Reservoir B project site. See Section 3.3 of this report for a description of those soils. 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 expansive soils that may be present underlying the tanks. 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. Similarly, overexcavation and removal of existing fill material under the proposed Pump Station will remove any potential expansive soil from under the structure.

5.2.2 Naturally Occurring Asbestos

As discussed in Section 3.5 above, NOA in excess of the threshold of 0.25% was encountered in the serpentinite tested along the proposed pipeline alignment. NOA was not detected in soils from the pump station site; however, those soils tested were colluvium and/or artificial fill resting above serpentinite. Thus, it is likely that regolithic soils of the serpentinite and serpentinite underlying the proposed pump station could also have NOA concentrations exceeding 0.25%. Thus, these rock and soil materials should be considered relatively hazardous to human health and best management practices (BMPs) to limit the ability of those soils to become aerosol during construction and/or disposal, should be implemented.

It is recommended that the Contractor utilize an environmental specialist or industrial hygienist with experience in working with NOA to develop a work plan for use during construction. That plan should be prepared and submitted to the Construction Manager prior to initiation of construction. It should include training for on-site personnel, methods to limit NOA exposure to PID staff at the WTP, BMPs for limiting the ability for asbestos to become airborne, testing and monitoring, etc.

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 pump station and Reservoir B areas 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 around the existing reservoir and along portions of the proposed pipeline alignment that are 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 development areas of the project elements. When buried improvements are encountered during construction, they should be removed and/or rerouted beyond construction limits, where possible. 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

No keyways or benching are anticipated to be required as part of construction.

5.3.4 Scarification and Compaction

Following site stripping and any overexcavation (as recommended in Sections 5.4 and 5.5 of this report), 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 and following the winter season, 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, mitigation measures such as 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 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 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 project sites was performed using CME-75 drill rigs utilizing 4.25-inch diameter solid-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 serpentinite at the proposed pump station site. It is our opinion that the underlying serpentinite should be excavatable with heavy grading equipment with moderate to high difficulty.

Exploration along the proposed pipeline alignment and at Reservoir B was performed with little to moderate difficulty. Where cobbles and boulders were encountered, some difficult

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

drilling conditions were experienced. Unless intact volcanic rock or large boulders are encountered, it is our opinion that excavation at Reservoir B and trenching along the proposed pipeline alignment should be possible using heavy grading equipment with moderate difficulty. If intact volcanic flows or large boulders are encountered, difficult excavation conditions could be experienced. As discussed in Section 5.4 of this report, additional site exploration is recommended at the Reservoir B site to evaluate the presence/extent of basaltic rock under the proposed tanks.

Blasting and other relatively unconventional excavation methods are not anticipated as necessary for these sites.

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			
Location	**Material**	**Shrinkage**	**Bulking**
Pump Station	Artificial Fill/ Soils	3% to 5%	
	Serpentinite		3% to 5%
Pipeline	Artificial Fill	1% to 3%	
	Serpentinite		3% to 5%
	Tuscan Formation		1% to 3%
Tank	Tuscan Formation		1% to 3%

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. However. this number could locally be larger, particularly at the Reservoir B site if large pieces of rock are found to be present under the proposed tanks. These factors should be included in volume calculations for on-site soils that are excavated and then recompacted 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 5.3.17 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.

Based on the direct shear data obtained from our laboratory testing and stability analyses performed for this study, we estimate that temporary construction should be stable up to an inclination of 0.5:1 at a height of less than 25 feet, provided no groundwater is exposed in those slopes. If groundwater is exposed, we recommend that an inclination of 1:1 be utilized for temporary cut slopes. We recommend that efforts be made during construction to limit exposure of temporary slopes 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 1.5:1 or flatter. If proposed unsupported cut slopes cannot be excavated at 1.5:1 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 1.5:1 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

Overexcavation is not anticipated as necessary for the proposed pipeline. For overexcavation recommendations for the proposed tanks and pump station, refer to Sections 5.4.2 and 5.5.2, respectively, of this report.

5.3.11 On-Site Soil Materials

It is our opinion that most of the near-surface soils encountered at the pump station and tank sites and along the proposed pipeline alignment 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

Sections 5.4.3 and 5.6.5 discuss more specific imported fill materials recommendations related to the proposed tank site and pipeline alignment, respectively.

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. Specific granular backfill material requirements for pipelines are presented in Section 5.6.5 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 material, we recommend it conforms and be placed per specifications presented in Section 19-3.062 of the Caltrans Standard Specifications (most current edition).

5.3.15 Placement & Compaction

This section provides general placement and compaction recommendations. If more stringent recommendations are made in Sections 5.4, 5.5, and/or 5.6, then those recommendations take precedent over those made in this section.

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], unless noted otherwise within this report. 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.

5.3.16 Frost Penetration

Frost heave is not typically a hazard in the Paradise and Magalia areas of Butte County. Therefore, no recommendations for frost protection have been provided herein.

5.3.17 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,

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

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 for the project sites are anticipated to be the following:

ANTICIPATED OSHA SOIL TYPES	
Location	**Soil Type**
Pump Station	A
Pipeline	A/B
Tanks	B

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 would 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.

5.3.18 Shoring

Preliminary design of braced shoring for trenches may be based on the preliminary shoring pressure diagrams provide on Plate 7 - Preliminary Shoring Pressure Diagrams. The preliminary shoring pressure diagrams provided on Plate 7 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. However, if thick layers of cohesionless materials are encountered during construction, 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)		

5.4 RESERVOIR B TANKS

5.4.1 Summary of Tank Foundation Design Recommendations

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

SUMMARY OF TANK FOUNDATION DESIGN RECOMMENDATIONS	
Foundation Element	**Recommended Value**
Shallow Perimeter Foundation System	
Anticipated foundation materials:	AB/MSE Raft
Minimum embedment depth:	18 inches
Allowable bearing pressure:	1,500 psf
Passive Pressure & Coefficient of Friction	
Ultimate Passive Pressure:	350 pcf
Base coefficient of friction:	0.35
Estimated Settlement	
Center of Tank:	4.9" – 8.1"
Edge of Tank:	1.3" – 1.6"

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

5.4.2 Additional Pre-construction Subsurface Exploration

As noted above in 2.4.2.3 Reservoir B Tanks, all of our subsurface exploration was performed around the site perimeter due to the presence of the existing reservoir. Contrary to what was depicted on the as-built plans for the reservoir, no intact volcanic rocks were encountered in our subsurface exploration or inferred from the geophysical surveys. Therefore, we recommend that additional exploration be performed after the existing reservoir is removed and before construction begins to confirm the actual conditions under the proposed tanks and to confirm or re-evaluate the foundation design recommendations made in this report, as may be necessary. The additional exploration program should consist of a combination of trenching and borings and should be planned and executed by VSI.

5.4.3 Overexcavation/Transition Lots

Transitions lots are those lots where a structure foundation will be supported partially by two different geologic materials, such as when a structure foundation is supported on artificial fill beneath one portion of the structure and a cemented rock beneath the remainder of the structure. Those two materials will settle at differing rates and magnitudes and could cause damage to the structure, structure performance, or performance of equipment within the structure due to differential settlement.

We recommend that the tanks and other structure foundations be founded entirely within native, undisturbed soils or competent rock. Structure foundations should not be founded on a combination of undisturbed Tuscan Formation and engineered fill materials (i.e., a transition lot condition). For the proposed tank sites, transition lots were not originally anticipated, as shown on Plates 4.3 through 4.5. However, if a transition lot is found to be present beneath the tanks or 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, except that all backfill should be compacted to 95% relative compaction;

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.

5.4.4 MSE Raft

VSI recommends the construction of a mechanically stabilized earth composite raft foundation (MSE raft) to provide a more uniform bearing layer immediately under the tanks and to help reduce total and differential settlement. 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 both the tanks and a minimum of 5 feet horizontally beyond the outside edge of the tank foundations. Plate 8 – Geosynthetic Composite Raft Foundation Illustration, provides details of the MSE raft.

5.4.5 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.

5.4.6 Allowable Bearing Pressures

It is assumed that all foundations for the proposed structures will rest entirely on either a uniform thickness of engineered fill or undisturbed Tuscan 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 1,500 pounds per square foot (psf). More specific bearing pressure recommendations can be provided, if desired, once further details of the structures are known. We have estimated that the proposed tanks will apply a total bearing pressure of less than 1,500 psf based on a water depth of 20 feet plus the weight of the tank itself. The perimeter ring footing should be designed to apply the same pressure of 1,500 psf or less.

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 2016 CBC, is applied. We recommend that VSI be allowed to observe foundation excavations to confirm projected site conditions.

5.4.7 Estimated Tank Settlements

Potential settlement was evaluated using SETTLE3D, developed by Rocscience (2012) and the data obtained from the geotechnical borings performed around the perimeter of the site. The results of the analyses indicate that center/edge tank settlements could vary from 5”/1.3” to 8.1”/1.6” for the range of conditions considered. This implies a range of differential settlements between the center of the tank and the edge of the tank of approximately 3.7” to 6.5”. As stated in previous sections of this report, VSI recommends that additional exploration be performed after the existing reservoir is removed and before construction begins to confirm the actual conditions under the proposed tanks and to re-evaluate the settlement if necessary.

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 resting on approved, properly prepared subgrade material. The 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.

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 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	60
Active	Flat	40
At-Rest	2:1	83
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 below-grade wall. 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 PUMP STATION

5.5.1 Summary of Pump Station Design Recommendations

The following table provides a summary of foundation design recommendations made in Section 5.5.

SUMMARY OF PUMP STATION DESIGN RECOMMENDATIONS	
Foundation Element	**Recommended Value**
Shallow Foundation System	
Anticipated foundation materials:	Structural Fill/Serpentinite
Minimum embedment depth:	18 inches
Allowable bearing pressure:	2,000 psf
Passive Pressure & Coefficient of Friction	
Ultimate Passive Pressure:	350 pcf
Base coefficient of friction:	0.30 to 0.40
Estimated Settlement	
Total:	<1.0”
Differential:	<0.5” in 50 ft
Slab-on-Grade	
Modulus of subgrade reaction (K_{1s})	250 pci

5.5.2 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.

The structure should be constructed so that the foundation systems rest completely on either approved structural fill or serpentinite bedrock. As depicted on Plates 4.1 and 4.2, it is anticipated that the bottoms of the pump cans will rest on serpentinite; however, the bottom of the foundation system for the remainder of the pump station may or may not extend down to the serpentinite bedrock. If excavations for the pump station building foundation system do not expose moderately weathered to fresh serpentinite, we recommend that the foundation excavations be deepened to remove all existing fill so that moderately weathered to fresh serpentinite is exposed. The foundations can then be constructed either on the exposed bedrock surface, or a two-sack sand-cement slurry can be poured up to the planned bottom elevations of the foundation system at which the foundations can be constructed.

A representative of VSI should observe and approve all Pump Station foundation excavations prior to placement of slurry and/or concrete.

5.5.3 Allowable Bearing Pressures

It is assumed that all foundations for the proposed structures will rest entirely on serpentinite or two-sack sand cement slurry established directly on the serpentinite, as discussed in Section 5.5.2. Isolated and continuous footing elements should be proportioned for dead loads plus probable maximum live load, and a maximum allowable bearing pressure of 2,000 pounds per square foot (psf). More specific bearing pressure recommendations can be provided, if desired, once further details of the structures are known.

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 2016 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.

5.5.4 Estimated Settlement

The anticipated total settlement for the pump station if construction occurs as recommended within this report, is estimated to be less than one inch. Differential settlement is estimated to be approximately ½-inch or less vertically over a horizontal distance of about 50 feet.

5.5.5 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 resting on approved, properly prepared subgrade material. The 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.

5.5.6 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	60
Active	Flat	40
At-Rest	2:1	83
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 below-grade walls. 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 Sections 5.5.9, 5.5.10, and 5.5.11, respectively.

5.5.7 Drainage Measures

Finish surface grades should be sloped away from the structure 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.

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 9 – 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 designed 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.

5.5.8 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.

5.5.9 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 foundations and slab on grade. If a membrane, such as polysheeting or PVC, is utilized between the pump station 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.

5.5.10 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.

5.5.11 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

5.5.12 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 PIPELINE & 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 10 – 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 11 – 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)**
Artificial fill	400
Tuscan Formation	1,000
Undisturbed Serpentinite	2,500

Anticipated locations of those earth materials are shown on Plates 3.2.1 through 3.2.2.

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, an ultimate 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

See Section 5.3.17 for a discussion regarding excavation and trench slopes.

5.6.4.2 Dewatering

Groundwater was not encountered within explorations advanced along the proposed pipeline alignment for this study. If construction is performed during winter or early spring or following a wet weather season, then shallow groundwater could be encountered within construction depths including areas in our explorations where groundwater was not observed. 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

See Section 5.3.18 for a discussion regarding a discussion regarding shoring considerations.

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 12 - 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 12 inches over the crown of the pipeline, and from trench wall to trench wall, as shown on Plate 12. In accordance with PID standards (PID, 2013), pipe zone backfill materials should consist of imported aggregate or sand having the following characteristics:

- Aggregate materials – 1.5" maximum size; or
- Sand materials – ¼-inch minus. VSI recommends that sand pipe zone materials have an SE of no less than 30.

Soils excavated along the pipeline alignment 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 12) 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

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 float rock 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 float rock 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 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.

7 ADDITIONAL SERVICES

This report and its associated recommendations were intended to assist WWE during design 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.

8 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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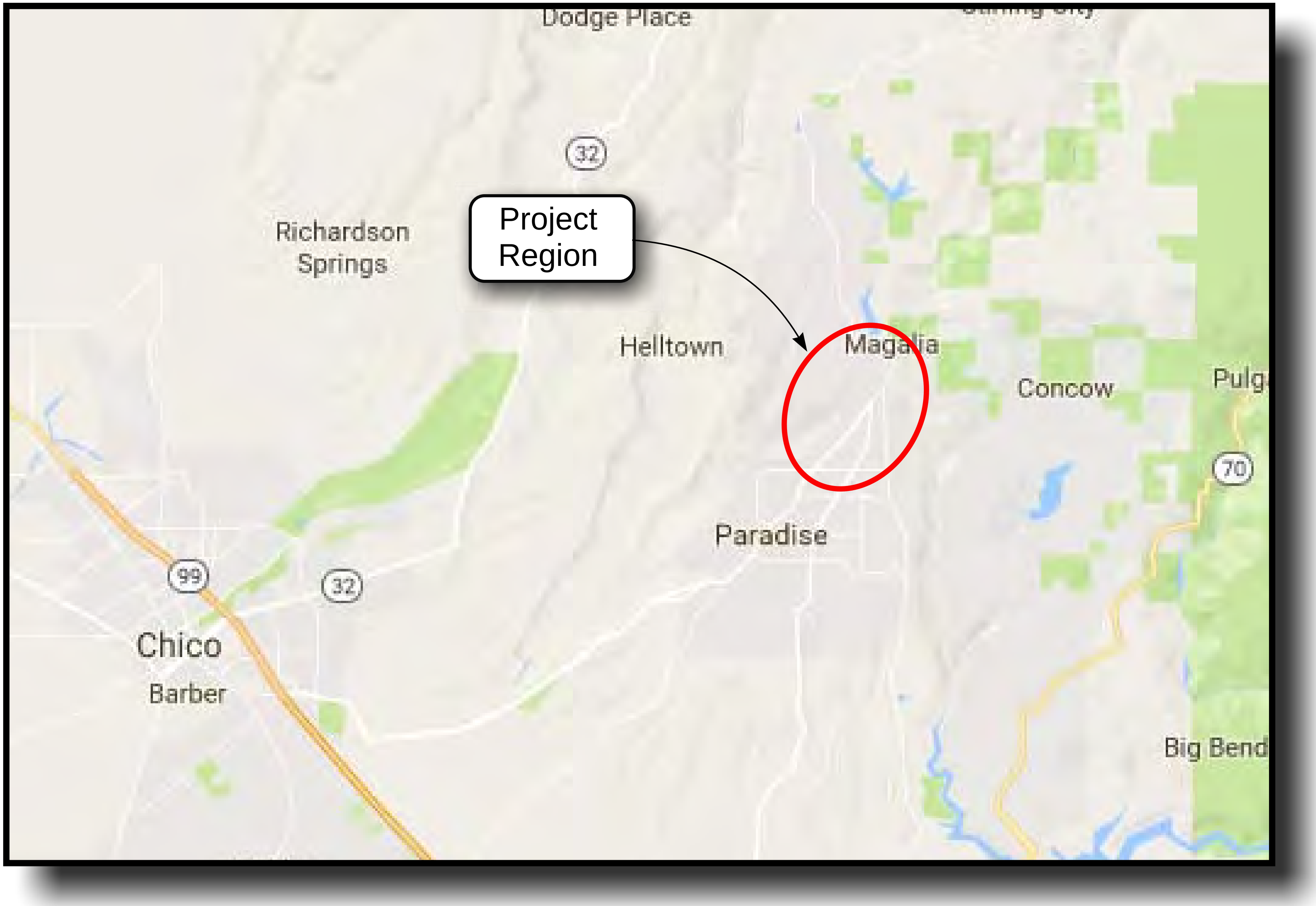


SITE LOCATION MAP

Reservoir B Replacement
Design-Level Study
Water Works Engineers
Paradise & Butte Co., California

Plate No. 1

VERTICAL SCIENCES, INC.

Project no. 170025

Scale undetermined
Base maps derived from Google Maps.

PROJECT ELEMENTS

Reservoir B Replacement Design-Level Study Water Works Engineers Paradise & Butte Co., California	Plate No. 2
VERTICAL SCIENCES, INC.	Project no. 170025

Scale undetermined
Base maps derived from Google Maps.

PS-2 Approximate Drill Hole Location this study

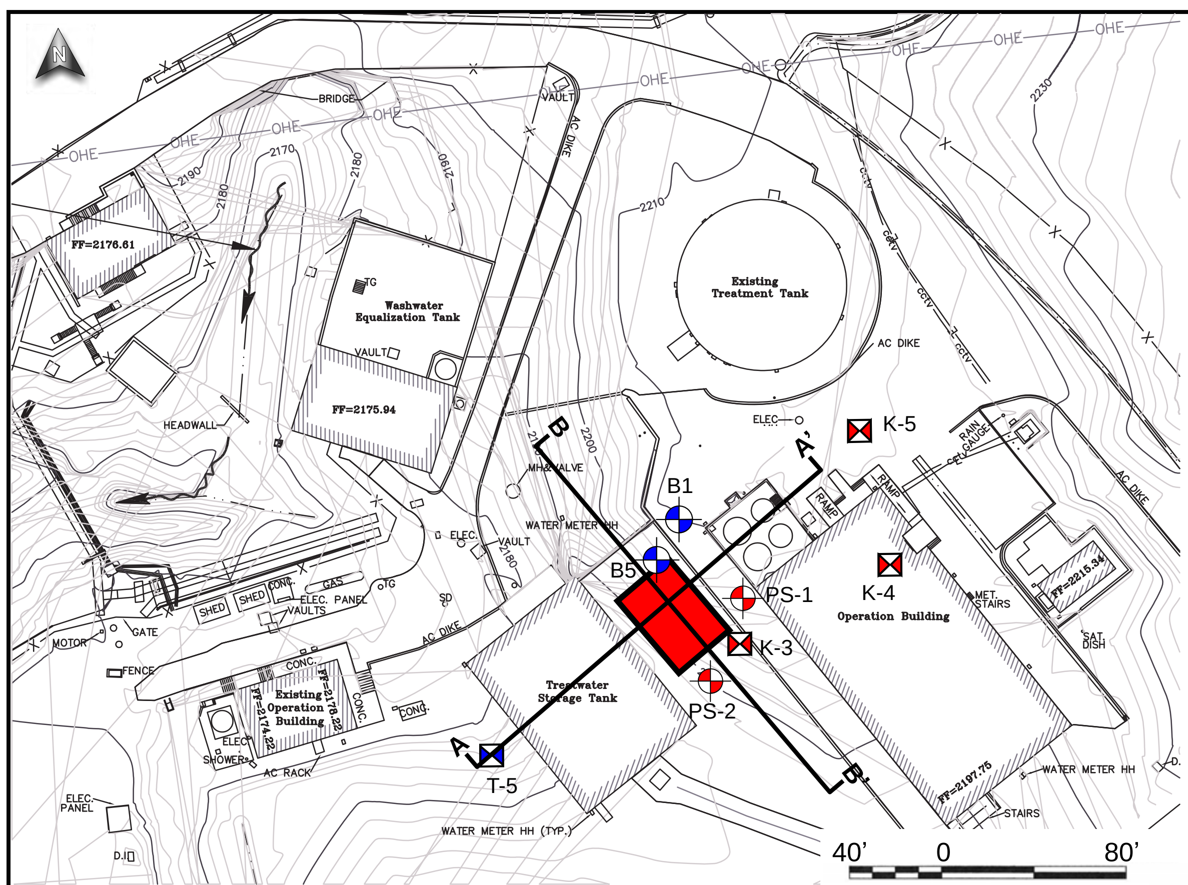
B-5 Approximate Drill Hole Location Taber (2015)

K-5 Approximate Test Pit Location Kleinfelder (1992)

T-5 Approximate Test Pit Location Taber (2015)

B B' Geotechnical Cross Section
See Plates 4.1 and 4.2 for Cross Sections

Formation Contact
Dashed where inferred & short dashed where concealed



af Artificial Fill

um Ultramafic Rocks Predominately Serpentinite

Scale undetermined
Base maps derived from Taber (2015) and WWE (2017)

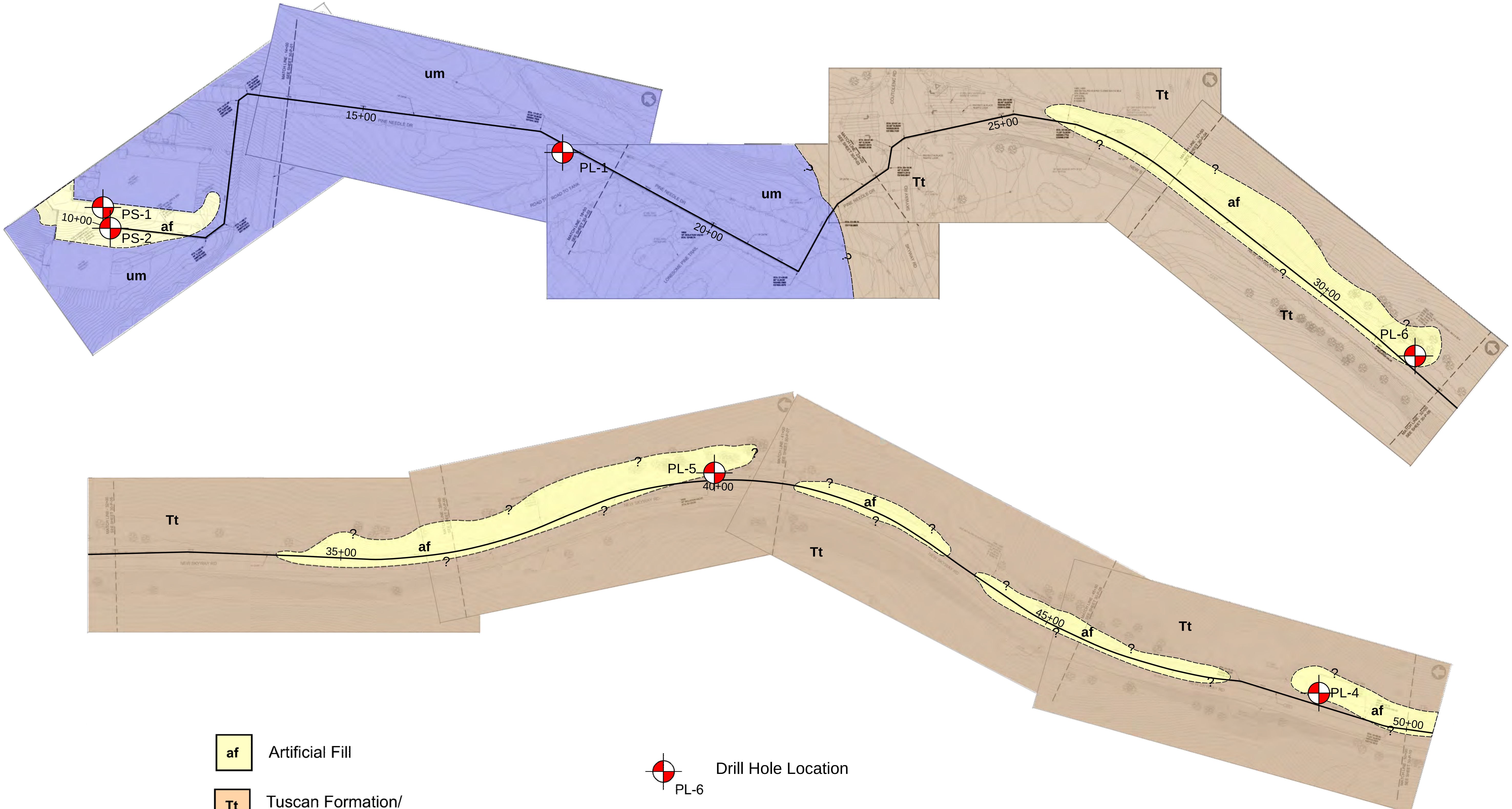
GEOTECHNICAL MAP - PUMP STATION

Reservoir B Replacement
Design-Level Study
Water Works Engineers
Paradise & Butte Co., California

Plate No. 3.1

VERTICAL SCIENCES, INC.

Project no. 170025



af	Artificial Fill
Tt	Tuscan Formation/ Volcanic Flows?
um	Ultramafic Rocks Predominately Serpentinite

PL-6 Drill Hole Location

Formation Contact
Dashed where inferred & short dashed where concealed
Queried where uncertain

GEOTECHNICAL MAP - PIPELINE

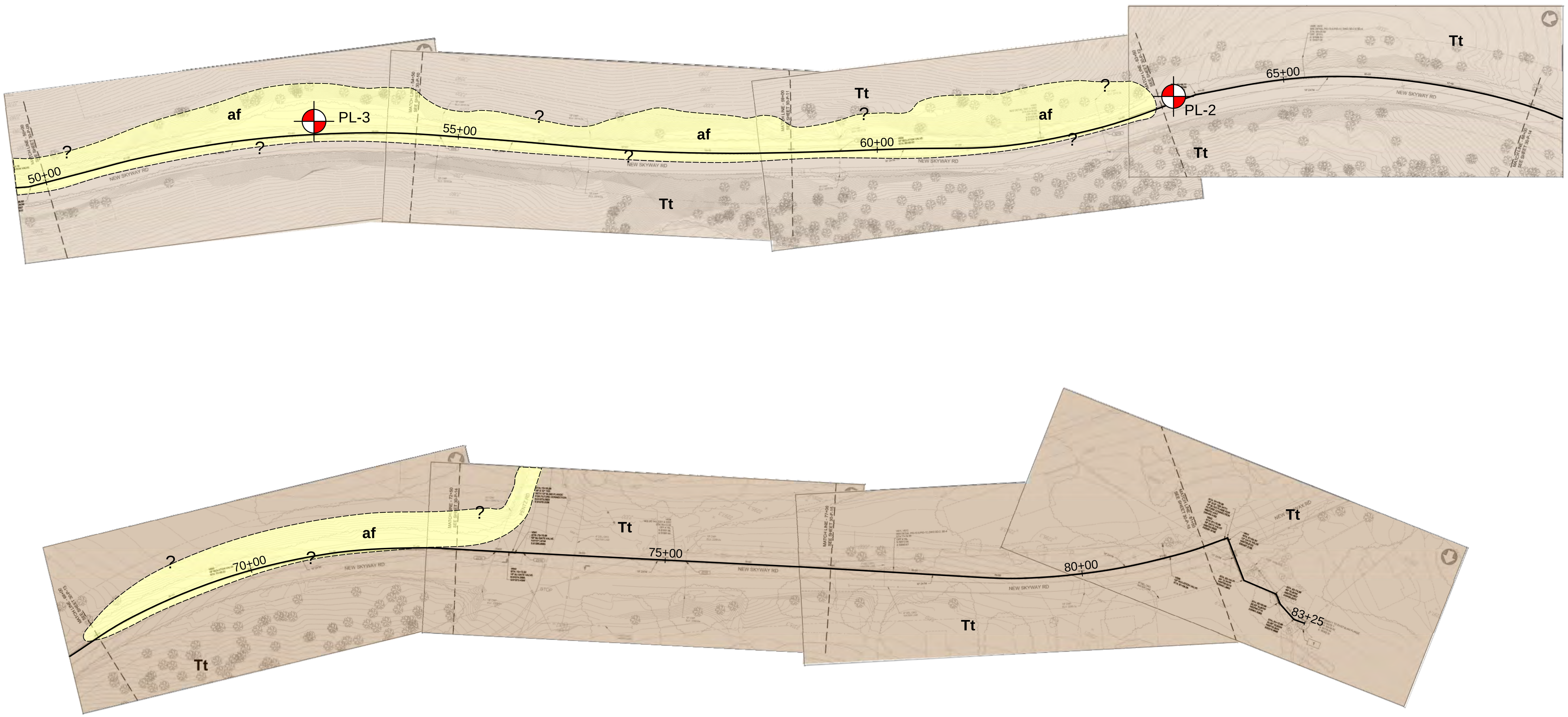
Reservoir B Replacement
Design-Level Study
Water Works Engineers
Paradise & Butte Co., California

Plate No.
3.2.1

VERTICAL SCIENCES, INC.

