GEOTECHNICAL REPORT

Backwash Equalization Tank Improvements
Paradise Irrigation District
Butte County, California

Submitted To:

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March 25, 2024
Project No. 2201.0155

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Redding, California 96001

Subject: **Geotechnical Report**
Backwash Equalization Tank Improvements
Paradise Irrigation District
Butte County, California

Dear Mr. Riess & Ms. Nilsen:

Bajada Geosciences, Inc., is pleased to submit this geotechnical report to Water Works Engineers for the design and construction of the proposed backwash equalization tank improvements at Paradise Irrigation District's water treatment plant located in Butte County, California. This geotechnical report discusses field mapping and explorations performed, laboratory testing results, subsurface conditions encountered, and geotechnical analyses associated with the study.

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

Sincerely,

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1 GENERAL

This report presents the results of our geotechnical study for the proposed backwash equalization tanks improvements at the Paradise Irrigation District (PID) water treatment plant (WTP), located in the Magalia area of Butte County California. The project site location is shown on Plate 1 – Site Location Map. Bajada Geosciences, Inc. (BAJADA) has prepared this report at the request of Water Works Engineers, LLC (WWE). Our services were performed in general accordance with our proposal dated September 27, 2022.

The following sections present our understanding of the project, the purpose of our study, and the geotechnical findings, conclusions, and recommendations for the project.

1.1 PROJECT UNDERSTANDING & LOCATION

We understand that PID has retained WWE to design a new backwash equalization tank at the WTP site. We understand that the proposed tank will be located adjacent to and northeast of the existing equalization tank at the site. Improvement locations are shown on Plate 2 – Project Elements. The site where the proposed tank is to be located is sloping and will likely require retaining walls ranging in height up to 20 feet to create the tank pad. We understand that the proposed tank will have dimensions that will be similar to the existing tank. Specific loading conditions for the tank are unknown but anticipated to be relatively light. We understand that the tank will be supported on a shallow ring footing foundation or on a stiffened concrete slab-on-grade.

The address of the site location is 13827 Pine Needle Drive, Magalia, California. Latitude and longitude for the approximate center of the pad are as follows:

APPROXIMATE PROJECT COORDINATES		
Coordinates	**Degrees, Minutes, Seconds**	**Decimal Degrees**
Latitude	39°48’54.32”	39.815091°
Longitude	-121°34’54.83”	-121.581910°

1.2 SCOPE OF SERVICES

Services performed for this study are in general conformance with the proposed scope of services presented in our September 27, 2022, proposal. Our scope of services included:

- Reconnaissance of the site surface conditions;
- Advancement of three exploratory test pits at selected locations shown on Plate 3 – Geotechnical Map. Exploration procedures and Logs of Test Pits 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 selected locations noted on Plate 3. Procedures used and results of those geophysical surveys are presented in Appendix C – Geophysical Refraction Surveys;
- Estimation of settlements for the proposed tank;
- 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;
    - 2022 California Building Code (CBC) seismic design criteria;
    - A geotechnical map presented as Plate 3;
    - Cross sections shown on Plates 4.1 & 4.2 – Geotechnical Sections A-A' & B-B', respectively;
    - Geotechnical recommendations for:
        - Site preparation, engineered fill, site drainage, and subgrades;
        - Suitability of on-site materials for use as engineered fill;
        - Foundation and slab-on-grade design;
        - Temporary excavations, shoring, and trench backfill;
        - Trench backfill and compaction recommendations; and
        - Lateral earth pressures for retaining wall design and construction.
    - Appendices that present a summary of our field investigation procedures, laboratory testing program, and geophysical refraction surveys.

1.3 PREVIOUS WORK PERFORMED & REFERENCES REVIEWED

Site-specific geotechnical evaluations have been performed at the WTP by Moore & Taber (1971), Kleinfelder (1992), Taber (2015), and Vertical Sciences (2018). Vertical Sciences (2018) performed a geotechnical study for the proposed new pump station located adjacent to the treated water storage tank, south of the project site. That study included evaluations of a pipeline extending to the WTP along Pine Needle Drive.

Kleinfelder (1992) performed subsurface exploration and refraction surveys across the WTP site to provide recommendations for design and construction of an expansion of the existing WTP facilities in use today. Those explorations and services included the existing backwash equalization site.

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. Taber (2015) performed coring of asphaltic concrete and subsurface exploration to help assess causes and mitigations for distress observed in site paving and structures.

In addition, numerous other geotechnical explorations have been performed at the adjacent Magalia Dam site and those studies are discussed in detail in Slate (2021).

Other references are made throughout this document. References cited, herein, can be found in Section 7 of this report.

2 FINDINGS

2.1 FIELD INVESTIGATION

Our field geotechnical investigation consisted of reconnaissance-level geologic mapping of the project site and subsurface exploration through advancement of three exploratory backhoe test pits to depths ranging from approximately 6 to 10 feet below existing grade. The test pits were advanced on December 15, 2022, using a mini-excavator affixed with a two-foot-wide bucket. The exploration locations are shown on Plate 3. Descriptions of soils encountered are presented on the Logs of Test Pits included in Appendix A.

2.2 SITE CONDITIONS

2.2.1 Surface Conditions

The project site is moderately to steeply sloping to the southwest. The existing equalization tank is located immediately southwest of the project site. Little Butte Creek, which is the discharge channel for the Magalia Reservoir spillway, is located immediately northwest of the site. Paved roads leading into the WTP and to the reservoir outlet works building border the north, northeast, and southeast portions of the project area. The proposed development area is fallow and covered with seasonal vegetation and a pine tree. The northern and eastern margins of the site are bounded by a chain link fence.

Based on topographic information estimated from open-source LiDAR data, elevations at the site range from 2,180 to 2,205 feet. Drainage occurs as sheetflow to the southwest into Little Butte Creek.

2.2.2 Subsurface Conditions

Subsurface conditions were explored using test pits at three locations at the site, as shown on Plate 3. Based on those test pits, the subsurface materials in the upper 4 to 6 feet of the soil column consist of moist to wet, slightly to highly plastic, clayey sand, sandy silt, sandy clay, sandy clay with gravel, and clay. Very fine pores were observed in near-surface soils with fine to medium roots present to depths of up to about 6 feet. Those materials appeared to be artificial fill and colluvial or regolithic soils.

Below a depth of 6 feet, moderately to highly weathered, poorly indurated, weak to hard, highly fractured, locally fissile serpentinzed pyroxenite rock was encountered. The observed serpentinzed pyroxenite had a blocky or disturbed to disintegrated structure.

Groundwater was observed in all three pits perched on top of competent rock material at depths ranging from about 5 to 7 feet.

Cross sections were prepared using the test pit data to graphically depict existing subsurface geological conditions at the site. The locations of the cross-sections are shown on Plate 3 and presented on Plates 4.1 and 4.2 – Geotechnical Section A-A' and B-B'.

2.3 GEOLOGIC CONDITIONS

2.3.1 Regional Geology

The project site is located near the boundaries of the Cascade Physiographic province and the Sierra Nevada Physiographic province of California.

The Sierra Nevada Physiographic province is bordered to the north by the Basin and Range, Modoc Plateau, and Cascade Range Physiographic provinces. To the west it is bordered by the Great Valley Physiographic province, to the east by the Basin and Range province, and to the south by the Mojave Desert province.

The Sierra Nevada Geologic/Geomorphic 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 (Schwichert 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 has been eroded by fluvial and later glacial processes to form the landscape we see today.

The Cascade Range Physiographic province is bordered to the north by the Basin and Range and continued Cascade Range physiographic provinces of Oregon. To the west it is bordered by both the Great Valley Physiographic province and the Klamath Mountains Physiographic province, to the east by the Modoc Plateau province, and to the south by the Sierra Nevada province.

The Cascade Range is a chain of volcanic cone structures which extend south through Washington and Oregon, into California. The range is dominated by Mt. Shasta, a glacier-mantled volcanic cone, rising 14,162 feet above sea level. The southern termination of the Cascade Range is marked by Lassen Peak, a lava dome volcano, which last erupted in the early 1900s. The Cascade Range is transected by deeply cut canyons of the Pit River, which flows through the range between these two major volcanic structures, after winding across the interior Modoc Plateau, on its way to the Sacramento River.

2.3.2 Local Geologic Setting

The project area is underlain by pre-Cenozoic metavolcanic rocks including latite, dacite, tuff, and greenstone, as shown on Plate 5 – Regional Geology (Saucedo & Wagner, 1992). Based on the data obtained from the test pits, there is up to approximately 10 feet of artificial fill, colluvium, and/or regolithic soils overlying the metavolcanic rocks, as shown on Plates 4.1 and 4.2.

2.3.3 Groundwater

Groundwater was observed perched on weathered rock material at depths ranging from about 4 to 5 feet in all three test pits. No groundwater was encountered in explorations advanced by Vertical Sciences (2018) or 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 treated water storage tank. No other reports of groundwater were found during this study.

Groundwater elevations at the project site will fluctuate over time. The depth to groundwater can vary throughout the year and from year to year. Intense and long duration precipitation or drought conditions, modification of topography, groundwater extraction, 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.

3 GEOLOGIC HAZARDS

3.1 REGULATORY SEISMIC SETTING

The State of California designates faults as Holocene-age or Pre-Holocene-age 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)**
Holocene-Active	Holocene	Within last 11,000 Years[1]
Pre-Holocene	Quaternary & Older	>11,000 Years[1]
Age Undetermined	Unknown	Unknown

[1] – Holocene is defined as 11,700 years before present by the International Commission on Stratigraphy. The California State Mining and Geology Board, which administers the review and application of the Alquist-Priolo Earthquake Fault Zoning Act, currently recognizes the Holocene as 11,000 years before present.

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 Holocene-active, pre-Holocene, or age undetermined. If an FER evaluates a fault as Holocene-active, then it is typically incorporated into a Special Studies Zone in accordance with the Alquist-Priolo Earthquake Fault Zoning Act (AP). AP Special Studies Zones require site-specific evaluation of fault location for structures for human occupancy and require a habitable structure setback if the fault is found traversing a project site.

The Magalia Fault has been mapped projecting through the project site, as shown on Plate 6 – Regional Fault Map. The period of last rupture of the Magalia fault is uncertain. It is known that the fault displaces Eocene-age paleochannels and older rocks (Gassaway, 1899; Lindgen, 1911; Logan, 1928, Clark, 1970). Faulting of younger rock materials has not been proven but is considered likely. According to Dudley (1988), the "scarp south of De Sabla Forebay suggests geologically young displacement." That scarp is underlain by Cenozoic-age Tuscan Formation, implying that faulting has occurred within the last approximately 2.6 million years. However, actual fault displacement of the Tuscan Formation at that location or elsewhere along the Magalia fault trace has never been verified. Thus, the Magalia fault should be considered Pre-Holocene.

The nearest Holocene-active fault system to the project site is the Indian Valley Fault Zone located approximately 40 miles northeast of the project site, and approximately 2 miles south of the City of Greenville, CA.

3.2 CBC SEISMIC DESIGN RECOMMENDATIONS

We understand that the proposed project will be designed and constructed under the 2022 California Building Code (CBC) criteria. At a minimum, structures should be designed in accordance with the following seismic design criteria:

CBC SEISMIC DESIGN PARAMETERS		
California Building Code	**Parameter**	**CBC Designation**
Site Coordinates	Latitude	39.815091°
	Longitude	-121.58191°
Table 1613.2.3(1)	Site Coefficient, F_a	1.2
Table 1613.2.3(2)	Site Coefficient, F_v	1.5
Section 16132.1 Figures 1613.2.1(2) through 1613.2.1(10)	Site Class Designation	C
	Seismic Factor, Site Class D at 0.2 Seconds, S_s	0.90
	Seismic Factor, Site Class D at 1.0 Seconds, S_1	0.28
Section 1613.2.3	Site Specific Response Parameter for Site Class D at 0.2 Seconds, S_{MS}	0.99
	Site Specific Response Parameter for Site Class D at 1.0 Seconds, S_{M1}	0.40
Section 1613.2.4	$S_{DS}=2/3S_{MS}$	0.66
	$S_{D1}=2/3S_{M1}$	0.27
Peak Ground Acceleration (PGA_M)		0.41g

3.3 PROBABILISTIC ESTIMATES OF STRONG GROUND MOTION

Probabilistic evaluations of horizontal strong ground motion that could affect the site were performed using attenuation evaluation methods provided by the U.S. Geological Survey (USGS, 2023b). The evaluations were performed using an estimated shear wave velocity in the upper 100 feet of the profile of 537 meters per second. Evaluations were performed for upper-bound (UBE) design-basis (DBE) probabilistic exposures, and maximum considered earthquake (MCEg). The UBE corresponds to horizontal ground accelerations having a 10 percent probability of exceedance in a 100-year exposure period, with a statistical return period of 949 years. The DBE corresponds to horizontal ground accelerations having a 10 percent probability of exceedance in a 50-year exposure period, with a statistical return period of 475 years. The MCEg corresponds to horizontal ground accelerations having a 2 percent probability of exceedance in a 50-year exposure period, with a statistical return period of 2,475 years. The results of these evaluations are presented in the following table:

PROBABILISTIC GROUND MOTION DATA				
Earthquake Level	**Probabilistic Estimate Exposure Period (years)**	**Probability of Exceedance (%)**	**Return Period (years)**	**Estimated Peak Horizontal Ground Acceleration (g)**
Maximum Considered Earthquake, geometric mean (MCEg)	50	2	2,475	0.292
Upper-Bound Ground-Motion	100	10	949	0.200
Design-Basis Ground-Motion	50	10	475	0.147

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

3.4 LIQUEFACTION & LATERAL SPREADING

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

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

If those criteria are present, then there is a potential that the soils could liquefy during a seismic event. The adverse effects of liquefaction include local and regional ground settlement, ground cracking and expulsion of water and sand, the partial or complete loss of bearing and confining forces used to support loads, amplification of seismic shaking, and lateral spreading. In general, the effects of liquefaction on the site 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.

The project site is underlain by metavolcanic rock that is not susceptible to liquefaction or lateral spreading.

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 2.2.2 and 2.3.2, ultramafic rocks in the form of serpentinite, chloritized serpentinite, and fissile serpentinzed pyroxenite were observed at the site of the proposed equalization tank. Soil samples taken during this study and tested for NOA found no NOA detected during the testing. Nearby samples tested by Vertical Sciences (2018) along Pine Needle Drive have been found to contain NOA's exceeding 6 percent. 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.

While we did not encounter NOA in our testing, its presence is noted in the area, as discussed above. If fibrous serpentinitic rocks are exposed during grading of the project, we recommend that those rocks be tested to evaluate the presence of NOA. If present, we recommend that an industrial hygienist be retained by the Contractor to develop methods of material handling that will reduce the potential for NOA to become aerosol during construction and to ensure worker safety.

3.6 EXPANSION POTENTIAL

There is a direct relationship between plasticity of a soil and the potential for expansive behavior, with expansion potential generally increasing as the Plasticity Index (PI) of a soil increases, as shown in the table below (from Day, 1999). Thus, granular soils typically have a low potential to be expansive, whereas fine-grained clay-rich soils can have a low to high expansion potential depending on various factors including the quantity and type of clay minerals present.

Atterberg limit testing was performed on three soil samples taken from the site. Results of the tests are as follows:

PLASTICITY INDEX TEST RESULTS		
Sample Location	**Sample Depth (ft)**	**Plasticity Index**
TP-1	5.5	32
TP-2	4	20
TP-3	4	17

As noted above, PIs ranged from 17 to 32. Soils having PIs in that range have a medium to high expansion potential, as noted in the following table (Day, 1999).

EXPANSION POTENTIAL – PLASTICITY INDEX CORRELATION	
Plasticity Index (PI)	**Correlated Expansion Potential**
0 – 10	Very Low
10 – 15	Low
15 – 25	Medium
25 – 35	High
35+	Very High
Taken from Day (1999)	

3.7 SOIL CHEMISTRY

Two samples of near-surface soils were subjected to chemical analysis for assessment of corrosion and reactivity with concrete. The samples were tested for soluble sulfates and chlorides. Test results are presented below.

SOIL CHEMISTRY RESULTS					
Sample Location	**Sample Depth**	**Sulfates (ppm)**	**Chlorides (ppm)**	**pH**	**Resistivity (ohms-cm)**
TP-1	0' – 4'	24.7	7.5	6.27	2,950
TP-3	0' – 3.5'	3.9	6.3	6.09	2,950

According to the American Concrete Institute publication ACI-318 , a sulfate concentration below 0.10 percent by weight (1,000 ppm) is considered negligible. A chloride content of less than 500 ppm is generally considered non-corrosive to reinforced concrete. Based on the results of the soil chemistry tests, the site soils have a low potential for corrosion of concrete due to sulfates and chlorides.

Minimum resistivity testing was also performed on the soil samples. A commonly accepted correlation between soil resistivity and corrosivity towards ferrous metals (NACE Corrosion Basics, 1984) is provided below:

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

Thus, according to the table above, the soils are estimated to be moderately corrosive to ferrous metals based upon the soil resistivity value measured for this study.

4 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Recommendations presented, herein, are based upon project information provided by WWE along with stated assumptions. Changes in the project element configurations from those studied during this investigation, as noted on Plate 2, may require supplemental recommendations.

4.2 GEOLOGIC HAZARDS

It is our opinion that, aside from NOA, geologic hazards should not impact the proposed project. As noted above, NOA was not encountered in testing during this study but was encountered at a nearby site on the property. As discussed in Section 3.5, no action needs to be taken regarding NOA at this time but during construction, if fibrous serpentinite is observed, then it is recommended that the Contractor retain an industrial hygienist to help reduce risks of handling and placement of NOA-bearing earth materials.

In addition, as noted in Section 3.6, soils with a high expansion potential were encountered during this study. It is our opinion that these soils should not impact the project because excavations for the proposed tank will be made through those soils and expose the underlying nonexpansive rock materials.

4.3 SITE PREPARATION AND GRADING

4.3.1 Stripping

Prior to general site grading and/or construction of planned improvements at the site, existing vegetation, organic topsoil, debris, and deleterious materials should be stripped and disposed of off-site or outside the construction limits. Stripping depths on the order of 2 to 3 inches should be anticipated for the projects with locally deeper stripping possible depending on the conditions encountered during grading.

4.3.2 Existing Utilities, Wells, and/or Foundations

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

4.3.3 Keying and Benching

The proposed improvements should not involve the need to key and bench slopes.

4.3.4 Wet/Unstable Soil Conditions

Perched groundwater was observed at the soil-bedrock interface within test pits excavated at the site. It is likely that near-surface perched groundwater levels will exist during construction and could impact construction. It is likely that the Contractor can channel and dispose of the groundwater using conventional trash pumps during grading; however, the means and methods of controlling the groundwater are entirely the responsibility of the Contractor.

Perched groundwater and/or wet soil conditions due to precipitation, snow melt, or on-site water discharge and usage could hinder equipment access as well as efforts to compact site soils to a specified level of compaction. If over optimum soil moisture content conditions are encountered during construction, disking to aerate, replacement with imported material, chemical treatment, stabilization with a geotextile fabric or grid, and/or other methods will likely be required to facilitate earthwork operations. The applicable method of stabilization is the Contractor's responsibility and will depend on the Contractor's capabilities and experience, as well as other project-related factors beyond the scope of this investigation. Therefore, if over-optimum moisture within the soil is encountered during construction, BAJADA should review these conditions (as well as the Contractor's capabilities) and, if requested, provide recommendations for their treatment.

4.3.5 Site Drainage

Grading should be performed in such a manner that provides a positive surface gradient away from all structures. The ponding of water should not be allowed adjacent to structures or retaining walls. Surface runoff should be directed toward engineered collection systems. Discharge from structures should also be collected, conveyed, and discharged at least 20 feet away from structures.

4.3.6 Excavation Characteristics

Exploration at the site was performed using a Kubota K040-4 mini-excavator equipped with a two-foot-wide bucket and hydraulic angle blade. Penetration of underlying soil materials was performed with little to moderate difficulty. It is our opinion that these soils should be excavatable with heavy grading equipment in good condition and operated by experienced personnel with moderate difficulty.

Geophysical refraction surveys performed during this study found that rock materials with seismic velocities of up to at least 9,400 feet per second (ft/sec) are present within 5 feet of the ground surface in the areas where the surveys were conducted. The methods utilized and results obtained during those geophysical surveys are presented in Appendix C. Metavolcanic rocks with seismic velocities of 9,000 ft/sec or greater are typically not considered rippable using conventional heavy grading equipment (Caterpillar, 2018). Plate 7 – Caterpillar Ripping Chart, indicates that basaltic rock (likely the closest rock type to the pyroxene at the site) is not rippable using a D9R/D9T bulldozer equipped with a single ripping shank.

