GEOTECHNICAL EXPLORATION
DENTRO DE LOMAS PUMP STATION PROJECT
APN 127-581-06-00
RAINBOW MUNICIPAL WATER DISTRICT (RMWD)
BONSALL AREA, SAN DIEGO COUNTY
CALIFORNIA

Prepared For **RAINBOW MUNICIPAL WATER DISTRICT**
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Project Number 12600.004

September 2, 2022

Leighton 60th Anniversary

Leighton Consulting, Inc.
A Leighton Group Company

September 2, 2022
Project No. 12600.004

Rainbow Municipal Water District
3707 Old Hwy 395
Fallbrook, CA 92589-9017

Attention: Mr. Malik Tamimi, P.E.

Subject: Geotechnical Exploration
Dentro De Lomas Pump Station Project,
APN 127-581-06-00, NEC Dentro de Lomas and Vista del Mar Road
Rainbow Municipal Water District (RMWD)
Bonsall Area, San Diego County, California

In accordance with your authorization, we have performed a geotechnical exploration for the subject project. This report presents our findings and provides our geotechnical recommendations for the design and construction of the proposed improvements. Based on the results of this exploration, the site is generally underlain by granitic bedrock. From a geotechnical perspective, the constructability of the proposed improvements is considered feasible provided the recommendations included in this report are implemented during design and construction phases.

The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to call our office.

Respectfully submitted,

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1.0 INTRODUCTION

1.1 Site Description

The proposed Dentro de Lomas Pump Station (PS) is located immediately northeast of the intersection of Dentro de Lomas and Vista del Mar Road, in the Community of Bonsall, San Diego County, California. More specifically, the site of the proposed PS is in the southwest corner of assessor parcel number APN 127-581-06-00 accessible via a graded private roadway connecting to Dentro de Lomas Road (see Figure 1). The site is moderately to steeply sloping to the south with an elevated granitic hill slope extending above the site to the north.

1.2 Project Description

Preliminary site plans provided indicate a proposed 106-foot by 40-foot building pad cut into the existing slope at an elevation of approximately 456 feet msl (see Figure 3). Plans also show one building housing a pump station & electrical room, a prone pump barrel/casing, temporary generator pad, perimeter CMU retaining wall(s) and parking with associated pump assemblies, piping, valves and support equipment. The building is anticipated to consist of reinforced masonry block walls.

1.3 Purpose and Scope of Exploration

The purpose of our exploration is to (1) evaluate geotechnical engineering characteristics of the earth materials at the site, and (2) provide geotechnical recommendations for design and construction of the proposed improvements. As described in our proposal, the scope of our evaluation included the following tasks:

- <u>Field Exploration</u>: Our field exploration consisted of two (2) hand excavated auger borings, geologic field mapping and measuring of the general jointing and rock fractures present.
- <u>Geophysical Survey</u>: This study was performed by our subconsultant (Atlas) and consisted of two (2) P-wave seismic refraction traverse lines.
- <u>Geotechnical Laboratory Tests</u>: Geotechnical laboratory tests were performed on selected soil samples collected during our field exploration. This laboratory testing program was designed to evaluate general physical and engineering characteristics of the site soils.
- <u>Engineering Analysis</u>: Data obtained from our background review, field exploration, and geotechnical laboratory testing program was evaluated to develop geotechnical conclusions and recommendations for the proposed pump station improoevments' design and construction.

- Report Preparation: Results of this evaluation have been summarized in this report, presenting our findings, conclusions and geotechnical recommendations for the proposed pump station improevments.

This report does not address the potential for hazardous materials at this site. Important information about limitations of geotechnical reports is presented in Appendix D.

1.4 Field Exploration

Our field exploration consisted of the excavation of two (2) hand auger borings in representative areas within the site as shown on Figure 3. The hand augers were excavated due to the limited equipment accessibility from the slope, rock outcrops, and vegetation coverage. During the auger excavation, bulk soil samples were collected and sent to our geotechnical laboratory for further testing and evaluation. Sampling of the borings was conducted by an engineering geologist from our office. The logs of borings are presented in Appendix A.

Field mapping included the collection of representative orientations from pervasive jointing and rock fracture structures as observed from the rock outcrops exposed at the ground surface. The results of our field mapping are included on Figure 3.