Project no.
170025

Scale undetermined
Base map derived from WWE (2018)



af — Artificial Fill

Tt — Tuscan Formation/
Volcanic Flows?

um — Ultramafic Rocks
Predominately Serpentinite

PL-6 — Drill Hole Location

Formation Contact
Dashed where inferred &
short dashed where concealed
Queried where uncertain

Scale undetermined
Base map derived from WWE (2018)

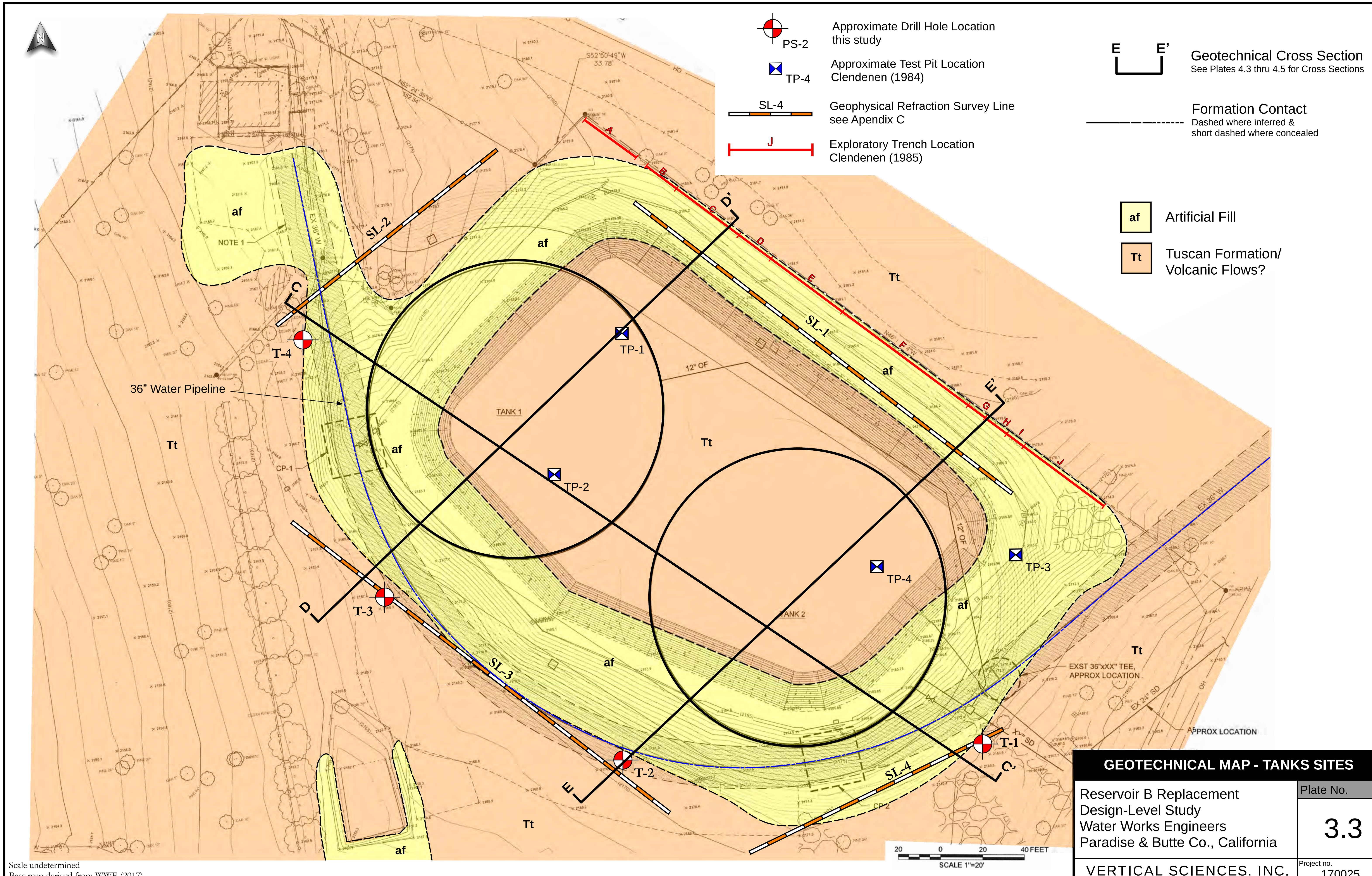
GEOTECHNICAL MAP - PIPELINE

Reservoir B Replacement
Design-Level Study
Water Works Engineers
Paradise & Butte Co., California

Plate No.
3.2.2

VERTICAL SCIENCES, INC.

Project no.
170025



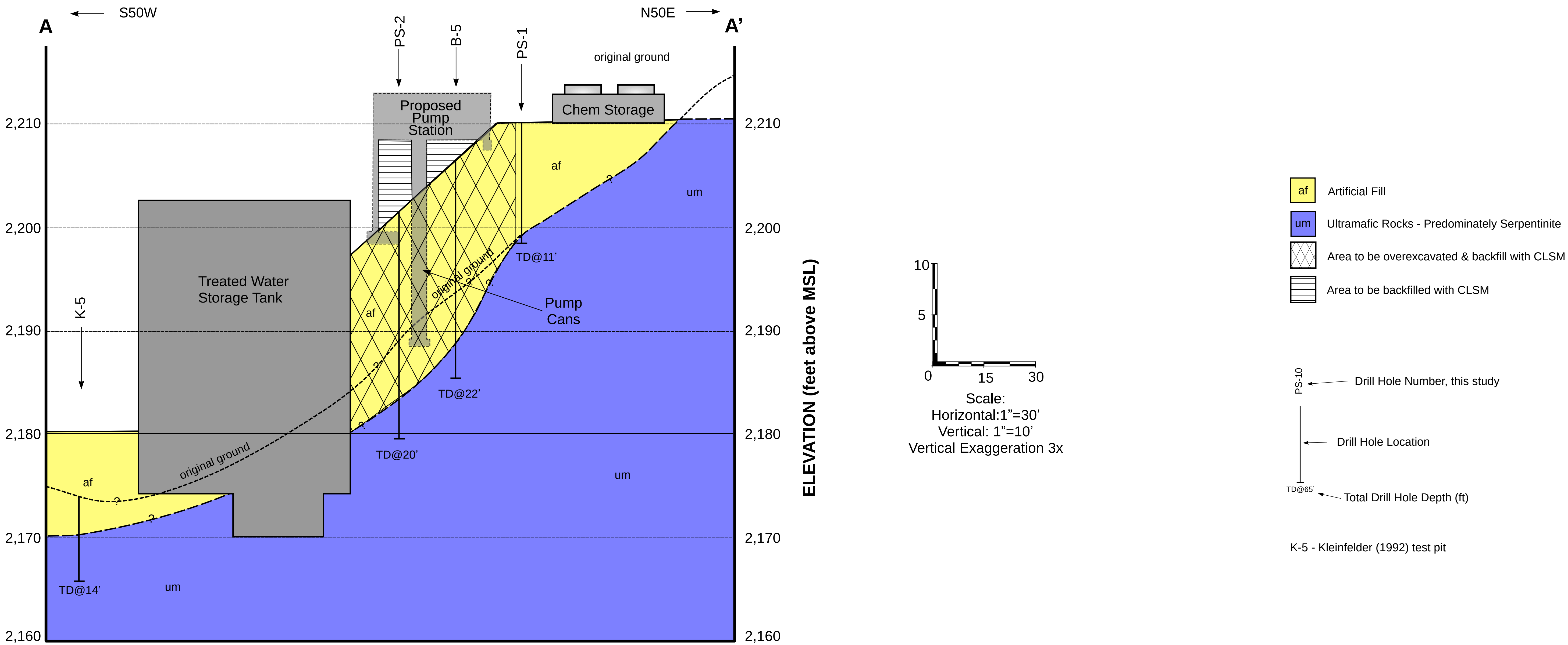
GEOTECHNICAL MAP - TANKS SITES

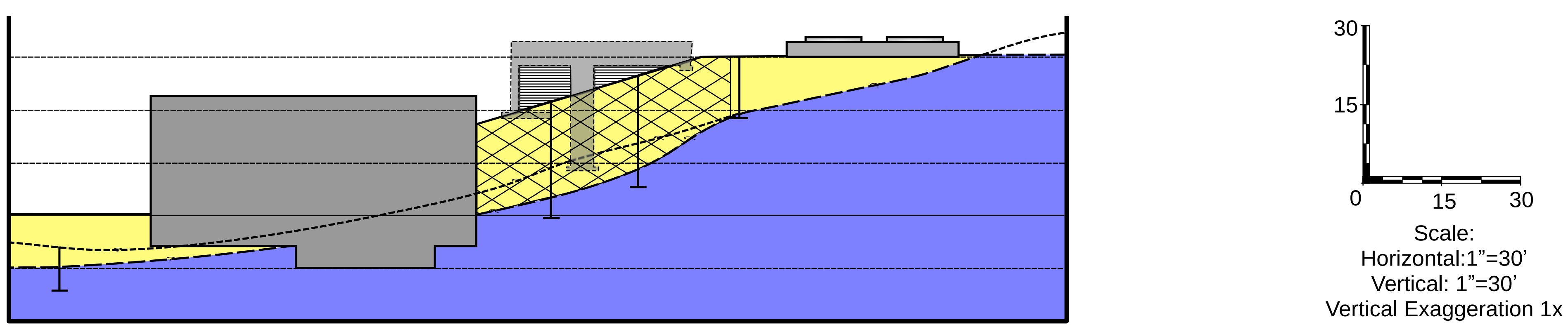
Reservoir B Replacement
Design-Level Study
Water Works Engineers
Paradise & Butte Co., California

Plate No.
3.3

VERTICAL SCIENCES, INC.

Project no.
170025



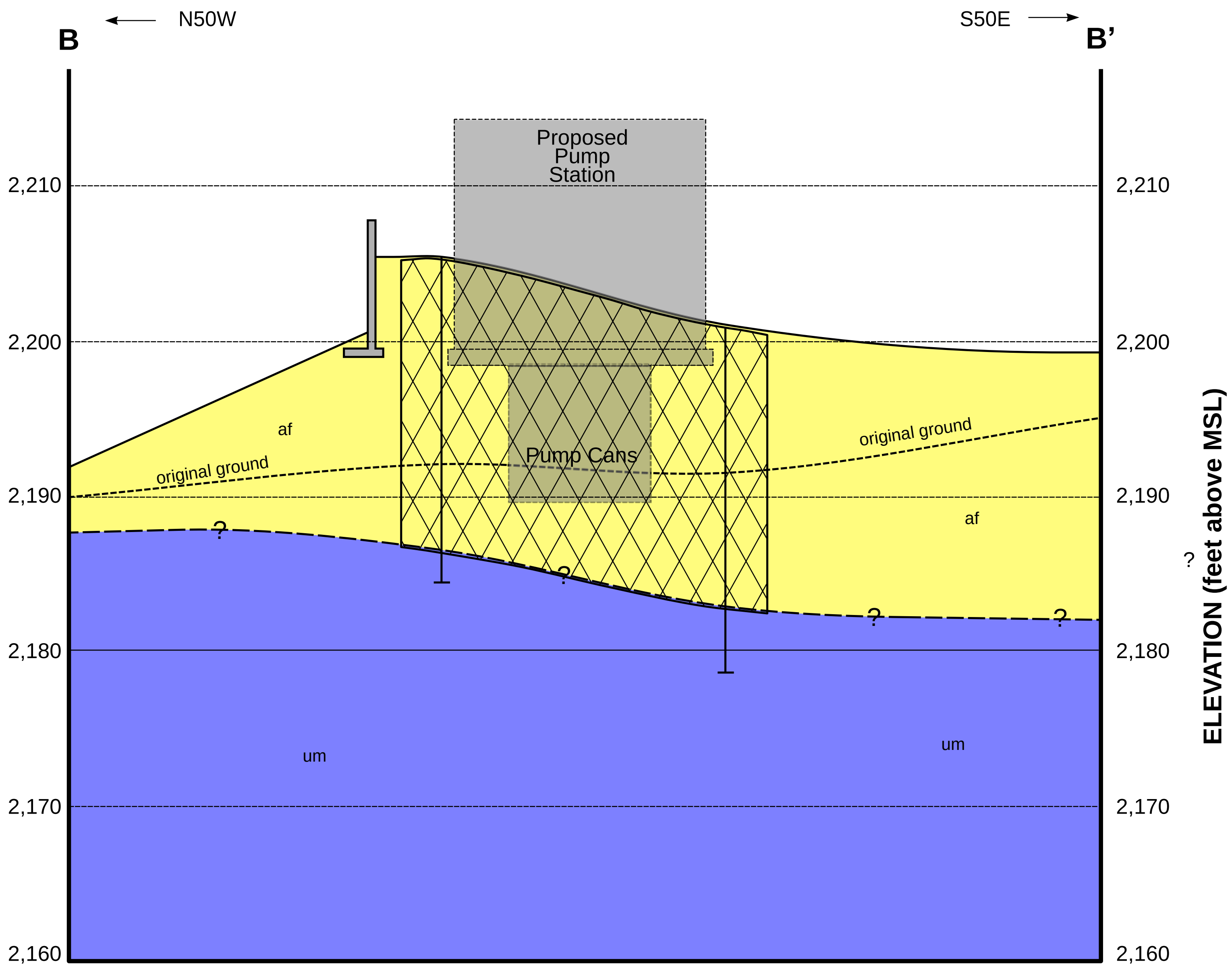


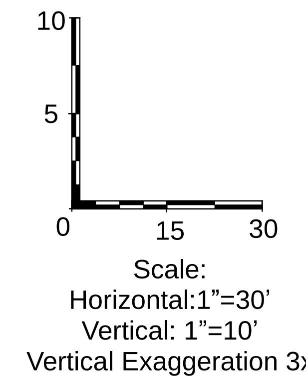
GEOTECHNICAL SECTION A-A”

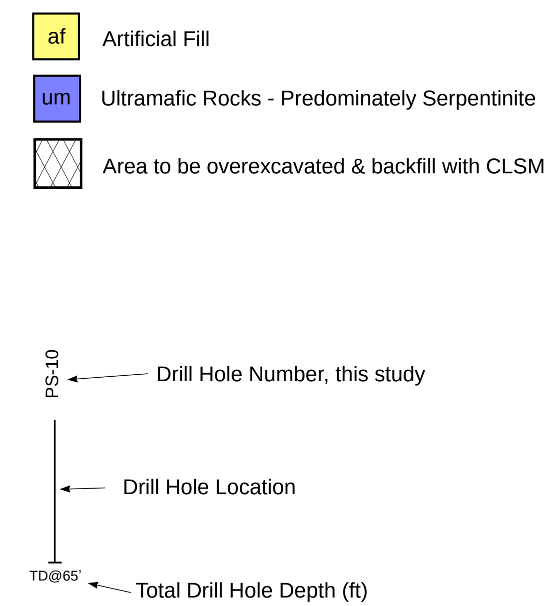
Reservoir B Replacement
Design-Level Study
Water Works Engineers
Paradise & Butte Co., California

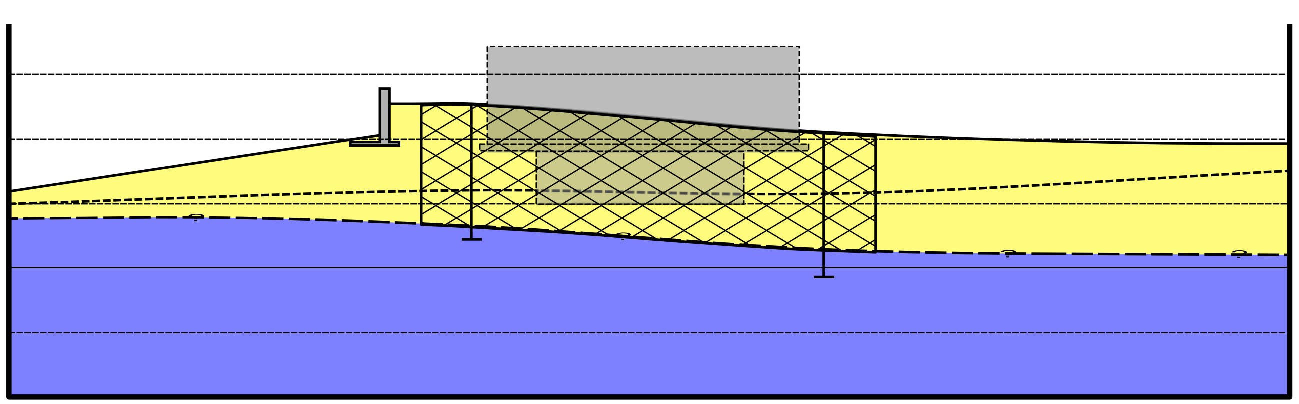
VERTICAL SCIENCES, INC.

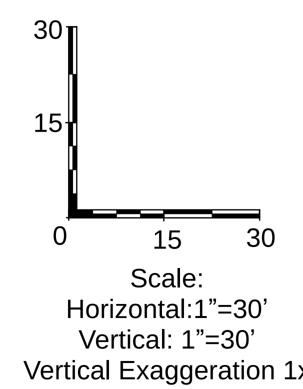
Plate No.
4.1
Project no. 170025









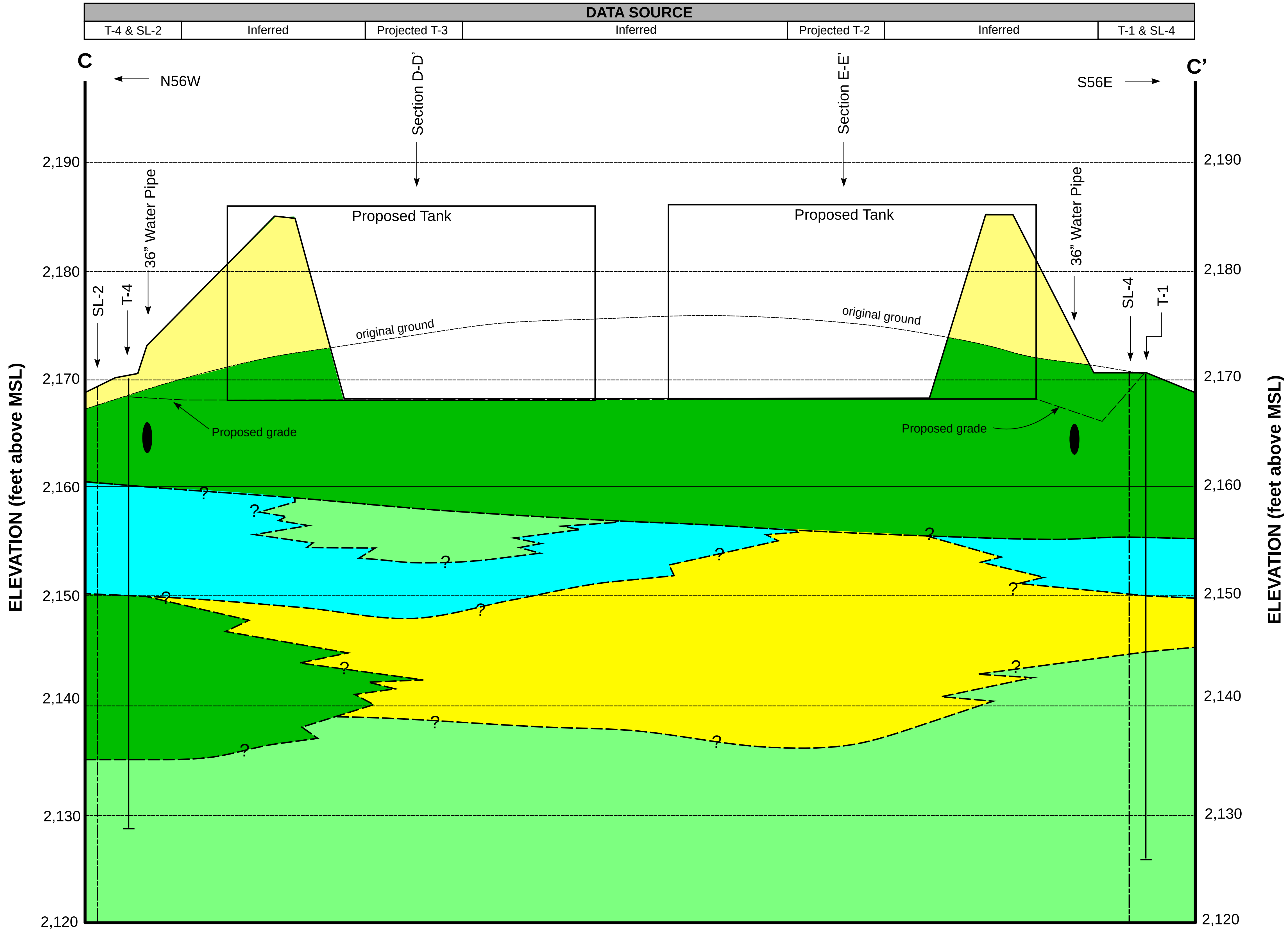


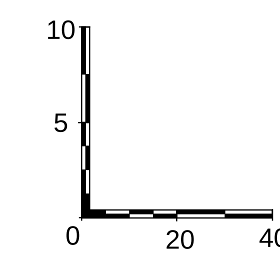
GEOTECHNICAL SECTION B-B'

Reservoir B Replacement
Design-Level Study
Water Works Engineers
Paradise & Butte Co., California

VERTICAL SCIENCES, INC.

Plate No.
4.2
Project no. 170025





Scale:
Horizontal:1"=40'
Vertical: 1"=10'
Vertical Exaggeration 4x

Artificial Fill
- Interbedded sandy and clayey sediments (USCS: SC & CL) with abundant cobbles and boulders

Tuscan Formation
- Predominately sandy sediments (USCS: SC & SM)
- Predominately silty sediments (USCS: ML)
- Interbedded sandy and clayey sediments (USCS: SC & CL)
- Predominately fine-grained sediments (USCS: CL)

Seismic Refraction Survey Line Number — SL-4

T-4 — Drill Hole Number
Drill Hole Location
TD@65' — Total Drill Hole Depth (ft)

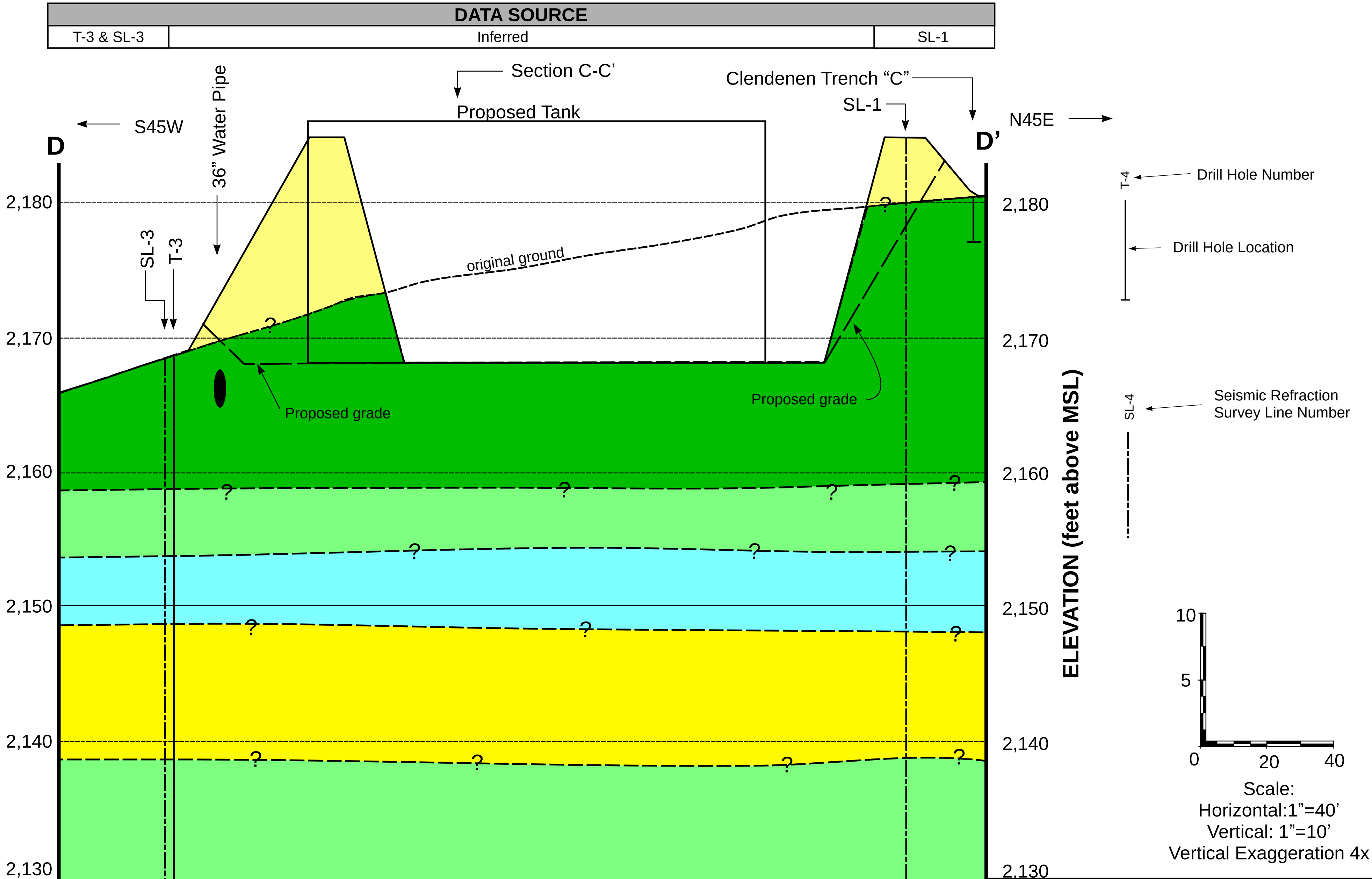
GEOTECHNICAL SECTION C-C'

Reservoir B Replacement
Design-Level Study
Water Works Engineers
Paradise & Butte Co., California

Plate No.
4.3

VERTICAL SCIENCES, INC.

Project no. 170025



Artificial Fill — Interbedded sandy and clayey sediments (USCS: SC & CL) with abundant cobbles and boulders

Tuscan Formation
- Predominately sandy sediments (USCS: SC & SM)
- Predominately silty sediments (USCS: ML)
- Interbedded sandy and clayey sediments (USCS: SC & CL)
- Predominately fine-grained sediments (USCS: CL)

GEOTECHNICAL SECTION D-D'

Reservoir B Replacement
Design-Level Study
Water Works Engineers
Paradise & Butte Co., California

Plate No. 4.4

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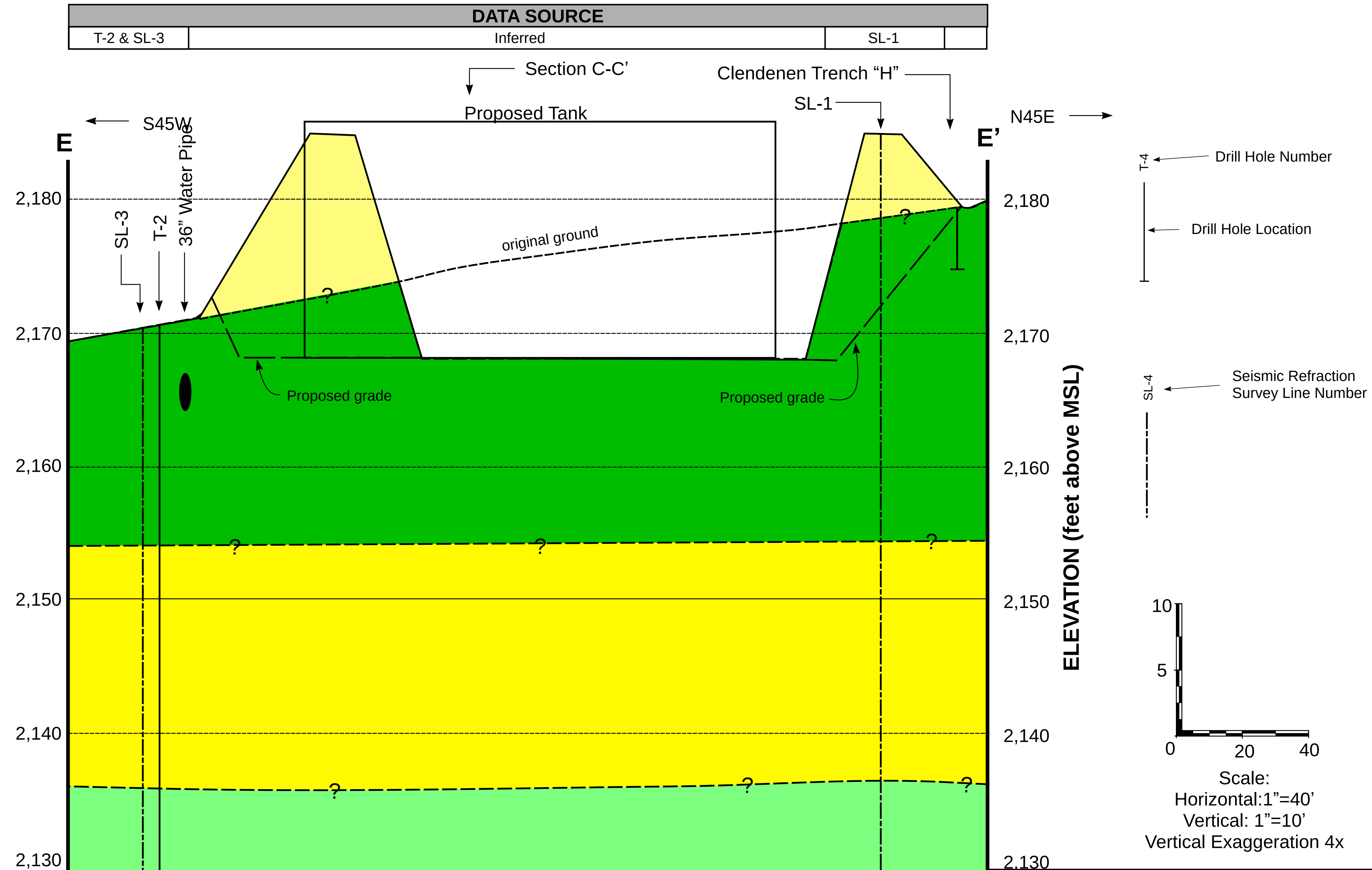
Project no. 170025



Artificial Fill — Interbedded sandy and clayey sediments (USCS: SC & CL) with abundant cobbles and boulders

Tuscan Formation:
- Predominately sandy sediments (USCS: SC & SM)
- Predominately silty sediments (USCS: ML)
- Interbedded sandy and clayey sediments (USCS: SC & CL)
- Predominately fine-grained sediments (USCS: CL)

GEOTECHNICAL SECTION E-E'

Reservoir B Replacement
Design-Level Study
Water Works Engineers
Paradise & Butte Co., California