On that basis, nonconventional excavation methods will likely be needed to construct this project. On some projects, those methods could include blasting; however, we do not recommend the use of explosives or any other method that can generate strong ground motions during this project due to the presence of the hydraulically-constructed Magalia Dam, located immediately to the north of the site. Numerous studies have been performed on the dam and found that in its present state, it is susceptible to liquefaction (Slate, 2021). Ground accelerations that can trigger liquefaction and, thus, explosives or other construction methods that can generate strong ground motions should not be utilized for this project. Methods such as the use of hoe rams, expansive demolition agents, etc., are examples of other nontraditional excavation methods that might be utilized on this project. Ultimately, the method of excavation of the rock is the Contractor's responsibility.

4.3.7 Overexcavation

Overexcavation is anticipated at the proposed pump station site. See section 4.4.1 for a discussion regarding overexcavation.

4.3.8 On-Site Soil Materials

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

4.3.9 Engineered Fill Materials and Placement

4.3.9.1 General Engineered Fill

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 minor quantities of clay are acceptable for use as imported general engineered fill. Gradation and plasticity recommendations for general engineered fill are presented in the table below.

4.3.9.2 Structural Fill

Structural fill materials are defined as those materials specifically intended for support of structures and pavements. General recommendations for structural fill are presented in the table below and should be considered minimum requirements.

All imported fill materials, whether General or Structural, should be sampled and tested prior to importation to the project site to verify that those materials meet the recommended material criteria, in accordance with applicable test procedures to verify material suitability, as shown in the following table.

IMPORTED FILL RECOMMENDATIONS				
GRADATION				
Sieve Size	General Fill	Structural 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	<1%	<1%	D2974	NA
SOIL CHEMISTRY	Chloride <500 ppm	Sulfate <1,000 ppm	Resistivity >2,000 ohm-cm	pH 6-7

4.3.10 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 that it conform and be placed per specifications presented in Section 19-3.062 of the Caltrans Standard Specifications (most current edition).

4.3.11 Placement & Compaction

Soil and/or soil-aggregate mixtures used for general engineered fill should be uniformly moisture-conditioned to near the optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to at least 90% relative compaction in accordance with standard test method ASTM D1557[1]. All structural fill should be placed in the same manner and compacted to at least 95% relative compaction per ASTM D1557. It is recommended that fill materials be placed and compacted uniformly in elevation around buried structures and that the vertical elevation differential of contiguous lifts diverge no more than three feet around the structure during compaction. Testing should be performed to verify that the relative compactions are being obtained as recommended herein. Compaction testing, at a minimum, should consist of one test per every 250 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.

4.4 FOUNDATIONS & SLABS

4.4.1 Transition Lots

Transitions lots are those sites where a structure foundation will be supported partially by two different geologic materials, such as artificial fill beneath one portion of the structure and undisturbed native soil beneath the remainder of the structure. Those two materials could cause structures to settle at differing rates and magnitudes. The resulting differential settlement could cause damage to the structure, structure performance, or performance of equipment within the structure.

It is not anticipated that transition lots will be present at the site. We assume that the proposed tank will be supported on undisturbed rock materials present beneath the site or on a layer of compacted sand or gravel as determined by the project designers.

4.4.2 Foundation Subgrade Preparation

The subgrade for all foundations should be smooth and unyielding prior to the placement of concrete or any aggregate base or other structural fill material. If soft and yielding areas are found, BAJADA should review these conditions and, if requested, provide recommendations for their treatment. We recommend that all foundation excavations be observed and tested by a licensed geotechnical engineering consultant to confirm projected site conditions and the requirements of this report.

4.4.3 General Foundation Design Considerations

The foundations for all structures should be designed by the project civil/structural engineer in accordance with the recommendations presented in this report.

[1] This test method (ASTM D1557) applies wherever relative compaction, maximum dry density, or optimum moisture content is referenced within this report.

4.4.4 Bearing Pressure and Settlement

All foundations should be supported on firm, undisturbed rock materials underlying the site. An allowable bearing pressure of 4,000 psf for isolated or continuous footing foundations or structural slabs can be used for design of tank foundations supported on intact rock.

The anticipated total settlement for foundations under static (i.e., non-seismic) loading conditions, if construction occurs as recommended within this report, should be relatively low (less than ½-inch) for concrete footings or slabs resting on intact rock.

4.4.5 Sliding Resistance

Ultimate sliding resistance generated between concrete and intact rock or approved compacted granular engineered fill soil can be estimated by multiplying the total dead weight structural loads by a friction coefficient of 0.40. If a membrane, such as polysheeting or PVC, is utilized between the fill pad and concrete footings/slabs, then the coefficient of friction between concrete and the sheeting should be established through consultation with the membrane manufacturer.

4.4.6 Passive Resistance

Ultimate passive resistance developed from lateral bearing of foundation elements 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 of 390 pcf. An appropriate safety factor should be applied to this value.

4.4.7 Safety Factors

Sliding resistance and passive lateral pressure may be used together in conjunction with the following recommended safety factors. A minimum factor of safety of 1.5 is recommended for sliding resistance where passive pressure is neglected; a minimum factor of safety of 2.0 is recommended for sliding resistance where passive pressure is included.

4.4.8 Frost Penetration

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

4.4.9 Slab-on-Grade Design

All ground-supported concrete slabs should be designed by a structural or civil engineer to support the anticipated loading conditions. In addition to anticipated structural loads, the design considerations include, but are not limited to, concrete mix design, structural reinforcement, joint spacing, crack control, slab underlayment, moisture control and corrosion resistance. Reinforcement for slabs should meet all applicable code requirements.

Reinforcement should be placed in the slab per the design requirements of the structural or civil engineer 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. 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.

Soil materials supporting structural concrete slabs should be uniformly moisture-conditioned to near the optimum moisture content and compacted to at least 95% relative compaction.

4.5 RETAINING WALLS

4.5.1 General

The following recommendations for design of retaining walls or other buried earth-retaining structures at the project site are based on the assumption that the foundation subgrade will consist of intact rock and the wall backfill soils will consist of competent granular materials or approved imported fill, respectively. The suitability of the foundation subgrade and wall backfill materials and conditions are subject to inspection and verification/testing by BAJADA prior to construction to confirm that the material properties correlate with the recommended design parameters presented below.

4.5.2 Allowable Bearing Pressure

Retaining wall footings resting at least 2 feet below the lowest adjacent finished grade on intact rock may be designed using a maximum allowable toe pressure of 4,000 psf.

4.5.3 Lateral Earth Pressures

Retaining walls and other buried earth-retaining structures utilized in this project should be designed to resist the appropriate earth pressures exerted by the retained, compacted backfill plus any additional lateral force that will be applied to the wall such as seismic loading and surface loads placed at or near the wall. The recommended equivalent fluid weights are presented in the table below. Walls that are not free to deflect should be designed to resist at-rest earth pressures.

LATERAL EARTH PRESSURES UNDER STATIC CONDITIONS		
Lateral Earth Pressure Condition	Slope Inclination Above Structure[1]	Equivalent Fluid Weight (pcf)
		Drained
At-Rest	Flat	60
Active	Flat	40
At-Rest	2:1	80
Active	2:1	60
[1] – horizontal:vertical		

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

The tabulated active pressure values are based on Rankine lateral earth pressure assumptions for granular soil with a phi-angle (φ) of 31 degrees, a soil unit weight (γ) of 125 pounds per cubic foot (pcf), and a vertical back wall, and do not provide for surcharge conditions resulting from foundations, vehicle traffic, or compaction equipment. As noted in the table, the equivalent fluid weights are drained values and, therefore, do not provide for hydrostatic forces (for example, standing water in the backfill materials), which must be considered separately.

Foundation loads not considered as surcharges should bear behind a 1:1 (horizontal to vertical) line projected upward from the base of the wall. If conditions such as surcharges resulting from footings or hydrostatic forces are expected, BAJADA 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.

4.5.4 Drainage Measures

Drainage measures should be constructed behind the proposed retaining wall to reduce the potential for groundwater accumulation. To help reduce the potential for the buildup of hydrostatic forces behind the wall, a granular free-draining backfill, at least 2 feet thick should be placed behind the wall, as shown on Plate 8 – 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 3 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 of the *Caltrans Standard Specifications*, most current edition;
- Pea gravel having a nominal diameter of 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 BAJADA for approval. In addition, manufactured drainage systems should be attached to the retaining wall face as opposed to the excavated slope face. This requires that provisions to protect the integrity of the drainage panels will need to be made while fill materials are being placed behind the walls.

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

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

4.5.5 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 above ground surface (ft)

Peak ground acceleration (PGA) values for the site are provided in Section 3.2. 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.

4.5.6 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 BAJADA should observe all foundation excavations prior to concrete placement.

4.6 ROCK ANCHORS

We understand that temporary measures might be needed to retain the rock materials behind proposed construction so that construction can be performed. To do so, we understand that the rock materials might be retained by rock anchors. We understand that the project structural engineers require rock-grout bond strengths to help design the rock anchors so that anchor length and spacings can be evaluated. The rock-bond strength is typically estimated from the rock unconfined compressive strength using the following equations:

$t_u = s_u/10$ – Ultimate Bond Stress
$t_a = s_u/30$ – Working Bond Stress

Where s_u = rock unconfined compressive strength (psi; Transportation Research Board, 2012).

We performed two unconfined compression (Uc) tests on rock samples taken at the site during our study for the equalization tanks. Those tests resulted in Uc values of 7,450 and 9,400 psi. This is a small sampling of rock strengths and we recommend lowering the value to 4,500 psi or lower for calculation of bond strength to account for variations in rock weathering and consistencies.

The length of the rock anchors can be computed using recommendations from the Post-Tensioning Institute (2004). The following is the equation typically used for calculating the bonded length for the anchors:

$$L_b = Q/\pi*d*t_a$$

Where:
L_b = bond length
Q = design load at head of anchor
$\pi = 3.1415$
d = diameter of the drill hole; and
t_a = working bond stress along the interface between rock and grout.

In addition to the bonded length of the anchors, an unbonded length should be included in design in accordance with recommendations of the Post-Tensioning Institute (2004).

4.7 PIPELINES & TRENCH BACKFILL

4.7.1 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 or 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, respectively (feet)
C_d, C_t	=	See below
c	=	Soil cohesion (psf)

Plate 9 – 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 (ft)

The value Kμ' is dependent on the backfill type, degree of compaction, and moisture content. Where trench backfill materials are compacted as recommended in Section 4.6.6 – Placement and Compaction, 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)	0.150
Sand & Gravel (SM, GM)	0.165
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 10 – Vertical Soil Pressures Induced by Live Loads.

4.7.2 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. We recommend that an E'_n value of 600 and 2,000 psi be used for design when in soil and rock, respectively.

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:

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

4.7.3 *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 4.5.5. For design of thrust block resistance, an ultimate passive lateral earth pressure of 390 psf/ft of depth may be used. An appropriate factor of safety should be applied to this value.

4.7.4 *Excavations, Trenches, Dewatering, & Shoring*

4.7.4.1 Excavation and Trench Slopes

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

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

OSHA SOIL TYPE DETERMINATIONS	
Stable Rock	Natural solid mineral matter that can be excavated with vertical sides and remain intact while exposed. It is usually identified by a rock name such as granite or sandstone. Determining whether a deposit is of this type may be difficult unless it is known whether cracks exist and whether or not the cracks run into or away from the excavation.
Type A Soils	Cohesive soils with an unconfined compressive strength of 1.5 tons per square foot (tsf) (144 kPa) or greater. Examples of Type A cohesive soils are often: clay, silty clay, sandy clay, clay loam and, in some cases, silty clay loam and sandy clay loam. (No soil is Type A if it is fissured, is subject to vibration of any type, has previously been disturbed, is part of a sloped, layered system where the layers dip into the excavation on a slope of 4 horizontal to 1 vertical (4H:1V) or greater, or has seeping water.
Type B Soils	Cohesive soils with an unconfined compressive strength greater than 0.5 tsf (48 kPa) but less than 1.5 tsf (144 kPa). Examples of other Type B soils are: angular gravel; silt; silt loam; previously disturbed soils unless otherwise classified as Type C; soils that meet the

OSHA SOIL TYPE DETERMINATIONS	
Type B Soils	unconfined compressive strength or cementation requirements of Type A soils but are fissured or subject to vibration; dry unstable rock; and layered systems sloping into the trench at a slope less than 4H:1V (only if the material would be classified as a Type B soil).
Type C Soils	Cohesive soils with an unconfined compressive strength of 0.5 tsf (48 kPa) or less. Other Type C soils include granular soils such as gravel, sand and loamy sand, submerged soil, soil from which water is freely seeping, and submerged rock that is not stable. Also included in this classification is material in a sloped, layered system where the layers dip into the excavation or have a slope of four horizontal to one vertical (4H:1V) or greater.
Layered Geological Strata	Where soils are configured in layers, i.e., where a layered geologic structure exists, the soil must be classified on the basis of the soil classification of the weakest soil layer. Each layer may be classified individually if a more stable layer lies below a less stable layer, i.e., where a Type C soil rests on top of stable rock.

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

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

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

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

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

4.7.4.2 Dewatering

Perched groundwater was encountered within explorations advanced for this study. It is likely that near-surface perched groundwater levels will exist during construction and 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. When 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.

4.7.4.3 Shoring

Preliminary design of braced shoring for trenches may be based on the preliminary shoring pressure diagrams provide on Plate 11 - Preliminary Shoring Pressure Diagrams. The preliminary shoring pressure diagrams provided on Plate 11 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.

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

Hydraulic speed shores and trench box shoring in flowing, running, or raveling ground conditions should not be allowed. Furthermore, soils subject to running, flowing, or raveling will have insufficient strength and stand-up time to safely hold full-depth vertical excavations long enough for complete trench box or speed-shore installations. Vertical excavations in such soils will most likely experience excavation wall loss and related undermining of adjacent pavements, utilities, structures, and improvements. Therefore, as a precautionary measure, shoring with trench boxes in flowing, running, or raveling soils will require very careful interior excavation through the trench box so that there are no unsupported vertical excavation faces as the trench box is incrementally lowered into place. Additionally, pre-advancing/driving steel backer plates in soil around the exterior perimeter of the trench box and ahead of excavations within the trench box may be necessary to maintain stable sidewalls and protect adjacent pavement, utilities, and structures. Shoring with speed shores in running or fast raveling ground will require solid sheet backing to provide full face support.

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

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

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

4.7.5 *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.

4.7.5.1 Pipe Zone Backfill

The pipe zone, as discussed herein, is that cross-sectional area that extends from the bottom of the trench to 6 inches over the crown of the pipeline, and from trench wall to trench wall, as shown on Plate 12. Pipe zone backfill materials should consist of imported soil having an SE of no less than 30 and having a particle size no greater than ½-inch in maximum dimension, per Section 306-1.2.1 of the Greenbook. Some on-site soils might meet these specifications; however, most of those soils will likely not meet these recommendations.

4.7.5.2 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 4.3.9 – Engineered Fill Materials and Placement, of this report. Those imported materials should be free of deleterious materials, organic debris, or clasts exceeding 3 inches in diameter in any direction.

4.7.5.3 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.

4.7.6 *Placement & Compaction*

Trench backfill should be placed and compacted in accordance with recommendations presented Section 4.3.9 – Engineered Fill Materials and Placement, of this report. 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 200 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.

4.7.7 *Trench Subgrade Stabilization*

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

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

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

5 REVIEW OF PLANS AND SPECIFICATIONS

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

6 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 specifically addressed and described herein (see Section 1.1 – 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 in order 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, BAJADA 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 BAJADA 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 BAJADA 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, BAJADA shall be notified of such occurrence in order to review current conditions. Depending on that review, BAJADA 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 BAJADA of such intended use. Based on the intended use as well as other site-related factors, BAJADA 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 BAJADA from any liability arising from the unauthorized use of this report.

7 REFERENCES

American Association of Civil Engineers (2017), Minimum Design Loads and Associated Criteria for Buildings and Other Structures, ASCE/SEI 7-16, 889 p.

American Concrete Institute (2019), Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary on Building Code Requirements for Structural Concrete (ACI 318 R-19), 623 p.

Boore, D.M. and Atkinson, G.M. (2007), Boore-Atkinson NGA Ground Motion Relations for the Geometric Mean Horizontal Component of Peak and Spectral Ground Motion Parameters, PEER 2007/01, Pacific Earthquake Engineering Research Center, Berkeley, California.

Burnett, J. L., Jenkins, C. W. (1962). Geologic Map of California: Chico sheet, California Division of Mines and Geology.

California Building Standards Commission (2023), 2022 California Building Code Standards, accessed at: https://www.dgs.ca.gov/BSC/Codes.

California Department of Water Resources (2023), Water Data Library, accessed at: http://www.water.ca.gov/waterdatalibrary/.

California Geological Survey (2002), California Geomorphic Provinces, Note 36, California Geological Survey, 3 p.

Campbell, K.W. and Bozorgnia, Y. (2007), Campbell-Bozorgnia NGA Ground Motion Relations for the Geometric Mean Horizontal Component of Peak and Spectral Ground Motion Parameters, PEER 2007/02, Pacific Earthquake Engineering Research Center, Berkeley, California.

Chiou, B.S., and Youngs, R.R. (2006), Chiou and Youngs PEER-NGA Empirical Ground Motion Model for the Average Horizontal Component of Peak Acceleration and Pseudo-Spectral Acceleration for Spectral Periods of 0.01 to 10 Seconds, Pacific Earthquake Engineering Research Center, Berkeley, California.

Clark, W.B. (1970), Gold Districts of California, California Division of Mines and Geology Bulletin 193, pp. 88.

Day, R. (1999), Geotechnical and Foundation Engineering, Design and Construction, McGraw – Hill, New York, NY 10121-2298.

Dudley, T. (1988), Memorandum of Fault Evaluation, Magalia Fault, unpublished DSOD publication, January 15, 9 p. with attachments.

Gassaway, E.M. (1899), The Magalia California Drift Mine, Mining and Scientific Press, July 31, pp 372-401.

Geotracker (2023), State Water Resources Control Board, Geotracker Database accessed at http://geotracker.waterboards.ca.gov/.

Hart, E.W. and Bryant, W.A. (1997), Fault-Rupture Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps, California Division of Mines and Geology Special Publication 42, with supplements 1 and 2 added in 1999, 38 p.

Helley, E. J., and Harwood, D. S. (1985). Geologic map of the Late Cenozoic deposits of the Sacramento Valley and Northern Sierran Foothills, California Scale: 1:63,360.

Hinds, N.E. (1952), Evolution of the California Landscape, California Division of Mines and Geology Bulletin 158, pp. 145-152.

Jennings, C.W. (1994), Fault Activity Map of California and Adjacent Area, with Locations and Ages of Recent Volcanic Eruptions, California Division of Mines and Geology, Geologic Data Map No. 6, Scale 1:750,000.

Kleinfelder (1992), Geotechnical Investigation Report, Proposed Plant Expansion, Paradise Irrigation District, Magalia, California, unpublished consultant's report prepared for Brown & Caldwell, dated February 13, 52 p. with plates and appendices.

Lindgren, W. (1911), The Tertiary Gravels of the Sierra Nevada of California, U.S. Geological Survey Professional Paper 73, pp. 73.

Logan, C.A. (1928), Report XXIV of the State Mineralogist covering Mining in California: California State Division of Mines and Mining, Volume 24, Number 3, pp. 191-207Marliave, C., 1932, Geological Report on Magalia Dam Situated on Little Butte Creek in Butte County, December.

Olmsted, F. H., Davis, G. H. (1961), Geologic features and ground-water storage capacity of the Sacramento Valley, California. Geologic Features and Ground-Water Storage Capacity of the Sacramento Valley California. doi:10.3133/wsp1497

Post-Tensioning Institute (2004), Recommendations for Prestressed Rock and Soil Anchors, 4th Ed., Phoenix, Arizona.

Seed, H. B. (1979), "Considerations in the Earthquake-Resistant Design of Earth and Rockfill Dams," Geotechnique, Vol. 29, No. 3, pp. 215-263.

Seed, H.B., and Whitman, R. (1970), Design of Earth Retaining Structures for Dynamic Loads, ASCE Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures, p. 103-147.