1.5 Laboratory Testing

Laboratory tests were performed on representative samples to provide a basis for development of geotechnical design parameters. Selected samples were tested to determine the following parameters: maximum dry density and optimum moisture content, gradation, sand equivalent, soluble sulfate content and chloride, pH and resistivity. The results of our laboratory testing are presented in Appendix B.

2.0 SUMMARY OF GEOTECHNICAL FINDINGS

A summary of our findings from research of pertinent literature, site-specific field exploration, geotechnical laboratory testing and engineering analysis, is discussed in this section.

2.1 Site Geology/Subsurface Soils Conditions

As shown on Figure 2, *Regional Geology Map*, and confirmed by our exploration, the site is underlain by plutonic Monzogranite bedrock (Kmm map unit). Surface soils/colluvium overlying the bedrock (upper 2 to 4 feet) generally consist of relatively loose silty sand (SM) with Sand Equivalent (SE) of 31 and very low expansion potential. Detailed descriptions of the earth materials encountered in each hand auger are provided on the logs of borings in Appendix A.

The weathered Granitic bedrock is exposed at multiple locations within the project site as angular resistant fractured bedrock outcrops. Fractures and jointing within the exposed bedrock unit appears random with no pervasive out of slope prevalence. Based on the results of our geophysical study performed for this site, P-wave velocity rates indicate that rippable conditions may be expected within the upper 5 feet to 10 feet below ground surface using Caterpillar D-9 dozer with a single shank (or equivalent). Excavation is likely to be significantly more difficult in the granitic rock at depths greater than 10 feet using conventional heavy equipment (D-9 dozer or Cat 235 trackhoe excavator with rock bucket) and that special rock breaking and/or blasting will likely be required. The complete geophysical study along with graphical presentation of both vertical and lateral velocities (tomography model) is included in Appendix C.

2.2 Surface and Groundwater

Surface water was not observed during our field exploration. Our review of Department of Water Resource groundwater data indicate a historical high groundwater depth of approximately 94 feet BGS for Well 10S03W33L001S within bedrock deposits located approximately 2000 feet to the south. However, groundwater may fluctuate seasonally and be directly-impacted by other factors not observed at the time of our field explorations.

2.3 Regional Faulting and Fault Activity

The subject site, like the rest of Southern California, is located within a seismically active region as a result of being located near the active margin between the North American and Pacific tectonic plates. The principal source of seismic activity on this site is movement along the northwest-trending regional fault systems such as the Lake Elsinore, San Andreas, and San Jacinto. Based on our review of published geologic maps (Hart, 2007), the site is not located within an Earthquake Fault Zone as created by the Alquist-Priolo Earthquake Fault Zoning Act. The nearest active fault is the Pala strand of the Elsinore fault zone, located approximately 12.4 miles to the east-northeast.

2.4 Seismic Coefficients

Strong ground shaking can be expected at the site during moderate to severe earthquakes in this general region. This is common to virtually all of Southern California. The intensity of ground shaking at a given location depends primarily upon earthquake magnitude, site distance from the source, and site response (soil type) characteristics. Based on ASCE 7-16 as the Design Code Reference Document and site Class C, the 2019 CBC seismic coefficients for this site are as listed in the following table:

Table 1. 2019 CBC Site Categorization and Seismic Coefficients

Parameters	
Site Longitude (decimal degrees)	-117.204608
Site Latitude (decimal degrees)	33.265138
Site Class Definition	**C**
Mapped Spectral Response Acceleration at 0.2s Period, S_s	0.96
Mapped Spectral Response Acceleration at 1s Period, S_1	0.35
Short Period Site Coefficient at 0.2s Period, F_a	1.2
Long Period Site Coefficient at 1s Period, F_v	1.5
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	1.15
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	0.53
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	**0.77**
Design Spectral Response Acceleration at 1s Period, S_{D1}	**0.35**

The results of the analysis also indicate that the site modified Peak Ground Acceleration (PGAm) is 0.50g.

2.5 Secondary Seismic Hazards

Secondary hazards such as seiches and tsunamis, landsliding, rockfalls, and ground rupture should be considered very low to non-existent for this site based on our field observations and review of referenced geologic maps. Due to the lack of shallow groundwater and the density of the bedrock subgrade, the potential for liquefaction-induced and dynamic “Dry-Sand” settlement are both also considered negligible or non-existent on this site.