Plate No. 4.5

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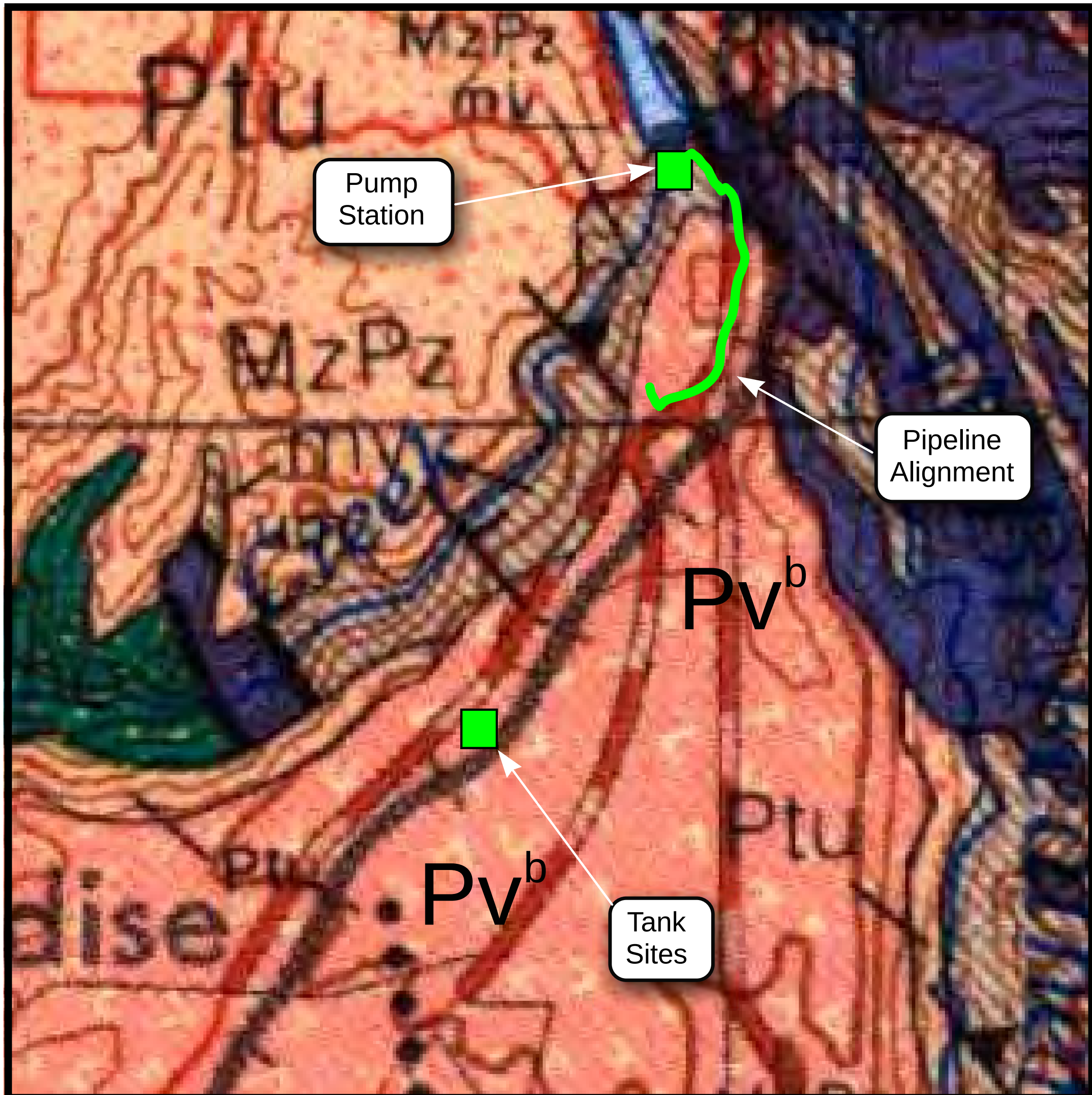
Project no. 170025



Ptu	Tuscan Formation
Pv^b	Volcanic Rock - Basalt
Jmo	Monte de Oro Formation
um	Ultramafic Rocks
MzPz	Metavolcanic Rocks

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

REGIONAL GEOLOGIC MAP

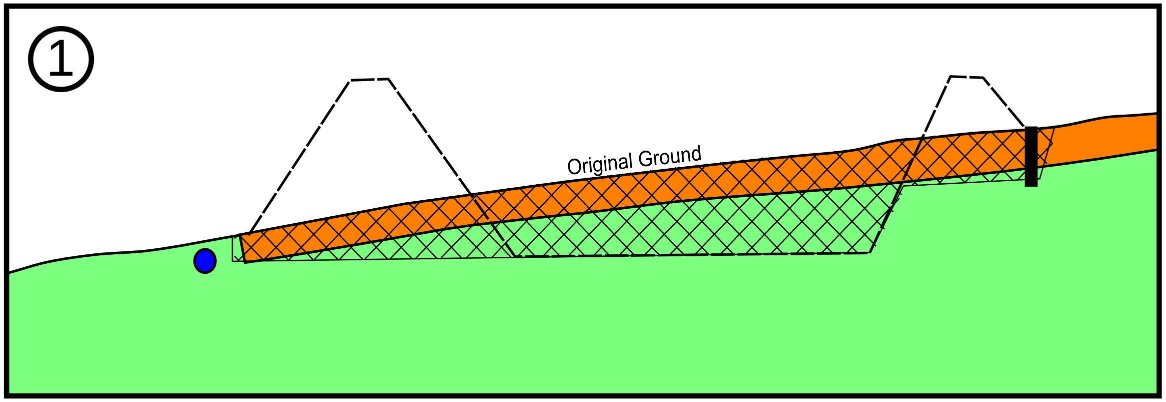
Reservoir B Replacement
Design-Level Study
Water Works Engineers
Paradise & Butte Co., California

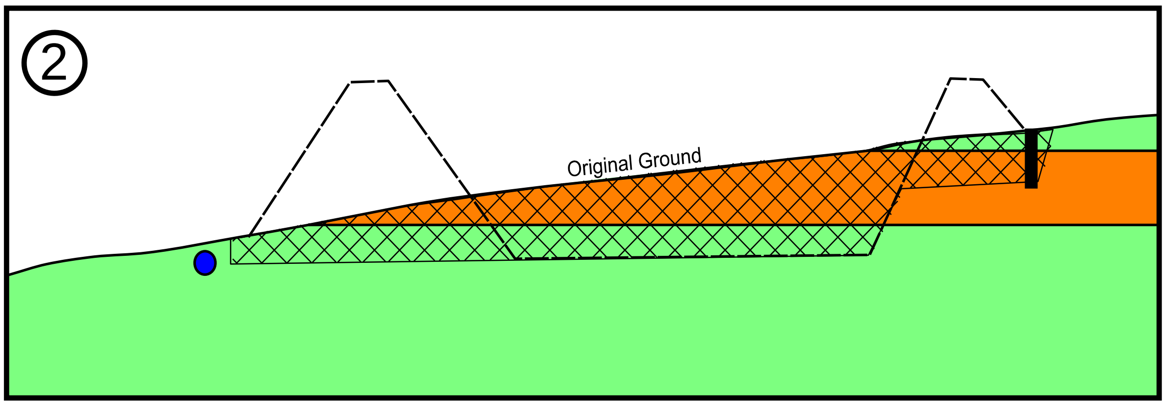
Plate No. 5

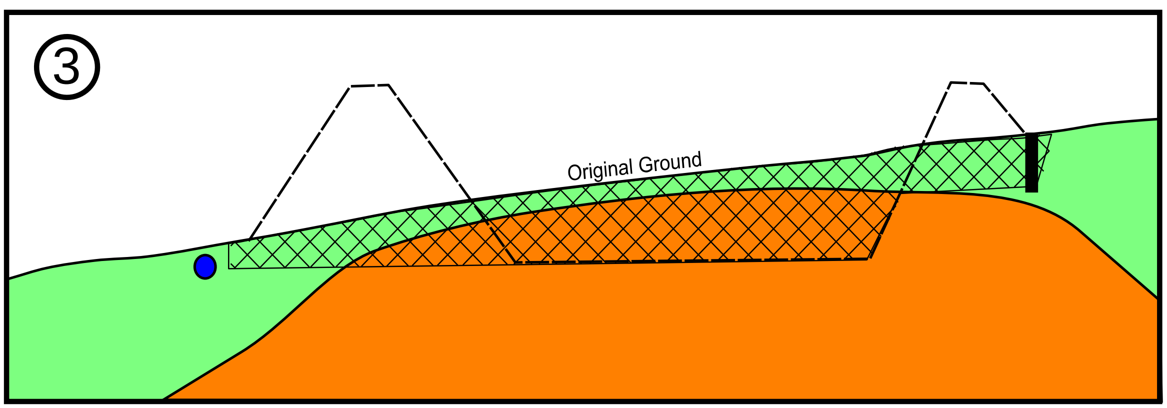
VERTICAL SCIENCES, INC.

Project no. 170025

Scale undetermined
Base maps derived from Saucedo & Wagner (1992)







Volcanic Flow Unit (vesicular, columnar basalt)

Soils of the Tuscan Formation

Possible Area of Excavation & Overexcavation

3 Possible Scenario

VOLCANIC ROCK SCENARIOS

Reservoir B Replacement
Design-Level Study
Water Works Engineers
Paradise & Butte Co., California

Plate No. 6

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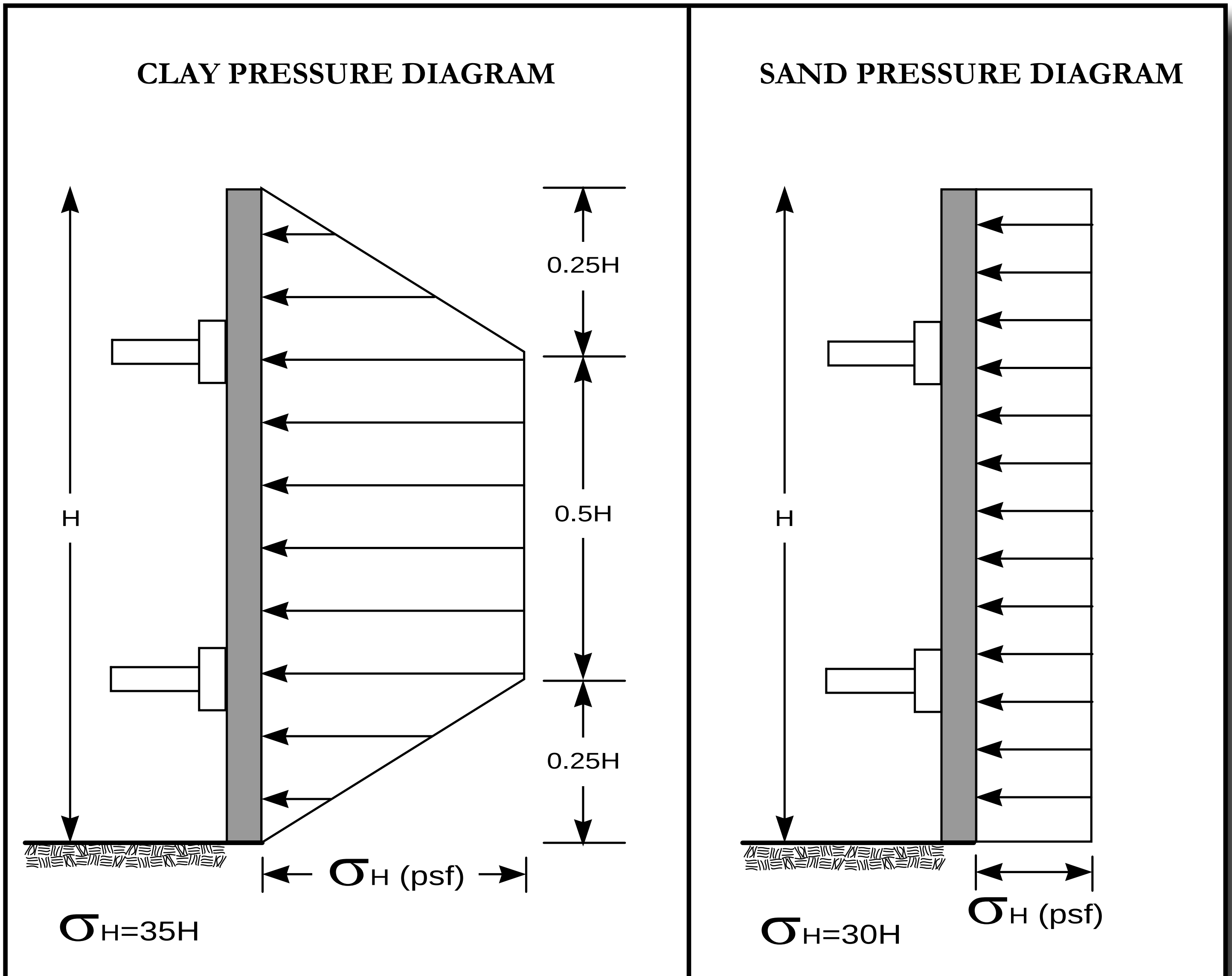
Project no. 170025



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.

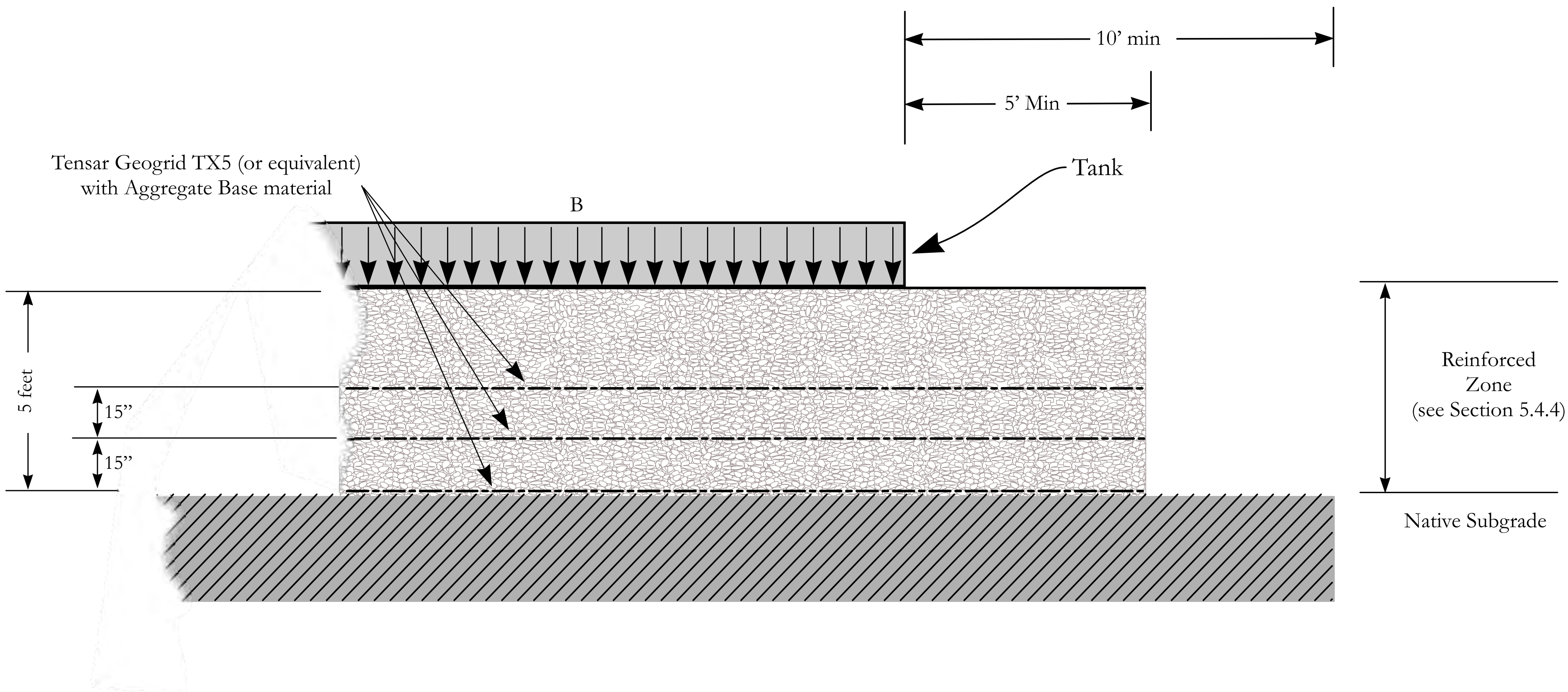
PRELIMINARY SHORING PRESSURE DIAGRAMS

Reservoir B Replacement
Design-Level Study
Water Works Engineers
Paradise & Butte Co., California

Plate No. 7

VERTICAL SCIENCES, INC.

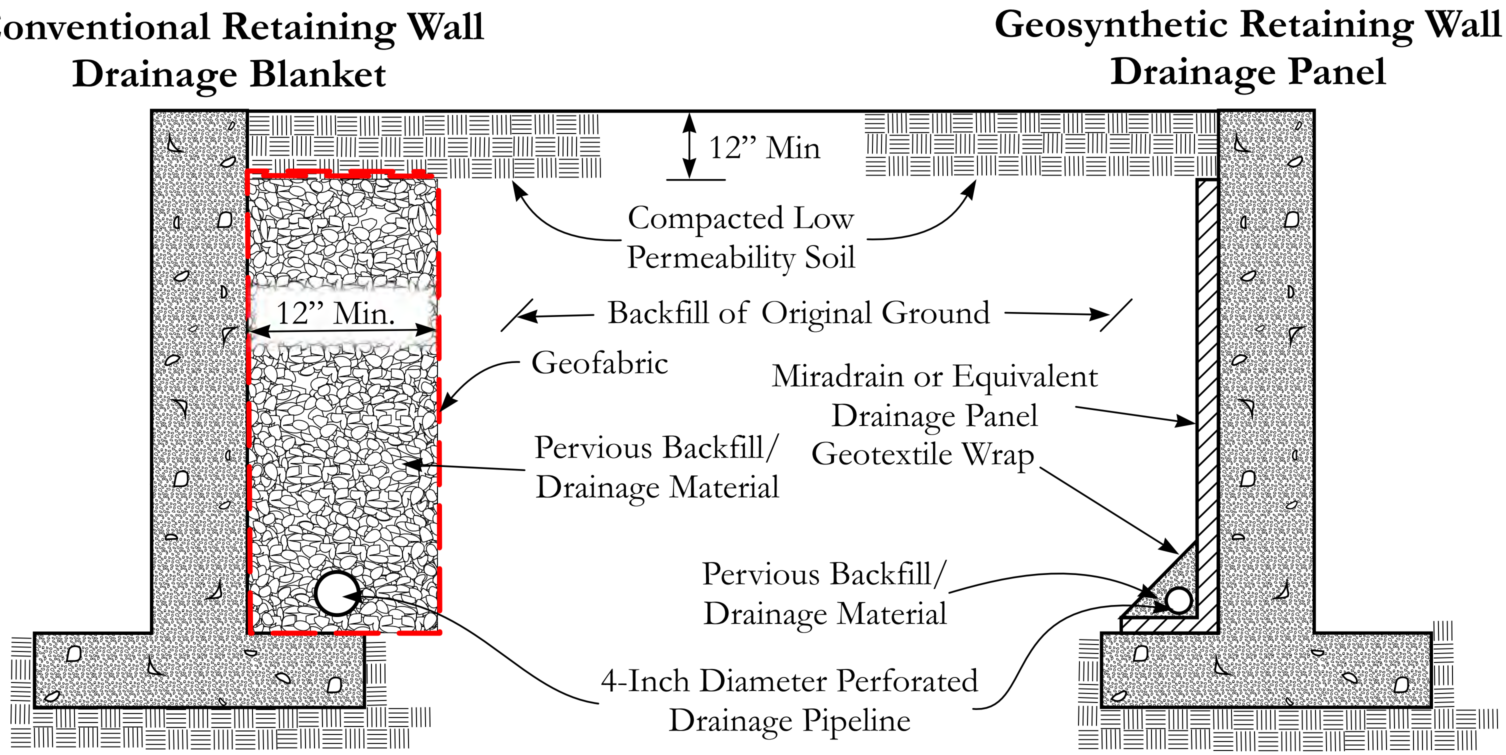
Project no. 170025



Not to Scale

GEOSYNTHETIC RAFT FOUNDATION

Reservoir B Replacement Design-Level Study Water Works Engineers Paradise & Butte Co., California	Plate No. 8
VERTICAL SCIENCES, INC.	Project no. 170025



General Notes

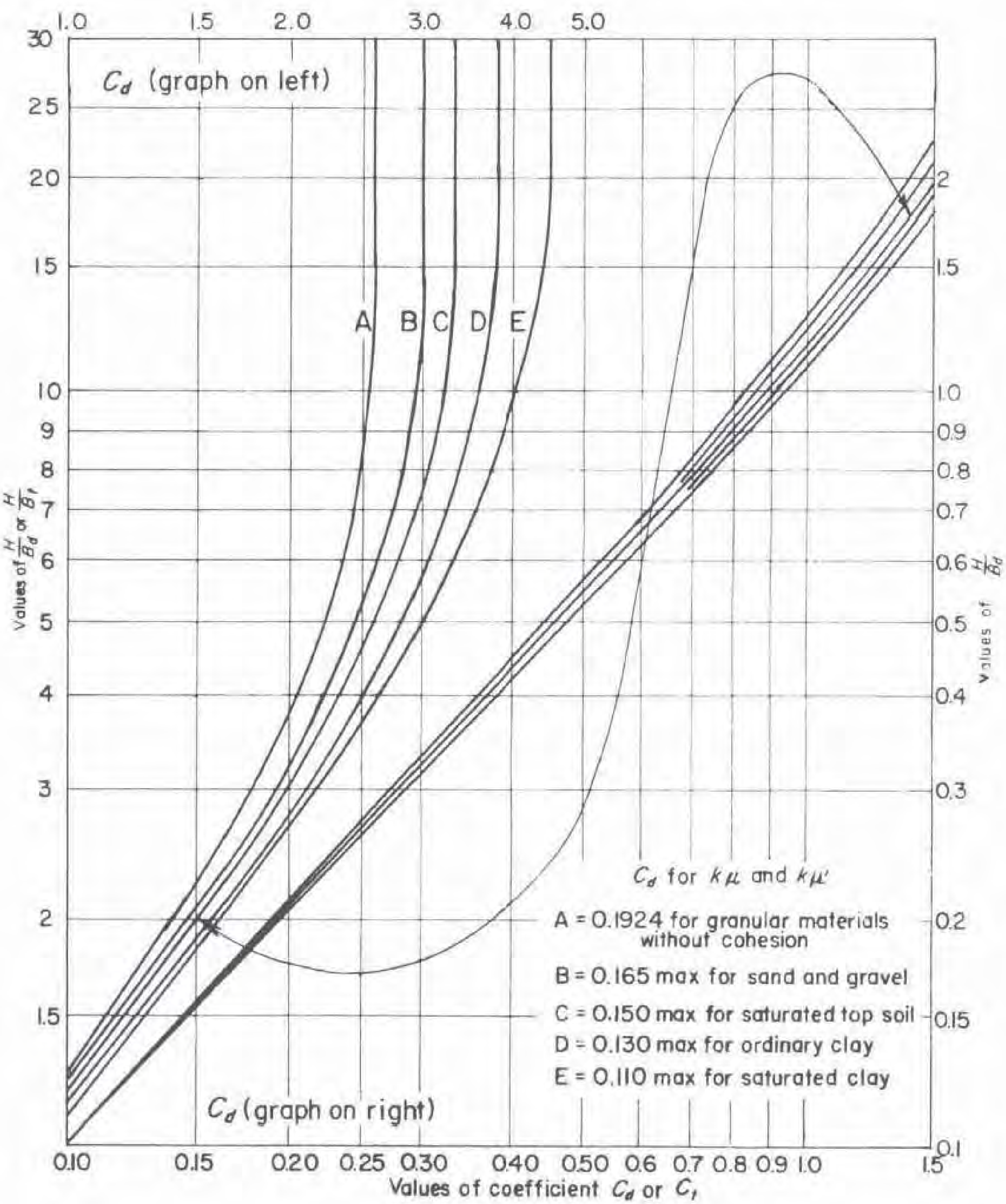
Pervious backfill/drainage material should conform to Pervious Backfill per Greenbook specifications, Class 2 Permeable
Material per Caltrans Standard Specifications, pea gravel having a nominal 1/4-inch diameter, or crushed stone sized
between 1/4-inch and 1/2-inch.

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.

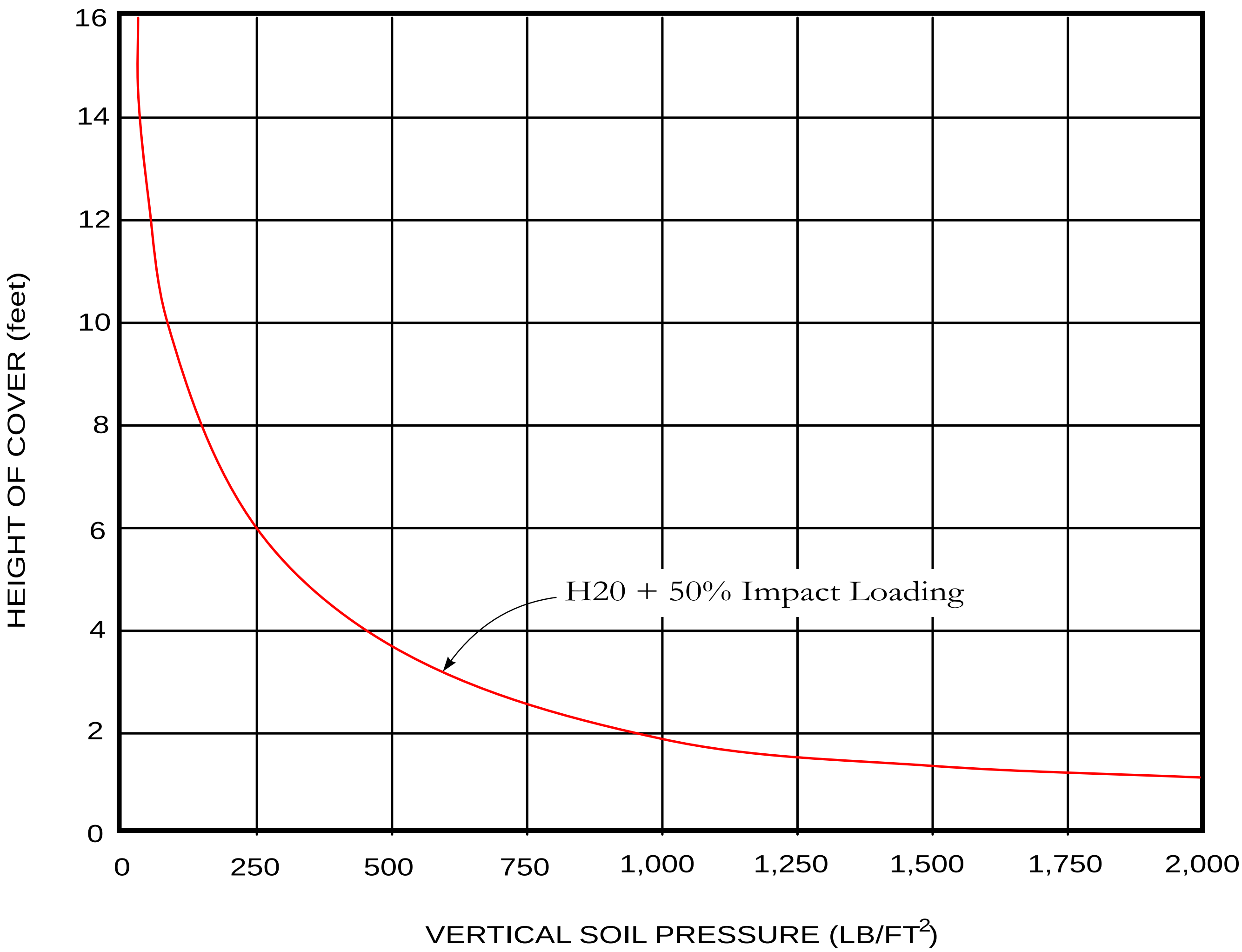
RETAINING WALL DETAILS	
Reservoir B Replacement Design-Level Study Water Works Engineers Paradise & Butte Co., California	Plate No. 9
VERTICAL SCIENCES, INC.	Project no. 170025



Curve from ASCE (1982)

MARSTON'S LOAD COEFFICIENTS

Reservoir B Replacement Design-Level Study Water Works Engineers Paradise & Butte Co., California	Plate No. 10
VERTICAL SCIENCES, INC.	Project no. 170025



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

VERTICAL SOIL PRESSURES INDUCED BY LIVE LOADS

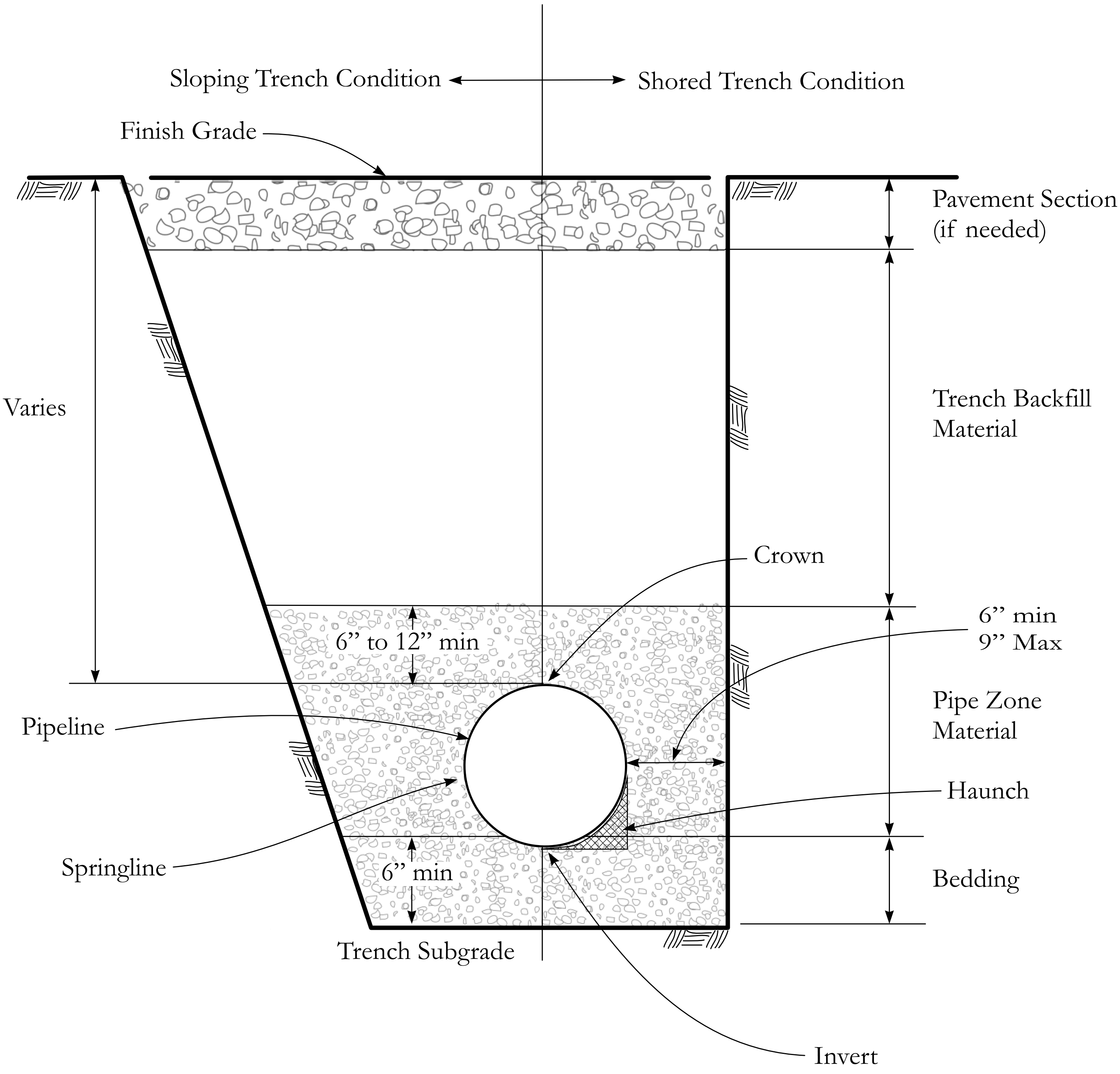
Reservoir B Replacement
Design-Level Study
Water Works Engineers
Paradise & Butte Co., California

Plate No.
11

VERTICAL SCIENCES, INC.