Slate Geotechnical Consultants (2021), Geotechnical Data Report, Revision 0, Magalia Dam Seismic Retrofit Report, Butte County, California, unpublished consultant's report prepared for Paradise Irrigation District, dated June 18, 464 p.

Taber (2015), Structure and Pavement Distress Evaluation – Revised Draft, Paradise Irrigation District Water Treatment Plant, 13888 Pine Needle Drive, Magalia, California, unpublished consultant's report prepared for AECOM, dated September 8, 66 p.

Toppozada and Branum (2002), *Bulletin of the Seismological Society of America*; October 2002; v. 92; no. 7; p. 2555-2601.

Toppozada, T. R., and D. Branum (2002), *California M >= 5.5 earthquakes, history and areas damaged*, in Lee, W. H., Kanamori, H. and Jennings, P., International Handbook of Earthquake and Engineering Seismology, International Association of Seismology and Physics of the Earth's Interior.

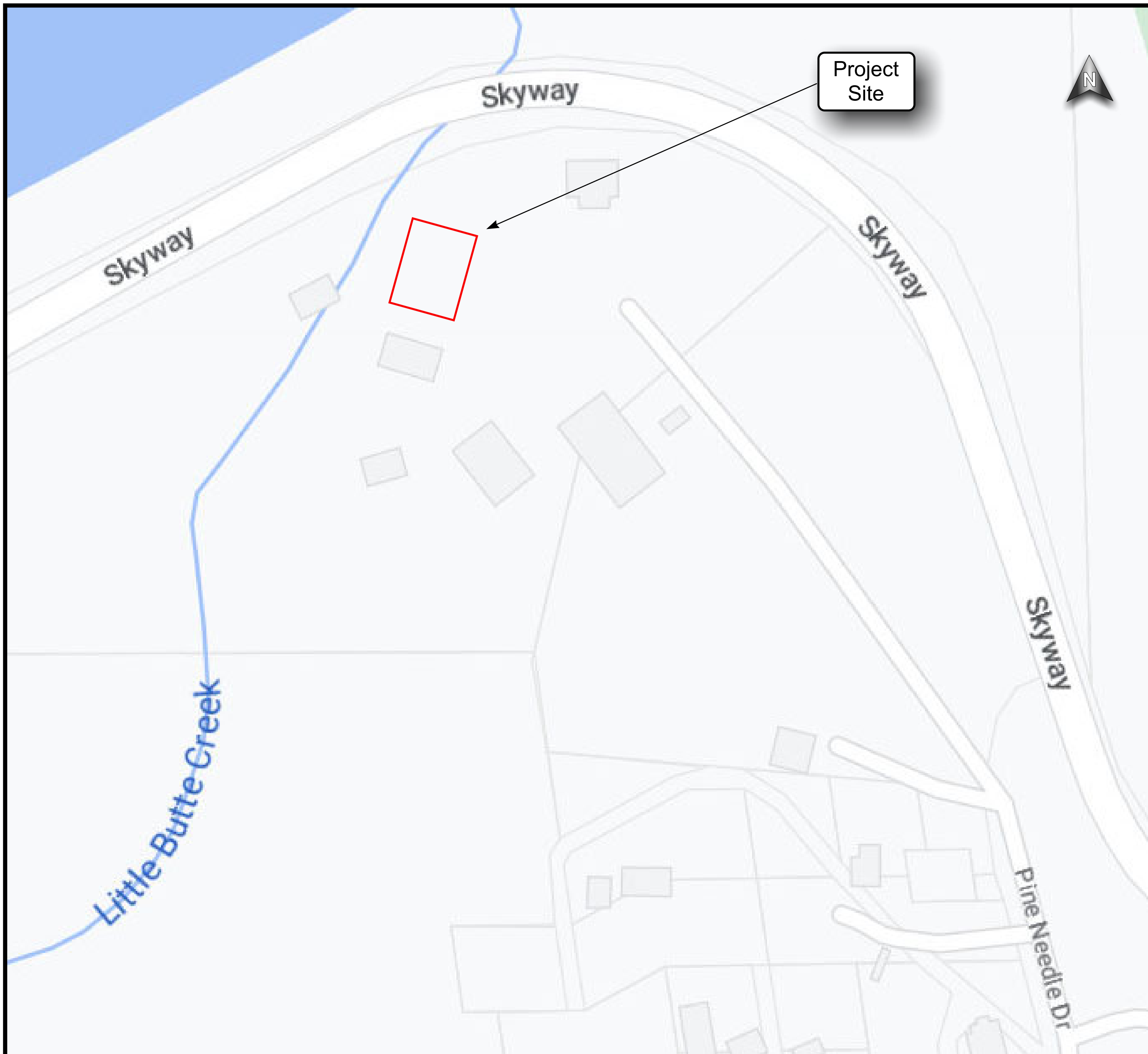
Transportation Research Board (2012), Rockfall, Characterization and Control, Washington D.C., p. 471 – 472.

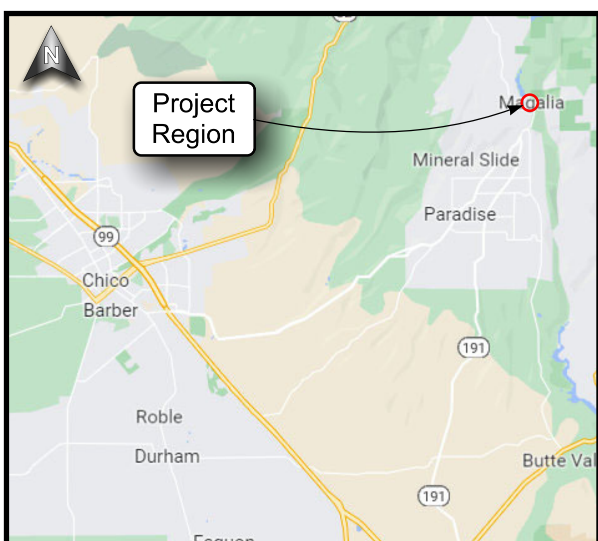
U.S. Geological Survey (2023), Unified Hazard Tool website accessed at: https://earthquake.usgs.gov/hazards/interactive/.

Vertical Sciences, Inc. (2018), Paradise Irrigation District, Reservoir B Replacement Study, Paradise Irrigation District, Town of Paradise and Butte County, California, dated July 31, 153 p.

Youd T.L., Idriss I.M., Andrus R.D., Arango I., Castro G., Christian J.T., et al. 2001.

Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. Journal of Geotechnical and Geoenvironmental Engineering 127(10): 817-833





SITE LOCATION MAP

Backwash Equalization Tank Paradise Irrigation District Water Works Engineers Magalia, California	Plate No. 1
BAJADA Geosciences, Inc.	Project no. 2201.0155

Base maps from Google Maps. Scale undetermined.

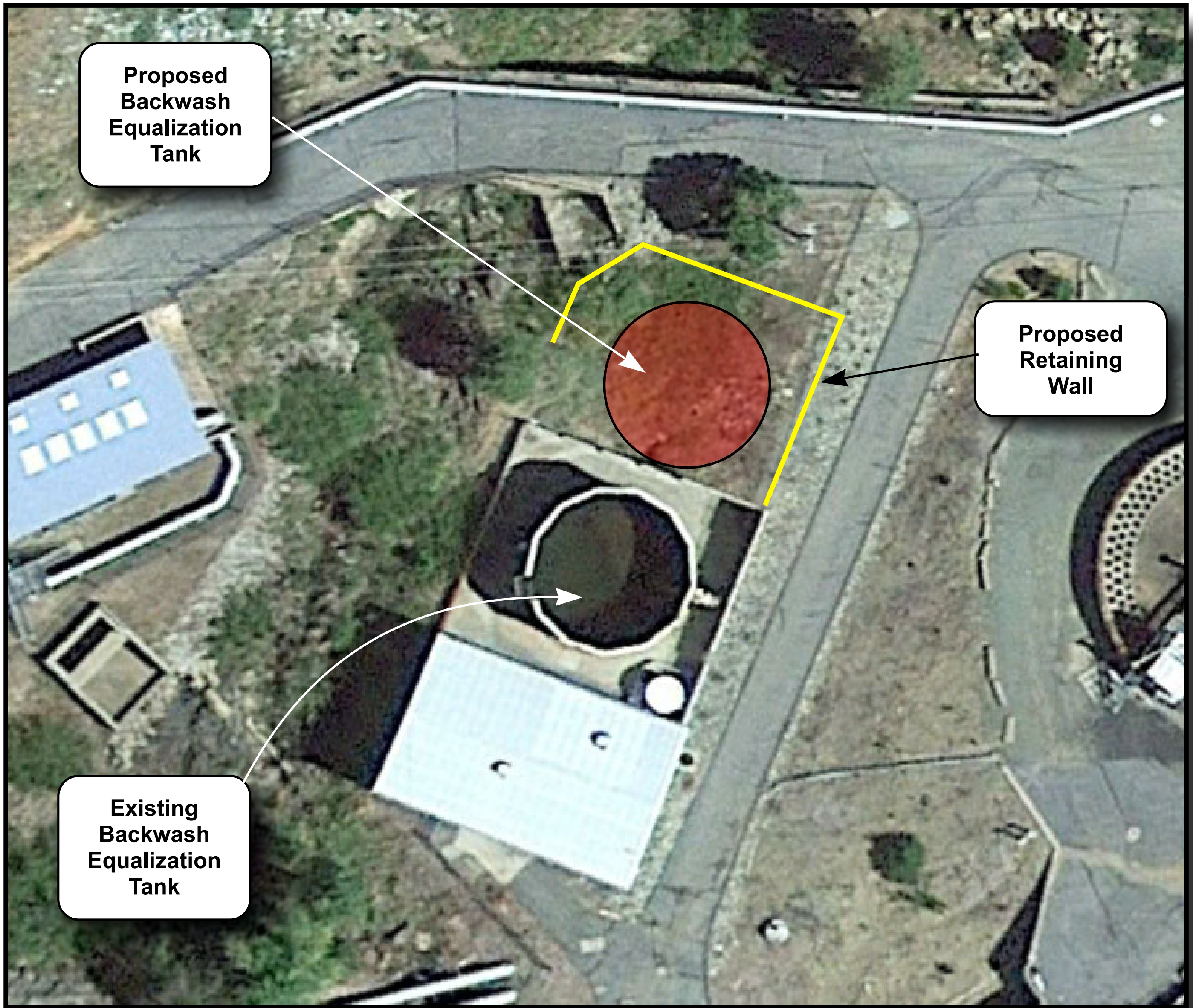
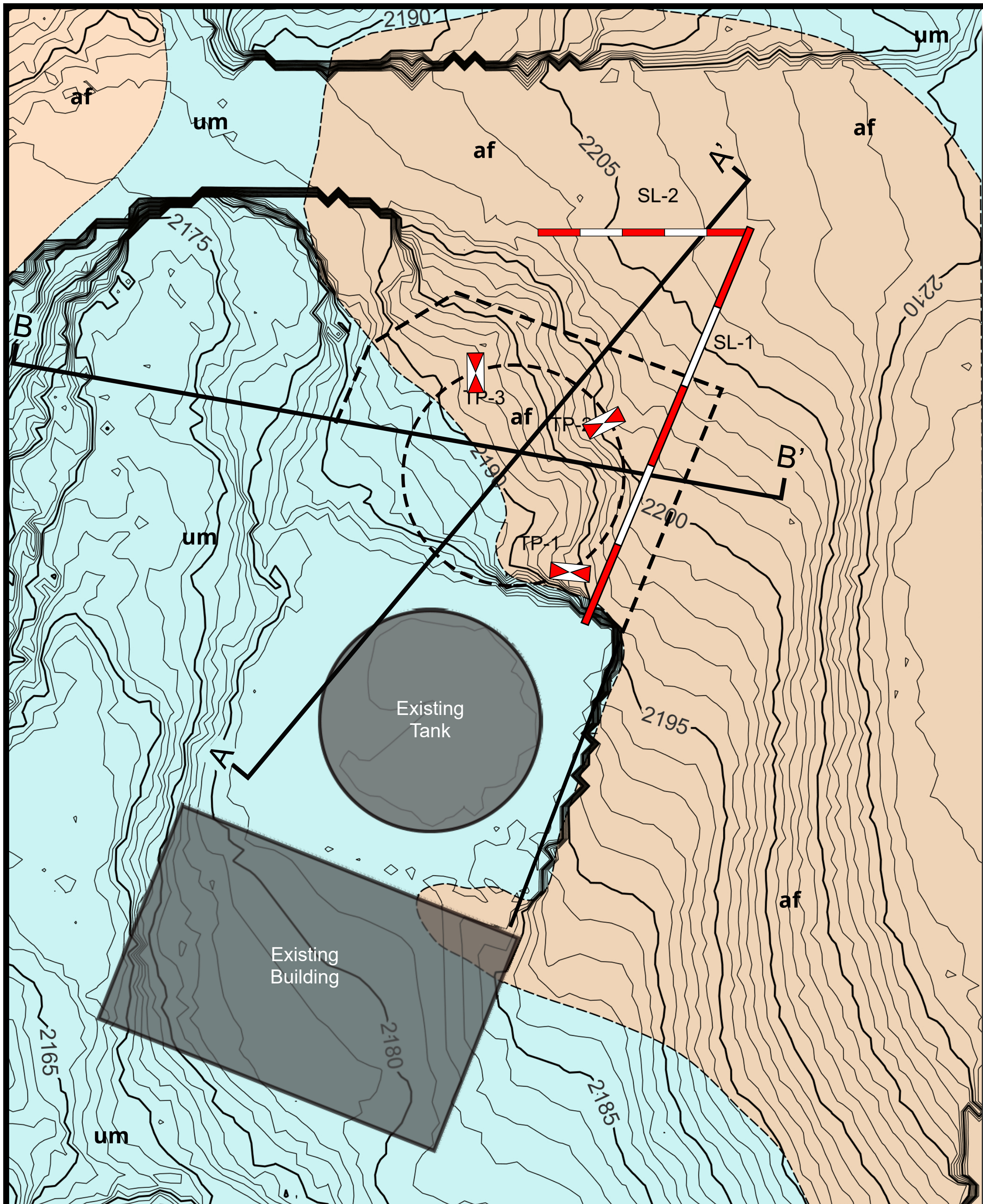


Image from Google Maps. Scale undetermined.

PROJECT ELEMENTS	
Backwash Equalization Tank Paradise Irrigation District Water Works Engineers Magalia, California	Plate No. 2
BAJADA Geosciences, Inc.	Project no. 2201.0155



af Artificial Fill

um Ultramafic (generally serpentinite) Rocks

TP-3 Test pit location

SL-2 Geophysical Refraction Survey Line

B B' Cross Section
see Plates 4.1 and 4.2

0 15 30
Scale: 1"=30'
1:360

GEOTECHNICAL MAP

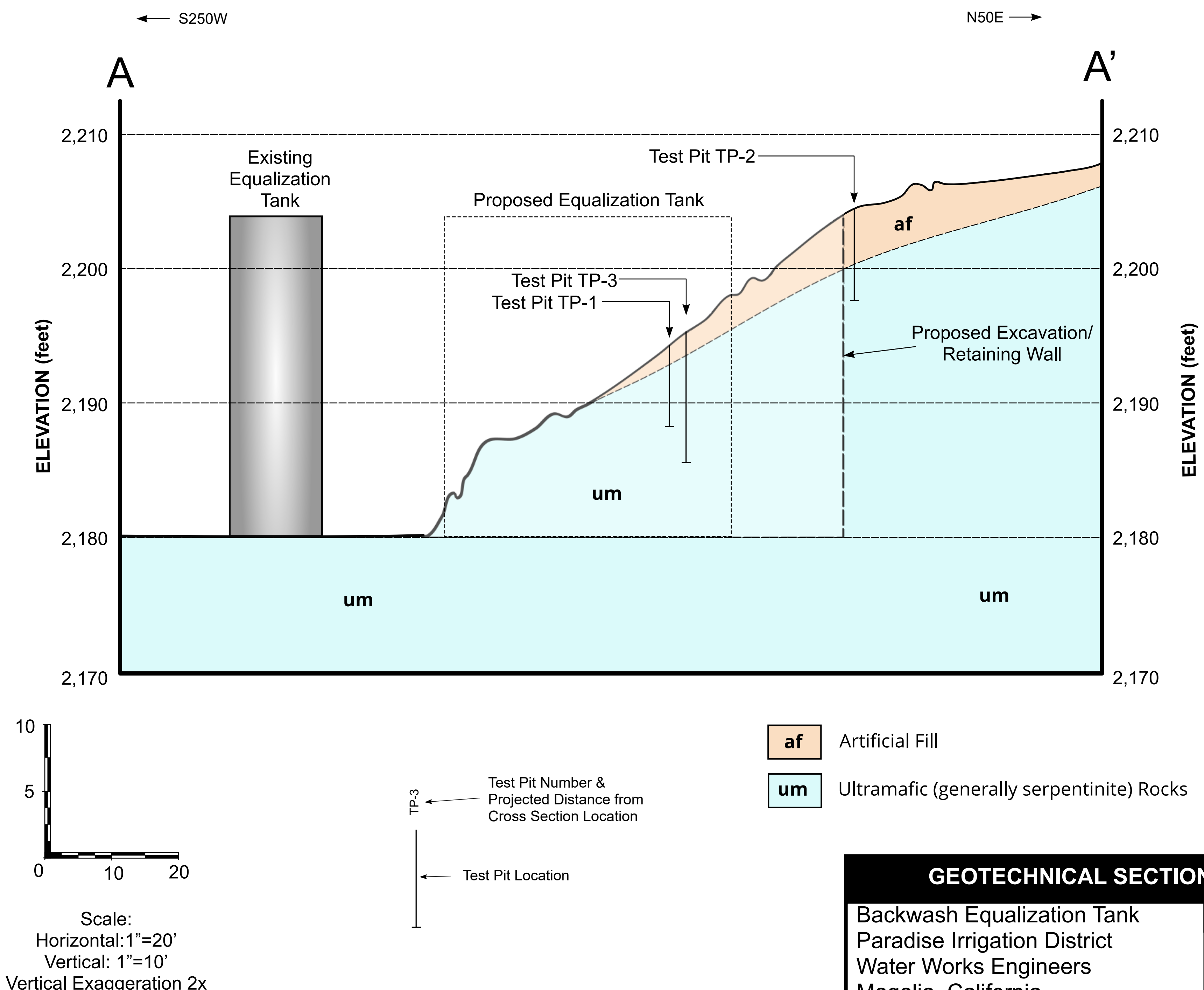
Backwash Equalization Tank
Paradise Irrigation District
Water Works Engineers
Magalia, California

Plate No. 3

BAJADA Geosciences, Inc.

Project no. 2201.0155

Topography from open source LiDAR.