3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 General

The proposed improvements appear feasible from a geotechnical viewpoint provided that the following recommendations are incorporated into the design and construction phases of development. The weathered bedrock within the depth explored may be considered as CalOSHA Type B soils, and sloped excavations will be required to protect workers, if shoring and/or shields are not used. The artificial fill and topsoil deposits should be considered as CalOSHA Type C soils with appropriate shoring and/or shields necessary in trenches and excavations.

3.2 Earthwork Considerations

Earthwork associated with the proposed site improvements should be performed in accordance with applicable RMWD Specifications, "Standard Specifications for Public Works Construction" (Greenbook, latest edition) and the recommendations included in the text of this report.

3.2.1 General

Site grading and trench excavation should be performed in accordance with the project plans, specifications, and all applicable OSHA requirements. The contractor should be responsible for providing the "competent person" required by OSHA standards. Contractors should be advised that onsite sandy soils could make excavations unsafe and hence necessary safety precautions should be taken at all times.

3.2.2 Excavation Characteristics

As indicated in Section 2.1 above, we anticipate the granitic bedrock to be rippable to a depth of 5 to 10 feet below existing grades with conventional heavy earth moving equipment in good operating conditions (Caterpillar D9L or D10 with single shank ripper and rock teeth). Very difficult to unrippable rock will likely exist at depths greater than 10 feet.

3.2.3 Pipe Subgrade Preparation

Prior to pipe installation, the subgrade should be firm/stable to provide uniform seating and support to the entire section of the pipe placed on bedding material.

3.2.4 Building / Pad Subgrade Preparation

No remedial grading/Over-exxcavation (OX) is required if structures or pavement are founded entirely on dense/competent granitic rock. If pad/foundation subgrade become disturbed or loose due to construction activities, the exposed surface should be scarified a minimum of 8 inches, moisture conditioned and compacted to a minimum 90 percent of the maximum dry density in accordance with ASTM D1557. Additional remedial removals may be necessary based on prevailing subgrade conditions during grading.

3.2.5 Backfill

Prior to backfilling, pipes should be bedded in and covered with a uniform, granular material that has a Sand Equivalent (SE) of 30 or greater, and a gradation meeting requirements of the pipe manufacturer. Approved pipe bedding material should be water-densified in-place provided appropriate water evacuation is utilized. Onsite soils can be used of met requirmenst for bedding material. A minimum cover of 12 inches of bedding material should be provided above the top of the pipe.

Native granular soils are generally considered suitable as backfill materials over the pipe bedding zone. However, organic soils and oversized materials generated during excavation (i.e. greater than 3 inches) are considered unsuitable for use in trench backfill. Suitable materials should be placed in thin lifts moisture conditioned, as necessary, and mechanically compacted to a minimum of 90 percent relative compaction per ASTM D 1557 or as required per District standard specifications. The actual lift thickness should depend on the compaction equipment used. For hand-directed mechanical equipment such as vibratory plates or tampers, the maximum lift thickness should not exceed 4 inches. The contractor should not use jetting to compact trench backfill unless approved by RMWD and the jetting procedures and soils requirements comply with the “GreenBook”.

Import soils and/or borrow sites, if needed, should be evaluated by the geotechnical consultant prior to import. Import soils should generally be uncontaminated, granular in nature, free of organic material (loss on ignition less-than 2 percent), have very low expansion potential (EI<21) and have a low corrosion impact to the proposed improvements.

3.3 Slope Construction

The proposed pad will require cut slopes into the granitic bedrock to a maximum height of approximately 20 feet. This slope should be constructed at 1:1 (horizontal to vertical) gradient to provide surficial and globally stability. The upper 2 to 4 feet of the slope may expose overburden soils/colluvium, which should be cut back to 2:1 gradient and protected or landscaped with drought tolerant vegetation as soon as possible after grading to minimize the potential for erosion. Brow ditches should be constructed at the top of cut slopes. Drainage should be directed such that surface runoff on the slope face is minimized.

3.4 Foundation Design Criteria

3.4.1 Bearing Capacity

A net allowable bearing capacity of 3,000 psf, or a modulus of subgrade reaction of 250 pci may be used for design of footings of appurtenant structures founded into a minimum of 2 feet of compacted fill or dense bedrock. A minimum base width of 18 inches for continuous footings and a minimum bearing area of 3 square feet (1.75 ft by 1.75 ft) for pad foundations should be used. Additionally, an increase of one-third may be applied when considering short-term live loads (e.g. seismic and wind).