Project no.
170025

Derived from Moser & Feldman (2008)



Not to Scale

TRENCH NOMENCLATURE

Reservoir B Replacement
Design-Level Study
Water Works Engineers
Paradise & Butte Co., California

Plate No. 12

VERTICAL SCIENCES, INC.

Project no. 170025

APPENDIX A

Subsurface Exploration

APPENDIX A
SUBSURFACE EXPLORATION

The subsurface exploration program for this study consisted of the advancement of twelve exploratory drill holes at selected locations at the project element sites, as on Plates 3.1 through 3.3. Prior to exploration, drilling permits were obtained from Butte County Environmental Health and encroachment permits obtained from Butte County Public Works Department. The drill holes were advanced on November 6 through 10, 2017 using a CME 75 drill rig provided by Geo-EX Subsurface Exploration of Dixon, California. The drill hole was advanced using solid-stem flight augers.

Samples of soils were collected from selected depth increments in the 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 Plate A-1.1. 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 log describes 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. Soils generated by drilling operations were spoiled at each drill hole location.

The drill hole log is presented as Plates A-2.1 through A-12. A legend to the drill hole logs is presented as Plate A-1.1.

LOG OF EXPLORATION: Expl. No.

PROJECT: CGI's Project Name | **EXPL. VENDOR:** Expl. Subcontractor | **SURFACE ELEVATION:** Expl. Elevation

PROJECT NO.: CGI's Project No. | **EXPL. METHOD:** Method of Expl. | **TOTAL DEPTH OF HOLE:** Total Depth of Expl.

LOCATION: General Location | **LOGGED BY:** CGI's Logger | **DEPTH TO WATER:** Depth to Water

START DATE: Date Started | **CHECKED BY:** CGI'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							

LOG OF EXPLORATION: DH-T1

PROJECT: PID Reservoir B
PROJECT NO.: 170025
LOCATION: Butte County, CA
START DATE: November 6, 2017
END DATE: November 6, 2017

EXPL. VENDOR: Geo-EX
EXPL. METHOD: 4.25" SSA
LOGGED BY: J.Bianchin
CHECKED BY: J.Everett
HAMMER TYPE: 140-Lb

SURFACE ELEVATION: 2,170 Feet
DEPTH OF HOLE: 44.5 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					ML	Clayey SILT, moderate reddish brown, dry, slightly plastic, with fine sand, angular fine to coarse gravel, cobbles, and boulders.							
			B1										Consol Soil Chem
5			1	(39)	CL	Silty CLAY, moderate reddish brown, moist, very stiff, slightly plastic to plastic, with trace fine sand and trace fine roots.		91.7	28.4		50	25	PI
10			2	(50:5”)		At 10 feet: mottled moderate yellowish brown, with fine to coarse saprolitic sand grains.		76.6	27.2				
15			3	(75)	ML	Clayey SILT, moderate brown mottled greyish green, dry, dense to very dense, with fine sand.		85.9	29.8				
20			4	(33)	SC/ SM	Clayey to Silty SAND, grey mottled moderate brown, moist, medium dense, fine to coarse grained, slightly plastic, with organic fragments.		71.3	35.8				Consol

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-2.1a

LOG OF EXPLORATION: DH-T1

PROJECT: PID Reservoir B
PROJECT NO.: 170025
LOCATION: Butte County, CA
START DATE: November 6, 2017
END DATE: November 6, 2017

EXPL. VENDOR: Geo-EX
EXPL. METHOD: 4.25" SSA
LOGGED BY: J.Bianchin
CHECKED BY: J.Everett
HAMMER TYPE: 140-Lb

SURFACE ELEVATION: 2,170 Feet
DEPTH OF HOLE: 44.5 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			5	(31)	CL	Silty CLAY, moderate yellowish brown mottled moderate reddish brown and brown, moist, very stiff, plastic, with organic fragments.		75.0	42.9				
30			6	(20)		At 30 feet: very moist to wet, with trace thin roots.							
35			7	(38)				68.2	47.1				
40			8	(23)				61.7	57.1				Consol
			9	17									
			10	(16)									
45						Bottom of Drill Hole at a Depth of 44.5 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-2.1b

LOG OF EXPLORATION: DH-T2

PROJECT: PID Reservoir B
PROJECT NO.: 170025
LOCATION: Butte County, CA
START DATE: November 8, 2017
END DATE: November 9, 2017

EXPL. VENDOR: Geo-EX
EXPL. METHOD: 4.25" SSA
LOGGED BY: J.Bianchin
CHECKED BY: J.Everett
HAMMER TYPE: 140-Lb

SURFACE ELEVATION: 2,172 Feet
DEPTH OF HOLE: 41.5 feet
DEPTH TO WATER: 7 feet
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					CL	Gravel (2") Silty CLAY with Gravel, moderate brownish red, moist, plastic, with subangular to angular gravel, cobbles, and boulders							
5			1	(50:5")		At 5 feet: hard.	▼	68.9	20.2				
10			2	(50:5")		At 10 feet: with trace fine to medium roots.		80.0	24.4				
15			3	(44)	SC	Clayey SAND, moderate brown mottled grey and tan, mist, medium dense, with saprolitic sand fine to coarse sand grains.		83.5	27.7				
20			4	(26)		At 20 feet: with angular fine to coarse sand and organic fragments.		72.7	43.9				

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-2.2a

LOG OF EXPLORATION: DH-T2

PROJECT:	PID Reservoir B	**EXPL. VENDOR:**	Geo-EX	**SURFACE ELEVATION:**	2,172 Feet
PROJECT NO.:	170025	**EXPL. METHOD:**	4.25" SSA	**DEPTH OF HOLE:**	41.5 feet
LOCATION:	Butte County, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	7 feet
START DATE:	November 8, 2017	**CHECKED BY:**	J.Everett	**BACKFILLED WITH:**	Cement Grout
END DATE:	November 9, 2017	**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	(18)	SC	Clayey SAND, moderate brown mottled grey and tan, mist, medium dense, with saprolitic sand fine to coarse sand grains.		66.0	50.3				Also obtained grab sample
30			6	(75)		At 30 feet: greyish brown, very dense, fine to medium grained, with trace subangular fine gravel.			37.9				
35			7	(15)	SC/ CL	Clayey SAND, greyish brown mottled grey and tan, moist, loose to medium dense, interbedded with Silty CLAY, moderate yellowish brown to brown, moist, medium stiff to stiff, plastic, with local trace fine sand.			44.0				
40			8	(29)					39.2				
						Bottom of Drill Hole at a Depth of 41.5 feet							
45													

LOG OF EXPLORATION: DH-T3

PROJECT: PID Reservoir B
PROJECT NO.: 170025
LOCATION: Butte County, CA
START DATE: November 8, 2017
END DATE: November 9, 2017

EXPL. VENDOR: Geo-EX
EXPL. METHOD: 4.25" SSA
LOGGED BY: J.Bianchin
CHECKED BY: J.Everett
HAMMER TYPE: 140-Lb

SURFACE ELEVATION: 2,166 Feet
DEPTH OF HOLE: 41.5 feet
DEPTH TO WATER: 20 feet
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					CL	Gravel (2") Silty CLAY with Gravel, moderate brownish red, moist, plastic, with abundant subangular to angular gravel, cobbles, and boulders.							
5			1	(35)		At 5 feet: moderate brown to moderate yellowish brown, moist, very stiff, with trace to moderate angular fine to medium gravel.		83.8	34.0				DS
10			2	(59)	CL/ SC	Silty CLAY with Gravel, moderate brownish red, moist, hard, plastic, interbedded with Clayey SAND, moderate brown, moist, very dense, fine grained, with saprolitic sand and gravel clasts.		87.7	32.7				Consol
15			3	(48)	ML	Clayey SILT, greenish grey mottled moderate brown, moist, very dense, slightly plastic.		80.6	34.7				
20			4	(20)	SC	Clayey SAND, moderate brown mottled moderate yellowish brown, moist, medium dense, fine to medium grained with trace angular fine gravel, and trace organic fragments.	▼	74.6	39.8				Consol

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LOG OF EXPLORATION: DH-T3

PROJECT:	PID Reservoir B	**EXPL. VENDOR:**	Geo-EX	**SURFACE ELEVATION:**	2,166 Feet
PROJECT NO.:	170025	**EXPL. METHOD:**	4.25" SSA	**DEPTH OF HOLE:**	41.5 feet
LOCATION:	Butte County, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	20 feet
START DATE:	November 8, 2017	**CHECKED BY:**	J.Everett	**BACKFILLED WITH:**	Cement Grout
END DATE:	November 9, 2017	**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	(50:3")	SC	Clayey SAND, moderate brown mottled moderate yellowish brown, moist, medium dense, fine to medium grained with trace angular fine gravel, and trace organic fragments.		71.7	46.9				
30				(32)									
35			6	(41)		At 35 feet: with subrounded to subangular fine to medium gravel, caliche mottling, and mangenese oxide staining.		72.8	43.3				
40			7	(58)				75.2	42.3				
						Bottom of Drill Hole at a Depth of 41.5 feet							
45													

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-2.3b

LOG OF EXPLORATION: DH-T4

PROJECT:	PID Reservoir B	**EXPL. VENDOR:**	Geo-EX	**SURFACE ELEVATION:**	2,170 Feet
PROJECT NO.:	170025	**EXPL. METHOD:**	4.25" SSA	**DEPTH OF HOLE:**	41.5 feet
LOCATION:	Butte County, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	November 9, 2017	**CHECKED BY:**	J.Everett	**BACKFILLED WITH:**	Cement Grout
END DATE:	November 9, 2017	**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					CL/ ML	Silty CLAY to Clayey SILT, reddish brown, dry, slightly plastic, with trace fine sand.							
5			1	34		At 5 feet: very stiff to medium dense.			27.7		58	37	PI
10			2	27	ML	Clayey SILT, moderate reddish brown, dry, medium dense, with trace fine sand and trace subrounded gravel.			28.7				
15			3	9		At 15 feet: moist.			34.8				
20			4	4	CL	Silty CLAY, moderate yellowish brown mottled moderate brown, moist, soft, plastic, with trace fine sand.			48.0				

LOG OF EXPLORATION: DH-T4

PROJECT:	PID Reservoir B	**EXPL. VENDOR:**	Geo-EX	**SURFACE ELEVATION:**	2,170 Feet
PROJECT NO.:	170025	**EXPL. METHOD:**	4.25" SSA	**DEPTH OF HOLE:**	41.5 feet
LOCATION:	Butte County, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	November 9, 2017	**CHECKED BY:**	J.Everett	**BACKFILLED WITH:**	Cement Grout
END DATE:	November 9, 2017	**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	5	CL	Silty CLAY, moderate yellowish brown mottled moderate brown, moist, soft, plastic, with trace fine sand.			48.4				
30			6	12					47.0				
35			7	23	CL/ SC	Silty CLAY, moderate yellowish brown mottled moderate brown, moist, soft, plastic, with trace fine sand interbedded with Clayey SAND, moderate brown, moist, medium dense, slightly plastic, with fine to medium gravel.			49.2				
40			8	30		At 40 feet: with subrounded to subangular fine to medium gravel.			45.9				
						Bottom of Drill Hole at a Depth of 41.5 feet							
45													

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-2.4b

LOG OF EXPLORATION: DH-PS1

PROJECT: PID Reservoir B
PROJECT NO.: 170025
LOCATION: Butte County, CA
START DATE: December 6, 2017
END DATE: December 6, 2017

EXPL. VENDOR: Geo-EX
EXPL. METHOD: 4.25" SSA
LOGGED BY: J.Bianchin
CHECKED BY: J.Everett
HAMMER TYPE: 140-Lb

SURFACE ELEVATION: 2,207 Feet
DEPTH OF HOLE: 11.5 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						ARTIFICIAL FILL (af) Asphaltic Concrete (3 inches); Aggregate Base (8 inches)							
			B1		SC/ CL	Clayey SAND to Sandy CLAY with Gravel, moderate yellowish brown, dry, slightly plastic, fine grained with abundant fine to coarse subangular gravel and cobbles.							Soil Chem NOA
5			1	(77)		Very dense to hard.							
10			2	(50:1")	ML	Sandy SILT, moderate brown, dry, very dense, fine grained with trace to moderate subangular fine to coarse gravel, cobbles, and possibly boulders.							Very hard drilling 11': crossed over to rotary wash.
						Drilling terminated due to practical refusal. Bottom of drill hole at a depth of 11.5 feet							
15													
20													

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LOG OF EXPLORATION: DH-PS2

PROJECT:	PID Reservoir B	**EXPL. VENDOR:**	Geo-EX	**SURFACE ELEVATION:**	2,201 Feet
PROJECT NO.:	170025	**EXPL. METHOD:**	4.25" SSA	**DEPTH OF HOLE:**	22.0 feet
LOCATION:	Butte County, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	December 10, 2017	**CHECKED BY:**	J.Everett	**BACKFILLED WITH:**	Cement Grout
END DATE:	December 10, 2017	**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			B1			ARTIFICIAL FILL (af) Abundant gravel, cobbles, and boulders with abundant fine to coarse roots in upper 18 inches.							
5			1	(45)	SC	Clayey SAND with Gravel, moderate greenish brown, moist, very dense, slightly plastic, fine grained, with angular fine to medium gravel.		105.7	8.4				
10			2	(34)	CL	Silty CLAY with Gravel, moderate greenish brown, moist, stiff, with abundant fine to medium subrounded gravel and trace fine sand.		103.8	15.7				DS
15			3	(18)		At 15 feet: wet, plastic, less gravel.							
					Rx	SERPENTINITE, buff to greenish grey, moderately weathered, moderately fractured, poorly indurated.							
20			4	(50:2")									
			5	50:1"									Very hard drilling
						Drilling terminated due to practical refusal. Bottom of drill hole at a depth of 22 feet							

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PLATE NO.: A-2.6

LOG OF EXPLORATION: DH-PL1

PROJECT: PID Reservoir B
PROJECT NO.: 170025
LOCATION: Butte County, CA
START DATE: December 10, 2017
END DATE: December 10, 2017

EXPL. VENDOR: Geo-EX
EXPL. METHOD: 4.25" SSA
LOGGED BY: J.Bianchin
CHECKED BY: J.Everett
HAMMER TYPE: 140-Lb

SURFACE ELEVATION: 2,280 Feet
DEPTH OF HOLE: 11.5 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						ARTIFICIAL FILL (af) Ashlatic Concrete (4.5"), Aggregate Base (6").							
			B1		Rx/ SM	SERPENTINITE, grey to greenish grey, dry, highly weathered, moderately fractured, poorly indurated, decomposed to Silty SAND.							NOA
5			1	(77)				121.0	4.1	6.1			GS
10			2	(50)	ML			109.4	4.1				
						Bottom of drill hole at a depth of 11.5 feet							
15													
20													

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-PL2

PROJECT:	PID Reservoir B	**EXPL. VENDOR:**	Geo-EX	**SURFACE ELEVATION:**	2,345 Feet
PROJECT NO.:	170025	**EXPL. METHOD:**	4.25" SSA	**DEPTH OF HOLE:**	11.5 feet
LOCATION:	Butte County, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	December 10, 2017	**CHECKED BY:**	J.Everett	**BACKFILLED WITH:**	Cement Grout
END DATE:	December 10, 2017	**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					CL	ARTIFICIAL FILL (af) Silty CLAY, reddish brown, dry, slightly plastic, with moderate to abundant fine to coarse roots.							
			B1							68			Curve, GS
5			1	(54)		At 5 feet: hard.		75.8	23.1				
10			2	17		At 10 feet: very stiff.							
						Bottom of drill hole at a depth of 11.5 feet							
15													
20													

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-2.8

LOG OF EXPLORATION: DH-PL3

PROJECT:	PID Reservoir B	**EXPL. VENDOR:**	Geo-EX	**SURFACE ELEVATION:**	2,345 Feet
PROJECT NO.:	170025	**EXPL. METHOD:**	4.25" SSA	**DEPTH OF HOLE:**	11.5 feet
LOCATION:	Butte County, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	December 10, 2017	**CHECKED BY:**	J.Everett	**BACKFILLED WITH:**	Cement Grout
END DATE:	December 10, 2017	**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					CL	ARTIFICIAL FILL (af) Silty CLAY, reddish brown, dry, slightly plastic, with moderate to abundant fine to coarse roots.							
5			1	(62)		At 5 feet: hard.		81.7	8.4				
10			2	28		At 10 feet: very stiff.							
						Bottom of drill hole at a depth of 11.5 feet							
15													
20													

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-2.9

LOG OF EXPLORATION: DH-PL4

PROJECT:	PID Reservoir B	**EXPL. VENDOR:**	Geo-EX	**SURFACE ELEVATION:**	2,345 Feet
PROJECT NO.:	170025	**EXPL. METHOD:**	4.25" SSA	**DEPTH OF HOLE:**	11.5 feet
LOCATION:	Butte County, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	December 10, 2017	**CHECKED BY:**	J.Everett	**BACKFILLED WITH:**	Cement Grout
END DATE:	December 10, 2017	**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			B1		CL	ARTIFICIAL FILL (af) Silty CLAY, reddish brown, dry, slightly plastic, with moderate to abundant fine to coarse roots.				44			Curve, GS
5			1	(50:3")	Rx	AGGLOMERATE, highly weathered, highly fractured, poorly indurated, weak.		118.6	6.9				
10			2	50:4"									
						Bottom of drill hole at a depth of 11.5 feet							
15													
20													

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-2.10

LOG OF EXPLORATION: DH-PL5

PROJECT:	PID Reservoir B	**EXPL. VENDOR:**	Geo-EX	**SURFACE ELEVATION:**	2,345 Feet
PROJECT NO.:	170025	**EXPL. METHOD:**	4.25" SSA	**DEPTH OF HOLE:**	11.5 feet
LOCATION:	Butte County, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	December 10, 2017	**CHECKED BY:**	J.Everett	**BACKFILLED WITH:**	Cement Grout
END DATE:	December 10, 2017	**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					CL	AGGLOMERATE, highly weathered, highly fractured, poorly indurated, weak weathered to Silty CLAY, moderate reddish brown, dry, with trace fine sand.							
5			1	(50:4”)	SC	Clayey SAND, moderate greyish brown, dry, very dense, fine to medium grained.		82.8	28.3				
10			2	92									
						Bottom of drill hole at a depth of 11.5 feet							
15													
20													

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-2.11

LOG OF EXPLORATION: DH-PL6

PROJECT:	PID Reservoir B	**EXPL. VENDOR:**	Geo-EX	**SURFACE ELEVATION:**	2,345 Feet
PROJECT NO.:	170025	**EXPL. METHOD:**	4.25" SSA	**DEPTH OF HOLE:**	11.5 feet
LOCATION:	Butte County, CA	**LOGGED BY:**	J.Bianchin	**DEPTH TO WATER:**	Not Encountered
START DATE:	December 10, 2017	**CHECKED BY:**	J.Everett	**BACKFILLED WITH:**	Cement Grout
END DATE:	December 10, 2017	**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					ML	AGGLOMERATE, highly weathered, highly fractured, poorly indurated, weak: weathered to Clayey SILT with Gravel, moderate brown to reddish brown, dry, with trace fine sand and subrounded fine to medium gravel.							
5			1	(16)		At 5 feet: medium dense.				55			GS
10			2	25									
						Bottom of drill hole at a depth of 11.5 feet							
15													
20													

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-2.12

APPENDIX B

Laboratory Testing

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 four selected soil samples in accordance with standard test method ASTM D422. The grain size distribution data are shown on the attached plates labeled *Particle Size Distribution*.

Plasticity Index Tests

Atterberg Limits (plastic limit, liquid limit, and plasticity index) tests were performed on two 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*.

Direct Shear Tests

Consolidated-drained direct shear testing was performed on two selected samplse 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.*

Consolidation

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

Maximum Density-Optimum Moisture Content

Five maximum density-optimum moisture content tests were performed on selected samples in accordance with standard test method ASTM D1557. Results of those tests are presented on attached plates labeled *Compaction Test Report*.

Naturally Occurring Asbestos

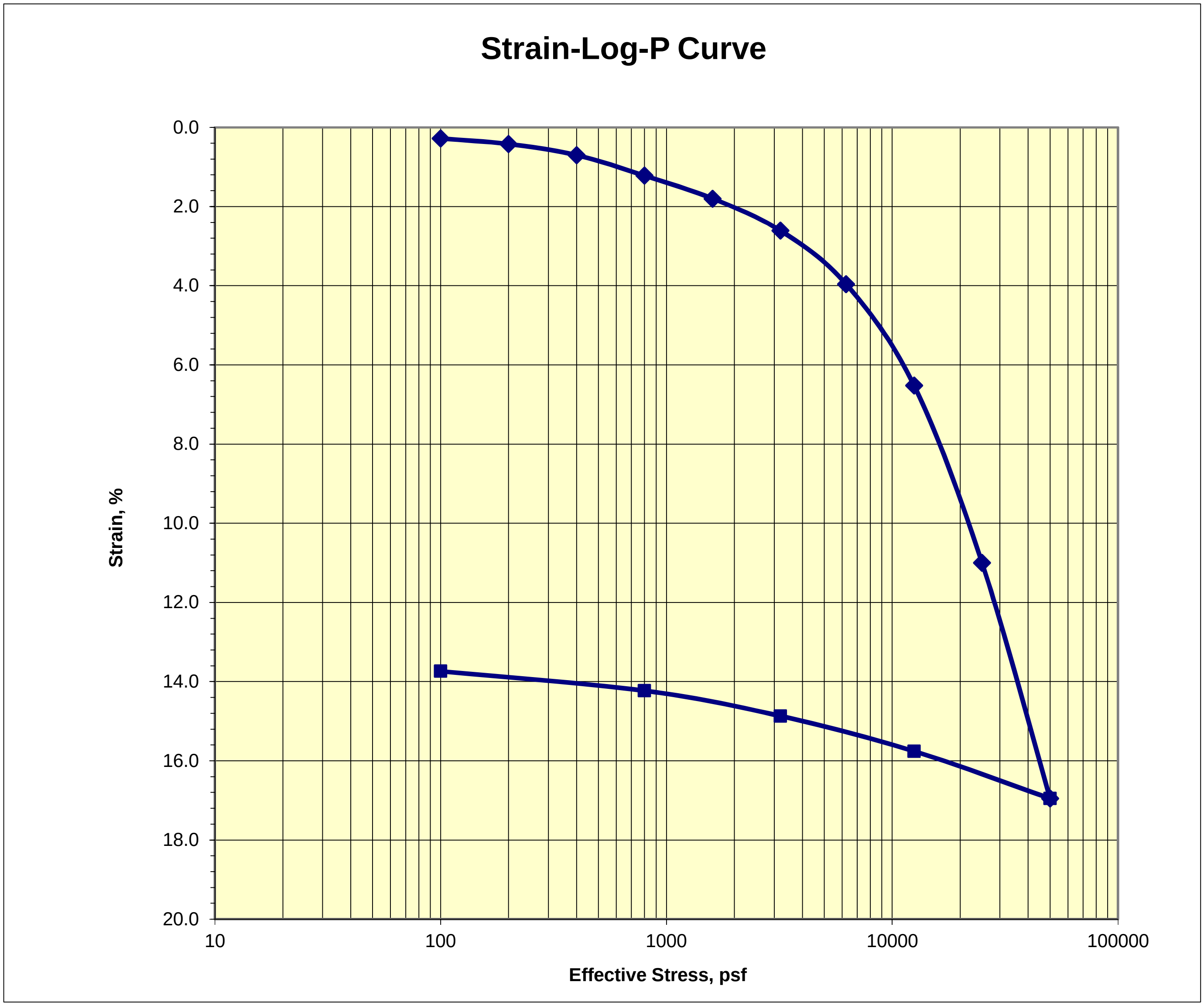
Two tests were performed on selected soil samples to evaluate the presence of naturally occurring asbestos. The tests were performed in accordance with standard test method CARD 435. Results of the tests are presented in this appendix.

Soil-Chemistry for Corrosion

Two tests were performed on 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.:	968-004	**Boring:**	T-1	**Run By:**	MD
Client:	Vertical Sciences	**Sample:**		**Reduced:**	PJ
Project:	170025	**Depth, ft.:**	20	**Checked:**	PJ/DC
Soil Type:	Reddish Brown Silty SAND (slightly plastic)			**Date:**	1/5/2018



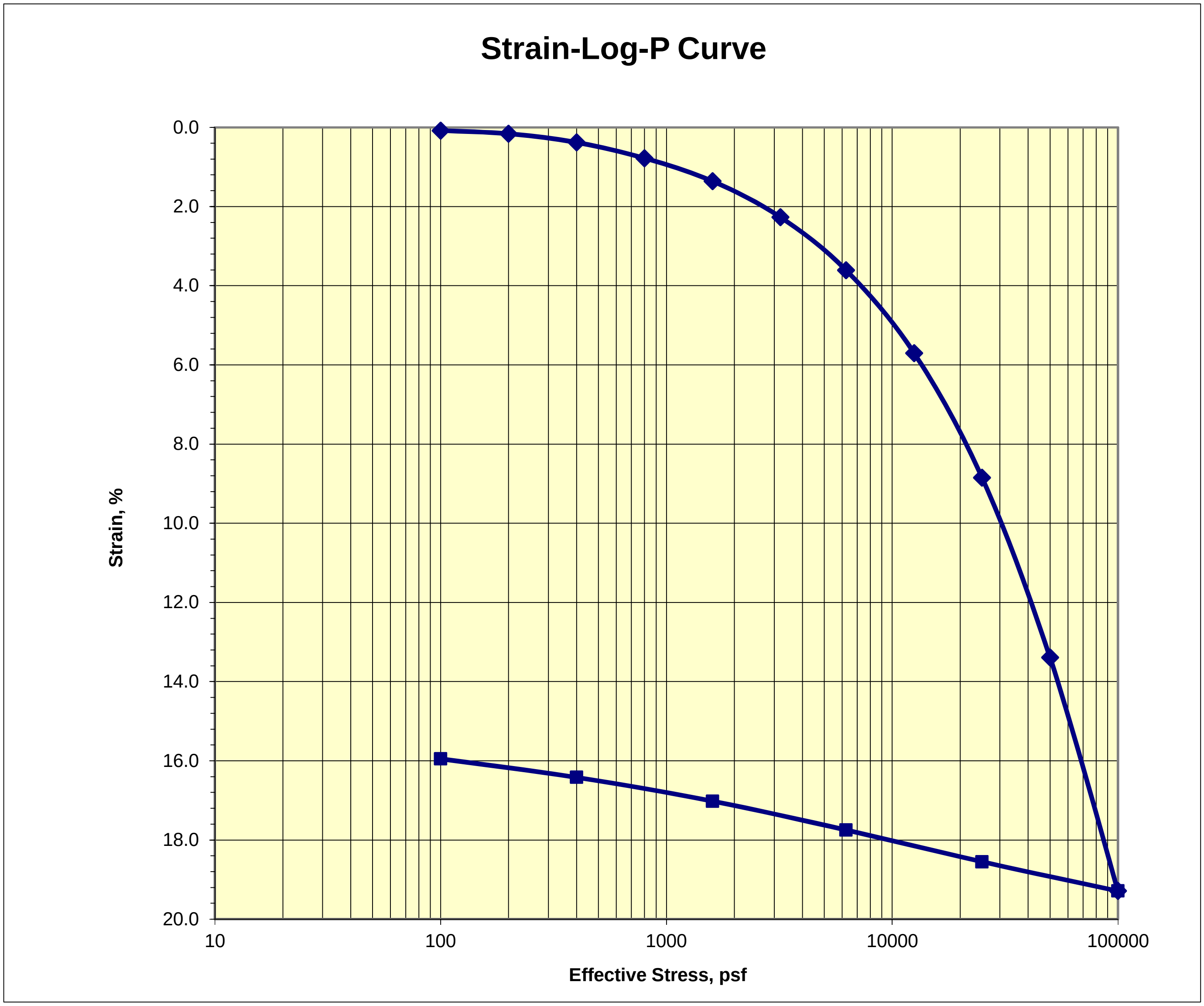
Assumed Gs 2.75	**Initial**	**Final**
Moisture %:	35.8	40.3
Dry Density, pcf:	71.3	81.4
Void Ratio:	1.407	1.109
% Saturation:	70.1	100.0

Remarks:

COOPER TESTING LABORATORY

Consolidation Test
ASTM D2435

Job No.:	968-004	**Boring:**	T-1	**Run By:**	MD
Client:	Vertical Sciences, Inc.	**Sample:**		**Reduced:**	PJ
Project:	170025	**Depth, ft.:**	40	**Checked:**	PJ/DC
Soil Type:	Grayish Brown Silty SAND w/ Gravel (Siltstone)			**Date:**	1/18/2018