GEOTECHNICAL SECTION A-A'

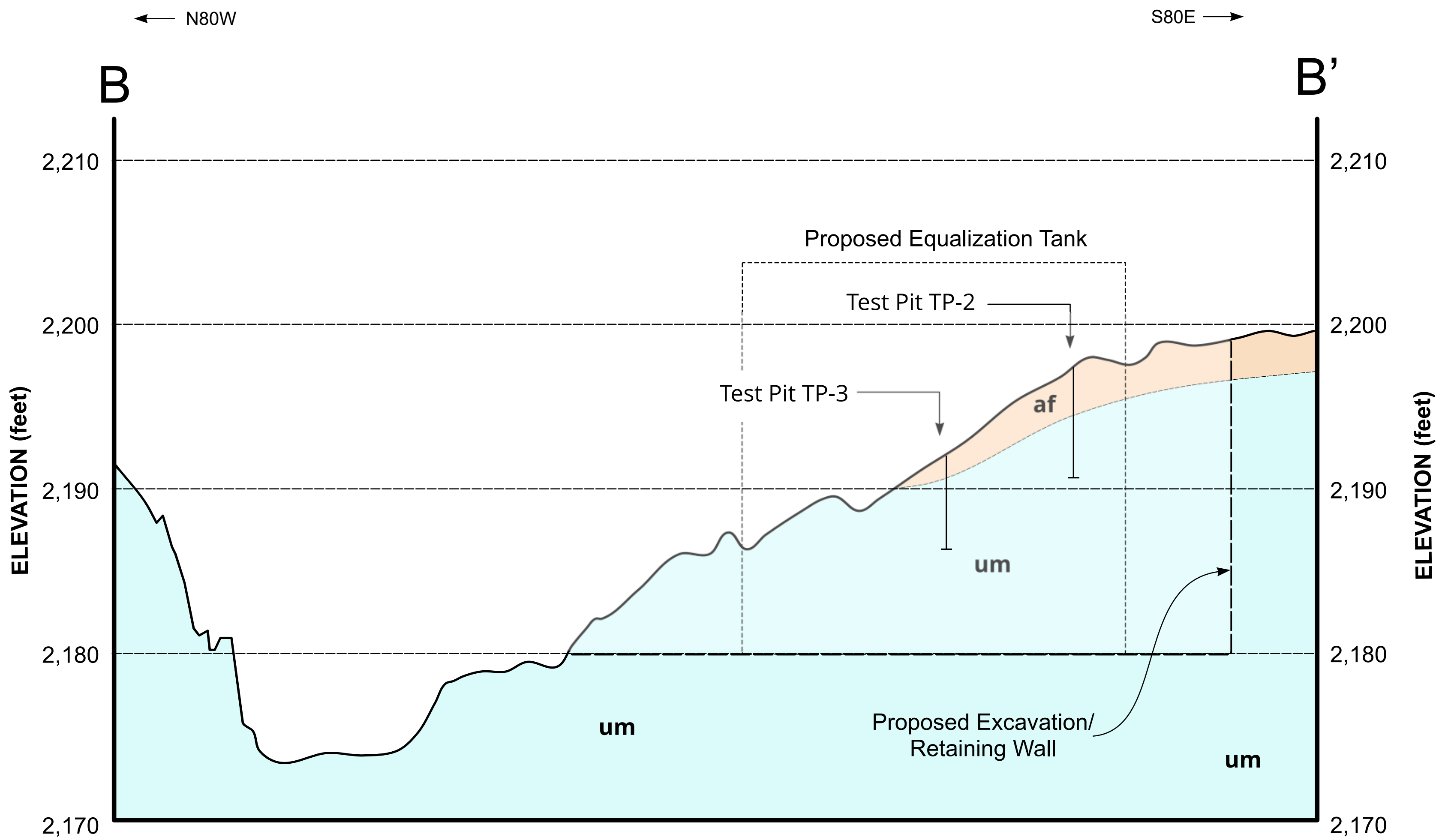
Backwash Equalization Tank
Paradise Irrigation District
Water Works Engineers
Magalia, California

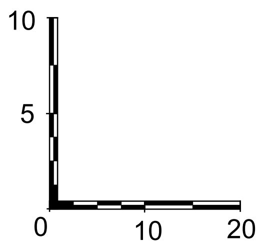
Plate No. 4.1

BAJADA Geosciences, Inc.

Project no. 2201.0155

See Plate 3 for cross section location..





Scale:
Horizontal:1"=20'
Vertical: 1"=10'
Vertical Exaggeration 2x

TP-3 Test Pit Number & Projected Distance from Cross Section Location

Test Pit Location

af	Artificial Fill
um	Ultramafic (generally serpentinite) Rocks

See Plate 3 for cross section location..

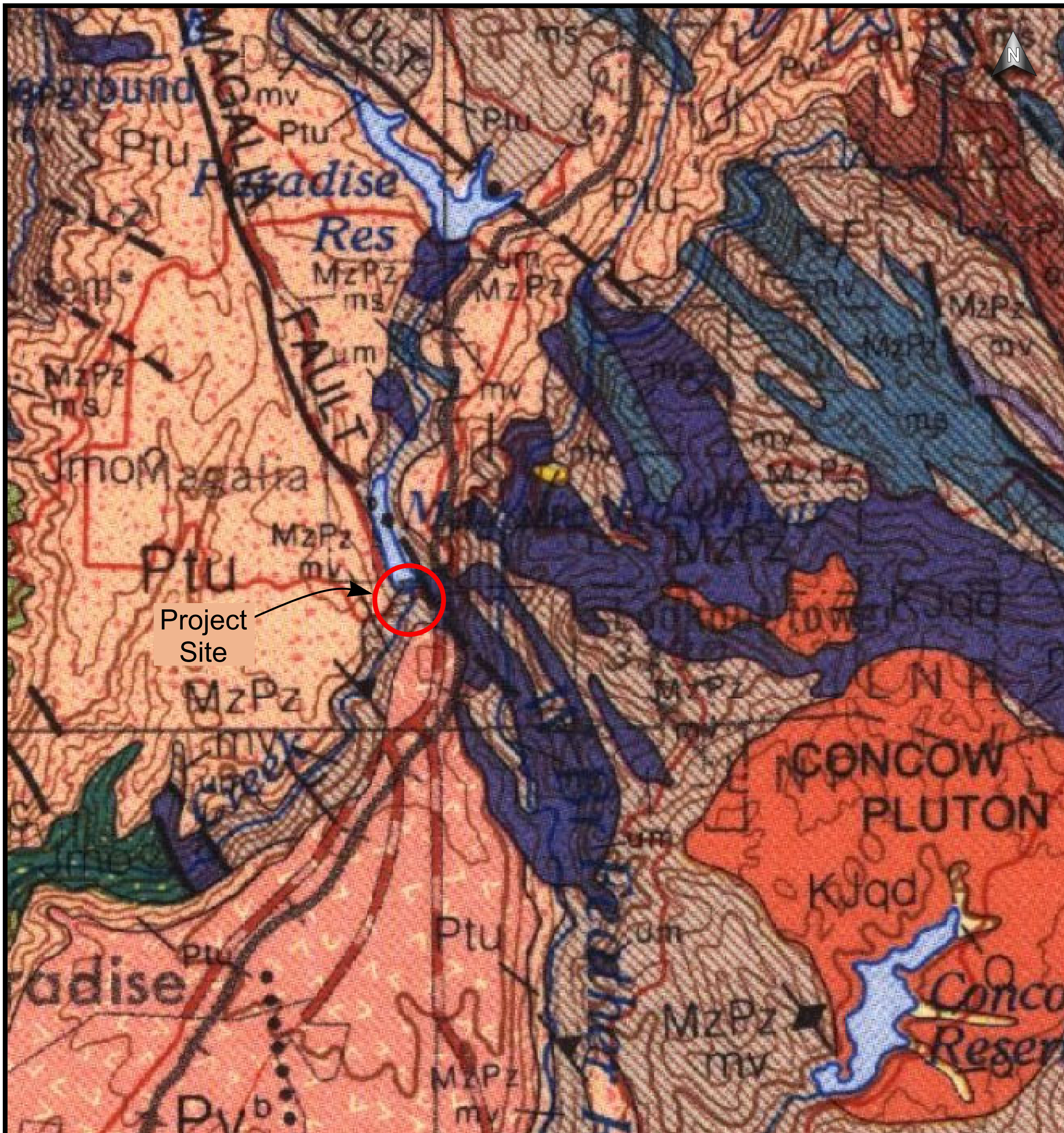
GEOTECHNICAL SECTION B-B'

Backwash Equalization Tank
Paradise Irrigation District
Water Works Engineers
Magalia, California

Plate No. 4.2

BAJADA Geosciences, Inc.

Project no. 2201.0155

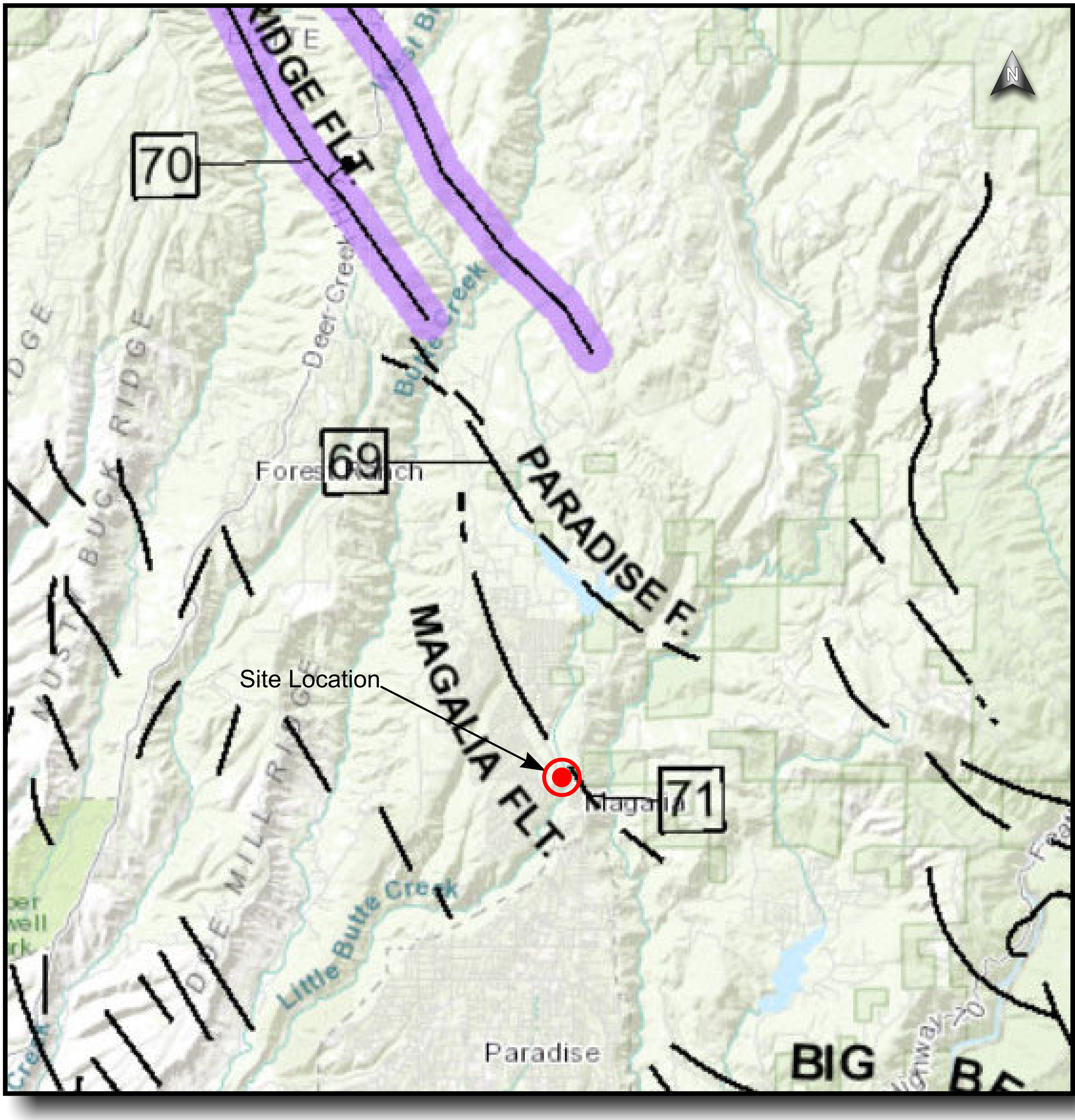


- Pv — Pliocene volcanic rocks *(a-andesite; b-basalt)*
- Ptu, Pnt — Tuscan Formation *(Lahars, volcaniclastic sediments, and tuff)* Pnt-Nomlaki Tuff
- Kc — Chico Formation
- Jv — Jurassic volcanic rocks *(Pyroclastic rocks and flows)*
- KJqd — Quartz diorite, tonalite, trondhjemite, quartz monzonite
- mv — mv - metavolcanic rocks
- um — um - ultramafic rocks

Base Map sourced from Saucedo & Wagner (1992). Scale undetermined.

REGIONAL GEOLOGY

Backwash Equalization Tank Paradise Irrigation District Water Works Engineers Magalia, California	Plate No. 5
BAJADA Geosciences, Inc.	Project no. 2201.0155



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)	

Base map from USGS Interactive Fault Map (2023). Scale undetermined.

REGIONAL FAULT MAP

Backwash Tank Installation Project 13827 Pine Needle Drive Paradise Irrigation District Water Works Engineers Magalia, California	Plate No. 6
BAJADA Geosciences, Inc.	Project no. 2201.0155

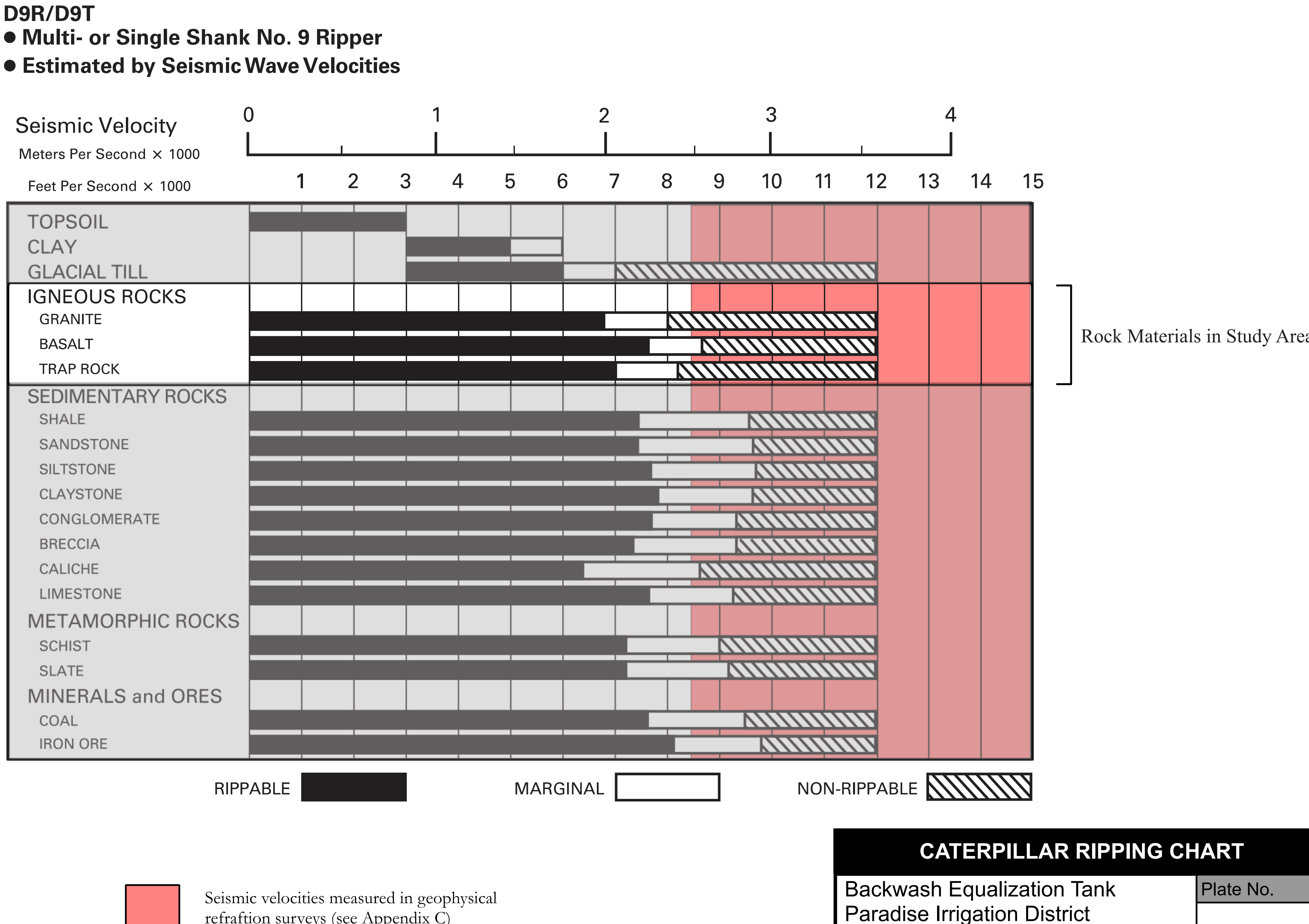
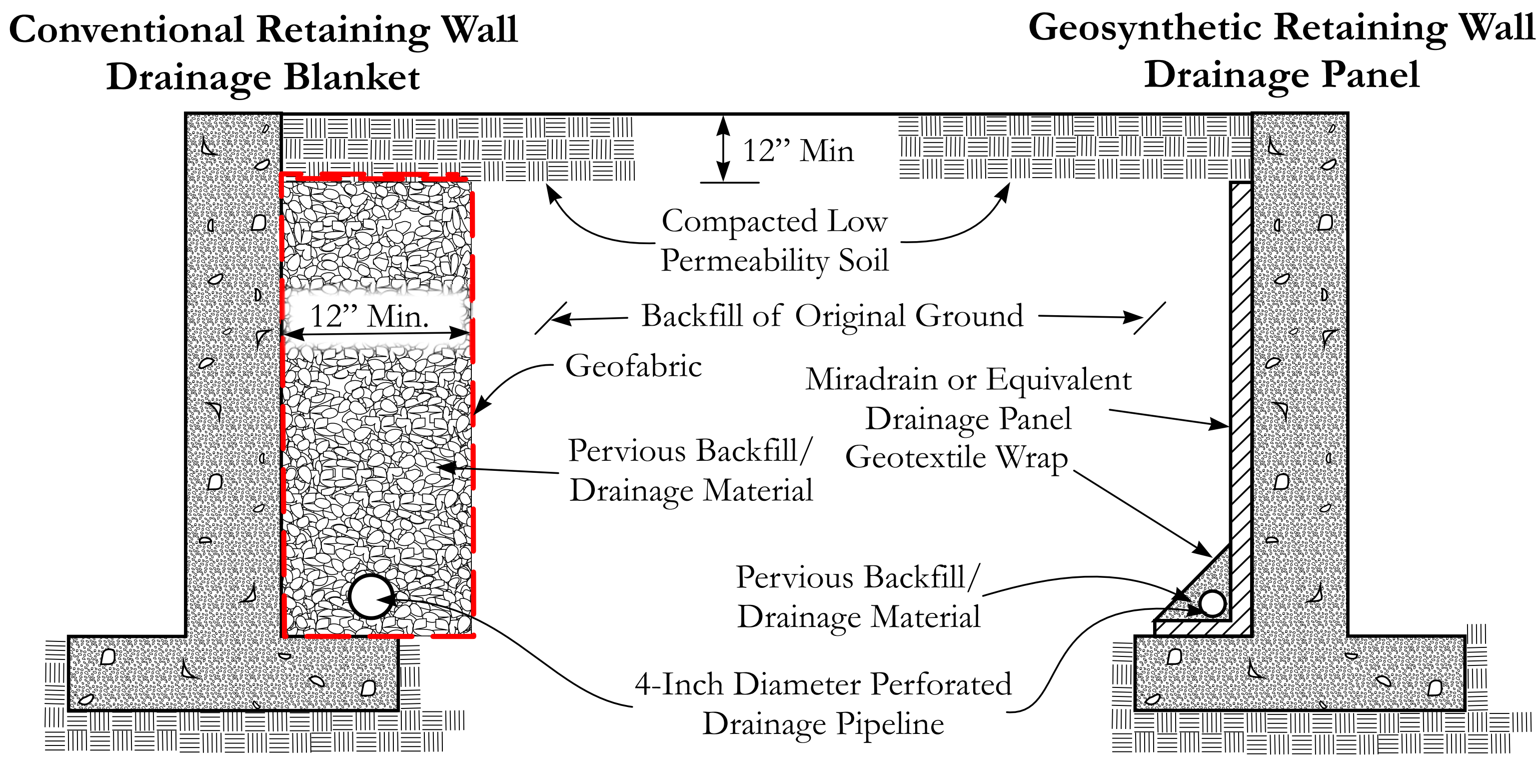


Chart from Caterpillar (2018).