3.4.2 Earth Pressures

Lateral loads on thrust blocks and other appurtenant structures may be resisted by passive soil pressure and friction, in combination. An allowable passive pressure based on an equivalent fluid pressure of 300 pounds-per-cubic-foot (pcf), not to exceed 3,000 pounds per square foot (psf) can be used if the pipe is embedded in the dense alluvium or compacted fill (minimum 2 feet embedment). This equivalent fluid pressure may be doubled for isolated thrust blocks. We have not applied a factor-of-safety to these values. A soil-pipeline surface friction of 0.20 for PVC pipes may be applied.

A modulus of soil reaction (E') of 1,200 pci can be used to estimate the stiffness of the soil bedding backfill at the sides and below buried flexible pipelines, if applicable, for the purpose of evaluating deflection caused by weight of the backfill over the pipe. This value assumes that the proposed pipeline is embedded at least 5 feet below exiting grades and a granular bedding material with an average relative compaction of 90 percent or more (per ASTM D1557) is placed.

3.5 Pipeline Design

3.5.1 Soils Parameters

Structural design of pipes requires proper evaluation of possible loads acting on the pipe, including dead and live or transient loads. Stresses and strains induced on a buried pipe depend on many factors, including the type of pipe, depth and width of trench, bedding and embedment conditions, soil density, angle of internal friction, coefficient of passive earth pressure, and coefficient of friction at the interface between the backfill and in-situ soils. We recommend the following soil parameters for the proposed pipe design:

Table 2. Soil Parameters for Pipe Design

Soil Parameters	Recommended Values
Average Compacted fill moist unit weight, (pcf)	120 - 130

Angle of internal friction of soils (degrees)	34 to 36
Soil cohesion, c (psf)	100
Sliding friction between pipe and native soils	0.20
Coefficient of friction between backfill and native soils	0.40

3.5.2 External Loads on Flexible Pipe by Soil

Structural design of pipes requires proper evaluation of possible loads acting on the pipe, including dead and live or transient loads. Stresses and strains induced on a buried flexible pipe depend on many factors. The magnitude of the load supported depends on the amount of backfill, type of soil, and pipe stiffness. The approximate dead load per unit length can be calculated from the following formula:

$$W = C\gamma\ BD$$

Where,

W External soil load on pipe: (pounds per foot of pipe)
C Unit less load coefficient (C = 1.4 for 5 feet deep trench, and 1.8 for 10 feet deep trench, assuming a trench width of 3 feet just above the pipe)
γ Total unit weight of soil above pipe (pounds-per-cubic-foot)
B Width of the trench (width just above top of the pipe, in feet)
D Pipe diameter (feet)

In addition to the load from backfill (above equation), loads due to embankments (if applicable) and other loads (live loads) should be considered.

3.6 Retaining Walls / Buried Structures

For design of retaining walls and/or underground structures, our geotechnical design parameters are presented in Table 3 below:

Table 3. Retaining Wall Design Earth Pressures

Retaining Wall Condition (Level Backfill)	Equivalent Fluid Pressure (pounds-per-cubic-foot)*
Active (cantilever)	35
At-Rest (braced)	55
Passive Resistance (compacted fill)	300**

*Only for level and drained properly, compacted backfill.
**Allowable passive resistance should not exceed 3,000 psf in any event.

Cantilever walls that are designed to yield at least 0.001H, where H is equal to the wall height, may be designed using the active condition. Rigid walls and walls braced at the top should be designed using the at-rest condition. Passive pressure is used to compute soil resistance to lateral structural movement. In addition, for sliding resistance, a frictional resistance coefficient of 0.45 may be used for concrete cast directly on soil. Lateral passive resistance should be taken into account only where soil providing passive resistance, embedded against the foundation elements, will remain intact during the design life of the retaining wall. The project Structural Engineer should apply the applicable factors of safety and/or load factors during design, as specified by the California Building Code.

3.7 Preliminary Pavement Design

Where required for light service vehicle traffic, we recommend that a minimum of 3-inch HMA layer placed on top of 4-inch aggregate base. Alternatively, 6-inch PCC pavement may be used in areas subject to heavy truckloads. The PCC pavement should be placed on a minimum 4-inch aggregate base. The PCC pavement should have a minimum of 28-day compressive strength of 3,250 psi. Design and placement of concrete materials should be follow applicable ACI and RMWD standards.