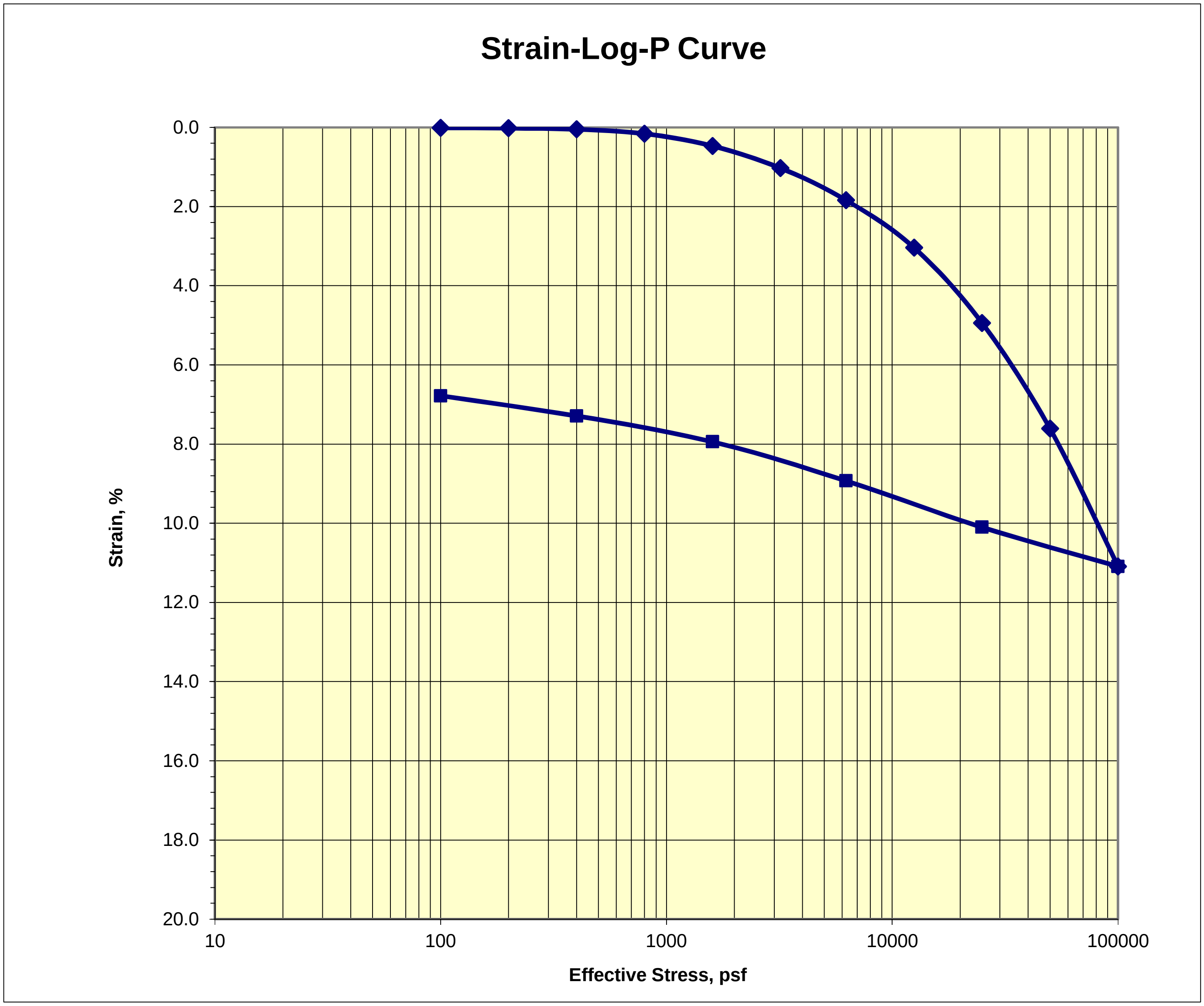
Assumed Gs 2.75	**Initial**	**Final**
Moisture %:	42.2	34.4
Dry Density, pcf:	74.6	88.2
Void Ratio:	1.301	0.946
% Saturation:	89.2	100.0

Remarks:

Consolidation Test
ASTM D2435

COOPER TESTING LABORATORY

Job No.:	968-004	**Boring:**	T-3	**Run By:**	MD
Client:	Vertical Sciences, Inc.	**Sample:**		**Reduced:**	PJ
Project:	170025	**Depth, ft.:**	10	**Checked:**	PJ/DC
Soil Type:	Yellowish Red Sandy CLAY/ Clayey SAND			**Date:**	1/17/2018



Assumed Gs 2.8	Initial	Final
Moisture %:	32.7	32.1
Dry Density, pcf:	87.7	92.1
Void Ratio:	0.994	0.898
% Saturation:	92.0	100.0

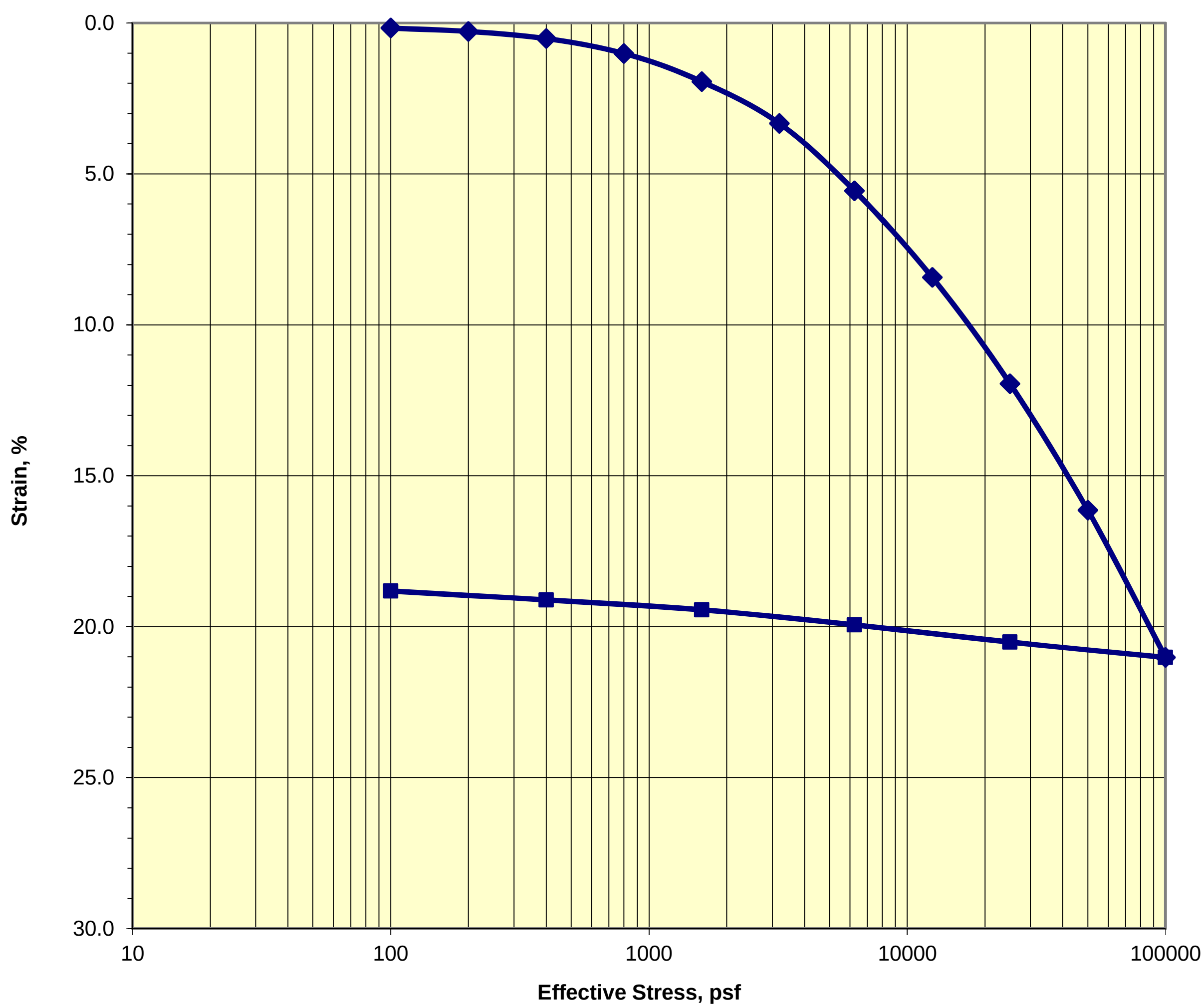
Remarks:

Consolidation Test

ASTM D2435

Job No.:	968-004	**Boring:**	T-3	**Run By:**	MD
Client:	Vertical Sciences, Inc.	**Sample:**		**Reduced:**	PJ
Project:	170025	**Depth, ft.:**	20	**Checked:**	PJ/DC
Soil Type:	Olive Brown Silty SAND			**Date:**	1/18/2018

Strain-Log-P Curve



Assumed Gs 2.75	Initial	Final
Moisture %:	39.8	32.5
Dry Density, pcf:	74.6	90.6
Void Ratio:	1.301	0.894
% Saturation:	84.3	100.0

Remarks:

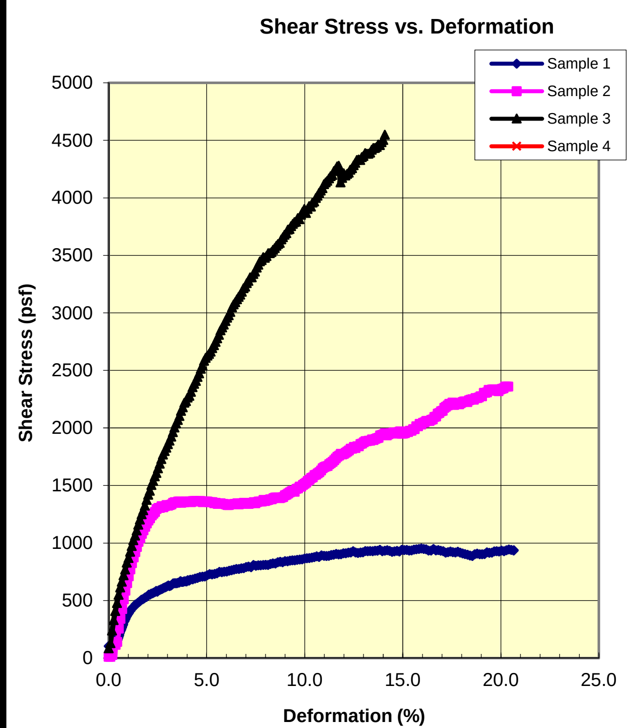
Consolidated Drained Direct Shear (ASTM D3080)

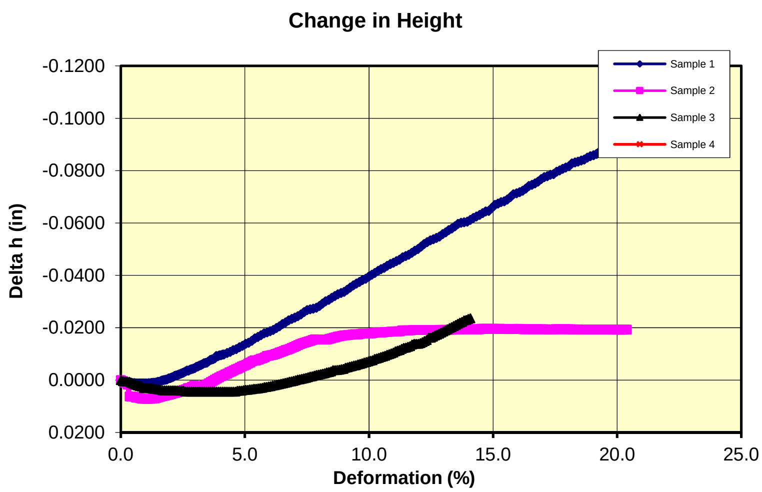
COOPER TESTING LABORATORY

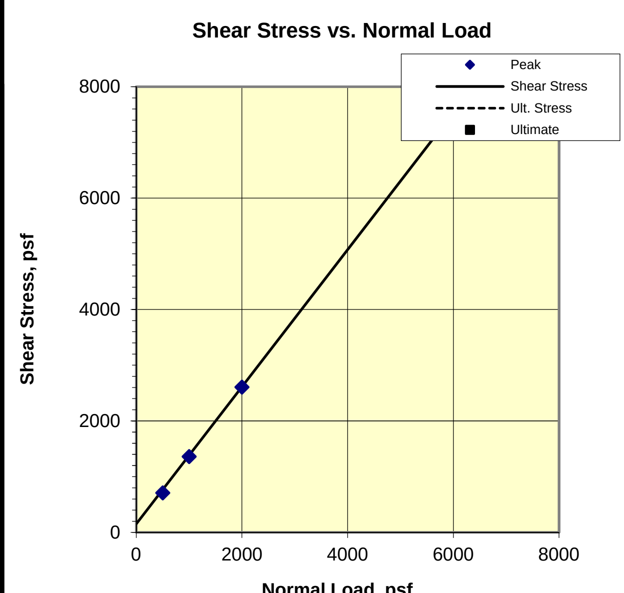
CTL Job #:	968-004	Project #:	170025	By:	MD
Client:	Vertical Sciences, Inc.	Date:	1/11/2018	Checked:	PJ
Project Name:	PID Reservoir B Project	Remolding Info:			

Phi (deg)	>45	Ult. Phi (deg)	
Cohesion (psf)	150	Ult. Cohesion (psf)	

Specimen Data	1	2	3	4
Boring:	PS2	PS2	PS2	
Sample:	2	2	2	
Depth (ft):	10	10	10	
Visual Description:	Brown Clayey GRAVEL w/ Sand	Brown Clayey GRAVEL w/ Sand	Brown Clayey GRAVEL w/ Sand	
Normal Load (psf)	500	1000	2000	
Dry Mass of Specimen (g)	139.5	142.5	143.0	
Initial Height (in)	1.00	1.01	1.00	
Initial Diameter (in)	2.42	2.42	2.42	
Initial Void Ratio	0.459	0.437	0.429	
Initial Moisture (%)	13.7	13.4	13.6	
Initial Wet Density (pcf)	131.4	133.0	134.1	
Initial Dry Density (pcf)	115.5	117.3	118.0	
Initial Saturation (%)	80.6	82.9	85.9	
ΔHeight Consol (in)	0.0087	0.0283	0.0218	
At Test Void Ratio	0.446	0.396	0.398	
At Test Moisture (%)	16.1	14.2	14.6	
At Test Wet Density (pcf)	135.3	137.9	138.2	
At Test Dry Density (pcf)	116.6	120.7	120.6	
At Test Saturation (%)	97.2	97.1	99.2	
Strain Rate (%/min)	0.01	0.01	0.01	
Strengths Picked at	5%	5%	5%	
Shear Stress (psf)	708	1360	2608	
ΔHeight (in) at 5%	-0.0131	0.0012	0.0039	
Ultimate Stress (psf)				







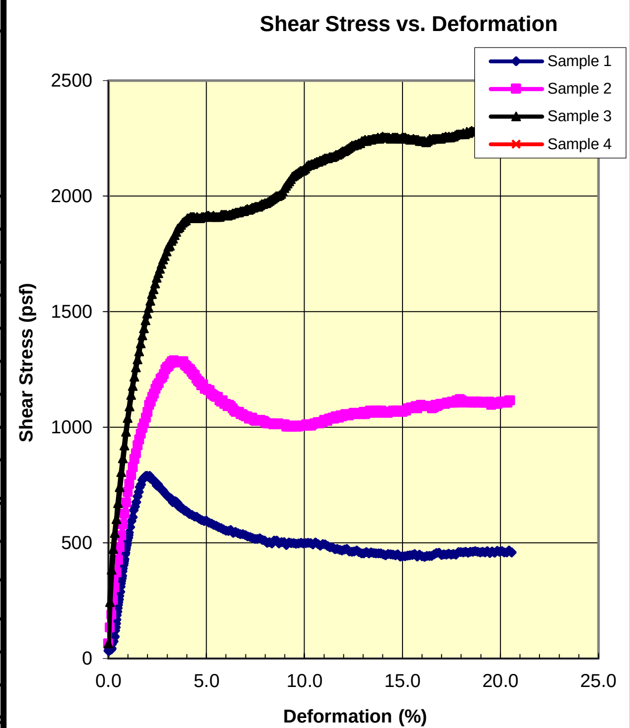
Remarks: Gravel in shear plane on all 3 samples may influence results. Due to the pre determined shear plane gravel can be problematic with the Direct Shear test.

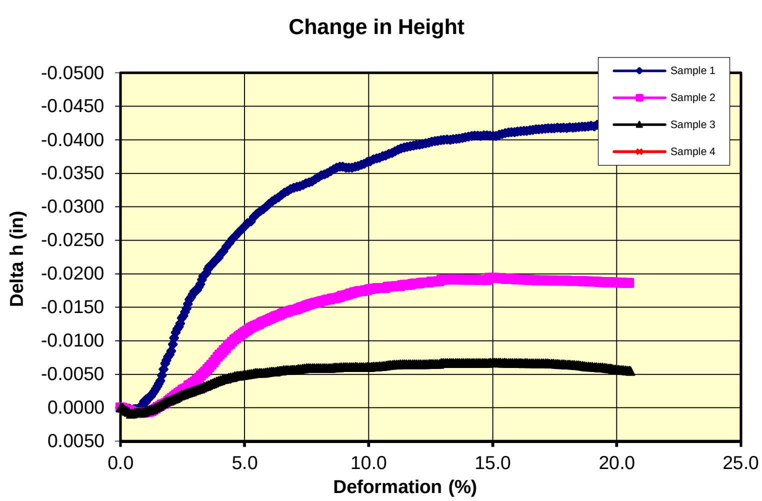
Consolidated Drained Direct Shear (ASTM D3080)

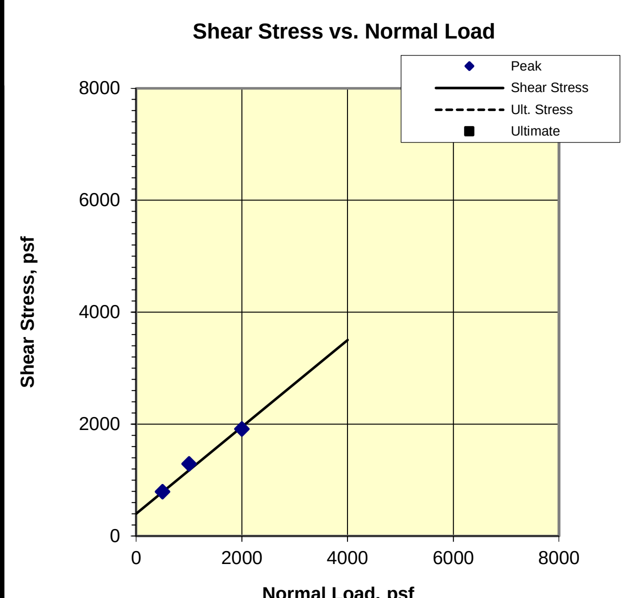
CTL Job #:	968-004	Project #:	170025	By:	MD
Client:	Vertical Sciences, Inc.	Date:	1/2/2018	Checked:	PJ
Project Name:	PID Reservoir B Project	Remolding Info:			

Phi (deg)	37.8	Ult. Phi (deg)	
Cohesion (psf)	400	Ult. Cohesion (psf)	

Specimen Data	1	2	3	4
Boring:	T-3	T-3	T-3	
Sample:	1	1	1	
Depth (ft):	5	5	5	
Visual Description:	Red Clayey SAND/ Sandy CLAY	Red Clayey SAND/ Sandy CLAY	Red Clayey SAND/ Sandy CLAY	
Normal Load (psf)	500	1000	2000	
Dry Mass of Specimen (g)	104.0	103.2	101.8	
Initial Height (in)	1.02	1.00	1.01	
Initial Diameter (in)	2.43	2.43	2.43	
Initial Void Ratio	1.086	1.061	1.112	
Initial Moisture (%)	34.0	32.5	34.0	
Initial Wet Density (pcf)	112.3	112.4	110.9	
Initial Dry Density (pcf)	83.8	84.8	82.8	
Initial Saturation (%)	87.6	85.8	85.7	
ΔHeight Consol (in)	0.0034	0.0053	0.0191	
At Test Void Ratio	1.079	1.050	1.072	
At Test Moisture (%)	37.7	36.4	38.3	
At Test Wet Density (pcf)	115.8	116.3	116.6	
At Test Dry Density (pcf)	84.1	85.3	84.4	
At Test Saturation (%)	97.9	97.0	99.9	
Strain Rate (%/min)	0.01	0.01	0.01	
Strengths Picked at	Peak	Peak	Peak	
Shear Stress (psf)	790	1289	1914	
ΔHeight (in) at Peak	-0.0079	-0.0051	-0.0055	
Ultimate Stress (psf)				







Remarks:

COOPER TESTING LABORATORY

Corrosivity Tests Summary

CTL #: 968-004 Date: 1/5/2018 Tested By: PJ Checked: PJ
Client: Vertical Sciences Project: PID Resrvoir B Project Proj. No: 170025
Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP		Sulfide	Moisture	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg	mg/kg	%		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	%	
Boring	Sample, No.	Depth, ft.	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
T1	-	3-5	-	-	23,653	<2	16	0.0016	6.1	-	-	-	17.4	Red CLAY w/ Sand (Silty)
PS1	-	1-5	-	-	5,299	<2	96	0.0096	6.6	-	-	-	7.2	OLIve Brown Clayey GRAVEL w/ Sand

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.: 3195-007
Report No.: 0300-001
Date: 12/22/17

Project: Paradise B Reservoir (Job No. 170025)
Butte County, California

Submitted by: Client
Submitted Date: 12/13/17

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

Sample #	Description	Dry Density p.c.f.	Moisture Content %	Liquid Limit	Plastic Limit	Plastic Index
T1-1 @ 5.0'	Red Sandy Clay with Gravel (visual)	91.7	28.4	50	25	25
T1-2 @ 10.0'	Red Sandy Clay with Gravel (visual)	76.6	27.2	---	---	---
T1-3 @ 15.0'	Mottled Red Sandy Clay with Gravel (visual)	85.9	29.8	---	---	---
T1-5 @ 25.0'	Mottled Brown & Gray Sandy Clay (visual)	75.0	42.9	---	---	---
T1-7 @ 35.0'	Mottled Brown & Gray Sandy Clay (visual)	68.2	47.1	---	---	---
T1-9 @ 41.5'	Brown & White Sandy Clay (visual)	61.7	57.1	---	---	---
T2-1 @ 5.0'	Red Sandy Clay (visual)	68.9	20.2	---	---	---
T2-2 @ 10.0'	Red Sandy Clay with Gravel (visual)	80.0	24.4			
T2-3 @ 15.0'	Red Sandy Gravel (visual)	83.5	27.7	---	---	---
T2-4 @ 20.0'	Mottled Gray & Brown Sandy Clay (visual)	72.7	43.9	---	---	---
T2-5 @ 25.0'	Mottled Brown Sandy Clay (visual)	66.0	50.3	---	---	---
T2-6 @ 30.0'	Reddish Brown Sandy Clay (visual)	---	37.9	---	---	---
T2-7 @ 35.0'	Reddish Brown Sandy Clay (visual)	---	44.0	---	---	---
T2-8@ 40.0'	Reddish Brown Sandy Clay (visual)	---	39.2	---	---	---

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865 Cotting Lane, Suite A
Vacaville, California 95688
(707) 447-4025, fax 447-4143

		Page No.:	2 of 3
Client:	Vertical Sciences, Inc. P.O. Box 491535 Redding, CA 96049	**Client No.:** **Report No.:** **Date:**	3195-007 0300-001 12/22/17
Project:	Paradise B Reservoir (Job No. 170025) Butte County, California	**Submitted by:** **Submitted Date:**	Client 12/13/17

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

Sample #	Description	Dry Density p.c.f.	Moisture Content %	Liquid Limit	Plastic Limit	Plastic Index
T3-3 @ 15.0'	Red Sandy Clay with Gravel (visual)	80.6	34.7	---	---	---
T3-5 @ 25.0'	Brown Sandy Clay with Gravel (visual)	71.7	46.9	---	---	---
T3-6 @ 35.0'	Dark Brown Sandy Clay with Gravel (visual)	72.8	43.3	---	---	---
T3-7 @ 40.0'	Mottled Reddish Brown Sandy Clay with Gravel (visual)	75.2	42.3	---	---	---
T4-1 @ 5.0'	Red Clay (visual)	---	27.7	58	21	37
T4-2 @ 10.0'	Red Clay (visual)	---	28.7	---	---	---
T4-3 @ 15.0'	Red Sandy Clay (visual)	---	34.8	---	---	---
T4-4 @ 20.0'	Red Sandy Clay (visual)	---	48.0	---	---	---
T4-5 @ 25.0'	Reddish Brown Sandy Clay (visual)	---	48.4	---	---	---
T4-6 @ 30.0'	Reddish Brown Sandy Clay (visual)	---	47.0	---	---	---
T4-7 @ 35.0'	Reddish Brown Sandy Clay (visual)	---	49.2	---	---	---
T4-8 @ 40.0'	Brown Sandy Clay (visual)	---	45.9	---	---	---
PS2-1 @ 5.0'	Brown Sandy Gravely Clay (visual)	105.7	8.4	---	---	---
PS2-3 @ 10.0'	Brown Sandy Clay with Gravel (visual)	103.8	15.7	---	---	---

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Date: 12/22/17

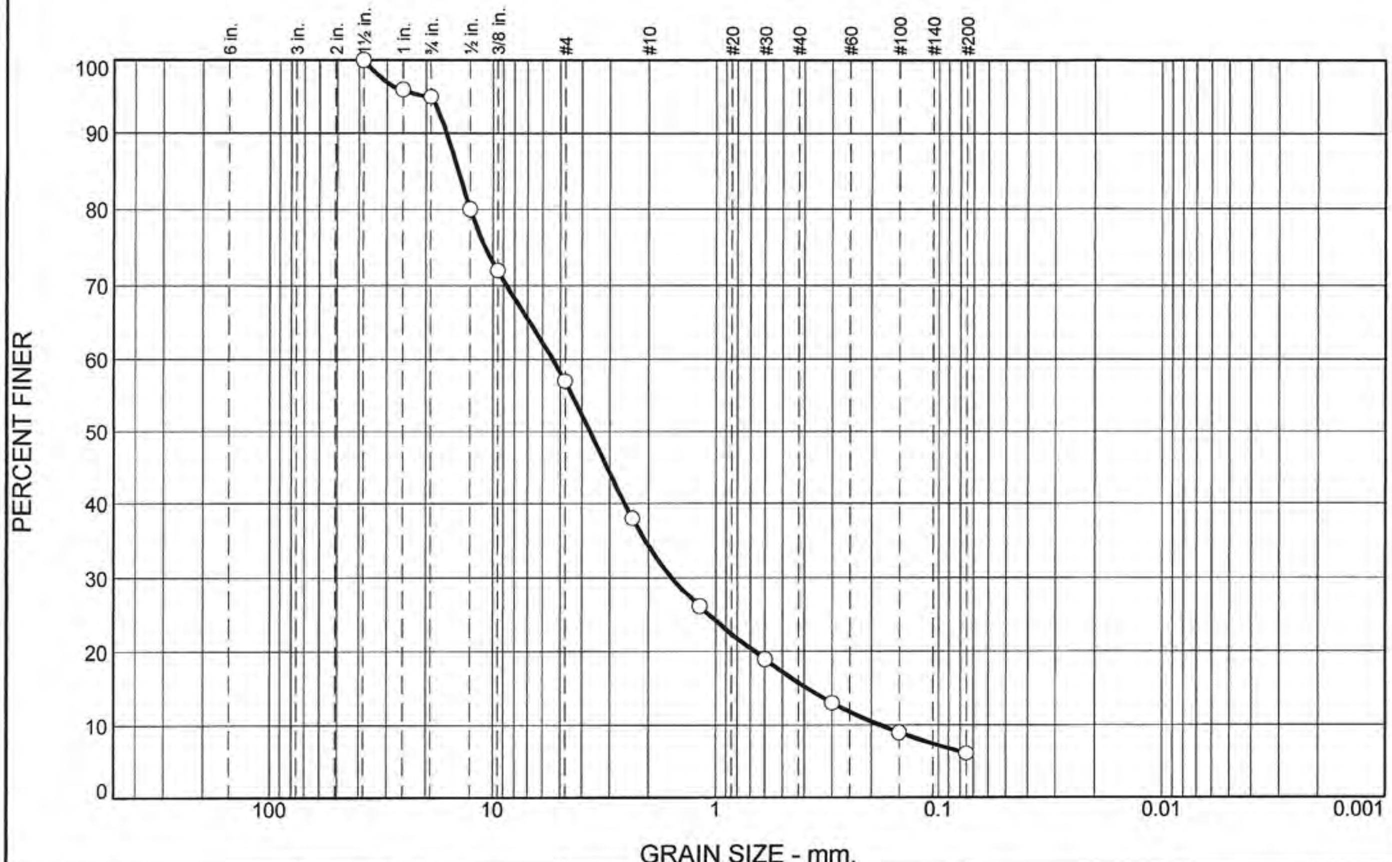
Project: Paradise B Reservoir (Job No. 170025)
Butte County, California

Submitted by: Client
Submitted Date: 12/13/17

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

Sample #	Description	Dry Density p.c.f.	Moisture Content %	Liquid Limit	Plastic Limit	Plastic Index
PL1-1 @ 5.0'	Gray Sand with Silt & Gravel (visual)	121.0	4.1	---	---	---
PL1-2 @ 10.0'	Gray Gravely Sand (visual)	109.4	4.1	---	---	---
PL2-1 @ 5.0'	Red Sandy Clay with Gravel (visual)	75.8	23.1	---	---	---
PL3-1 @ 5.0'	Red Sandy Clay (visual)	81.7	29.5	---	---	---
PL4-1 @ 5.0'	Olive Gray Sandy Clay with Gravel (visual)	118.6	6.9	---	---	---
PL5-1 @ 5.0'	Brown Sandy Clay with Gravel (visual)	82.8	28.3	---	---	---
PL6-1 @ 5.0'				---	---	---

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	5	38	23	18	10	6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2"	100		
1"	96		
3/4"	95		
1/2"	80		
3/8"	72		
#4	57		
#8	38		
#16	26		
#30	19		
#50	13		
#100	9		
#200	6.1		

* (no specification provided)

Material Description

Gray Sand with Silt & Gravel (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 16.1520 D_{85}= 14.3444 D_{60}= 5.3889
D_{50}= 3.6667 D_{30}= 1.5703 D_{15}= 0.3857
D_{10}= 0.1841 C_u= 29.28 C_c= 2.49

Classification

USCS= SP-SC AASHTO=

Remarks

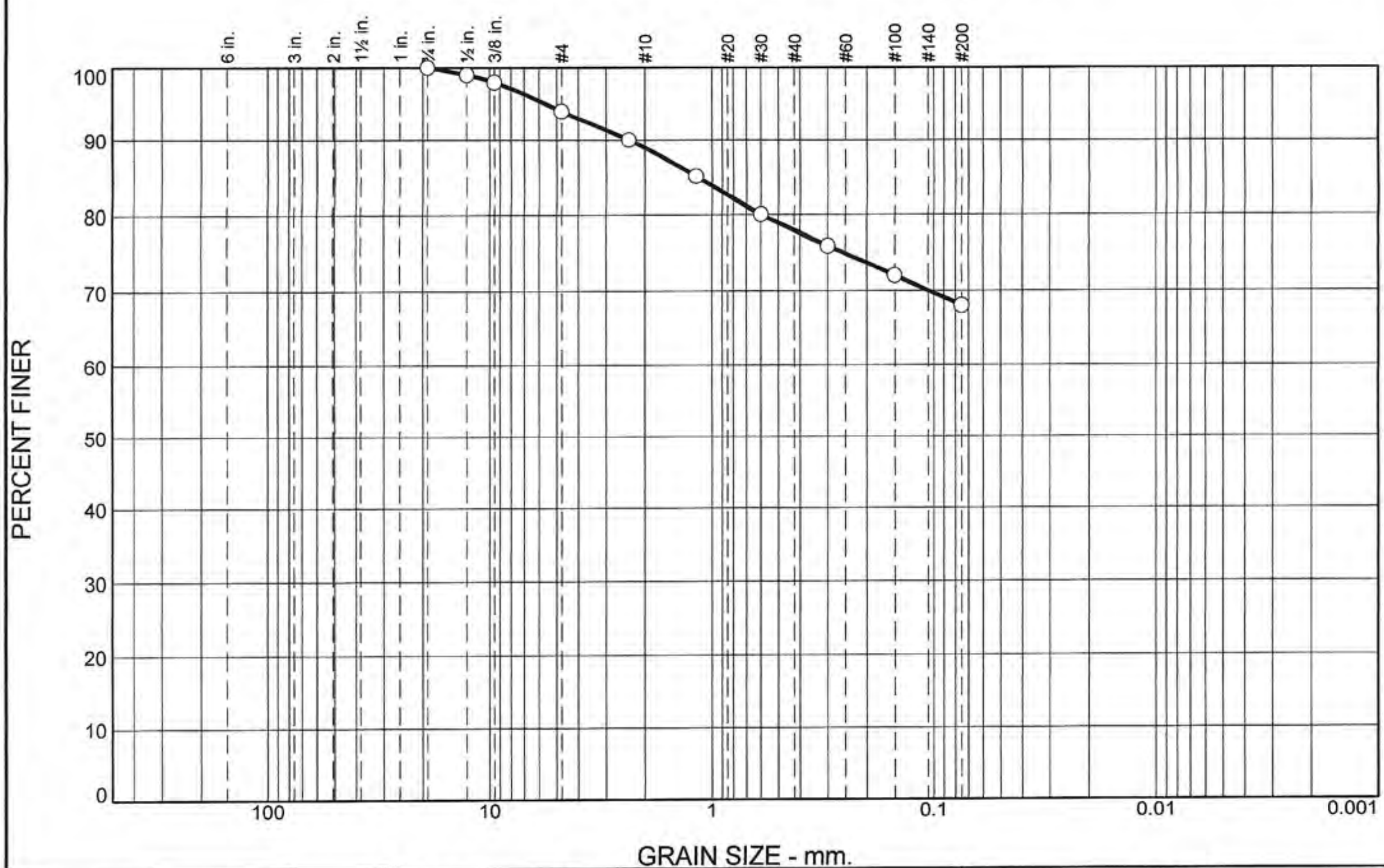
Material tested in accordance with ASTM D6913.