CATERPILLAR RIPPING CHART	
Backwash Equalization Tank Paradise Irrigation District Water Works Engineers Magalia, California	Plate No. 7
BAJADA Geosciences, Inc.	Project no. 2201.0155



General Notes

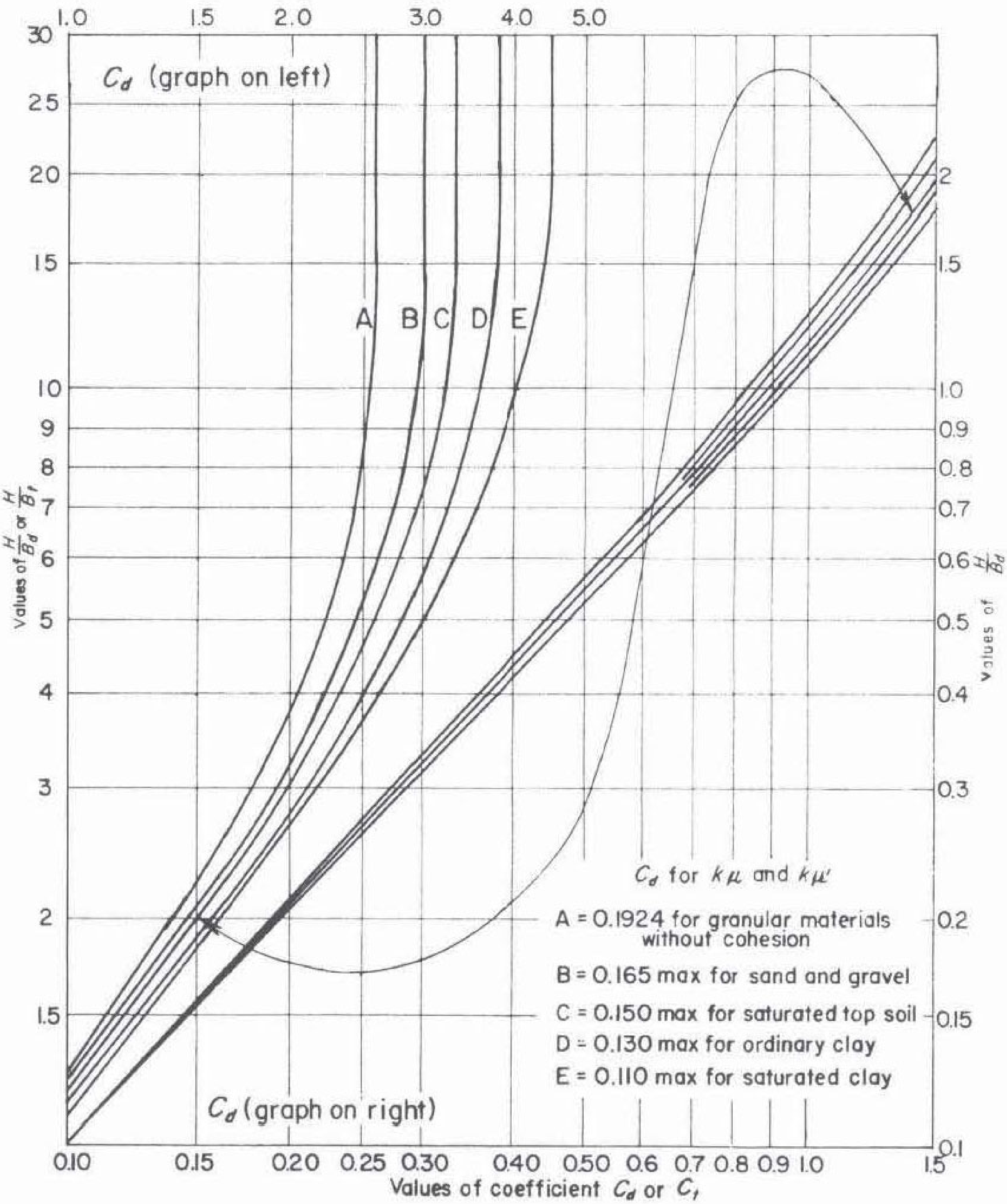
Pervious backfill/drainage material should conform to Pervious Backfill per Greenbook specifications, Class 2 Permeable Material per Caltrans Standard Specifications, pea gravel having a nominal 1/4-inch diameter, or crushed stone sized between 1/4-inch and 1/2-inch.

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.

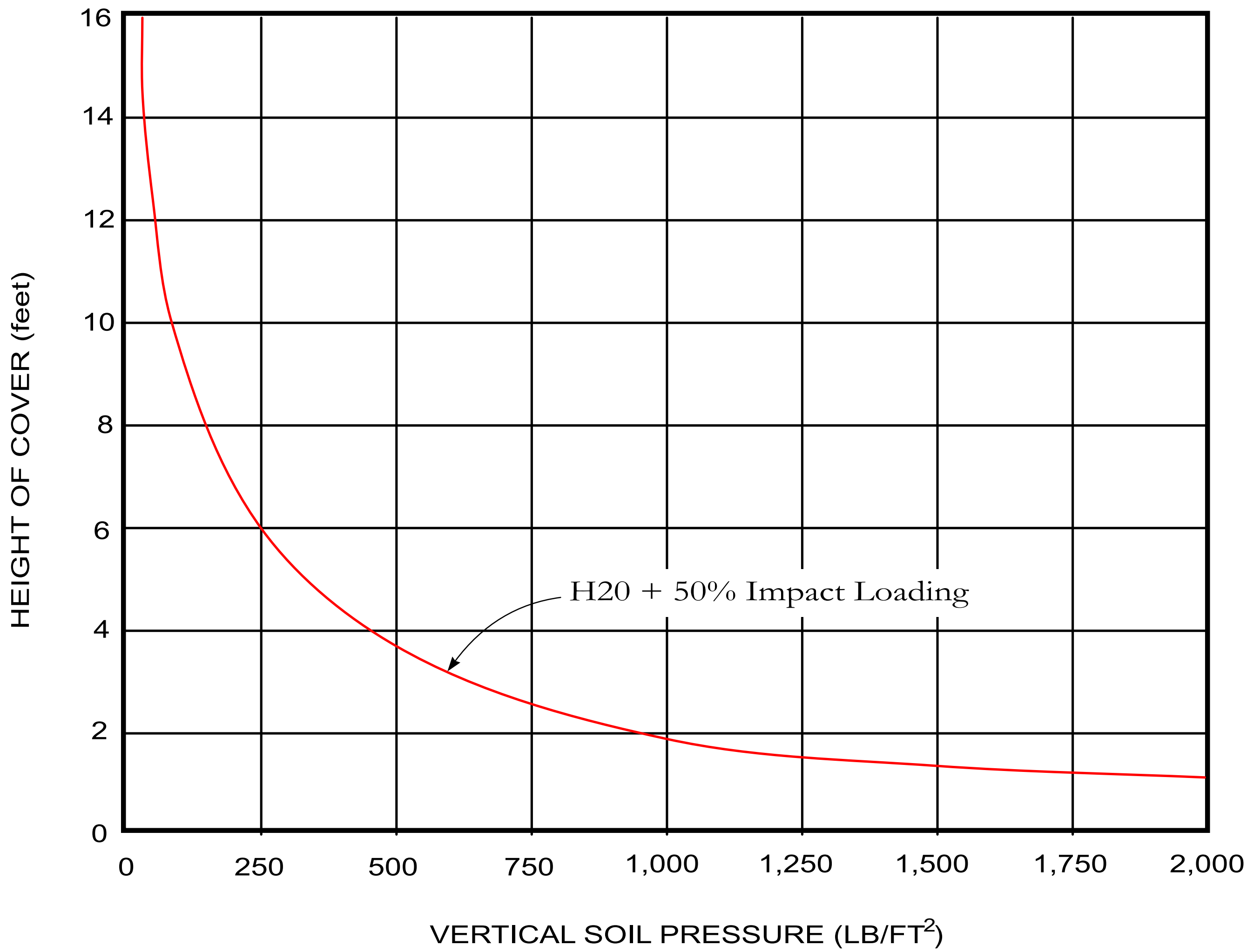
RETAINING WALL DETAILS	
Backwash Equalization Tank Paradise Irrigation District Water Works Engineers Magalia, California	Plate No. 8
BAJADA Geosciences, Inc.	Project no. 2201.0155



Curve from ASCE (1982)

MARSTON'S LOAD COEFFICIENTS

Backwash Equalization Tank Paradise Irrigation District Water Works Engineers Magalia, California	Plate No. 9
BAJADA Geosciences, Inc.	Project no. 2201.0155

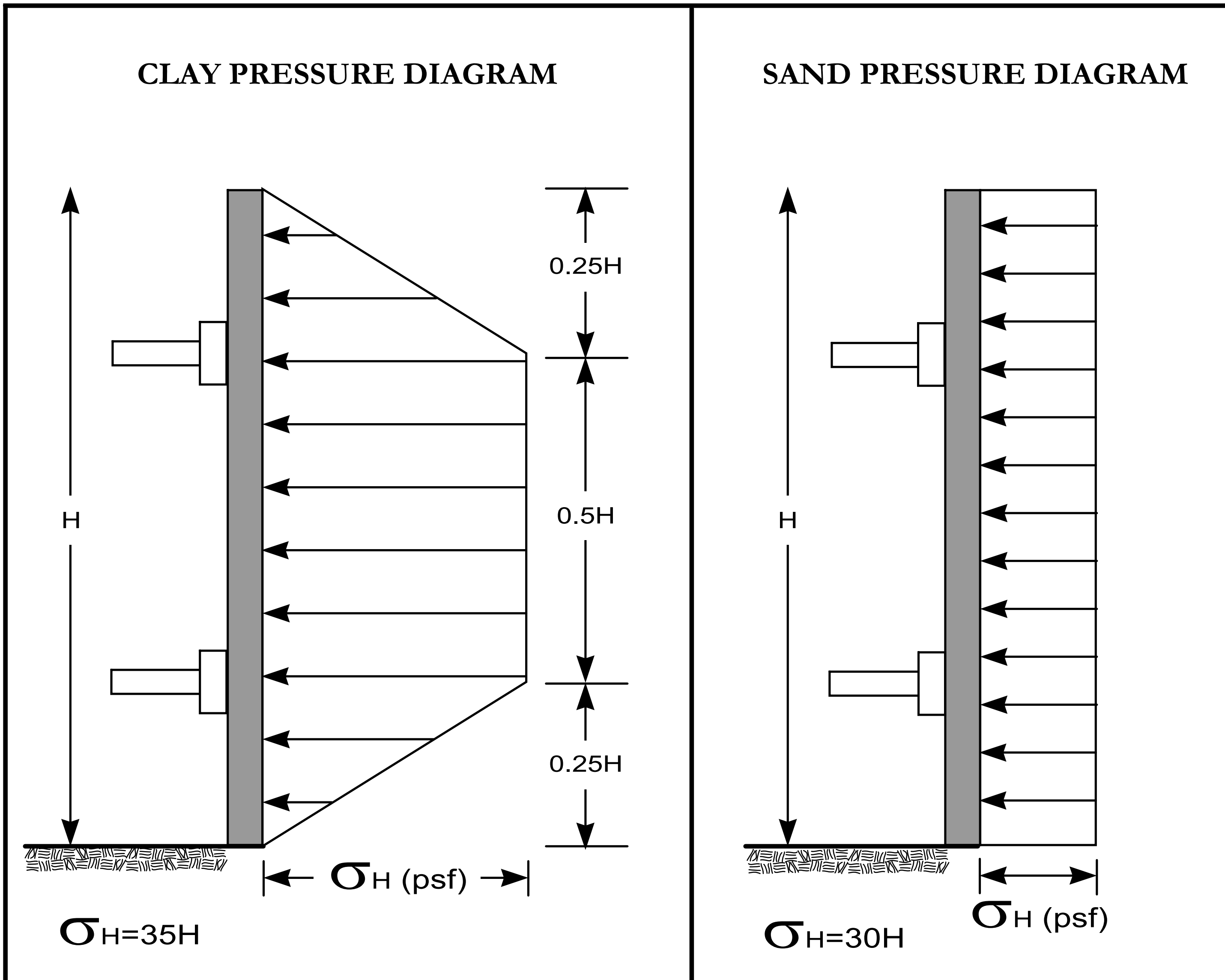


Apply vertical soil pressure to diameter of pipeline (horizontal projection
to calculate vertical load

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

Derived from Moser & Feldman (2008)

VERTICAL SOIL PRESSURES INDUCED BY LIVE LOADS	
Backwash Equalization Tank Paradise Irrigation District Water Works Engineers Magalia, California	Plate No. 10
BAJADA Geosciences, Inc.	Project no. 2201.0155



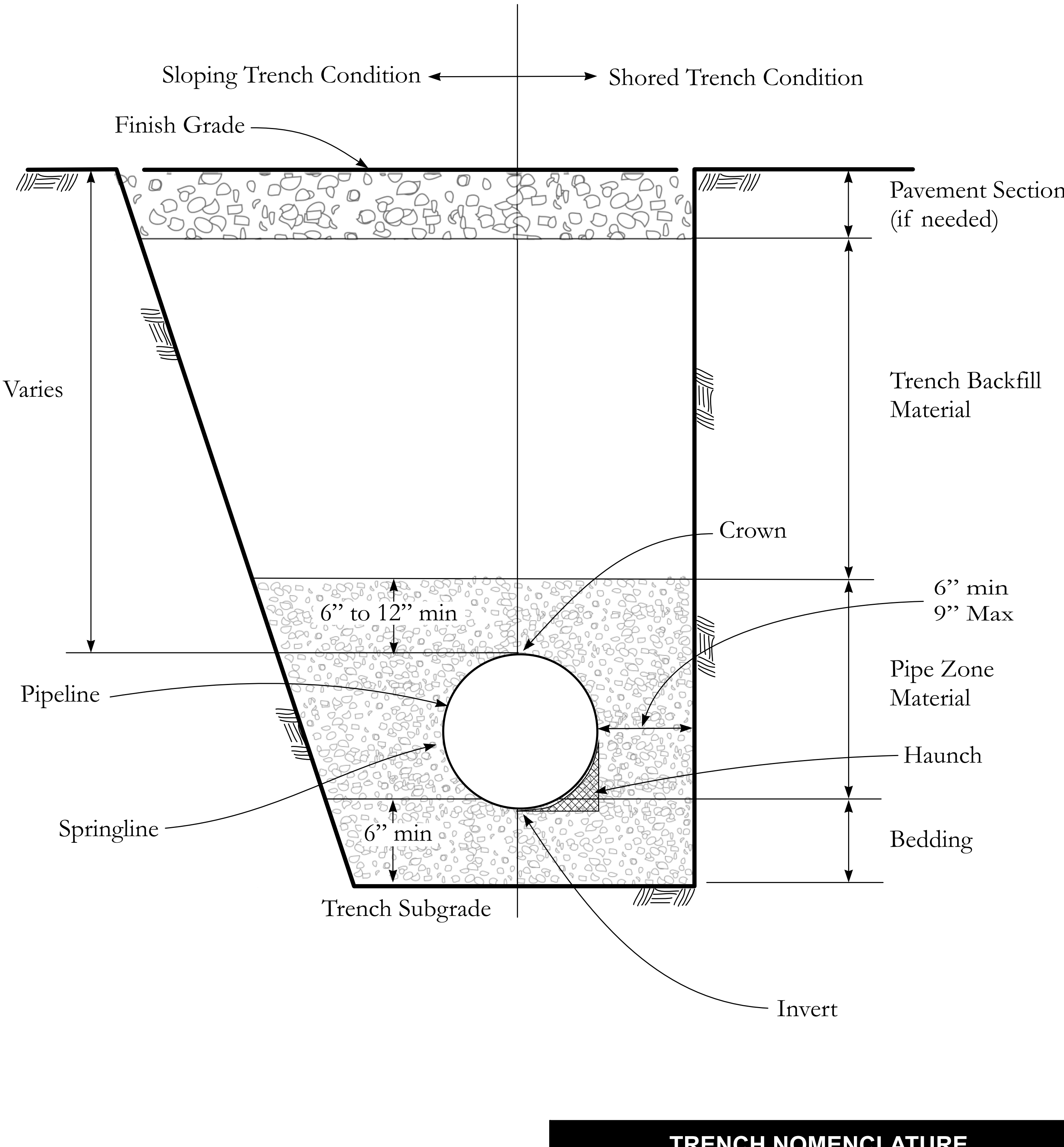
Preliminary shoring pressure diagrams are for excavations in unsaturated soils only.

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	
Backwash Equalization Tank Paradise Irrigation District Water Works Engineers Magalia, California	Plate No. 11
BAJADA Geosciences, Inc.	Project no. 2201.0155



Not to Scale

TRENCH NOMENCLATURE

Backwash Equalization Tank Paradise Irrigation District Water Works Engineers Magalia, California	Plate No. 12
BAJADA Geosciences, Inc.	Project no. 2201.0155

APPENDIX A

Subsurface Exploration

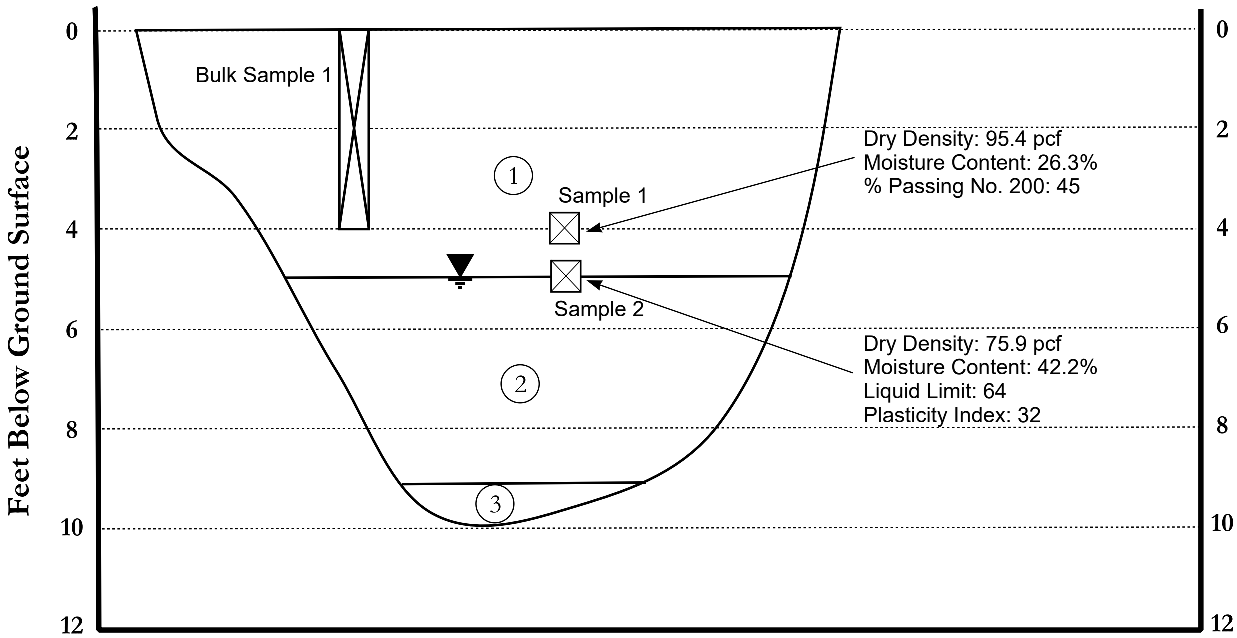
APPENDIX A
SUBSURFACE EXPLORATION

The subsurface exploration program for this study consisted of the advancement of three exploratory test pits at selected locations, as shown on Plate 3 of the report. Test pits were excavated using a Kubota KX040-4 Mini-excavator equipped with a 2-foot-wide bucket. Test pits were excavated on December 15, 2022, by TRG Excavating of Cottonwood, California.

The exploration logs describe the earth materials encountered in each test pit. The logs also show the location, exploration number, date of exploration, and the names of the logger and equipment used. A BAJADA geologist, using ASTM 2488 for visual soil classification, logged the explorations and samples. 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 test pits were backfilled with the excavated soils and tracked in place. No other densification efforts were made on those soils.

The test pit logs are presented as Plates A-1.1 and A-1.3. A legend to the test pit logs is presented as Plate A-2.1.