The upper 8 inches of subgrade soils should be moisture-conditioned to near optimum moisture content, compacted to at least 95 percent relative compaction (ASTM D1557) and kept in this condition until the pavement section is constructed. Minimum relative compaction requirements for aggregate base should be 95 percent of the maximum laboratory density as determined by ASTM D1557. If applicable, aggregate base should conform to Greenbook or Caltrans Class 2 aggregate base.

3.8 Corrosivity Evaluation

Sulfate ions in the soil can lower soil resistivity and can be highly aggressive to portland cement concrete by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. Potentially high sulfate content could also cause corrosion of the reinforcing steel in concrete. Table below summarizes current standards for concrete exposed to sulfate-containing solutions.

Table 4. Sulfate Concentration and Sulfate Exposure

Sulfate In Water (parts-per-million)	Water-Soluble Sulfate (SO_4) in soil (percentage by weight)	Sulfate Exposure
0-150	0.00 - 0.10	Negligible
150-1,500	0.10 - 0.20	Moderate (Seawater)
1,500-10,000	0.20 - 2.00	Severe
>10,000	Over 2.00	Very Severe

The sulfate content was determined in the laboratory for representative onsite soil sample. The results indicate that the water-soluble sulfate range is less than 0.1 percent by weight, which is considered Negligible as per Table 4 above. Based upon the test results, Type II cement or an equivalent may be used.

Many factors can affect corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor. Based on the findings of studies presented in ASTM STP 1013 titled “Effects of Soil Characteristics on Corrosion” (February 1989), the approximate relationship between soil resistivity and soil corrosiveness was developed as shown in Table below.

Table 5. Relationship between Soil Resistivity and Soil Corrosivity

Soil Resistivity (ohm-cm)	Classification of Soil Corrosiveness
0 to 900	Very Severely Corrosive
900 to 2,200	Severely Corrosive
2,200 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
10,000 to >100,000	Very Mildly Corrosive

Acidity is an important factor of soil corrosivity. The lower the pH (the more acidic the environment), the higher the soil corrosivity will be with respect to buried metallic structures and utilities. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures, due to protective surface films, which form on steel in high pH environments. The 6.5 pH of the site soils representative samples is below 7.0, which is considered acidic from a corrosion standpoint. Chloride and sulfate ion concentrations, and pH appear to play secondary roles in affecting corrosion potential. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried steel or reinforced concrete structures.

Based on minimum resistivity laboratory test results (see Table 6 below), the onsite soil is considered very mildly corrosive. Ferrous pipe can be protected by polyethylene bags, tape or coatings, di-electric fittings, concrete encasement or other means to separate the pipe from wet onsite soils. Further testing of import and possibly site soil corrosivity could be performed and specific recommendations for corrosion protection may need to be provided by a qualified corrosion engineer.

Table 6. Corrosion Sample Results

Boring	Sample Depth (ft)	Sulfate Content (ppm)	Chloride Content (ppm)	pH	Minimum Resistivity (ohm-cm)
HA-1	0-4.0	152	110	6.5	12000

3.9 Temporary Cut Slopes

The contractor is responsible for all temporary slopes and trenches excavated at the site and the design of any required temporary shoring. Shoring, bracing and benching should be performed by the contractor in accordance with the current edition of the *California Construction Safety Orders*, see:

http://www.dir.ca.gov/title8/sb4a6.html

During construction, exposed earth material conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor is responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Close coordination between the competent person and geotechnical consultant should be maintained to facilitate construction while providing safe excavations. Existing surface soils encountered are classified as OSHA soil Type C. Therefore, unshored temporary cut slopes should be no steeper than 1½:1 (horizontal:vertical), for a height no-greater-than (≤) 20 feet (*California Construction Safety Orders*, Appendix B to Section 1541.1, Table B-1). Encountered granitic rock may be classified as OSHA soil Type B. Existing weathered bedrock are classified as OSHA soil Type B. Therefore, unshored temporary cut slopes should be no steeper than 1:1 (horizontal:vertical), for a height no greater than (≤) 20 feet. These recommended temporary cut slopes assume a level ground surface for a distance equal to one-and-a-half (x1.5) the depth of excavation. For steeper temporary slopes, deeper excavations, and/or where slopes terrain exists within close proximity to excavation (<1.5xdepth), appropriate shoring methods or flatter slopes may be required to protect the workers in the excavation and adjacent improvements. Such methods should be implemented by the contractor and approved by the geotechnical consultant.