Location: PL1-1
Sample Number: 44 **Depth:** 5.0' **Date:** 12/22/17

Materials Testing, Inc.

Client: Vertical Sciences, Inc.
Project: Paradise B Reservoir (Job No. 170025)
Butte County, California
Project No: 3195-007 **Figure** 0300-002

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	6	5	11	10	68	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100		
1/2"	99		
3/8"	98		
#4	94		
#8	90		
#16	85		
#30	80		
#50	76		
#100	72		
#200	68		

* (no specification provided)

Material Description

Reddish Brown Sandy Clay (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 2.3600 D_{85}= 1.1800 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

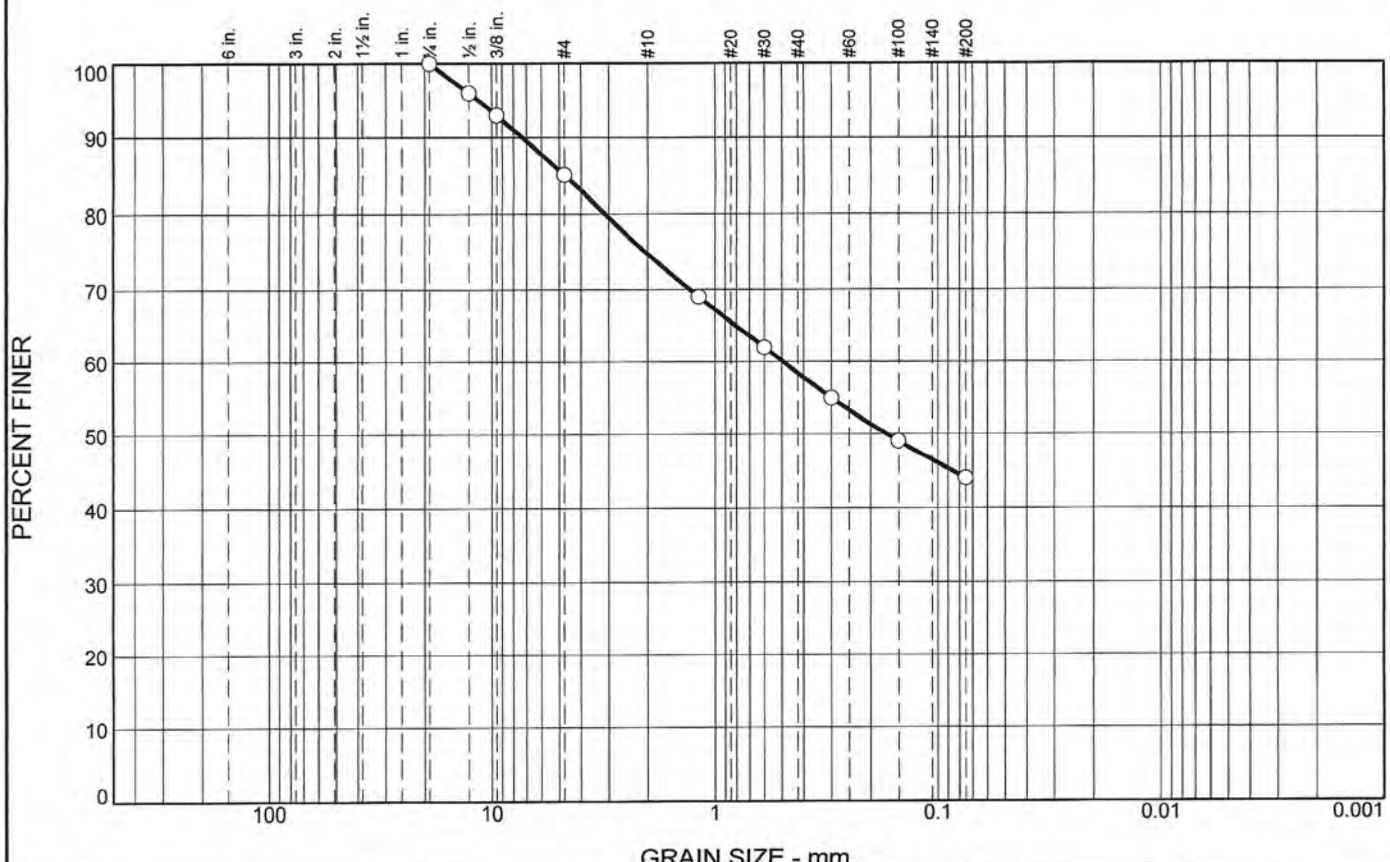
Material tested in accordance with ASTM D6913.

Location: PL2-B1
Sample Number: 46 **Depth:** 1.0-5.0' **Date:** 12/22/17

Materials Testing, Inc.

Client: Vertical Sciences, Inc.
Project: Paradise B Reservoir (Job No. 170025)
Butte County, California
Project No: 3195-007 **Figure** 0300-003

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	15	10	17	14	44	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100		
1/2"	96		
3/8"	93		
#4	85		
#16	69		
#30	62		
#50	55		
#100	49		
#200	44		

* (no specification provided)

Material Description

Olive Silty Sand with Gravel (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 7.2779 D_{85}= 4.7500 D_{60}= 0.4941
D_{50}= 0.1701 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

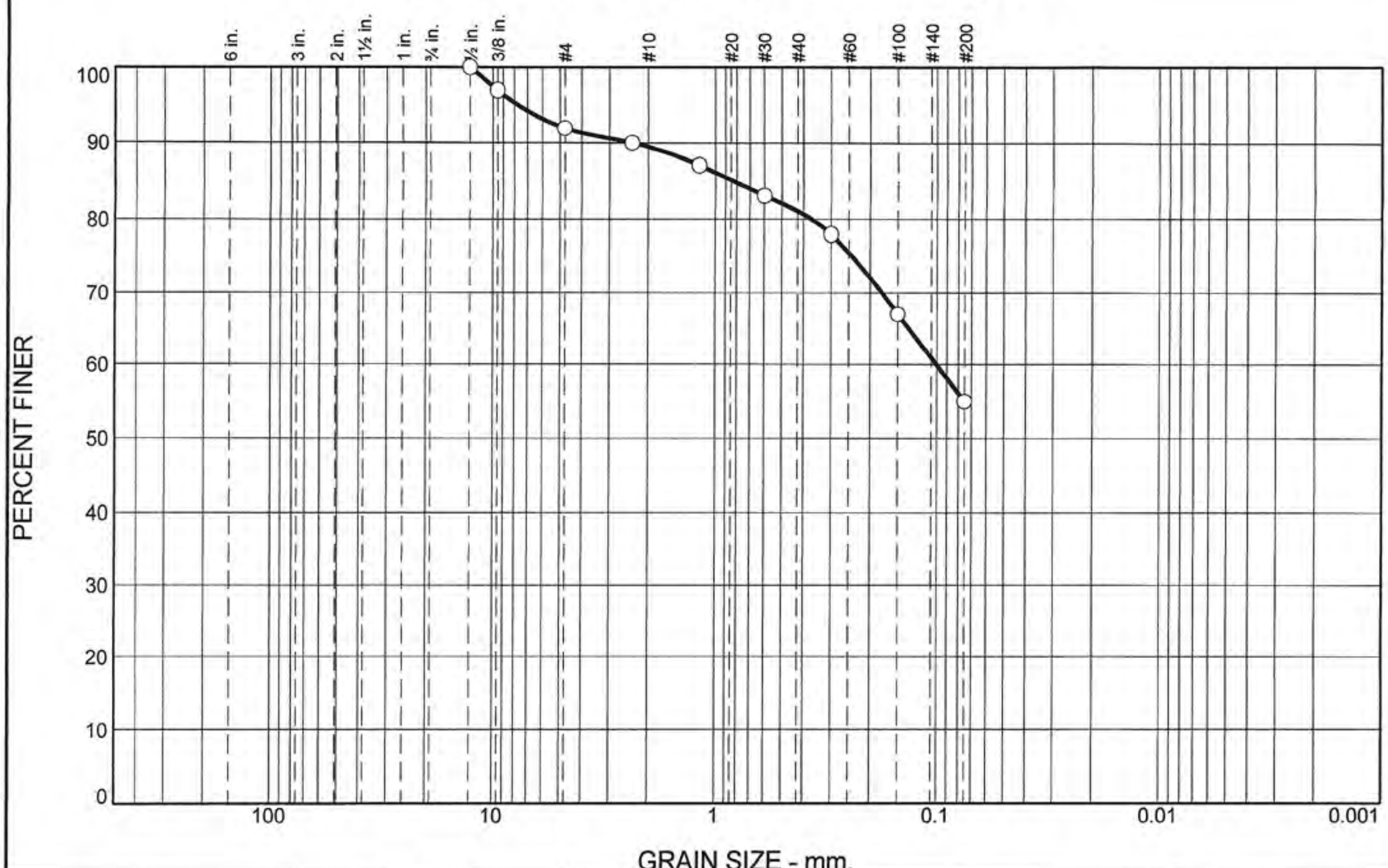
Material tested in accordance with ASTM D6913.

Location: PL4-B1
Sample Number: 51 **Depth:** 1.0-5.0' **Date:** 12/22/17

MTI Materials Testing, Inc.

Client: Vertical Sciences, Inc.
Project: Paradise B Reservoir (Job No. 170025)
Butte County, California
Project No: 3195-007 **Figure** 0300-004

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	8	3	8	26	55	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2"	100		
3/8"	97		
#4	92		
#8	90		
#16	87		
#30	83		
#50	78		
#100	67		
#200	55		

* (no specification provided)

Material Description

Brown & Black Sandy Clay (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 2.3600 D_{85}= 0.8411 D_{60}= 0.1004
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

Material tested in accordance with ASTM D6913.

Location: PL6-1
Sample Number: 56 **Depth:** 5.0' **Date:** 12/22/17

Materials Testing, Inc.

Client: Vertical Sciences, Inc.
Project: Paradise B Reservoir (Job No. 170025)
Butte County, California
Project No: 3195-007 **Figure** 0300-005

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						
3.0-10.0'							4.57	

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 94.1 pcf Optimum moisture = 26.1 %	Red Sandy Clay with Gravel (visual)

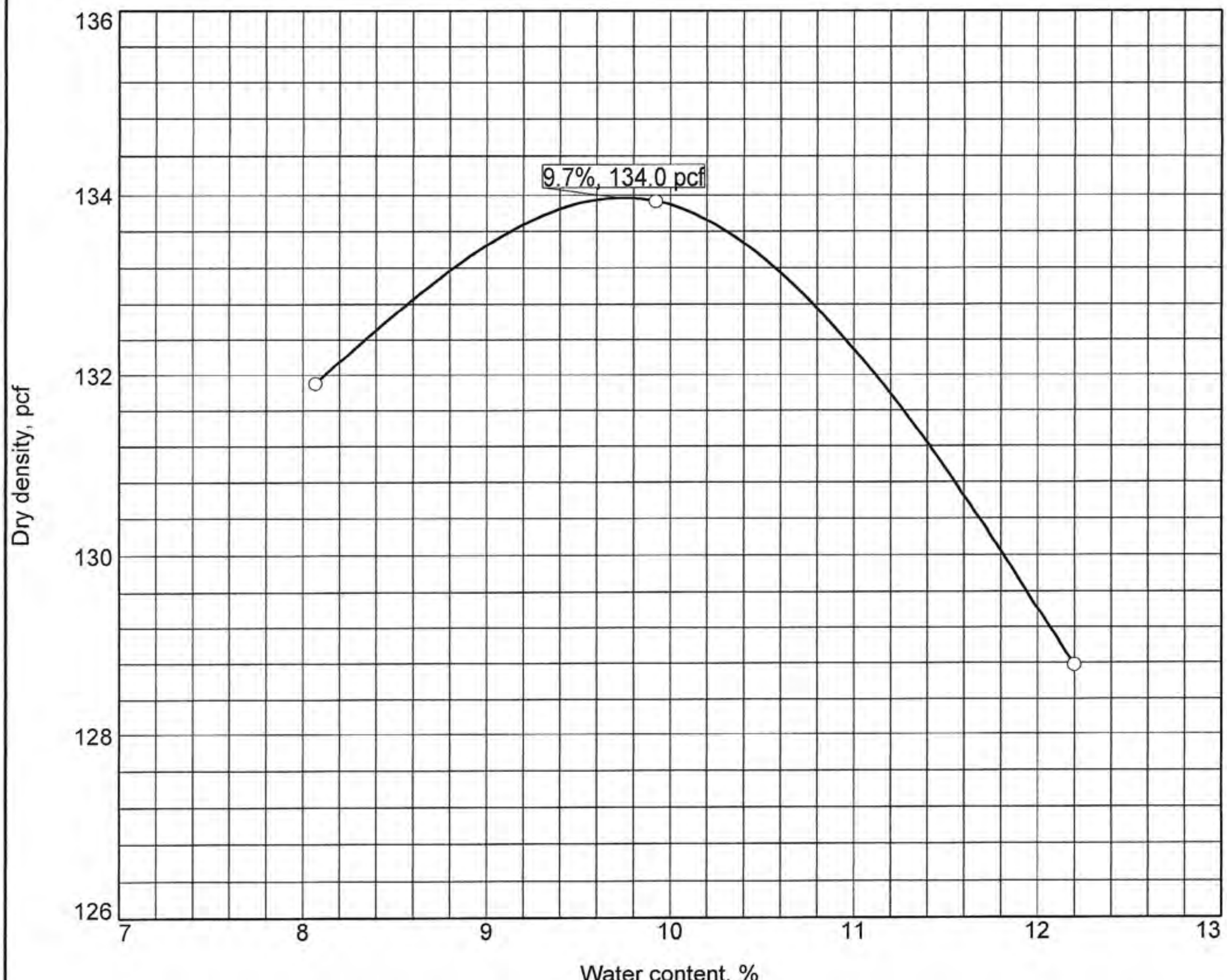
Project No. 3195-007 **Client:** Vertical Sciences, Inc.
Project: Paradise B Reservoir (Job No. 170025)
Butte County, California
○ **Location:** T1-B1 **Sample Number:** 11

Materials Testing, Inc.

Remarks:
Curve #1
(12/22/17)

Figure 0300-006

COMPACTION TEST REPORT



Test specification: ASTM D1557-12 Method B Modified
ASTM D4718-15 Oversize Corr. Applied to Each Test Point

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/8 in.	% < No.200
	USCS	AASHTO						
1.0-5.0'				2.6			12.9	

ROCK CORRECTED TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 134.0 pcf Optimum moisture = 9.7 %	Light Brown Sandy Clay with Gravel (visual)

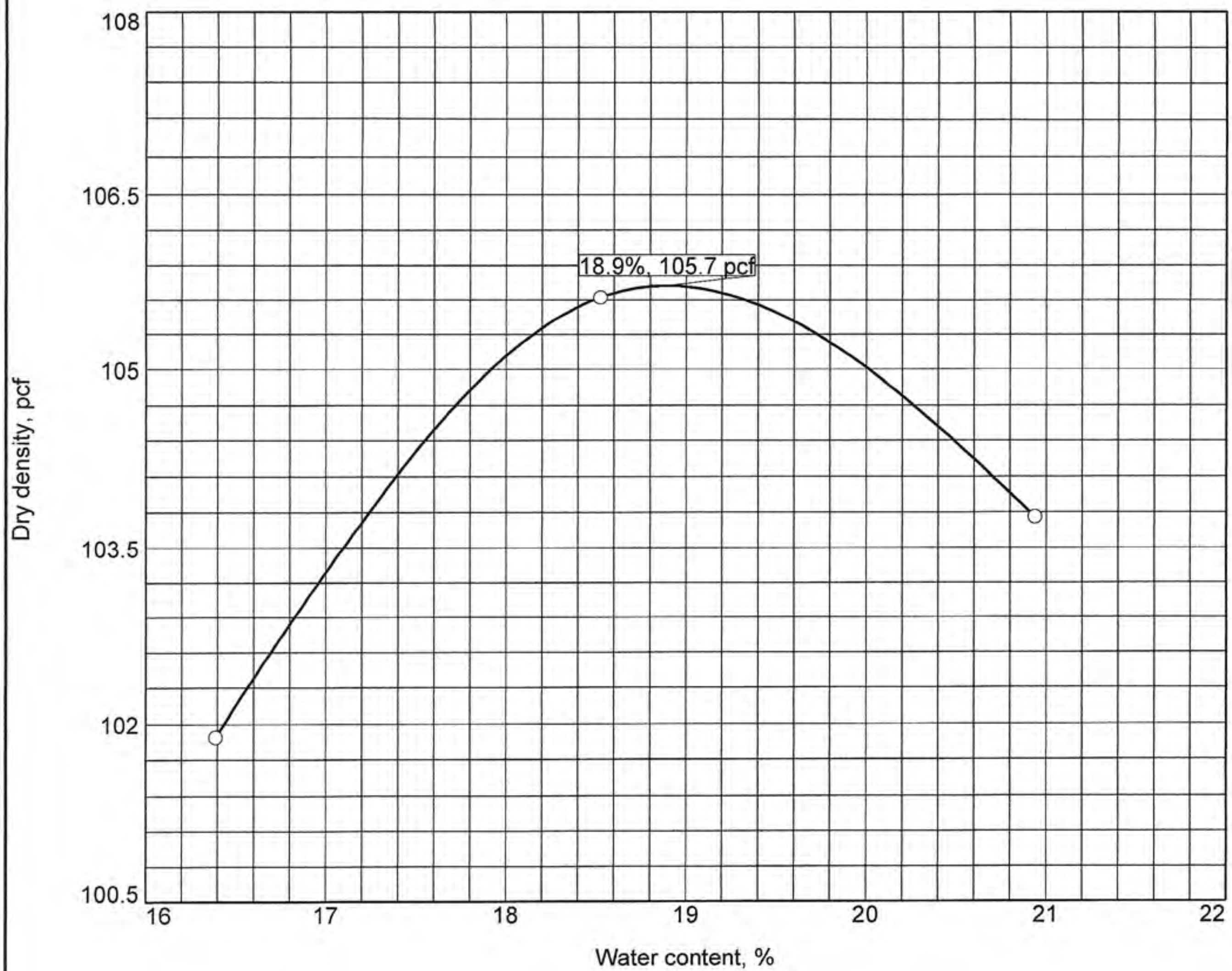
Project No. 3195-007 **Client:** Vertical Sciences, Inc.
Project: Paradise B Reservoir (Job No. 170025)
Butte County, California
○ **Location:** PS1-B1 **Sample Number:** 37

Materials Testing, Inc.

Remarks:
Curve #2
(12/22/17)

Figure 0300-007

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						
1.0-5.0'							3.5	

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 105.7 pcf Optimum moisture = 18.9 %	Reddish Brown Sandy Clay (visual)

Project No. 3195-007 **Client:** Vertical Sciences, Inc.
Project: Paradise B Reservoir (Job No. 170025)
Butte County, California
○ **Location:** PL2-B1 **Sample Number:** 46

Materials Testing, Inc.

Remarks:
Curve #3
(12/22/17)

Figure 0300-008

COMPACTION TEST REPORT

Test specification: ASTM D1557-12 Method B Modified
ASTM D4718-15 Oversize Corr. Applied to Each Test Point

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/8 in.	% < No.200
	USCS	AASHTO						
1.0-5.0'				2.6			5.7	

ROCK CORRECTED TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 134.8 pcf Optimum moisture = 9.0 %	Olive Silty Sand with Gravel (visual)

Project No. 3195-007 **Client:** Vertical Sciences, Inc.
Project: Paradise B Reservoir (Job No. 170025)
Butte County, California
○ **Location:** PL4-B1 **Sample Number:** 51

Materials Testing, Inc.

Remarks:
Curve #4
(12/22/17)

Figure 0300-009

ASBESTOS TEM LABORATORIES, INC.

CARB Method 435
Polarized Light Microscopy
Analytical Report

Laboratory Job # 96-02476

630 Bancroft Way
Berkeley, CA 94710
(510) 704-8930
FAX (510) 704-8429

Dec/21/2017

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

RE: LABORATORY JOB # 96-02476
Polarized light microscopy analytical results for 2 bulk sample(s).
Job Site: 170025
Job No.: PID Reservoir B Study

Enclosed please find the bulk material analytical results for one or more samples submitted for asbestos analysis. The analyses were performed in accordance with the California Air Resources Board (ARB) Method 435 for the determination of asbestos in serpentine aggregate samples.

Prior to analysis, samples are logged-in and all data pertinent to the sample recorded. The samples are checked for damage or disruption of any chain-of-custody seals. A unique laboratory ID number is assigned to each sample. A hard copy log-in sheet containing all pertinent information concerning the sample is generated. This and all other relevant paper work are kept with the sample throughout the analytical procedures to assure proper analysis.

Sample preparation follows a standard CARB 435 prep method. The entire sample is dried at 135-150 C and then crushed to ~3/8" gravel size using a Bico Chipmunk crusher. If the submitted sample is >1 pint, the sample was split using a 1/2" riffle splitter following ASTM Method C-702-98 to obtain a 1 pint aliquot. The entire 1 pint aliquot, or entire original sample, is then pulverized in a Bico Braun disc pulverizer calibrated to produce a nominal 200 mesh final product. If necessary, additional homogenization steps are undertaken using a 3/8" riffle splitter. Small aliquots are collected from throughout the pulverized material to create three separate microsope slide mounts containing the appropriate refractive index oil. The prepared slides are placed under a polarizing light microscope where standard mineralogical techniques are used to analyze the various materials present, including asbestos. If asbestos is identified and of less than 10% concentration by visual area estimate then an additional five sample mounts are prepared. Quantification of asbestos concentration is obtained using the standard CAL ARB Method 435 point count protocol. For samples observed to contain visible asbestos of less than 10% concentration, a point counting techinique is used with 50 points counted on each of eight sample mounts for a total of 400 points. The data is then compiled into standard report format and subjected to a thorough quality assurance check before the information is released to the client.

While the CARB 435 method has much to commend it, there are a number of situations where it fails to provide sufficient accuracy to make a definitive determination of the presence/absence of asbestos and/or an accurate count of the asbestos concentration present in a given sample. These problems include, but are not limited to, 1) statistical uncertainty with samples containing <1% asbestos when too few particles are counted, 2) definitive identification and discrimination between various fibrous amphibole minerals such as tremolite/actinolite/hornblende and the "Libby amphiboles" such as tremolite/winchite/richterite/arfvedsonite, and C) small asbestiform fibers which are near or below the resolution limit of the PLM microscope such as those found in various California coast range serpentine bodies. In these cases, further analysis by transmission electron microscopy is recommended to obtain a more accurate result.

Sincerely Yours,

R. Mc Bain

Lab Manager
ASBESTOS TEM LABORATORIES, INC.

--- These results relate only to the samples tested and must not be reproduced, except in full, without the approval of the laboratory. ---

POLARIZED LIGHT MICROSCOPY
CARB 435 ANALYTICAL REPORT

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

Samples Submittec 2
Samples Analyzed: 2
Job Site / No. PID Reservoir B Study
170025

Report No. 354995
Date Submitted: Dec-14-17
Date Reported: Dec-21-17

SAMPLE ID	POINTS COUNTED	ASBESTOS %	ASBESTOS TYPE	LOCATION / DESCRIPTION
PID PL1 @ 3' Lab ID # 96-02476-001	**24** / **400** - Total Points	**6 %**	**Chrysotile**	Serpentinite Chrysotile and Antigorite fibers observed.
PID PS1 1'- 5' Lab ID # 96-02476-002	**400** - Total Points	**<0.25%**	**None Detected**	Clayey sand, potentially derived from ultramafics. No Asbestos Detected
Lab ID #	- Total Points			
Lab ID #	- Total Points			
Lab ID #	- Total Points			
Lab ID #	- Total Points			
Lab ID #	- Total Points			
Lab ID #	- Total Points			
Lab ID #	- Total Points			
Lab ID #	- Total Points			

QC Reviewer R. Mel Bai

Analys Jo Ann Hnatko

354955

ASBESTOS TEM LABORATORIES CHAIN OF CUSTODY – www.asbestostemlabs.com

CALIFORNIA: 600 Bancroft Way, Ste. A, Berkeley, CA 94710 Phone (510) 704-8930 Fax (510) 704-8429
NEVADA: 1350 Freeport Blvd. #104, Sparks, NV 89431 Phone (775) 359-3377 Fax (775) 359-2798

Please print and send completed CoC with your samples. If you wish to email CoC, send the form as an attachment to Berkeley <coc@asbestostemlabs.com> or Reno <sehrlich@asbestostemlabs.com>.

Company: Consulting Associates of California	Contact: Gary Hennis	Phone/Fax: 925/285-4097	Email: ghennis@comcast.net
Address: 1 Casey Glen Court	City: Clayton	State: CA Zip: 94517	Country: United States
Job Site: Westside Courts Apartment Units 109, 209 & 309		Job No:	P.O. No:

Reporting: ☐ Fax ☐ Phone ☒ Email ☐ Mail ☐ FTP ☐ EDD/State Form ☐ Verbal ☐ Pickup **Billing:** ☐ Fax ☐ Email ☐ Mail ☐ Pre-Paid ☐ 3rd Party

Results Due:* ☒ 2 hr ☐ 4 hr ☐ 6 hr ☐ 8 hr ☐ 24 hr ☐ 48 hr ☐ 3 day ☐ 4 day ☐ 5 day ☐ 10 day ☐ Time due:____ * Contact lab to confirm TAT

Asbestos Air: ☒ PCM (NIOSH 7400A ☐ TEM AHERA ☐ TEM CARB Mod. AHERA ☐ TEM EPA *amate Level ☐ TEM NIOSH 7402, Issue 2 ☐ ISO 10312 ☐ ISO 13794

Asbestos Bulk: ☐ PLM Standard (EPA 600/R-93-1 ☐ PLM 400 PC ☐ PLM 1000 PC ☐ PLM 400 PC Grav. Red ☐ PLM 1000 PC Grav. Red ☐ TEM EPA Qualitative ☐ TEM EPA Quantitative ☐ TEM Chatfield (Semi-Quant) ☐ PLM Vermiculite Attic Insulation ☐ Custom Analysis: Type:

Asbestos Soils: ☐ CARB 435 Prep Only ☐ CARB 435 PLM 400 PC ☐ CARB 435 PLM 1000 PC ☐ EPA Soil Screening Qualitative ☐ TEM EPA/CARB Quantitative

Asbestos Dust: ☐ ASTM D-5755 Fiber Count ☐ ASTM D-5756 Wt. % ☐ ASTM D-5756 Mass ☐ ASTM D-6840-99 Dust Wipe ☐ Total Particulates (Grav.)

Asbestos Water: ☐ 100.2 Potable Drinking Water ☐ 100.1 Non Potable Water

Lead: ☐ Paint Chips ☐ Dust Wipe ☐ Air Cassette ☐ Soil **Lead Waste Characterization:** ☐ TTLC ☐ STLC ☐ TCLP

Sample Storage: *☐ No Test, Hold Sample Until:________ *☐ Post Test, Hold Sample Until:________

Custom Order: ☐ Reanalysis by:________ ☐ Sensitivity:________ ☐ Composite ☐ Other:

Sample #	Sample Type	Date Collected	Time On	Time Off	Total Time (min)	Flow Rate (lpm) On	Flow Rate (lpm) Off	Flow Rate (lpm) Average	Volume or Area Sampled	8 Hour TWA Requested	Description
WC-109-01	PCM	12-13	9:15	11:15	120	10	10	10	1200	☐	Clearance-Unit 109
WC-209-02	PCM	12-13	9:17	11:17	120	10	10	10	1200	☐	Clearance-Unit 209
WC-309-03	PCM	12-13	9:20	11:20	120	10	10	10	1200	☐	Clearance-Unit 309
										☐	
										☐	
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										☐	
										☐	
										☐	
										☐	
										☐	
										☐	

Submitted By	[signature]	Received By	[signature]
Date/Time Submitted	12-13-17	Date/Time Received	12/13/17 12:30
Submitted By		Received By	
Date/Time Submitted		Date/Time Received	

*All samples will be held for 3 months from the date of receipt at ATEM. Additional sample storage time may be obtained through ATEM Customer Service

APPENDIX C

Geophysical Refraction Survey Results

APPENDIX C
GEOPHYSICAL REFRACTION SURVEY RESULTS

Geophysical refraction surveys were performed along two survey lines at the project site. The surveys were performed on November 28, 2017 by Redpath Geophysics of Murphys, California. A VSI geologist assisted Redpath Geophysics during the surveys. The results of geophysical surveys and a discussion on methodology are included within this appendix.