LOG OF TEST PIT



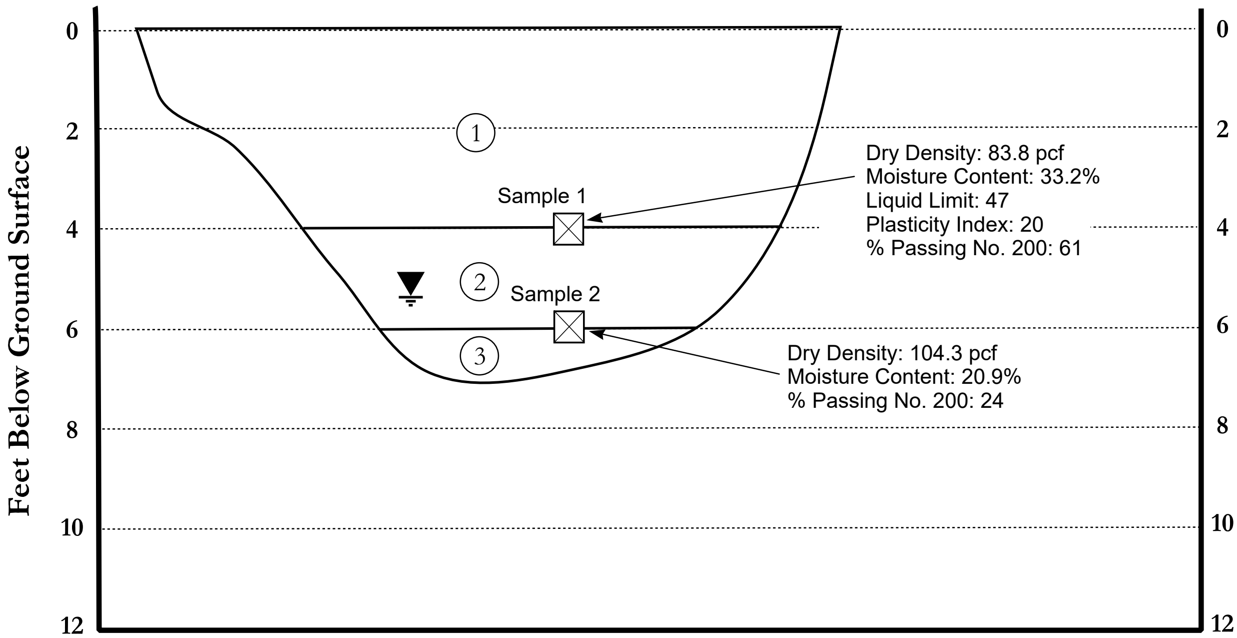
Soil Descriptions	
1	COLLUVIUM (Qc) Clayey SAND (SC), brown to dark brown with orangish brown mottling, moist abundant organics, with trace fine to medium angular to subangular gravels.
2	CLAY (CH), dark brown to blackish brown, wet, highly plastic, with trace fine to medium angular gravels, grades into soft, intensely weathered black serpentinite rock with abundant caliche.
3	IGNEOUS ROCK (RX) Serpentinzed Pyroxenite, dark greenish black to brownish black, highly weathered, highly fractured, highly friable.

Estimated Latitude: 38.8150793°
Estimated Longitude: -121.581981°
Measured using Solocator, Photos with Direction
Not considered survey quality

TEST PIT TP-1

Date Logged:	December 15, 2022	Backwash Equalization Tank Paradise Irrigation District Water Works Engineers Magalia, California	Plate No. A-1.1
Logged by:	Bryan Puleri		
Excavator:	TRG Excavation		
Excavated With:	Kubota K040-4	BAJADA Geosciences, Inc.	Project no. 2201.0155
Backfilled With:	Excavated Cuttings		
Depth to Water (ft):	5 Feet		

LOG OF TEST PIT



Soil Descriptions	
1	COLLUVIUM (Qc) Sandy CLAY (CL) brown to dark brown, with orangish brown mottling, moist, abundant organics, with trace fine to medium angular to subangular gravels.
2	CLAY (CL), dark brown to blackish brown, wet, plastic, with trace fine to medium angular gravels, grades into soft, intensly weathered black serpentinite rock with abundant caliche.
3	IGNEOUS ROCK (RX) Serpentinzed Pyroxenite, dark greenish black to brownish black, highly weathered, highly fractured, highly friable.

Estimated Latitude: 39.815113°
Estimated Longitude: -121.5819715°
Measured using Solocator, Photos with Direction
Not considered survey quality

TEST PIT TP-2

Date Logged:	December 15, 2022
Logged by:	Bryan Puleri
Excavator:	TRG Excavation
Excavated With:	Kubota K040-4
Backfilled With:	Excavated Cuttings
Depth to Water (ft):	5 Feet

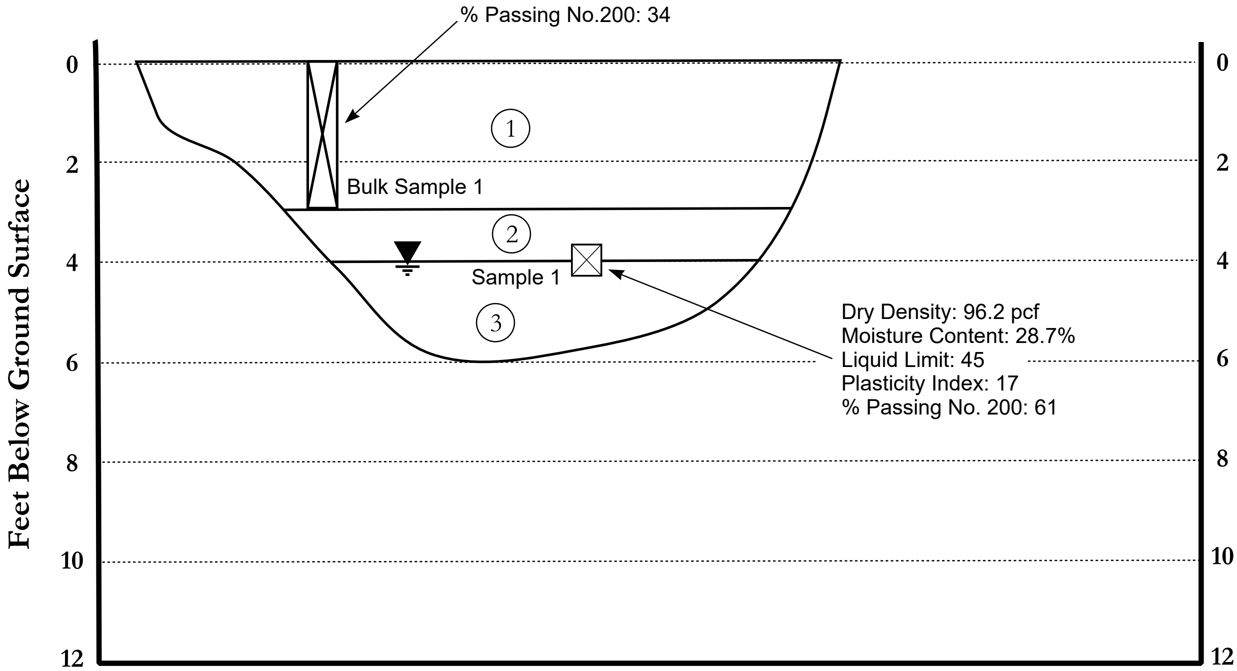
Backwash Equalization Tank
Paradise Irrigation District
Water Works Engineers
Magalia, California

BAJADA Geosciences, Inc.

Plate No. A-1.2

Project no. 2201.0155

LOG OF TEST PIT



Soil Descriptions	
1	COLLUVIUM DEPOSITS (Qc) Sandy CLAY (CL) brown to dark brown with orangish brown mottling, moist, abundant organics, with trace fine to medium angular to subangular gravels.
2	CLAY (CL), dark brown to blackish brown, wet, plastic, with trace fine to medium angular gravels, grades into soft, intensly weathered black serpentinite rock with abundant caliche.
3	IGNEOUS ROCK (RX) Serpentinzed Pyroxenite, dark greenish black to brownish black, highly weathered, highly fractured, highly friable.

Estimated Latitude: 39.815041°
Estimated Longitude: -121.582101°
Measured using Solocator, Photos with Direction
Not considered survey quality

TEST PIT TP-3

Date Logged:	December 15, 2022	Backwash Equalization Tank Paradise Irrigation District Water Works Engineers Magalia, California	Plate No. A-1.3
Logged by:	Bryan Puleri		
Excavator:	TRG Excavation		
Excavated With:	Kubota K040-4		
Backfilled With:	Excavated Cuttings	BAJADA Geosciences, Inc.	Project no. 2201.0155
Depth to Water (ft):	4 Feet		

Major Divisions			USCS Symbol	Description
COARSE-GRAINED SOILS More than 50% of sample or material is larger than the No. 200 Sieve (0.0029 inches)	GRAVELS More than 50% of the coarse fraction is retained on No. 4 sieve (0.187 inches)	GRAVELS Clean Gravels, few fines	GW	Well graded gravels and sand mixtures with little to no fines
			GP	Poorly graded gravels & gravel/sand mixtures with little to no fines
		GRAVELS With appreciable fines	GM	Silty gravels and poorly graded gravel/sand/silt mixtures
			GC	Clayey gravels and poorly graded gravel/sand/clay mixtures
	SANDS More than 50% of the coarse fraction passes the No. 4 sieve (0.187 inches)	SANDS Clean Sands, few fines	SW	Well graded sands and gravelly sands with little to no fines
			SP	Poorly graded sands and gravelly sands with little to no fines
		SANDS With appreciable fines	SM	Silty sands and poorly graded sand/gravel/silt mixtures
			SC	Clayey sands and poorly graded sand/gravel/clay mixtures
FINE-GRAINED SOILS More than 50% of sample or material is smaller than the No. 200 Sieve (0.0029 inches)	SILTS & CLAYS Liquid limit less than 50		ML	Inorganic silts with very fine sands, silty and/or clayey fine sands, clayey silts with slight plasticity
			CL	Inorganic clays with low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and clays with low plasticity
	SILTS & CLAYS Liquid limit greater than 50		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts
			CH	Inorganic clays with high plasticity, fat clays
			OH	Orgainic silts and clays with high plasticity
HIGHLY ORGANIC SOIL			PT	Peat, humus, swamp soil with high organic content

Samples

☒ Bulk or disturbed sample

■ Relatively undisturbed sample

Symbols

Groundwater

Caving

------- Contact Between Soil/Rock Layers

GENERAL NOTES

Dual symbols (such as ML/CL or SM/SC) are used to indicate borderline classifications.

In general, USCS designations shown on the logs were evaluated using visual methods. Actual designations (based on laboratory tests) may vary.

Logs represent general soil conditions observed on the date and locations indicated. No warranty is provided regarding soil continuity between locations.

Lines separating soil strata on logs are approximate. Actual transitions may be gradual and vary with depth.

TEST PIT LEGEND TO TERMS & SYMBOLS

Backwash Equalization Tank
Paradise Irrigation District
Water Works Engineers
Magalia, California

Plate No.

A-2

BAJADA Geosciences, Inc.

Project no.

2201.0155

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.

Grain Size Distribution

Grain size distribution was determined for five 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.*

Grain Size Distribution

Plasticity Index tests were performed on three selected soil samples in accordance with standard test method ASTM D4318. Results of the tests are presented on the attached plate labeled *Plasticity Chart and Data.*

Unconfined Compression Tests

Uniaxial unconfined compression tests were performed on two rock samples taken from the project site. The rocks were cored then tested in accordance with standard test method ASTM D7012, Method C. Results of those tests are presented on the attached plate labeled *Rock Core Compressive Strength Data.*

Soil Chemistry Tests for Corrosion

Two selected soil samples were tested to evaluate sulfate and chloride contents, pH, and resistivity. The tests were performed in accordance with standard test methods ASTM G51 and G75, and California Test Method 417 and 422. Test results are presented on the attached plates labeled *Corrosivity Test Summary.*

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: BAJADA Geosciences, Inc.
28301 Inwood Road
Shingletown, CA 96088

Project: Paradise Irrigation District Backwash Tank Project
#2201.0155
Magalia, California

Client No.: 3237-093
Figure No.: 0300-001
Date: 01/16/2023
Page No.: 1 of 1
Submitted by: Client

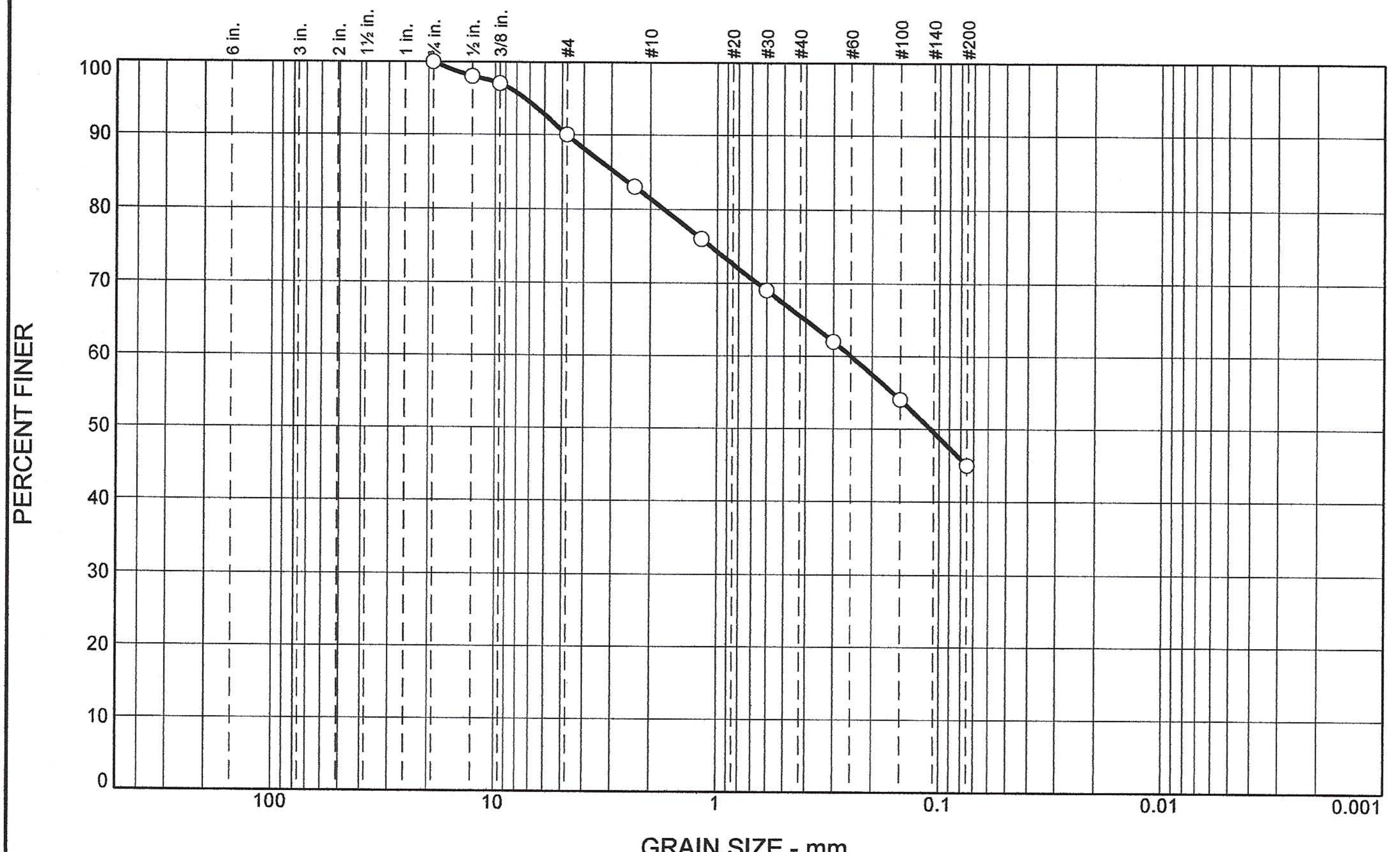
Date Submitted: 01/11/2023

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
TP-1, 1 @ 4.0’	Brown Silty Sand (visual)	95.4	26.3	---	---	---
TP-1, 2 @ 5.5’	Dark Grayish Brown Sandy Silt with Gravel (visual)	75.9	42.2	64	32	32
TP-2, 1 @ 4.0’	Dark Grayish Brown Sandy Clay (visual)	83.8	33.2	47	27	20
TP-2, 2 @ 6.0’	Olive Gray Clayey Sand (visual)	104.3	20.9	---	---	---
TP-3, 1 @ 4.0’	Olive Gray Sandy Silt with Gravel (visual)	96.2	28.7	45	28	17

Tested by John Hubbard.
The samples were tested according to the referenced standard test procedures and relate only to the items inspected or tested.
Results are not transferable and shall not be reproduced, except in full, without written permission from MTI.

PARTICLE SIZE DISTRIBUTION REPORT



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0	19	15	21	45	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100		
1/2"	98		
3/8"	97		
#4	90		
#8	83		
#16	76		
#30	69		
#50	62		
#100	54		
#200	45		

* (no specification provided)

Material Description

Brown Silty Sand (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 4.7500 D_{85}= 2.9096 D_{60}= 0.2495
D_{50}= 0.1096 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Material tested in accordance with ASTM D6913.

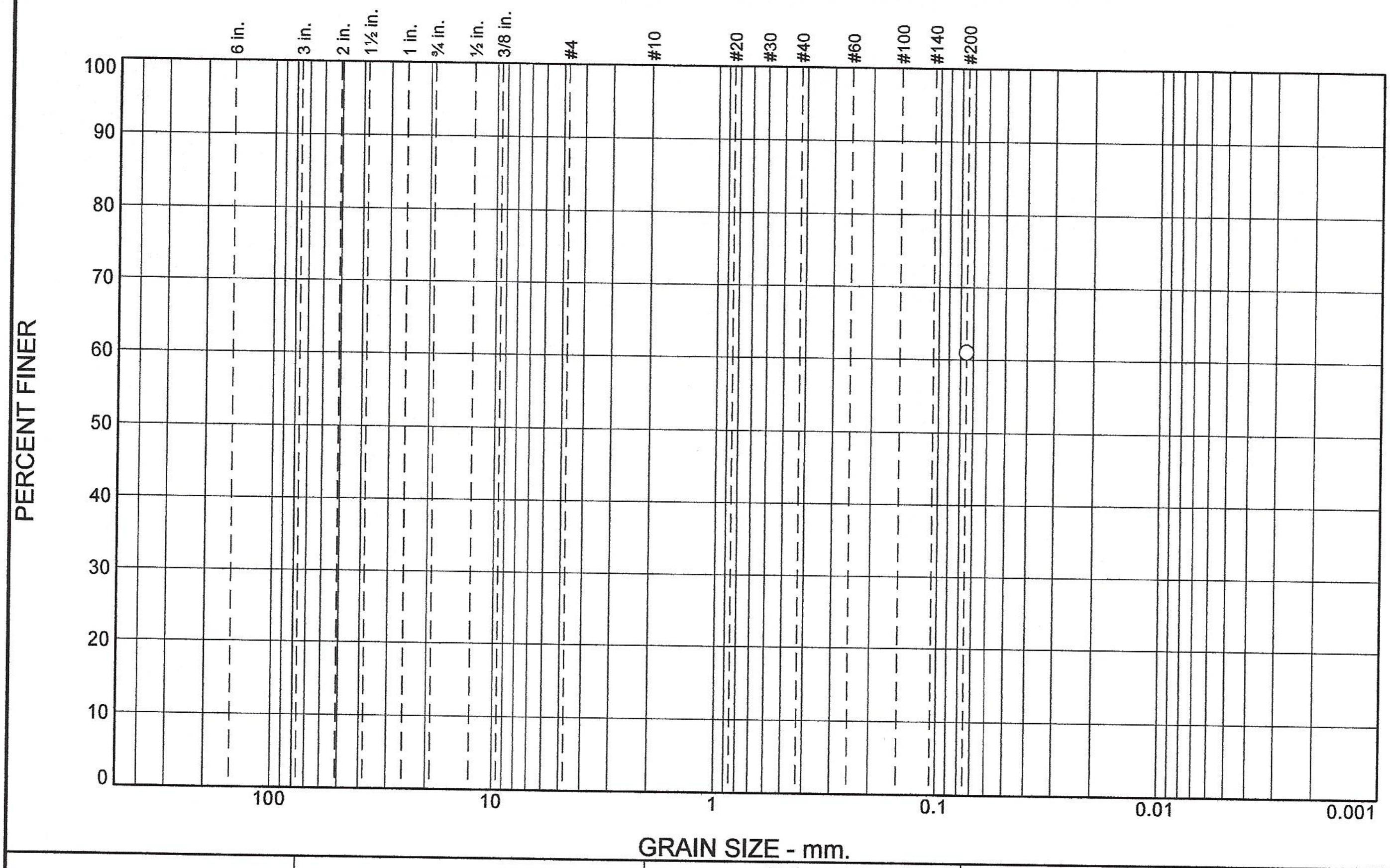
Location: TP-1, 1
Sample Number: 2 **Depth:** 4.0' **Date:** 01/16/2023

MATERIALS TESTING INCORPORATED M T I

Client: BAJADA Geosciences, Inc.
Project: Paradise Irrigation District Backwash Tank Project #2201.0155
Magalia, California
Project No: 3237-093 **Figure** 0300-002

Tested By: John Hubbard

PARTICLE SIZE DISTRIBUTION REPORT



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
				61	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	61		

* (no specification provided)

Material Description

Dark Grayish Brown Sandy Clay (visual)

Atterberg Limits

PL= 27 LL= 47 PI= 20

Coefficients

D_{90}= D_{85}= 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 D1140.