3.10 Temporary Shoring

If the sloped open cut excavation is not feasible based on requirements above and due to existing structures, excavations for pipelines should be supported by a temporary shoring system such as cross-braced hydraulic shoring, conventional shields, sheet piles, soldier piles and wood lagging. The choice should be left to the contractor's judgment since economic considerations and/or the individual contractor's construction experience may determine which method is more economical and/or appropriate. The contractor and shoring designer should also perform additional geotechnical studies as necessary to refine the means-and-methods of shoring construction.

The support of all adjacent existing structures during excavation and construction (including pavements) without distress is the contractor's responsibility. In addition, it should be the contractor's responsibility to undertake a pre-construction survey with benchmarks and photographs of the adjacent properties. Shoring systems should be designed by a California licensed civil or structural engineer. As preliminary design guidelines, we present the following geotechnical parameters for shoring design. The following lateral earth pressures are recommended for temporary shoring supporting encountered alignment soils with level ground behind the shoring. Passive pressure also may be used to compute lateral soil resistance, if necessary, for sheet piles. Earth pressures provided are ultimate values and a safety factor should be applied as appropriate.

Table 7. Static Lateral Earth Pressures

Conditions[1]	Static Equivalent Fluid Weight (pcf)
Active (cantilever)	35
At-Rest (braced)	55
Passive[2]	300

1. For temporary excavations only, with level backfill, not including surcharges
2. Passive equivalent fluid pressure may be doubled for isolated soldier piles spaced at least 2½ diameters on-center. Passive resistance should not exceed 3,000 pounds-per-square-foot (psf)

Determination of appropriate design conditions (active or at-rest) depends on shoring flexibility. If a rotation of more than 0.001 radian (0.06 degrees) is allowed, active pressure conditions apply; otherwise, at-rest condition governs.

Surcharge loads (dead or live) should be added to the indicated lateral earth pressures and should be applied uniformly, if such loads are within a horizontal distance that is less-than the exposed shoring height. The corresponding lateral earth pressure will approximately be 33-percent of the vertical surcharge for active conditions, and 50-percent for at-rest conditions. Surcharge pressures from concentrated loads should be evaluated after geometric constraints and loading conditions are determined on individual basis.

3.11 Additional Geotechnical Services

Recommendations are based on information available at the time our report was prepared and may change as plans are developed, or if supplemental subsurface exploration is authorized. Leighton Consulting, Inc. should review site, grading and foundation plans, when available, and comment further on geotechnical aspects of the project. Geotechnical observation and testing should be conducted during excavation and all phases of grading. Geotechnical conclusions and preliminary recommendations should be reviewed and verified by us (Leighton Consulting, Inc.) during construction, and revised accordingly if geotechnical conditions encountered vary from our findings and interpretations. Geotechnical observation and testing should be provided:

- To approve subgrade soils prior to placing bedding materials,
- During compaction of trench backfill,
- After excavation of all footings and prior to placement of concrete,
- During pavement subgrade and base and/or sub-base preparation, and
- When any unusual conditions are encountered.

4.0 LIMITATIONS

This report was necessarily based in part upon data obtained from a limited number of observances, site visits, soil samples, tests, analyses, histories of occurrences, spaced subsurface explorations and limited information on historical events and observations. Such information is necessarily incomplete. The nature of many sites is such that differing characteristics can be experienced within small distances and under various climatic conditions. Changes in subsurface conditions can and do occur over time. This exploration was performed with the understanding that the project as described in Section 1.2 of this report.

This report was prepared for Rainbow Municipal Water District based on Rainbow Municipal Water District's needs, directions, and requirements at the time of our investigation. This report is not authorized for use by, and is not to be relied upon by any party except Rainbow Municipal Water District, and its successors and assigns as owner of the property, with whom Leighton Consulting, Inc. has contracted for the work. Use of or reliance on this report by any other party is at that party's risk. Unauthorized use of or reliance on this report constitutes an agreement to defend and indemnify Leighton Consulting, Inc. from and against any liability which may arise as a result of such use or reliance, regardless of any fault, negligence, or strict liability of Leighton Consulting, Inc.

The client is referred to Appendix C regarding important information provided by the Geoprofessional Business Association (GBA) on geotechnical engineering studies and report and their applicability.