REDPATH
GEOPHYSICS
P.O. BOX 540 MURPHYS CALIFORNIA 95247
209-728-3705

Mr. James A. Bianchin
Vertical Sciences Inc.
P.O. Box 491535
Redding, CA 96049
via email: jim.bianchin@verticalsciences.com

8 December 2017

Dear Mr. Bianchin,

This letter presents the results of seismic refraction surveys that were conducted at the site of the Paradise Irrigation District Reservoir 'B' in Paradise, California. Four lines were surveyed on 28 November with your assistance. The general intent of the seismic surveys was to provide subsurface information that would assist in an assessment of the foundation conditions for proposed concrete storage tanks.

Seismic Line No. 1 (SL-1) consisted of 24 geophones spaced at 10-ft intervals, for a length of 230 ft, and ran along the top of the berm that forms the northeast boundary of the existing reservoir. SL-2 had 12 geophones at 10-ft intervals for a length of 110 ft and crossed the entry road to the reservoir. SL-3 was a 230-ft line parallel to the southwest perimeter of the reservoir, and SL-4 was a 110-ft line parallel to the southeast boundary. These locations are shown on the attached Google Earth view of the site.

A 16-lb sledgehammer striking a one-inch-thick slab of high-density polyethylene on the ground was used as the energy source. The hammer has an impact sensor attached to the handle that triggers the recording process. Signals from 9 hammer-points were recorded for SL-1 and SL-3, and signals from 7 hammer points were recorded along SL-2 and SL-4, i.e., at points every 3 or 4 geophones along each line and at a point offset beyond each end of the line. All data were recorded on a Geometrics model R24 Strataview™ digital seismograph configured to record 24 or 12 channels, as required, each of which consisted of 1024 samples at intervals of 125 microseconds, for a total recording time of about 125 milliseconds.

The geophones' natural frequency is 4.5 Hz. The seismograph has the capability of adding or 'stacking' the signals from repeated hammer blows in order to improve the signal-to-noise ratio, and as many as 4 hammer blows were stacked at any given point. The seismic records were viewed on the R24's LCD screen as they were acquired and paper copies were printed on its internal printer for examination in the field. The data are stored on the internal hard drive of the R24 and ultimately copied to 3-1/2-inch diskettes in SEG-2 binary format; the data are subsequently transferred from the diskettes to the analysis software.

.
First arrivals and travel times are picked using the Pickwin component of Geometrics' SeisImager™ software which compiles a time vs. distance file for subsequent analysis. The time vs. distance plots are analyzed with the Plotrefa portion of SeisImager in which a two- or three-layer solution is developed first and then used as a starting model for a tomographic inversion of the travel-time data.

The results of the surveys are presented on the attached profiles in the format of both a simple layered velocity cross-section (the "time-term inversion") and a tomographic inversion of the time vs. distance data. The tomographic profiles for all four lines share the same velocity/color scale of 1000 ft/sec to 4000 ft/sec with a contour interval of 100 ft/sec.

The SeisImager software has the capability of calculating and displaying the ray paths from the sources (hammer points) to receivers (geophones). I used this feature to trim the depth of the color cross-sections to be just slightly below the computed maximum depth of penetration of the seismic signals. The software is also capable of calculating travel times from each source to each geophone in the tomographic model, and then comparing the observed and calculated times. The root-mean-square (rms) difference between the observed and calculated times is a measure of the validity of the solution. Plots of the comparisons of observed and calculated times are attached. The rms differences were between 1 and 1.5 milliseconds; the overall quality of the raw data, i.e., the waveforms, was good.

A tomographic inversion of the data, by its nature, represents a velocity interface as somewhat of a velocity gradation rather than a sharp, distinct boundary. However, this does not preclude the possible presence of thin layers or lenses of material with a relatively high velocity embedded in the second or third layers; any such thin layers simply cannot be resolved with a refraction survey. Nevertheless, there is absolutely no indication of any material with velocities characteristic of basaltic rock (8000 to 12,000 ft/sec) on any of the four cross-sections.

Please do not hesitate to contact me if you have questions about any aspect of these surveys or the results.

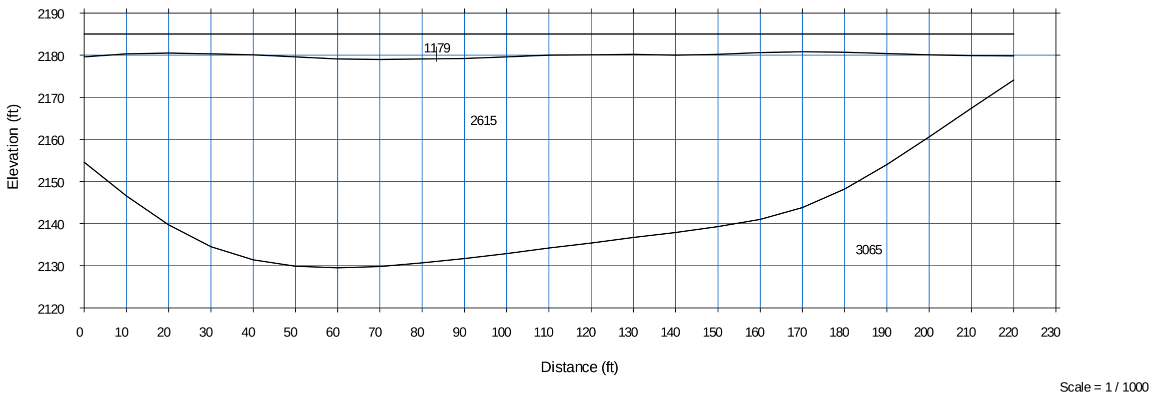
Sincerely,

Bruce B. Redpath

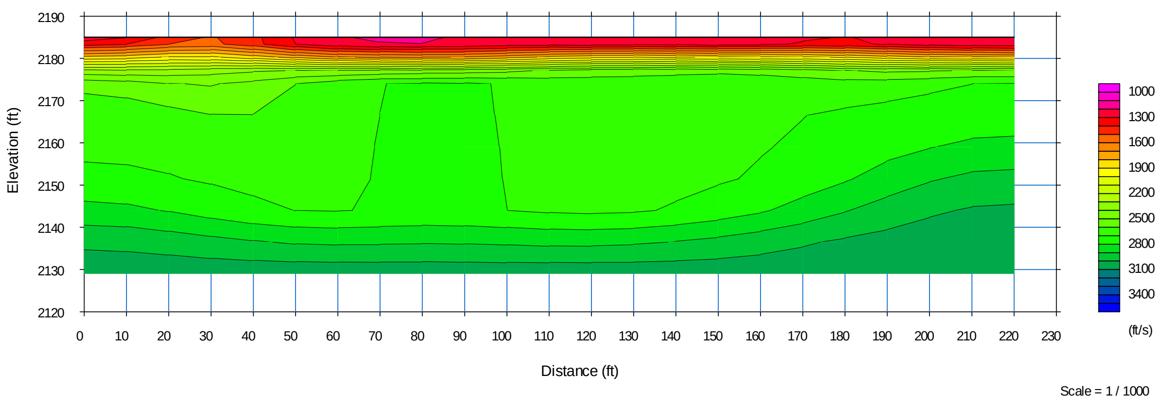
Bruce B. Redpath
California Registered Geophysicist GP-347

SE
0+00

NW
2+30

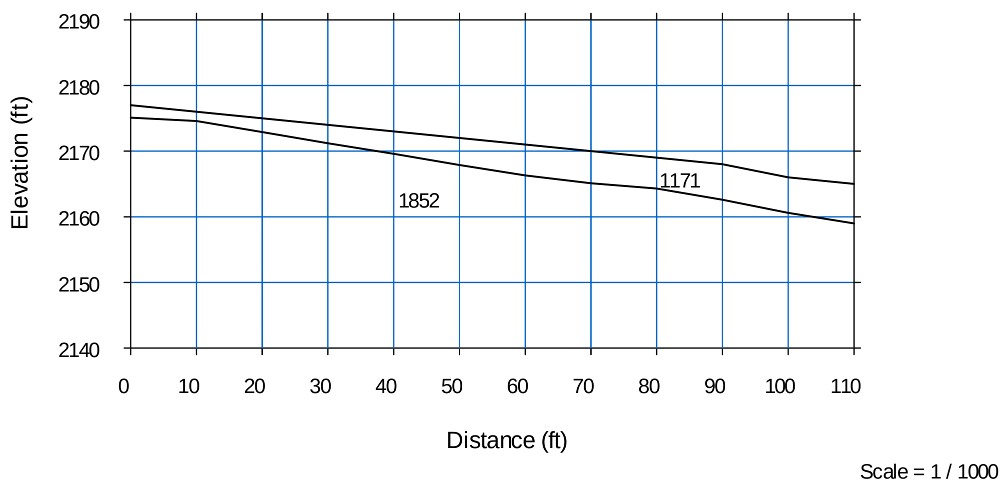


Time-Term Inversion of Time vs. Distance Data

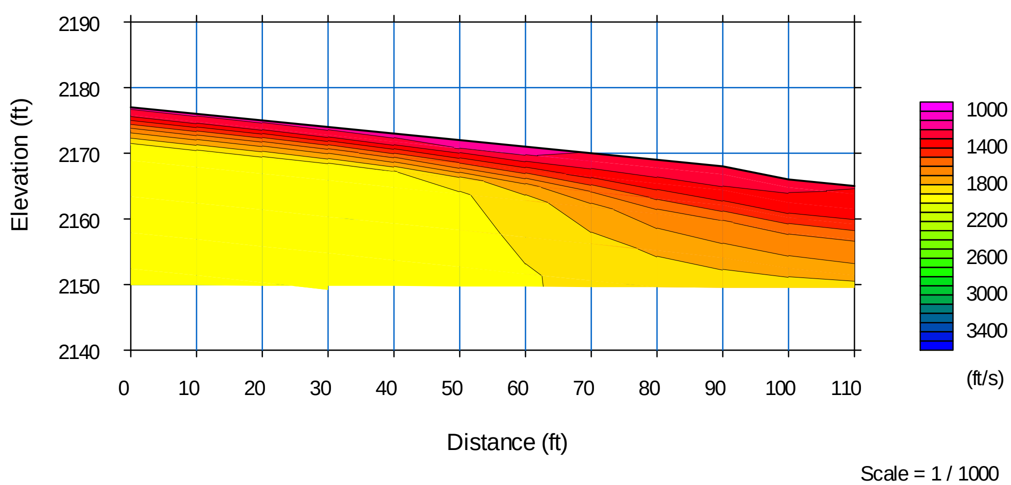


Tomographic Inversion of Time vs. Distance Data

Seismic Line No. 1
Paradise Irrigation District Reservoir B
Paradise – California – November 2107



Time-Term Inversion of Time vs. Distance Data

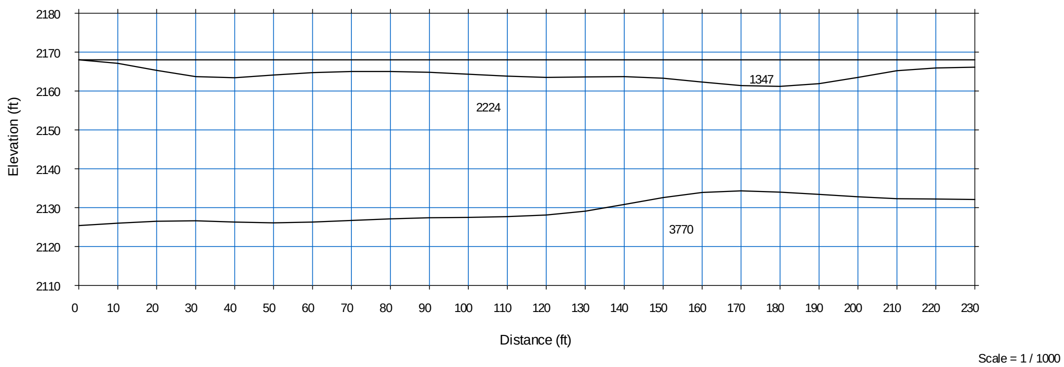


Tomographic Inversion of Time vs. Distance Data

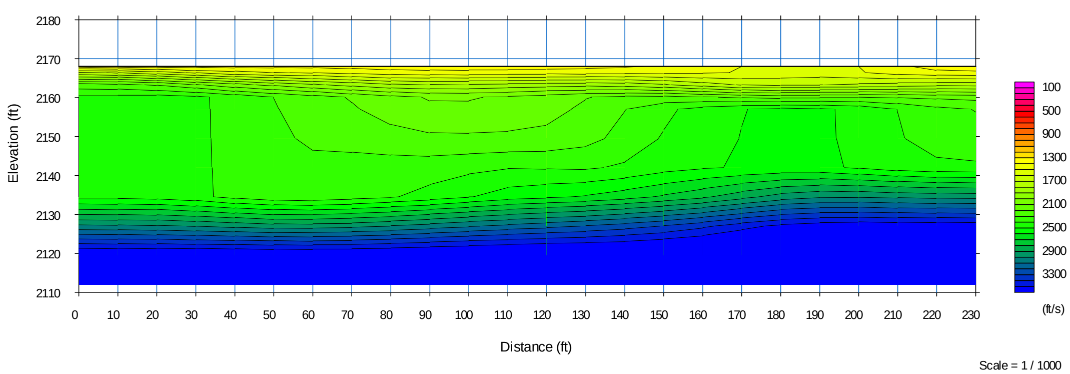
Seismic Line No. 2
Paradise Irrigation District Reservoir B
Paradise – California – November 2017

SE
0+00

NW
2+30

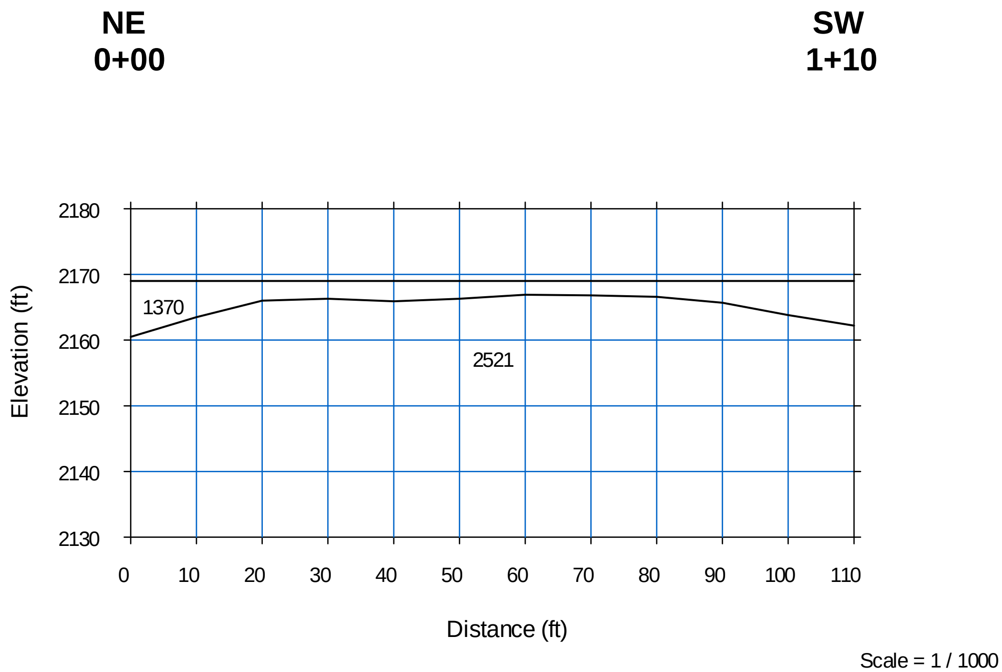


Time-Term Inversion of Time vs. Distance Data

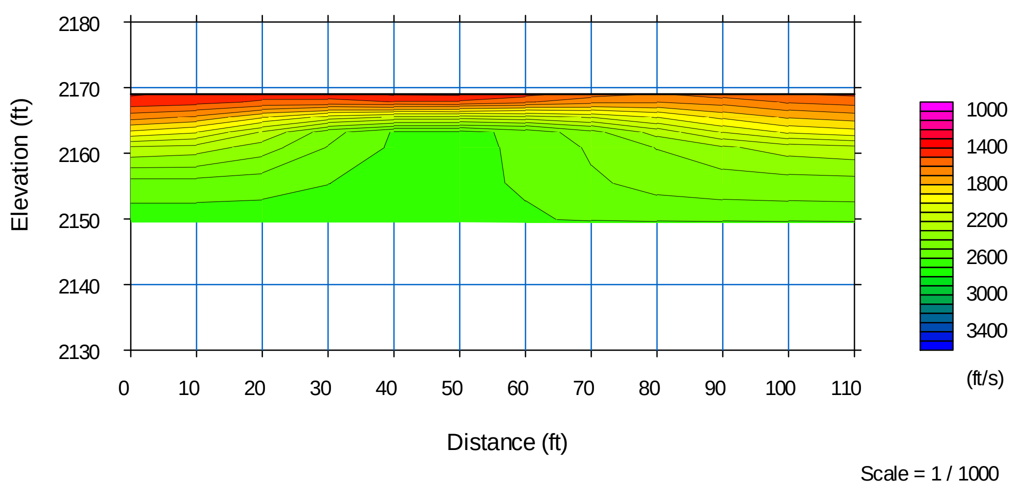


Tomographic Inversion of Time vs. Distance Data

Seismic Line No. 3
Paradise Irrigation District Reservoir B
Paradise – California – November 2017

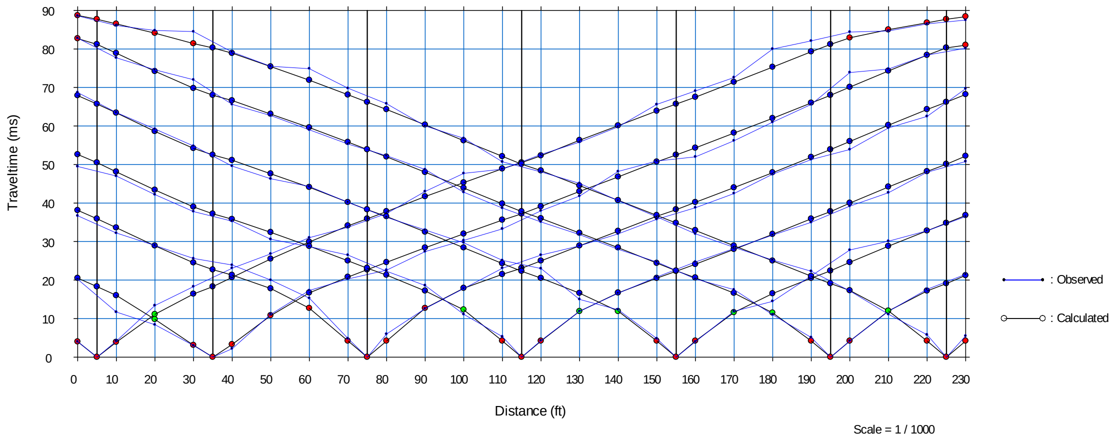


Time-Term Inversion of Time vs. Distance Data

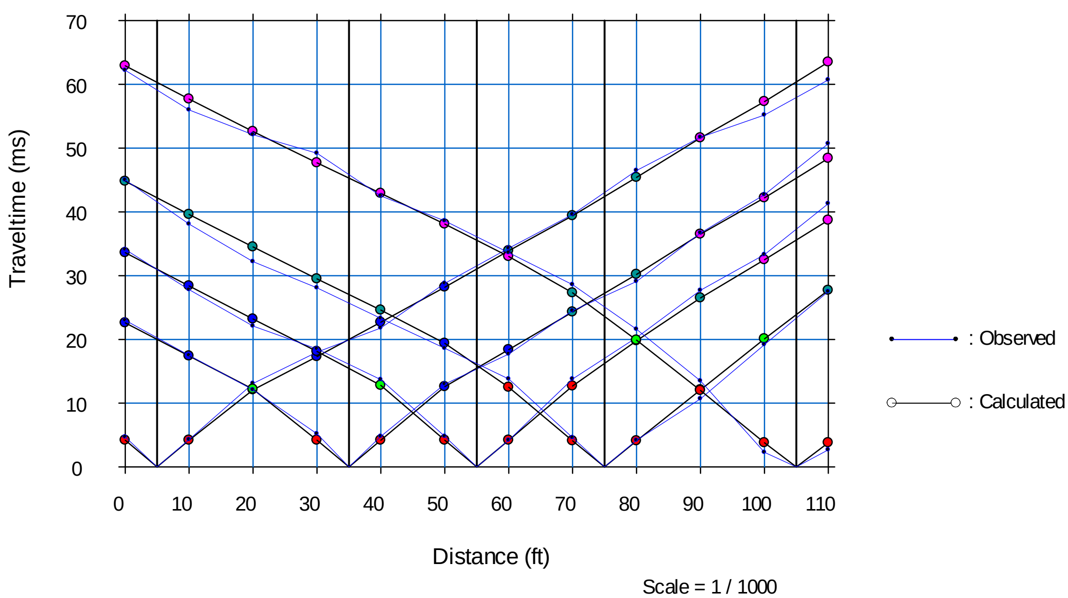


Tomographic Inversion of Time vs. Distance Data

Seismic Line No. 4
Paradise Irrigation District Reservoir B
Paradise – California – November 2017

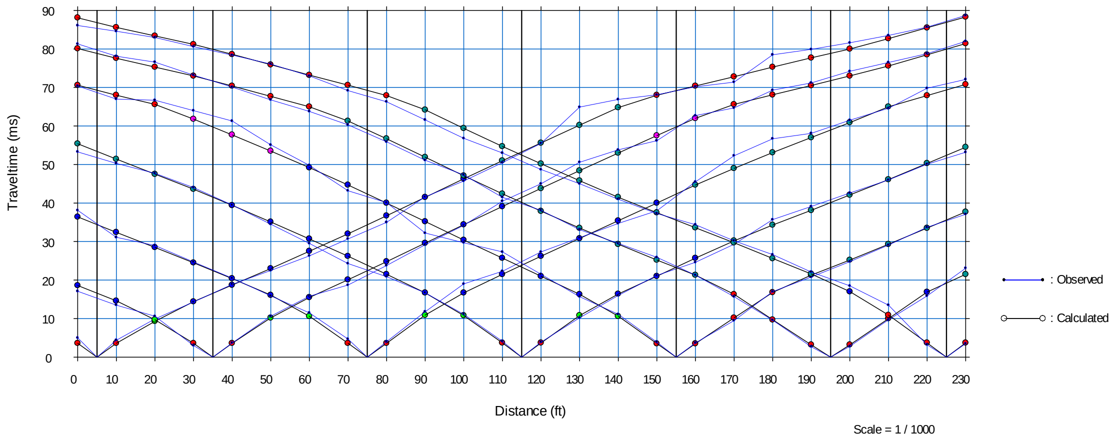


Seismic Refraction Line No. 1

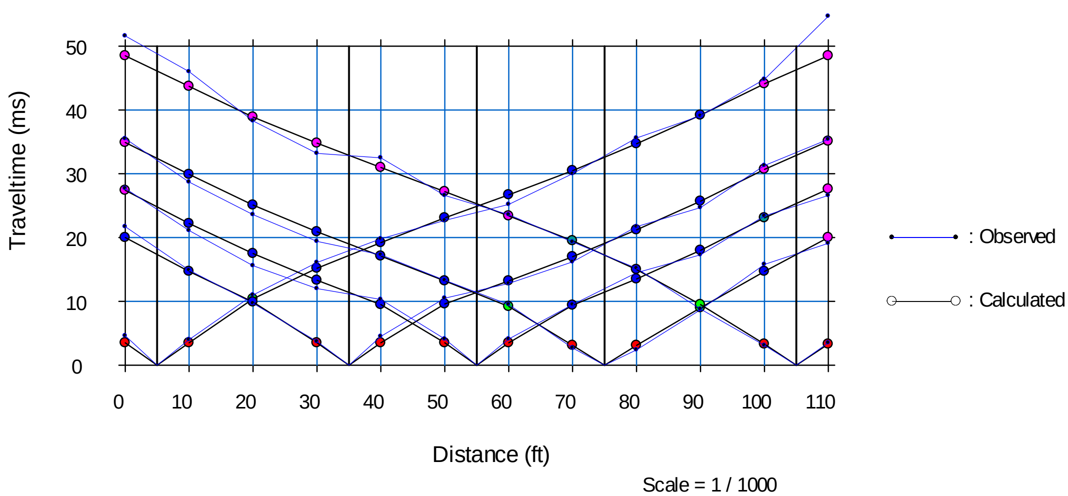


Seismic Refraction Line No. 2

Comparison of Observed and Calculated Travel Times
Paradise Irrigation District Reservoir B
Paradise – California – November 2017



Seismic Refraction Line No. 3



Seismic Refraction Line No. 4

Comparison of Observed and Calculated Travel Times
Paradise Irrigation District Reservoir B
Paradise – California – November 2017

SL-1
0+00
2+30
SL-2
0+00
1+10
SL-3
0+00
2+30
SL-4
0+00
1+10

APPENDIX D

Preliminary (Desktop) Geotechnical Report

July 31, 2017
170025

Mr. Sami Kader, P.E.
WATER WORKS ENGINEERS
1405 Victor Avenue, Suite A
Redding, CA 96003

Subject: **Preliminary (Desktop) Geotechnical Services**
Reservoir B Replacement – Planning & Design
Paradise Irrigation District
Town of Paradise & Unincorporated Butte County, California

Dear Mr. Kader:

Vertical Sciences, Inc. (VSI), is pleased to present this letter to Waterworks Engineers, LLC (WWE), providing preliminary (desktop) geotechnical services for the Paradise Irrigation District's (PID) Reservoir B Replacement project located in the Town of Paradise and Butte County, California, as shown on Plate 1 – Site Location Map. The following letter presents our understanding of the project, our observations made at project sites, a discussion regarding each alternative, a preliminary design recommendations for various project elements.

PROJECT UNDERSTANDING

We understand that the PID has an existing 3-million-gallon reservoir within its B pressure zone called the B Reservoir. Water stored in that reservoir is pumped to Reservoir A, which services a separate section of PID's service area. We understand that Reservoir B has insufficient capacity for its service area and to service Reservoir A. Because of that, one or more of the following changes are being considered to upgrade PID's water system:

1. Increase Reservoir B capacity; and/or
2. Install a new pipeline to Reservoir A directly from the WTP.

The following paragraphs discuss the alternatives in greater detail.

Reservoir B Improvements

We understand that currently Reservoir B is lined with a Hypalon liner and covered with a floating Hypalon material. We understand that the reservoir is 16 feet deep and was constructed around 1985 by excavating materials from within and along the northeastern margin of the basin and placing those materials as embankments around the southwestern, southeastern, and northwestern margin. Minor embankments are also present along the basin's northeastern margin.

We understand that two concepts are being considered by WWE to improve the B Reservoir:

1. Concept 1 – Covering the existing reservoir with an aluminum covered roof structure (to eliminate the floating cover); or
2. Concept 2 – Demolishing the existing reservoir and replacing it with two 5-million-gallon steel water storage tanks, or a single relatively larger tank.

These concepts are shown on Plate 2 – Reservoir B Concept Elements. With either concept, a portion of the reservoir will need to remain in operation during construction to service PID customers. To do so, a bifurcation berm will be constructed across the existing reservoir to allow about half of the reservoir to remain in service. Once the berm is constructed, then the other half of the reservoir can be constructed upon via one of the aforementioned concepts.

Upon completion of the initial construction, the other half of the existing reservoir can be covered or removed and replaced with the second tank. For the covering alternative, the berm may permanently remain in place but for the tanks alternative, the berm will be removed.

Pipeline to Reservoir A

This approximately 1.5-mile-long pipeline would extend from the WTP to Reservoir A using 16-inch diameter high density polyethylene (HDPE) or polyvinyl chloride (PVC) piping materials. The pipeline would extend from the WTP south along Pine Needle Drive to New Skyway, south along New Skyway to Skyway, continue south on Skyway for about 850 feet, then west and north to Reservoir A. This alignment is shown on Plate 3 - Proposed Pipeline Alignment.

An alternative to the proposed pipeline noted on Plate 3 is to construct the new pipeline within the same easement and adjacent to the existing 42-inch dimeter pipeline extending south from the WTP near Little Butte Creek. That alignment is shown on Plate 4 - Existing Pipeline Alignment. We understand that PID maintains a narrow easement along this alignment.

GEOLOGIC CONDITIONS & GEOLOGIC HAZARDS

Regional Geology

The project site is located near the contact between the Sierra Nevada and Cascade Physiographic provinces. The Sierra Nevada province is bordered to the north by the Cascade Physiographic province, to the east by the Modoc Plateau and Basin and Range Physiographic provinces, to the west by the Great Valley Physiographic province, and to the south by the Mojave Desert Physiographic province.

The Sierra Nevada province is dominated by the strongly asymmetric mountain range of the Sierra Nevada, which has a long, gentle western slope and a high, steep eastern escarpment (Bateman and Wahrhaftig, 1966). The geologic history of the Sierra Nevada can be divided into five broad phases.

The first phase consisted of the formation and accretion of an assemblage of metamorphic rocks to the ancestral western North American continent during the Sonoman Orogeny in latest Paleozoic to early Mesozoic time (Schichert and Snyder, 1981). In later Mesozoic time, the Paleozoic rocks were intruded and further metamorphosed by large masses of granitic rock, and the area was eroded to a depth of approximately 5 miles (Bateman and Wahrhaftig, 1966). Later in Cenozoic time, after a short period of inactivity, the area was uplifted and tilted as west-flowing rivers cut valleys into the ancestral Sierra Nevada. This was followed by Late Cenozoic volcanic activity that delivered copious amounts of material from volcanoes positioned along the crest and east of the range. Lastly, the area was eroded by fluvial and later glacial processes to form the landscape we see today.

Rocks within the Sierra Nevada are divided into two broad categories: the subjacent series and the superjacent series. The subjacent series rocks form the basement material of the Sierra Nevada and consist of Mesozoic granitic rocks and Mesozoic and Paleozoic metamorphic rocks. The superjacent series generally consist of Cenozoic sedimentary and volcanic rocks that now reside on the ridge tops.

The Cascade Range province extends from the northern end of the Sierra Nevada north to the Canadian border. In the project vicinity, the Cascade Range province is bounded to the west by the Great Valley province, to the east by the Modoc Plateau province, to the south by the Sierra Nevada province, and to the north by the Cascade Range extending through Oregon and Washington.

The Cascade Range province consists of a north-northwest-trending, relatively linear belt of active and dormant strata and shield volcanoes. The regional geologic conditions are dominated by andesitic, rhyolitic and basaltic volcanic rocks mantled with surficial deposits consisting of pyroclastic rocks, lahar deposits, alluvium, and local lacustrine sediments (Hinds, 1952).