Location: TP-2, 1
Sample Number: 4 **Depth:** 4.0'

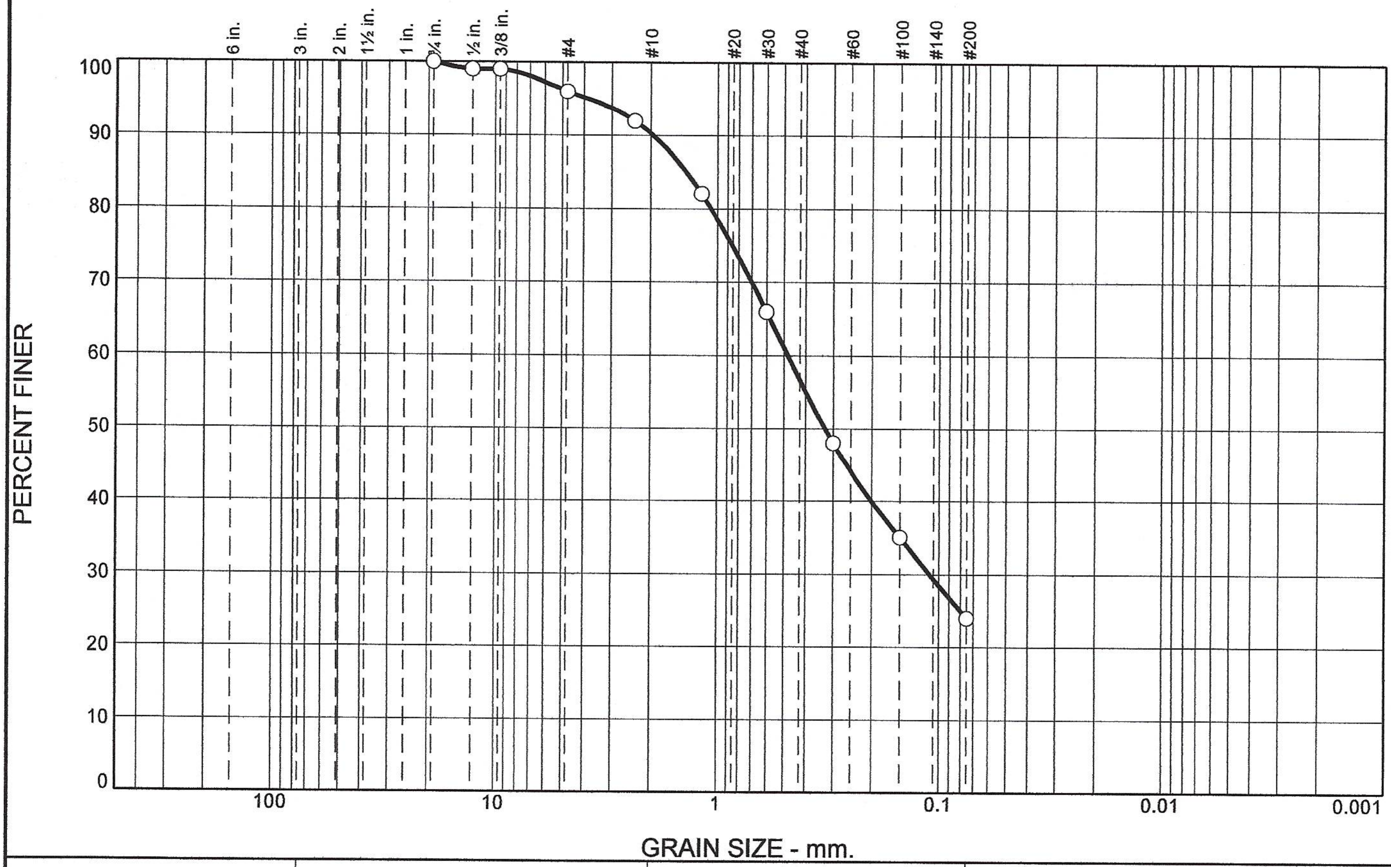
Date: 01/16/2023

MATERIALS TESTING INCORPORATED M T I

Client: BAJADA Geosciences, Inc.
Project: Paradise Irrigation District Backwash Tank Project #2201.0155
Magalia, California
Project No: 3237-093 **Figure** 0300-003

Tested By: John Hubbard

PARTICLE SIZE DISTRIBUTION REPORT



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0	10	33	33	24	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100		
1/2"	99		
3/8"	99		
#4	96		
#8	92		
#16	82		
#30	66		
#50	48		
#100	35		
#200	24		

* (no specification provided)

Material Description

Olive Gray Clayey Sand (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 1.9501 D_{85}= 1.3875 D_{60}= 0.4802

D_{50}= 0.3267 D_{30}= 0.1099 D_{15}=

D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

Material tested in accordance with ASTM D6913.

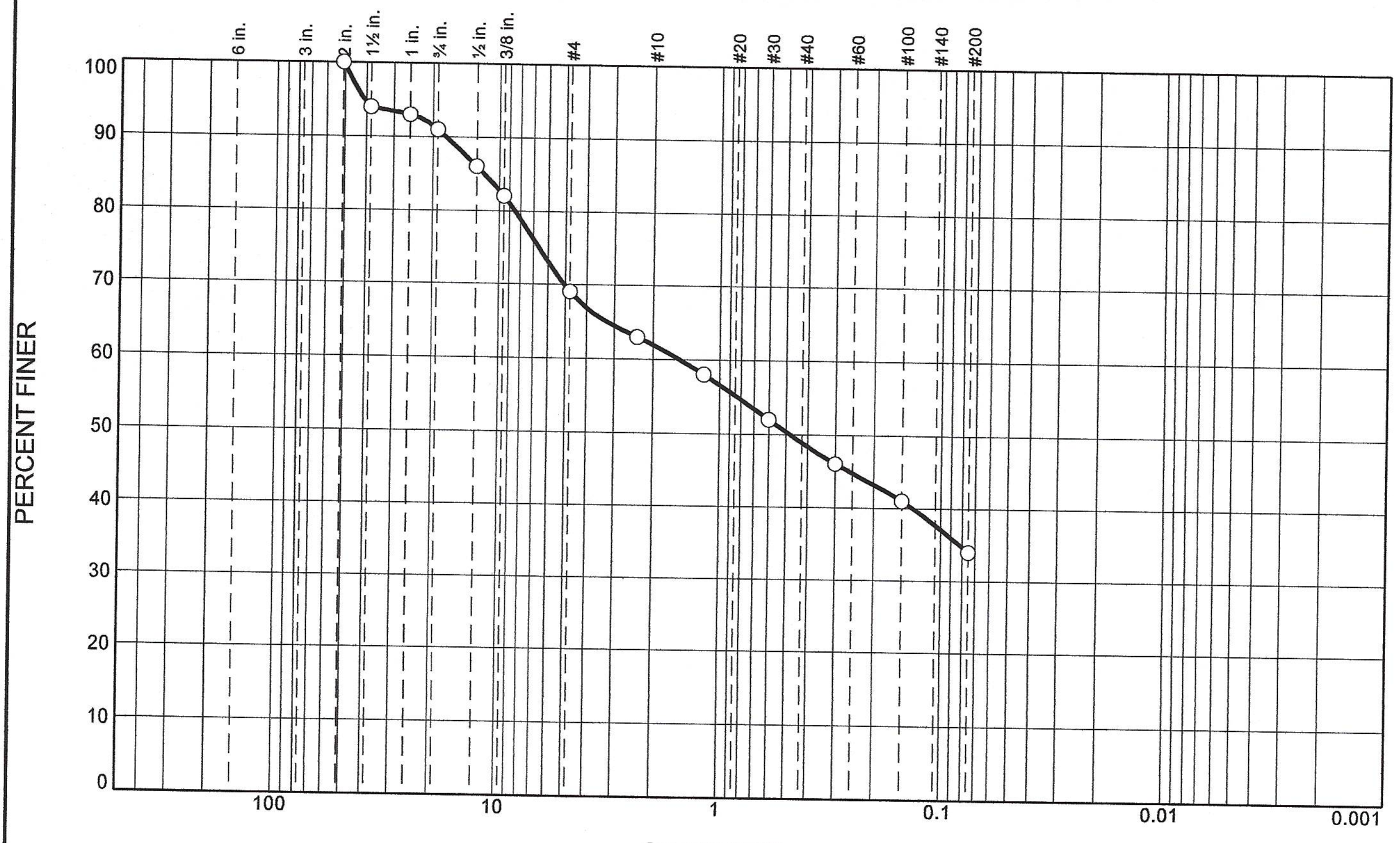
Location: TP-2, 2
Sample Number: 5 **Depth:** 6.0'

Date: 01/16/2023

Client: BAJADA Geosciences, Inc.
Project: Paradise Irrigation District Backwash Tank Project #2201.0155
Magalia, California
Project No: 3237-093 **Figure** 0300-004

Tested By: John Hubbard

PARTICLE SIZE DISTRIBUTION REPORT



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
0	38	13	15	34	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100		
1 1/2"	94		
1"	93		
3/4"	91		
1/2"	86		
3/8"	82		
#4	69		
#8	63		
#16	58		
#30	52		
#50	46		
#100	41		
#200	34		

* (no specification provided)

Material Description

Brown Clayey Sand with Gravel (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 17.4088 D_{85}= 11.7566 D_{60}= 1.5199
D_{50}= 0.4823 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

Material tested in accordance with ASTM D6913.

Location: TP-3, B1
Sample Number: 6 **Depth:** 0.0'-3.5'

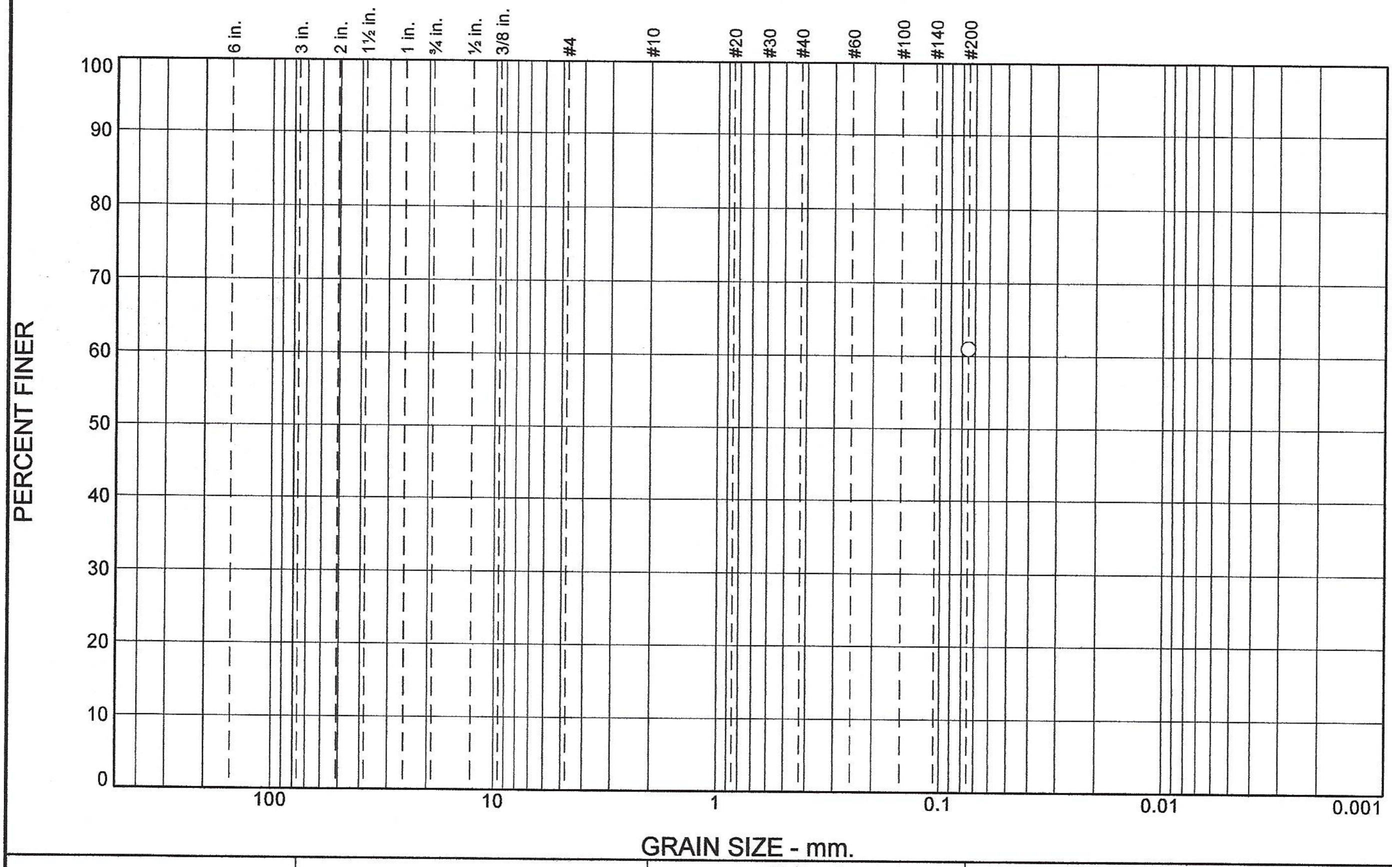
Date: 01/19/2023

MATERIALS TESTING INCORPORATED M T I

Client: BAJADA Geosciences, Inc.
Project: Paradise Irrigation District Backwash Tank Project #2201.0155
Magalia, California
Project No: 3237-093 **Figure** 0300-005

Tested By: Travis Fiscus

PARTICLE SIZE DISTRIBUTION REPORT



% +3"	% Gravel	% Sand		% Fines	
		Coarse	Fine	Silt	Clay
				61	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	61		

* (no specification provided)

Material Description

Olive Gray Sandy Silt with Gravel (visual)

Atterberg Limits

PL= 28 LL= 45 PI= 17

Coefficients

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

Classification

USCS= ML AASHTO=

Remarks

Material tested in accordance with ASTM D1140.

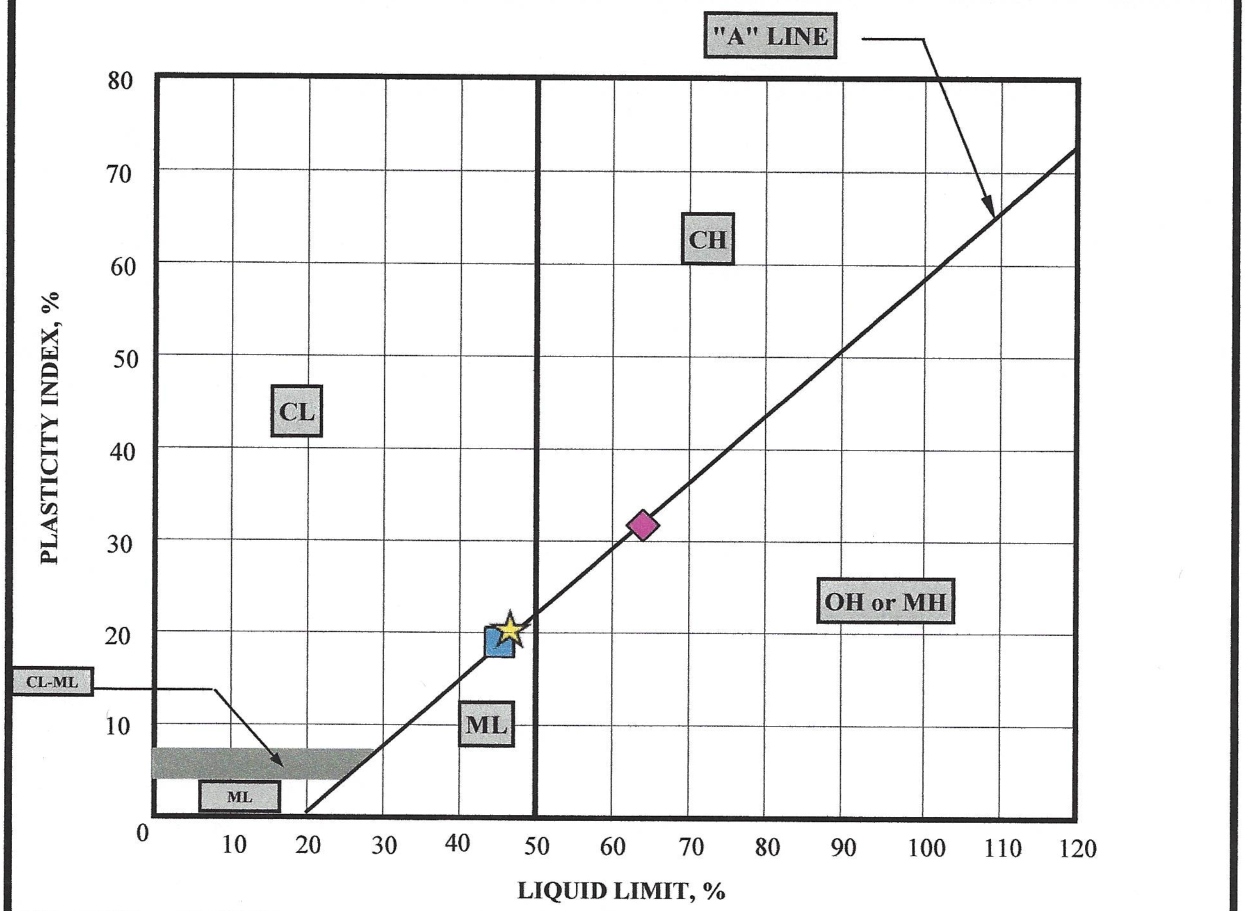
Location: TP-3, 1
Sample Number: 7 **Depth:** 4.0'

Date: 01/16/2023

MATERIALS TESTING INCORPORATED M T I

Client: BAJADA Geosciences, Inc.
Project: Paradise Irrigation District Backwash Tank Project #2201.0155
Magalia, California
Project No: 3237-093 **Figure** 0300-006

Tested By: John Hubbard



KEY SYMBOL	SAMPLE NUMBER	DEPTH	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, LL	PLASTIC LIMIT, PL	PLASTICITY INDEX, PI	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
◆	TP-1, 2	5.5'	N/A	64	32	32	N/A	MH
☆	TP-2 , 1	4.0'	N/A	47	27	20	N/A	CL
■	TP-3, 1	4.0'	N/A	45	28	17	N/A	ML

Note: Atterberg Limits tested in accordance with ASTM D4318.

MATERIALS TESTING INCORPORATED M T I

Materials Testing, Inc.

PLASTICITY CHART AND DATA

Paradise Irrigation District Backwash Tank

Project #2201.0155- Magalia California

Project No:	Date:	Figure No:
3237-093	1/19/2023	0300-007

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:	BAJADA Geosciences, Inc. 28301 Inwood Road Shingletown, CA 96088	Date:	01/23/23
		Client No:	3237-093
		Report No:	0100-008
Project:	Paradise Irrigation District Backwash Tank Project #2201.0155	Page No:	1 of 1
Location:	Magalia, California	Sampled By:	Client

ROCK CORE COMPRESSIVE STRENGTH DATA
(ASTM D7012 Method C)

Identification	Rock 2	Rock 3
Material		
Date Cored	01/17/23	01/17/23
End Preparation Date	01/17/23	01/17/23
Date Tested	01/23/23	01/23/23
Bagged Age in Days	6	6
Average Diameter, in	1.94	1.94
Cross Sect. Area, in^2	2.96	2.96
As Received Length, in	3.90	2.90
Trimmed Length, in	---	---
L/D Factor	2.01	1.49*
Maximum Load, lbs.	27,830	23,520
Compr. Strength, psi	**9,400**	**7,950**
Fracture Pattern, Type	Columnar with Vertical Cracking Through Ends	Columnar with Vertical Cracking Through Ends
Testing Technician	Travis Fiscus	Travis Fiscus

Notes:
Specimens prepared and tested in accordance with ASTM D4543.
*-Does not meet length to diameter requirements.

Tested by Travis Fiscus.

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

Date Reported 01/20/2023
Date Submitted 01/16/2023

To: Andy King
K.C. Engineerig
8798 Airport Rd.
Redding, CA 96002

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

The reported analysis was requested for the following location:
Location : 3237 BAJADA Site ID : #1 TP1,B1@0-4.
Thank you for your business.

* For future reference to this analysis please use SUN # 88859-184627.

EVALUATION FOR SOIL CORROSION

Soil pH	6.27		
Minimum Resistivity	2.95 ohm-cm (x1000)		
Chloride	7.5 ppm	0.00075	%
Sulfate-SO4	24.7ppm	0.00247	%

METHODS
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate-SO4 ASTM C1580, Chloride CA DOT Test #422m

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

Date Reported 01/20/2023
Date Submitted 01/16/2023

To: Andy King
K.C. Engineerig
8798 Airport Rd.
Redding, CA 96002

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

The reported analysis was requested for the following location:
Location : 3237 BAJADA Site ID : #6TP-3,B1@0-3.5.
Thank you for your business.