Local Geologic Conditions
According to Saucedo & Wagner (1992), the Reservoir B and proposed pipeline alignments are both underlain by Pleistocene-age volcanic flows that underlie the greater Paradise area. The existing pipeline easement is underlain by undifferentiated Paleozoic- to Mesozoic-age metavolcanic rocks, ultramafic rocks, and the Tuscan Formations. Locally at each project element site, artificial fill, colluvium, and alluvium might be present. The relationship of proposed project elements to geologic conditions is shown on Plate 5 - Regional Geologic Map.

Groundwater
Attempts to estimate the depth to groundwater was performed by accessing databases of the project area to review existing reported data. Those databases include the State's Geotracker (2017) and Envirostore (2017) sites, and the California Department of Water Resources Data Library (2017). The closest data site to any of the project elements was located at the intersection of Skyway and

Wagstaff Road (Hanover, 2011). At that location, groundwater was measured at depths ranging from about 2 to 12 feet deep. No other groundwater data were obtained during this study.

CBC Seismicity Recommendations
At a minimum, structures should be designed in accordance with the 2013 CBC seismic design criteria as follows:

CBC SEISMIC DESIGN PARAMETERS		
California Building Code	**Parameter**	**CBC Designation**
Site Coordinates	Latitude	39.784963°
	Longitude	-121.595223°
Section 1613.3.3 Table 1613.3.3(1)	Site Coefficient, F_a	1.143
Section 1613.3.3 Table 1613.3.3(2)	Site Coefficient, F_v	1.535
Section 1613.3.1 Figure 1613.3	Site Class Designation	C
	Seismic Factor, Site Class C at 0.2 Seconds, S_s	0.643g
	Seismic Factor, Site Class C at 1.0 Seconds, S_1	0.265g
Section 1613.3.3	Site Specific Response Parameter for Site Class C at 0.2 Seconds, S_{MS}	0.735g
	Site Specific Response Parameter for Site Class C at 1.0 Seconds, S_{M1}	0.407g
Section 1613.3.4	$S_{DS}=2/3S_{MS}$	0.490g
	$S_{D1}=2/3S_{M1}$	0.272g

The latitude and longitude used above correspond to the approximate center of the existing Reservoir B.

Faulting
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 Hazards Act (AP). AP Special Studies Zones require site-specific evaluation of fault location and require a structure setback if the fault is found traversing a project site.

Active faults have not been mapped within the project region. A number of potentially active and inactive faults have been mapped in the project area, as shown on Plate 6 – Regional Fault Map. None of those potentially active faults have been mapped projecting beneath or across proposed project improvements. Unnamed inactive faults have been mapped projecting across the existing pipeline easement located near Little Butte Creek.

Landslides

No landslides, incipient or otherwise, were observed at Reservoir B or along the proposed new pipeline route. According to the Butte County General Plan (Butte County, 2012), this area has a low to moderate potential for slope instability.

Geomorphic features on the slopes descending westerly from Magalia and Paradise down to Little Butte Creek, where the easement for the existing 42-inch diameter pipeline is present, imply that dormant, older landslide features might be present along those slopes. In those areas, hummocky and benchy terrain, along with larger arcuate-shaped features imply that past slope failures have occurred. According to the Butte County General Plan (Butte County, 2012), this area has a moderate potential for slope instability. These slopes have been mapped as being underlain by ultramafic, metavolcanic, and Chico Formation rock materials, some of which is known for slope instability. Landslides have been mapped within these rock materials west of the project site (Saucedo & Wagner, 1992).

Liquefaction

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. In order 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.

Relatively shallow soils overlying volcanic rocks are anticipated beneath Reservoir B, the existing pipeline alignment, and the proposed pipeline alignment. Those soils are generally rich in clay. Groundwater is anticipated to be located within the volcanic rock materials and not the soils. Because of this, the potential for liquefaction to occur at the various project elements is anticipated to be low. This should be verified during project design-level studies.

Expansion Potential

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. Plasticity Index (PI) testing of soils by NRCS (2017) reported a PI of about 26 for the Reservoir B site, the proposed pump station at the WTP, and for the proposed new pipeline alignment. Those PIs correlate to soils with a medium to high expansion potential (Day, 1999). PIs for the existing pipeline easement range from about 18 to 34, with most of the soils crossed having a PI of about 34. Those soils correlate to a high to very high expansion potential, 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)	

Naturally Occurring Asbestos (NOA)

Ultramafic rock, such as serpentinite, amphibolite, peridotite, dunite, pyroxenite, hornblendite, etc., can contain asbestiform minerals, which are fibrous, silica-rich crystals that can cause lung cancer, mesothelioma, asbestosis, and other health-related issues, if present. Typically, six minerals within ultramafic rocks are responsible for the primary, naturally occurring asbestiform concerns for health-related issues: chrysotile, tremolite, actinolite, anthophyllite, crocidolite, and amosite. These minerals may or may not be present in ultramafic rocks; thus, the presence of ultramafic rock does not automatically indicate that there is a health hazard. The presence of asbestiform minerals can sometimes be discerned in the field based on visual examination of rock exposures but, most often, must be confirmed using laboratory testing.

Naturally occurring asbestos can be hazardous to human health if it is disturbed, becomes airborne and is inhaled. If NOA is not disturbed and fibers are not released into the air, then it is typically not considered a health hazard. Inhalation is the primary exposure route of concern, because breathing asbestos fibers may cause them to become trapped in the lungs. Ingestion is another, albeit less common, pathway of concern, because swallowing asbestos fibers may also cause the fibers to be trapped in body tissues. Asbestos is not absorbed through the skin, so merely touching it does not pose a significant risk to human health. Asbestos fibers are not water soluble and do not move through groundwater to any appreciable extent. Based on studies of other insoluble particles of similar size, the expected migration rate of an asbestos fiber through soils by the forces of groundwater is approximately 1 to 10 centimeters (0.4 to 4 inches) per 3,000 to 40,000 years (New Hampshire DES, 2010). Thus, asbestos is not considered a groundwater contaminant.

Ultramafic rocks have been mapped and were observed only near the water treatment plant, north of the intersection of Skyway and New Skyway. Those rock materials are noted in Plate 5. The ultramafic rocks extend west of Skyway and down to at least Pine Needle Drive at the eastern flank of the water treatment plant. South of that intersection, ultramafic rocks have not been mapped nor observed within the project area.

If ultramafic rocks or soils derived from ultramafic rocks are encountered during exploration for design of the project or during construction, then testing for the presence of NOA should be performed by an appropriate professional licensed and/or certified to assess the presence of NOA using randomized multi-increment sampling methods. If NOA concentrations exceed the regulatory threshold, then mitigation measures are typically required to reduce the potential of inducing NOA to become aerosol. This includes consistent wetting of excavated soils, wetting excavation surfaces, use of surfactants or binding agents on soil and rock surfaces, and entombing NOA-bearing soils and rock materials as artificial fills within excavations (such as a pipeline trench).

DISCUSSION REGARDING EACH PROJECT ELEMENT

The following sections discuss geotechnical/geological issues, considerations, and/or constraints for each of the proposed project elements.

Reservoir B – Concept 1 (Covering Existing Reservoir)

Concept 1 consists of covering the existing lined reservoir with an aluminum-cladded and roofed structure. The foundations will be constructed within the existing embankment soils currently present surrounding the reservoir. Challenges for this concept include:

1. Difficult excavation conditions; and
2. Increased concrete volumes due to excavation overbreak.

The existing embankment soils were observed to contain locally abundant basaltic cobbles and boulders, which will make constructability more difficult. When excavating for spread foundations, equipment will need to contend with the cobbles and boulders which will slow excavation. In addition, overbreak (the amount of additional annular volume created during drilling or excavating due to removal of cobbles and boulders, as compared to planned excavation volumes) will likely be experienced as the excavator removes cobbles and boulders from the foundation area, resulting in increased volumes of concrete needed to fill the excavated area.

Reservoir B – Concept 2 (Construction of One or More Steel Tanks)

This concept consists of the design and construction of one relatively large (up to 10-million-gallon) or up to two somewhat smaller (3- to 5-million-gallon) steel tanks at the existing reservoir site. To do so, the existing reservoir embankment materials along the northwest, southwest, and southeast margins of the existing reservoir will be removed. Depending on where the tank(s) is/are situated, some engineered fill might be needed to create a pad area for construction of the tank. Challenges for this concept include:

1. Presence of rock in existing embankment soils;
2. Differential settlement beneath tank(s); and
3. Expansion potential of subgrade soils.

Embankment soils should be excavatable using conventional heavy grading equipment. However, cobbles and boulders within those materials will need to be removed if those materials are to be used as engineered fill. Thus, a relatively high shrinkage percentage of up to 30 percent or more could be experienced when estimating soil volumes for the site due to rock removal. The actual shrinkage percentage should be confirmed during design-level studies if this concept is pursued.

The most significant issue for performance of the tank(s) at this site is the potential for differential settlement to adversely affect the tank(s). Steel tanks are less sensitive to differential settlement relative to concrete tanks but this condition is still a potential concern. If tanks are partially situated on cut and partially situated on engineered fill materials, then settlement magnitudes and rates for those materials will be different, potentially leading to differential settlement. In addition, variable bedrock surface depths could also contribute to differential settlement potentials.

Typically, if differential settlement thresholds exceed allowable tolerances, then the following mitigations are typically applied:

- Overexcavating the cut side of the tank to a target depth and backfilling with engineered fill to create a uniform fill thickness beneath the tank(s);
- Deepening the foundation systems or use of deep foundations systems (such as piles or piers) to support the portion of the tank(s) extending over engineered fill;
- If necessary, the use of select relatively low-compressible materials, such as aggregate base or controlled low strength material (CLSM or soil-cement slurry), in place of the engineered fill; and/or
- Utilization of geogrid materials within engineered fills to distribute loads over a broader area.

Differential settlement potential should be evaluated during design-level geotechnical studies and, if necessary, mitigation alternatives developed for the project.

The soils in the Reservoir B area have been evaluated by NRCS to have a PI of about 25, which is at the moderate-high threshold for expansion potential. Expansion and contraction of soils can cause differential movement of the tank floor that could exceed structural tolerances. It is our opinion that there is a low potential of this occurring but design-level geotechnical studies should evaluate whether expansive soils will adversely affect performance of the tank(s) floor(s) or slab(s) and foundations. If it proves to be a potential problem, then typical mitigations include:

- Removal of 2 to 4 feet of soils beneath the slab and foundations and replacement of those materials with nonexpansive soils, such as aggregate base;
- Lime- or cement-treatment of soils to reduce their expansive potential, or
- Deepen foundations to extend below the active expansion/contraction zone for those soils.

Proposed Pipeline Alignment

The proposed pipeline alignment to Reservoir A extends beneath existing paved roadways for most of its alignment. The alignment, as proposed, could extend across a variety of geologic materials with varying excavation difficulties and material conditions. The primary issues we've identified with this alternative are:

1. Excavatability of volcanic rocks along the alignment; and
2. The amount of oversize materials (rocks greater than 3 to 6 inches in maximum dimension) generated during excavation that cannot be reused within the trench zone.

The primary area of concern regarding excavatability of underlying rock and soil materials along the proposed alignment extends south from the intersection of Coutolenc Road and New Skyway to the intersection of Skyway and New Skyway. This section of the alignment, shown on Plate 7 – Skyway Segment Evaluation, extends along cut slopes that expose variable volcanic rock materials.

Those rocks consisted of agglomerates, tuffaceous deposits, and minor massive basaltic and andesitic rocks, along with colluvial soils. The materials observed were moderately to highly weathered, poorly indurated, and locally fractured. No features on the cut slope faces implied that those slopes were created by blasting. It is our opinion, based on our site observations, that the proposed pipeline trench should be excavatable using conventional heavy grading equipment and that blasting or other unconventional excavation methods will likely not be needed. If relatively hard rock is encountered, it is likely it will be for short segments and that a large excavator equipped with a single-shank ripping tooth or a hydraulic hoe-ram, should be able to penetrate those materials.

If the proposed alignment is situated along the north-bound lane on the east side of Skyway, then much of the trench will likely be excavated within engineered fill materials used to create the road. Those materials should be excavatable using conventional heavy grading equipment due to a lower likelihood of encountering relatively hard rock than if the pipeline were to be located in the south-bound lane.

The agglomerate and relatively massive rock materials exposed within the cut slopes are anticipated to generate oversize rock materials that will not be suitable for reuse within the pipeline trench zone. Plate BB shows estimates of oversize rock that might be encountered during excavation. It might be possible to reduce the oversize materials if the pipeline is situated within the north-bound lane of Skyway where engineered fill is present. There's a potential that oversize rocks were screened from engineered fill when the roadway was constructed. This should be evaluated during design-level geotechnical evaluations.

The segments of the pipeline south of the intersection of New Skyway and Skyway are anticipated to be excavatable. Some oversize rock materials are likely to be encountered but at a volume lower than those noted above. Depending on the time of year and the intensity of the winter season, some shallow groundwater might be encountered. It is anticipated that if groundwater is encountered, it will likely seep into the excavation at relatively slow rates due to the clay-rich nature of the soils.

Existing Pipeline Easement

This alignment will extend across a wide range of geologic materials in relatively steep terrain. It is anticipated that these geologic materials will be excavatable and that locally, oversize materials will be encountered that will not be useable within the trench zone. The most significant concern regarding this alternative is the potential for slope instability to damage the proposed pipeline. As noted above, there are a number of geomorphic indicators that slope failures have occurred along this alignment and that dormant landslides are present. If design of a new pipeline along this alignment occurs, we recommend that design-level geotechnical studies be performed to evaluate the potential for landslides' impact the pipeline. This includes detailed geologic mapping, exploration of possible landslide features, extensive laboratory testing to characterize critical strengths of underlying soils and rocks, and stability analyses to estimate how stable or unstable those slopes and features are.

PRELIMINARY GEOTECHNICAL DESIGN RECOMMENDATIONS

The following sections provide preliminary geotechnical design recommendations that can be used on a planning-level basis to develop estimated costs for various project elements. These recommendations should not be used for design of the project; design-level geotechnical studies should be performed on those project elements that will be implemented within the project improvements.

Reservoir B

Our preliminary recommendation for allowable bearing pressure for shallow foundations at the Reservoir B site is 1,500 pounds per square foot. This is based on the potential for relatively clay-rich soils interspersed cobbles and boulders within the anticipated foundation zone. A design-level geotechnical investigation may result in a higher allowable bearing pressure. The preliminary allowable bearing pressure can be proportioned for dead loads plus probable maximum live load.

Pipeline to Reservoir A

External Loads on Buried Pipes. 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. The pipe should be designed to resist the imposed loads with a factor of safety and an allowable deflection as recommended by the pipeline manufacturer. The earth loads on the pipe can be estimated using formulas developed by Marston (1930) and Spangler (1982). When using Marston's formulas, a preliminary unit weight of the backfill materials can be assumed to be 125 pcf.

The pipe may be subject to surcharge pressures due to construction activities and traffic. Those surcharge pressures should be considered in the design of the pipe.

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. 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	2,500

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. Preliminary E'_n values for the earth materials along the alignment are anticipated to be 1,000 pounds per square inch. That value is suitable for use for planning-level design and cost estimating purposes. Future geotechnical design-level studies should be performed to confirm those values.

For trench widths less than five times the diameter of the pipe, the composite design E' (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' 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:

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

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 reported conditions encountered during our review of selected, available, published information collected during this study and from our site observations. No subsurface exploration or laboratory testing was performed by VSI to prepare this report. This study is applicable only to those project features described herein (see Section 1.1 – Project Understanding). The conclusions and recommendations presented in this report are based upon the findings at the points of exploration from other's studies, and interpolation and extrapolation of information between and beyond the points of observation, and are subject to confirmation based on the conditions revealed by future geotechnical exploration and by construction.

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 project 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.

CLOSURE

We appreciate the opportunity to participate on this project and look forward to working with you during design-level studies. If you have questions regarding this proposal or require additional information, please contact me at (530) 638-5263 at your convenience.

Regards,

VERTICAL SCIENCES, INC.

James A. Bianchin, C.E.G.
Principal Engineering Geologist

Jon Everett, P.E., G.E.
Principal Geotechnical Engineer

Attachments:

- Plate 1 – Site Location Map
- Plate 2 – Reservoir B Concepts
- Plate 3 – Proposed Pipeline Alignment
- Plate 4 – Existing Pipeline Easement Alignment
- Plate 5 – Regional Geologic Map
- Plate 6 – Regional Fault Map
- Plate 7 – Skyway Segment Evaluation

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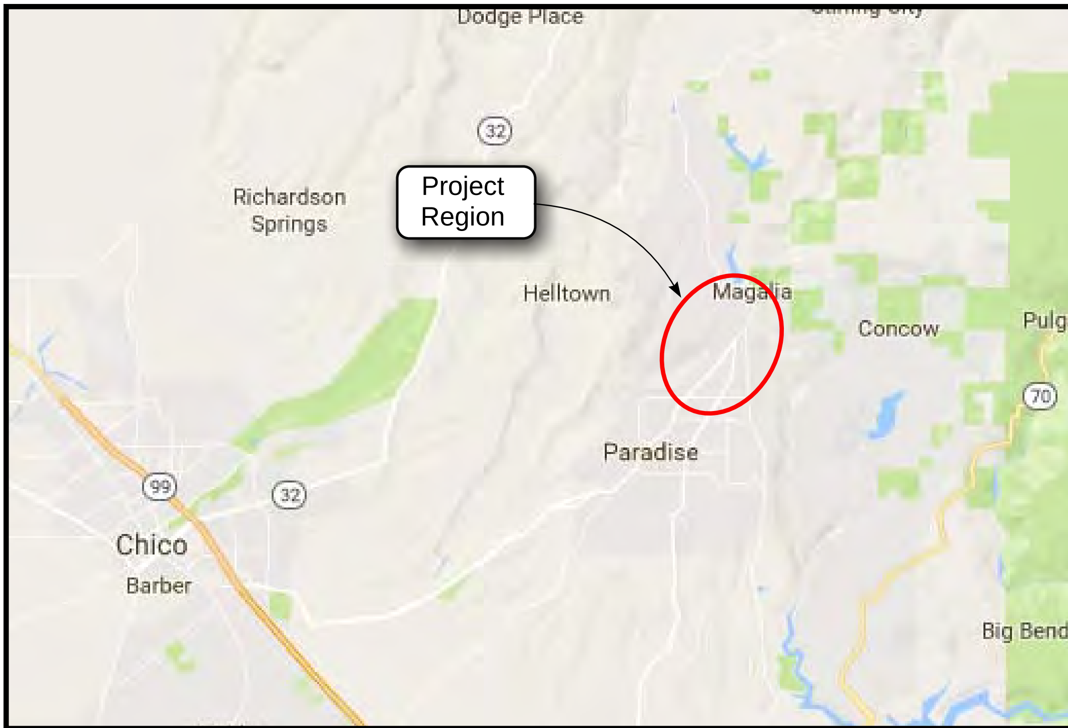
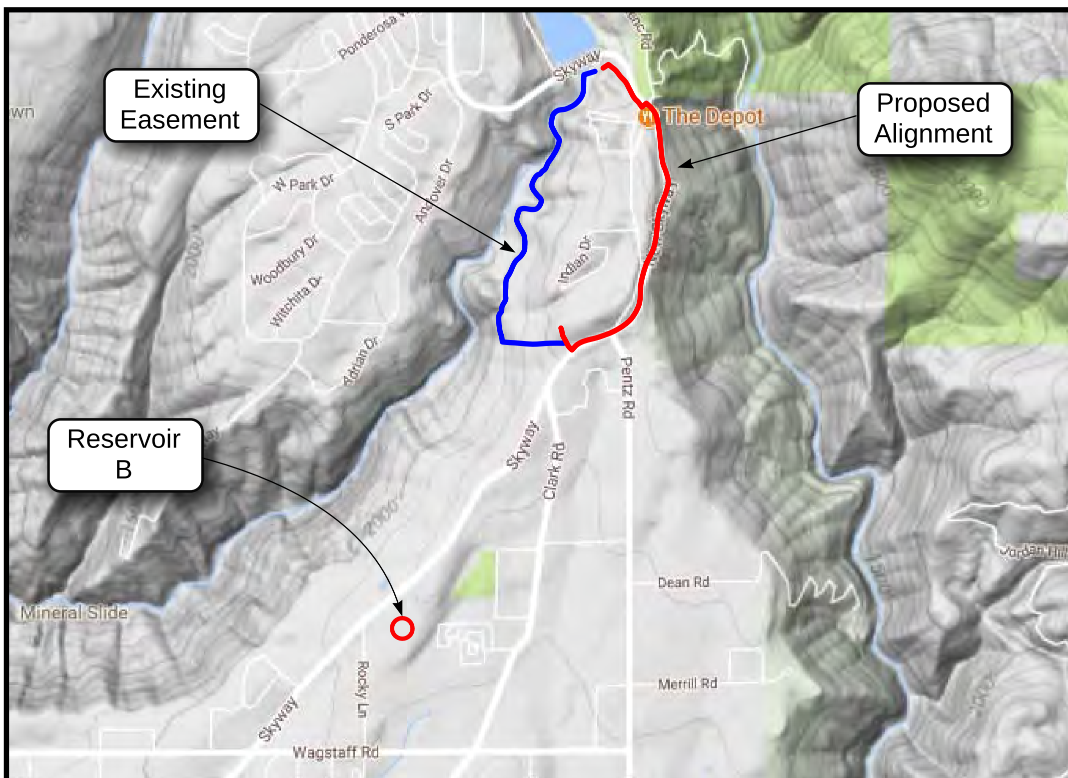
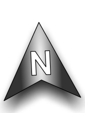




SITE LOCATION MAP	
Reservoir B Replacement Preliminary (Desktop) Study Water Works Engineers Paradise & Butte Co., California	Plate No. 1
VERTICAL SCIENCES, INC.	Project no. 170025

Scale undetermined
Base maps derived from Google Maps.



Scale undetermined
Imagery derived from Google Maps.

RESERVOIR B CONCEPT ELEMENTS	
Reservoir B Replacement Preliminary (Desktop) Study Water Works Engineers Paradise & Butte Co., California	Plate No. 2
VERTICAL SCIENCES, INC.	Project no. 170025

PROPOSED PIPELINE ALIGNMENT

Reservoir B Replacement
Preliminary (Desktop) Study
Water Works Engineers
Paradise & Butte Co., California

Plate No. 3

VERTICAL SCIENCES, INC.

Project no. 170025

Scale undetermined
Imagery derived from Google Maps.

EXISTING PIPELINE EASEMENT ALIGNMENT

Reservoir B Replacement Preliminary (Desktop) Study Water Works Engineers Paradise & Butte Co., California	Plate No. 4
VERTICAL SCIENCES, INC.	Project no. 170025

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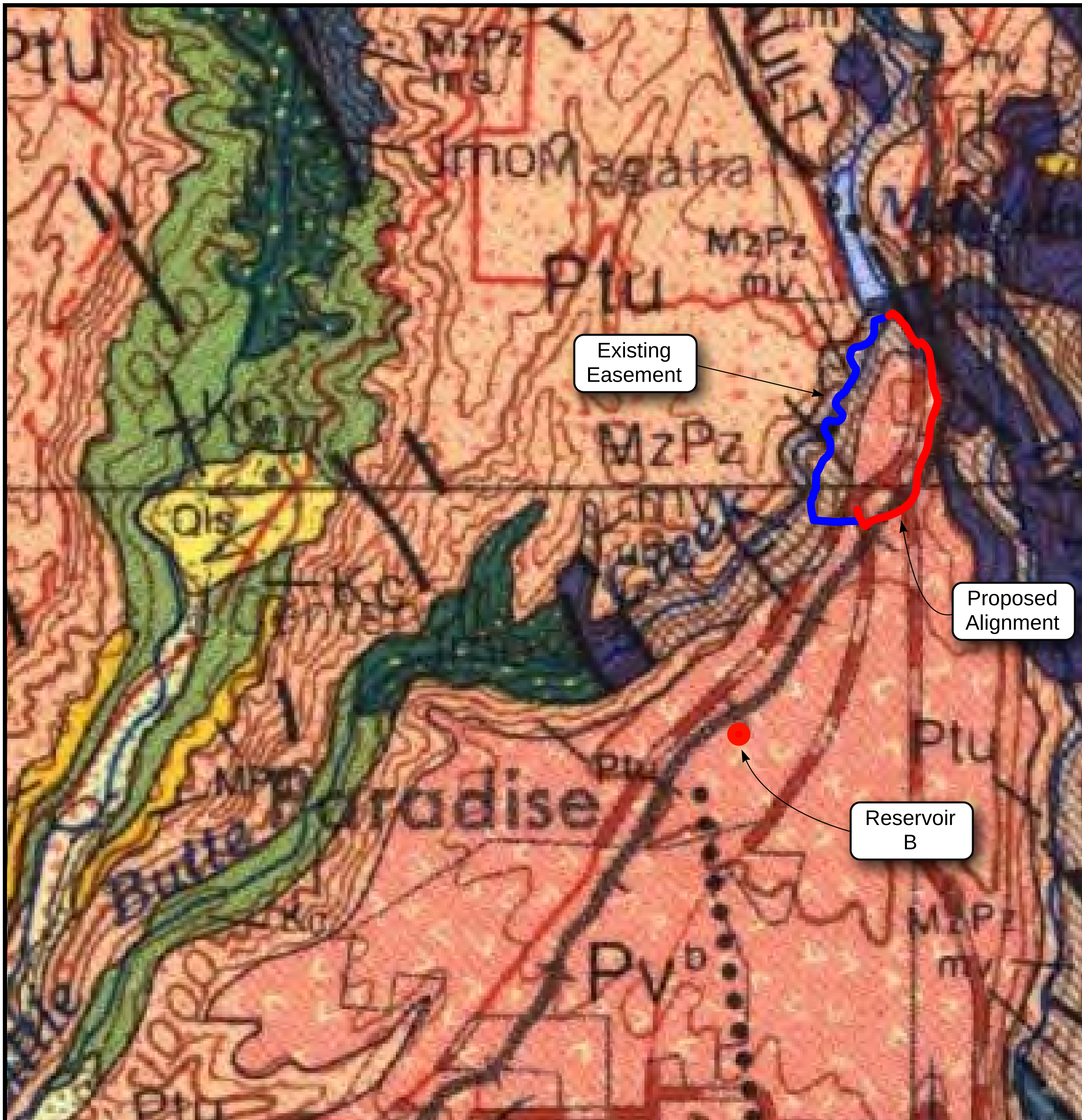
Imagery derived from Google Maps.



Ptu Pnt	Tuscan Formation
Pv	Pleistocene Volcanics, basalt
Kc	Chico Formation
mv	Metavolcanics
um	Ultra Mafic Rocks
Jmo	Monte de Oro Formation

MzPz

N

Scale undetermined
Base map from Saucedo & Wagner (1992)

REGIONAL GEOLOGIC MAP

Reservoir B Replacement
Preliminary (Desktop) Study
Water Works Engineers
Paradise & Butte Co., California

Plate No.
5

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Scale undetermined

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)

REGIONAL FAULT MAP

Reservoir B Replacement
Preliminary (Desktop) Study
Water Works Engineers
Paradise & Butte Co., California

Plate No. 6

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Map from California Geological Survey Fault Activity Map

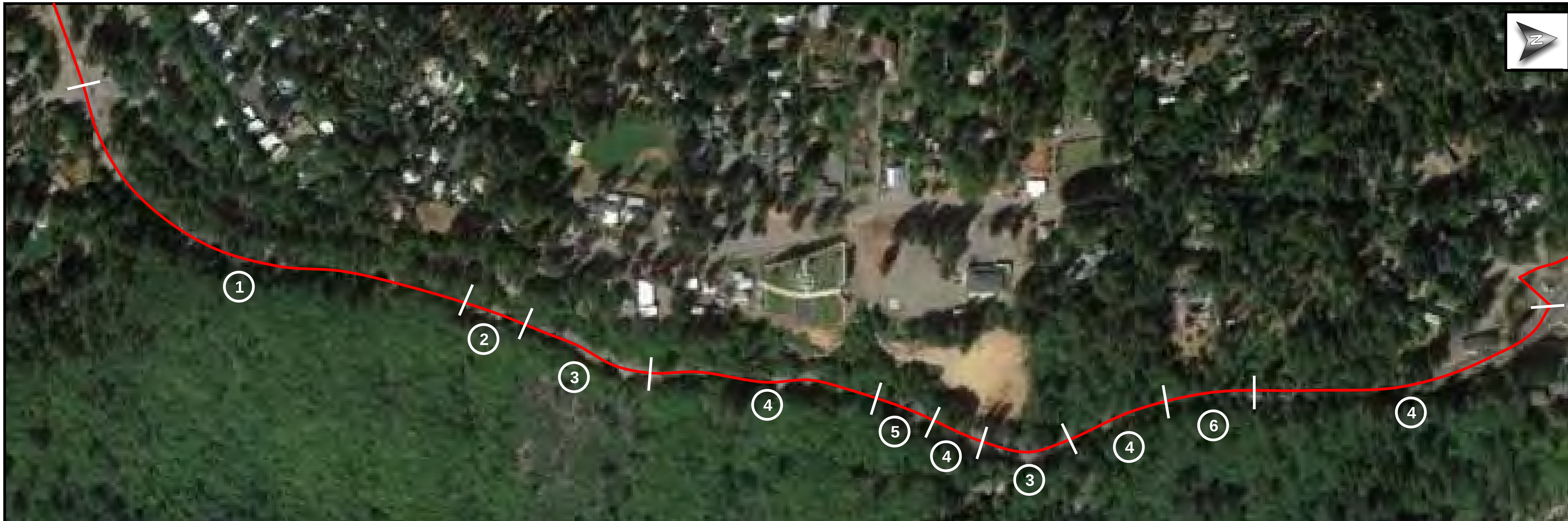

Image area shown above

Location:	1	2	3	4	5	6
Earth Materials Exposed	Massive andesite. Moderately to highly weathered, moderately hard, poorly indurated. Few agglomerate clasts	Andesite & zeolite tuff. Moderately weathered, massive, moderately hard to hard, poorly to moderately indurated.	Agglomerate. Moderately to highly weathered, thickly bedded with clasts up to 4 feet in diameter.	Colluvial & regolith soils with few to moderate gravel, cobbles, and boulders.	Agglomerate. Moderately to highly weathered, thickly bedded with clasts up to 4 feet in diameter.	Colluvial & regolith soils with moderate to abundant gravel, cobbles, and boulders up to at least 3 feet in diameter.
Estimated Excavation Difficulty	Moderate to high	High. Possible hoe ram needed	Low to moderate	Low	Low to moderate	Low to moderate
% Oversize	<10%	~40%	~40%	~20%	~30%	~30%

6 Location noted on map and in table above

0 150 300
Scale: 1"=300'
1:3,600

SKYWAY SEGMENT EVALUATION

Reservoir B Replacement
Preliminary (Desktop) Study
Water Works Engineers
Paradise & Butte Co., California

Plate No.
7

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