* For future reference to this analysis please use SUN # 88859-184628.

EVALUATION FOR SOIL CORROSION

Soil pH	6.09	
Minimum Resistivity	2.95 ohm-cm (x1000)	
Chloride	6.3 ppm	0.00063 %
Sulfate-SO4	3.9ppm	0.00039 %

METHODS
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate-SO4 ASTM C1580, Chloride CA DOT Test #422m

ASBESTOS TEM LABORATORIES, INC.

CARB Method 435
Polarized Light Microscopy
Analytical Report

Laboratory Job # 96-07605

3431 Ettie St.
Oakland, CA 94608
(510) 704-8930
FAX (510) 704-8429

ASBESTOS TEM LABORATORIES, INC

CA ELAP
Lab No. 1866

NVLAP® TESTING
NVLAP Lab Code: 101891-0
Oakland, CA

Jan/26/2023

Andy King
Materials Testing, Inc.
8798 Airport Road
Redding, CA 96002

RE: LABORATORY JOB # 96-07605
Polarized light microscopy analytical results for 3 bulk sample(s).
Job Site: 2201.0155
Job No.: Paradise Irrigation District Backwash Tank

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. ---

3431 Ettie St. • Oakland, CA 94608 • PH. (510) 704-8930 • FAX (510) 704-8429
With Branch Offices Located At: 1350 FREEPORT BLVD. UNIT 104, SPARKS, NV 89431

POLARIZED LIGHT MICROSCOPY
CARB 435 ANALYTICAL REPORT

Contact: Andy King
Address: Materials Testing, Inc.
8798 Airport Road
Redding, CA 96002

Samples Submittec 3
Samples Analyzed: 3
Job Site / No. Paradise Irrigation District Backwash Tank
2201.0155

Report No. 382159
Date Submitted: Jan-20-23
Date Reported: Jan-26-23

SAMPLE ID	POINTS COUNTED	ASBESTOS %	ASBESTOS TYPE	LOCATION / DESCRIPTION
TP-1, B1 @ 0-4.0' Lab ID # 96-07605-001	0 - Total Points	<0.25%	No Asbestos Detected	Exception #1 - No asbestos in 10 FOV on 3 slides
TP-3, B1 @ 0-3.5' Lab ID # 96-07605-002	0 - Total Points	<0.25%	No Asbestos Detected	Exception #1 - No asbestos in 10 FOV on 3 slides
R3 Lab ID # 96-07605-003	0 - Total Points	<0.25%	No Asbestos Detected	Exception #1 - No asbestos in 10 FOV on 3 slides
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. McBrien

Analyst Jo Ann Huerta

Asbestos TEM Laboratories, Inc.
3431 Ettie St., Oakland, CA 94608 PH. (510) 704-8930

ATEM LABORATORIES CHAIN OF CUSTODY

CALIFORNIA: 3431 Ettie Street Oakland, CA 94608 Phone (510) 704-8930 Fax (510) 704-8429
NEVADA: 1350 Freeport Blvd. #104, Sparks, NV 89431 Phone (775) 359-3377 Fax (775) 359-2798
You may also email this chain of custody to asbestostemlabs.ca@gmail.com

* denotes required field

Company: Materials Testing, Inc.	Contact:* Andy King	Phone: * 530-222-1116	Email: * aking@kcengr.com
Address: * 8798 Airport Road	City: * Redding	State: * Zip: 96002	Email:
Job Site:* Paradise Irrigation District Backwash Tank	Job #: 2201.0155	PO #:	Email:

Reporting *	■ Email ☐ Phone ☐ Fax ☐ Mail ☐ Pickup — Billing ■ Email ☐ Fax ☐ Mail ☐ Pre-Paid — Billing Email :
Results Due:*	☐ 2 HR ☐ 4 HR ☐ 6 HR ☐ 8 HR ☐ 24 HR ☐ 48 HR ☐ 3 DAY ■ 5 DAY ☐ 10 DAY ☐ Hold Samples (Until ______) ☐ After Hours: **
Asbestos Air	☐ PCM NIOSH 7400 ☐ A ☐ B ☐ TEM AHERA ☐ TEM CARB Mod. AHERA ☐ TEM EPA Yamate Level II ☐ TEM NIOSH 7402 ☐ ISO 10312 ☐ ISO 13794 ☐ Sensitivity ______
Asbestos Bulk	☐ PLM Standard (EPA 600/R-93-1) ☐ PLM 400 Point Count ☐ PLM 1000 PC ☐ PLM 400 PC Gravimetric Reduction ☐ PLM 1000 PC Grav. Red. ☐ TEM EPA Qualitative ☐ TEM EPA Quantitative
Asbestos Soils	☐ CARB 435 Prep Only — CARB 435 PLM ■ 400 PC ☐ 800 PC ☐ 1000 PC ☐ 1200 PC ☐ EPA Soil Screening Qualitative ☐ TEM-NOA EPA/CARB Quantitative ☐ Erionite
Asbestos Dust	☐ ASTM D-5755 Fiber Count ☐ ASTM D-5756 Wt. % ☐ ASTM D-5756 Mass ☐ ASTM D-6480 Dust Wipe ☐ Total Particulates (Gravimetric)
Asbestos Water	☐ 100.2 Potable Drinking Water ☐ 100.1 Non Potable Water *note that 100.2 will be used for all water samples unless otherwise requested*
Lead/Silica	☐ Lead Paint Chips EPA-SW-846 7000B ☐ Lead Dust Wipe EPA-SW-846 7000B ☐ Lead Air NIOSH 7082 ☐ Lead Soil EPA-SW-846 7000B — Crystalline Silica Air (NIOSH 7500) ☐ Single Species ☐ All Species — Crystalline Silica in Bulk (NIOSH 7500) ☐ Single Species ☐ All Species — Respirable Crystalline Silica in Bulk (NIOSH 7500) ☐ Single Species ☐ All Species
Custom/Other	☐ Custom Analysis ** ☐ TEM Chatfield (Semi-Quant) ☐ NIOSH 0500 ☐ NIOSH 0600 ☐ TTLC ☐ STLC ☐ TCLP
Special Instruct.	☐ Composite ☐ Prep Only ☐ 8 Hour TWA — 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	Hold Sample	Description *
TP-1, B1 @ 0-4.0'	Bulk	01/04/2023								☐	
TP-3, B1 @ 0-3.5'	Bulk	01/04/2023								☐	
R3	Rock	01/04/2023								☐	
										☐	
										☐	
										☐	
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										☐	
										☐	
										☐	

Submitted By * Jim Bianchin	Received By MTI
Date/Time Submitted * 01/11/2023	Date/Time Received 01/11/2023
Submitted By	Received By
Date/Time Submitted	Date/Time Received

** *For any special instructions, RUSH results or Custom Analysis, you must clarify these specifications AND, of more importance, contact us here at ATEM ahead of time to manage scheduling to meet your requests. **This includes dropping off samples for rush, same day analysis.** Drop off and processing of samples after hours cannot be accommodated without proper notification from you, and confirmation by ATEM staff. 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 Surveys

APPENDIX C
GEOPHYSICAL REFRACTION SURVEY

A geophysical refraction was performed at the site on December 19, 2022. The survey was performed by Redpath Geophysics of Murphys , California. The attached report presents the methods and results of that survey.

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

Mr. James A. Bianchin 3 January 2023
Bajada Geosciences, Inc
1300 Market St., Suite 201
Redding, CA 96001
via email: jim.bianchin@bajadageo.com

Dear Mr. Bianchin,

This letter presents the results of seismic refraction surveys that were conducted at the Paradise Irrigation District in Paradise, CA. The lines were surveyed on 19 December 2022 with the assistance of Bryan Puleri of Bajada Geosciences. The general intent of the surveys was to provide information that would assist in assessing the subsurface conditions for an expansion of the facilities.

Two short, 12-channel lines were surveyed, the locations of which are shown on the attached Google Earth view. Because of limited space, the first line (SL-1) used a 7-foot geophone interval, instead of the normal 10-ft spacing, for a total length of 77 feet. Line 2 (SL-2) was correspondingly limited to a length of 55 ft. A 16-lb sledgehammer striking a one-inch-thick slab of high-density polyethylene on the ground was used as the energy source. Signals from 5 hammer-points were recorded along each line and, where possible, 20 ft off the end of each line.

All data were recorded on a Geometrics model R24 Strataview™ digital seismograph configured to record 12 channels, each of which consisted of 1024 samples at intervals of 60 microseconds, for a total recording time of 62 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 5 hammer blows were stacked at a given hammer point. Data quality was generally good on SL-1, but somewhat questionable on SL-2.

The seismic records were viewed on a video monitor as they were acquired and paper copies were printed on its internal printer for retention as field copies. The data are stored on the internal hard drive on the R24 and ultimately copied to a 3-1/2-inch diskette in SEG-2 binary format; the data are then transferred from the diskettes to the analysis programs.

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 time vs. distance plot of the data suggested that a two-layer model was a plausible approximation of the subsurface velocity structure, and was used as the initial model for the tomographic inversion of the refraction data. The results of the SL-1 survey are presented on the attached profiles in the form of a layered model, designated as the time-term inversion, and a velocity cross-section based on a tomographic inversion of the time-distance data. The ground elevation was arbitrarily set at 100 ft on the cross sections at 0+77 on SL-1 and 0+55 on SL-2. The SeisImager software has the capability of calculating and displaying the ray paths from the sources (hammer points) to receivers (geophones). This feature was used to trim the depth of the color cross-section of SL-1 to be just slightly below the computed maximum depth of penetration of the seismic signals.

I have also included the time vs. distance plot showing observed travel times and those calculated on the basis of the tomographic model for SL-1. As can be seen, the agreement between the times is reasonably good along this line, but it was problematic on SL-2. The first arrivals along SL-2 were somewhat ambiguous and uncertain for signals from the interior hammer points, which was manifested by a large average difference between observed and calculated travel times for the tomographic inversion. I attribute this to a heterogeneous first layer, probably with a mix of soil, boulders, concrete, and nearby pavement along the line, which affected the travel paths. The quality of signals from the off-end hammer blows appeared to be relatively better, and I used the standard single-layer, critical-distance formula to compute depths at each end of SL-2; the result is shown by the red-dotted interface on the SL-2 profile. Also shown is the comparison of observed and calculated travel times for SL-2 from the initial Plotrefa inversions, which suggested that computer-generated solution was too questionable to use.

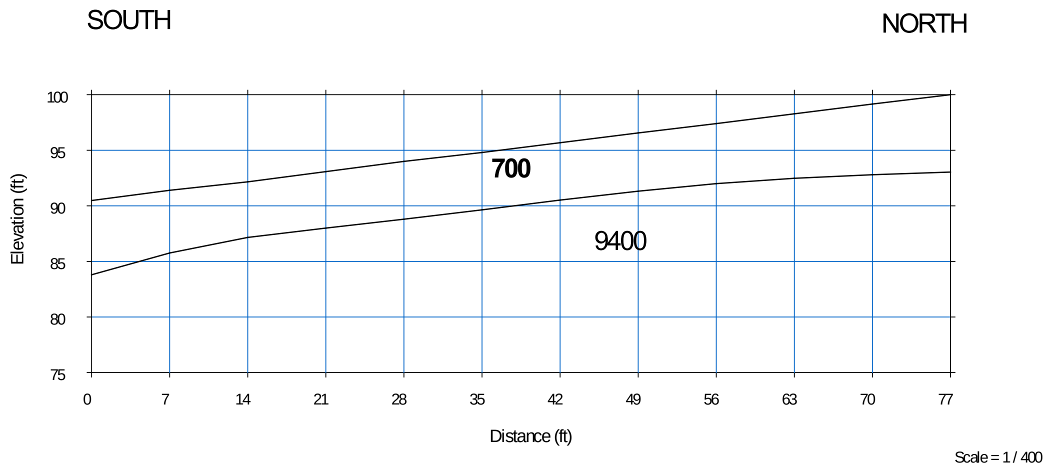
Please do not hesitate to contact me if you have questions about any aspect of these surveys or the results.

Sincerely,

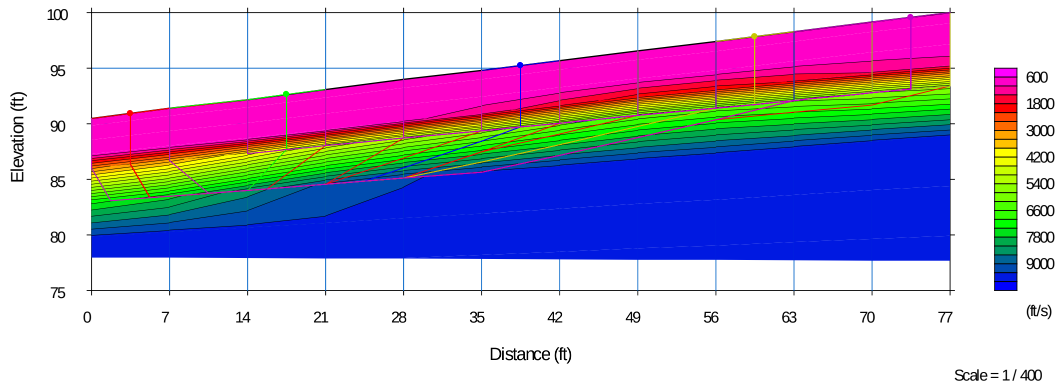
Bruce B. Redpath

California Registered Geophysicist GP-347

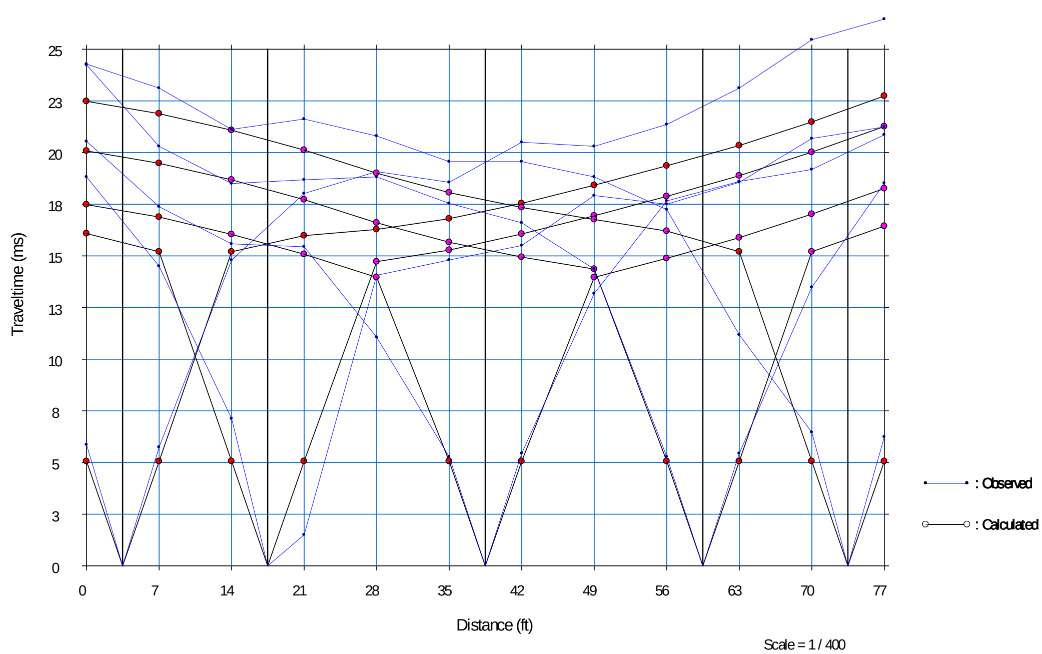

Paradise Irrigation District
Legend
0
55
77
39.815154N, 121.581626W
SL-2
SL-1
0
21.583°
Google Earth
200 ft
N



SL-1 Time-Term Inversioin of Time vs. Distance Data

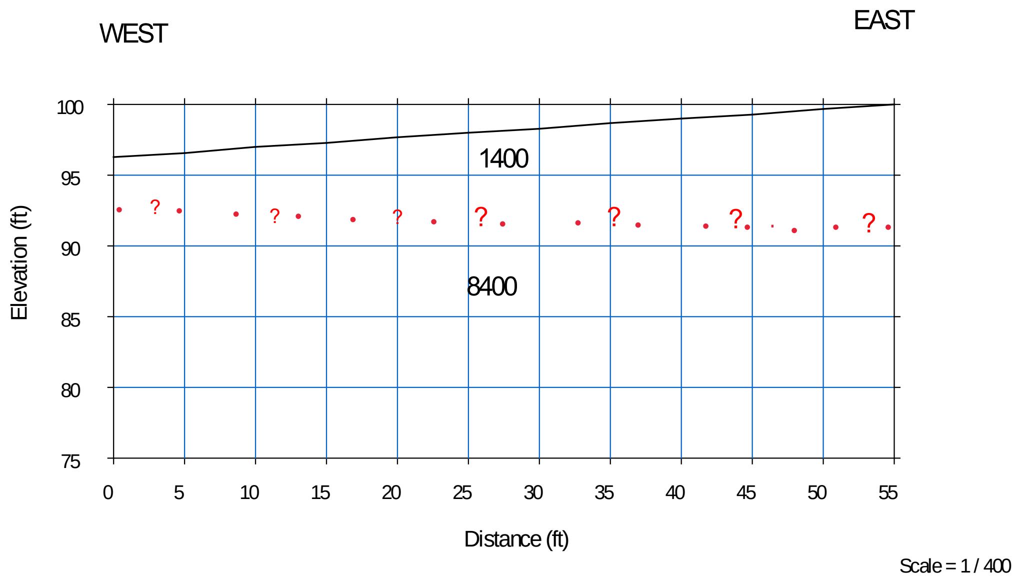


SL-1 Tomographic Inversion of Time vs. Distance Data

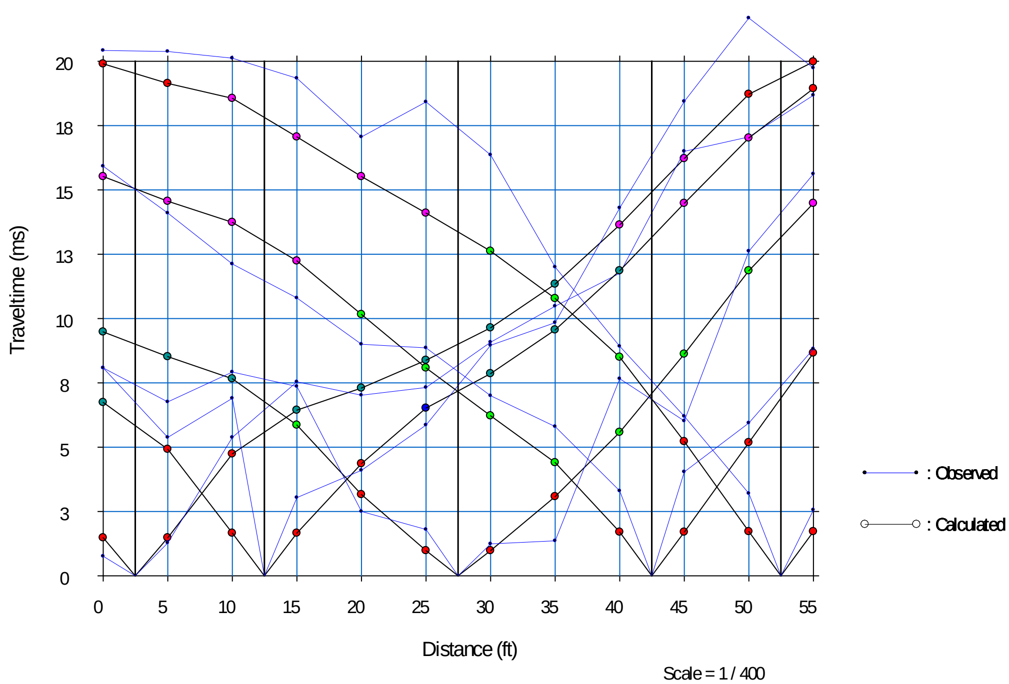


SL-1 Comparison of Observed and Calculated Travel Times

Seismic Line 1 - Paradise Irrigation District - December 2022



SL-2 Interpretation Based on Critical Distance and Off-end Data



SL-2 Observed vs. Times Calculated from 'Suspect' Tomographic Inversion

Seismic Line 2 - Paradise Irrigation District - December 2022