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Project: 12600.004	Eng/Geol: SIS/MSB
Scale:1 " = 2,000 '	Date: August 2022

Reference: © 2022 Microsoft Corporation © 2022 Maxar ©CNES (2022) Distribution Airbus

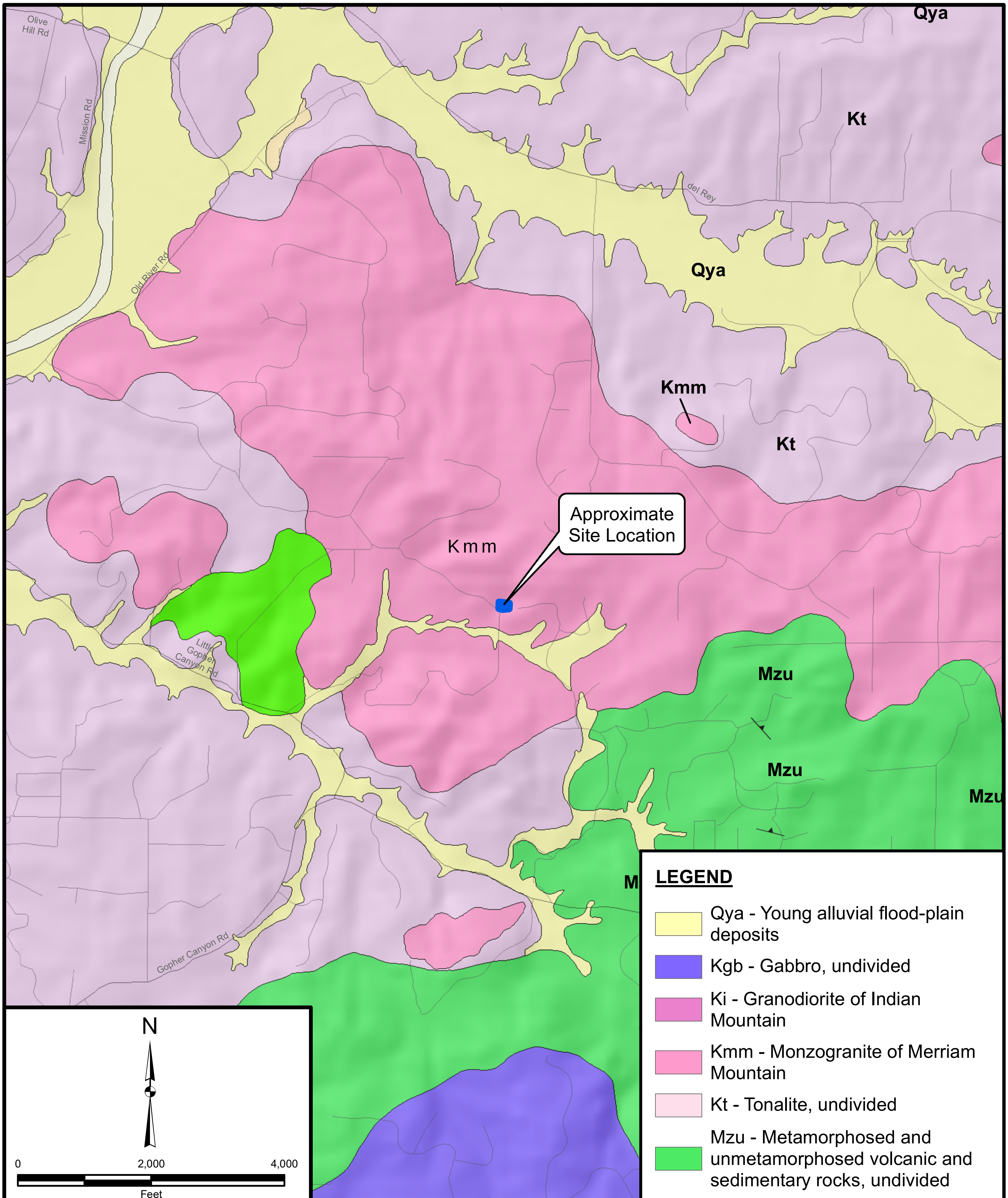
SITE LOCATION MAP

Rainbow Municipal Water District
Dentro De Lomas Pump Station
Bonsall, California

FIGURE 1

Leighton

Map Saved as V:\Drafting\12600\004\Maps\12600-004_F01_SLM_2022-08-08.mxd on 8/8/2022 9:47:28 AM Author: KVM (btran)



Project: 12600.004	Eng/Geol: SIS/MSB
Scale:1 " = 2,000 '	Date: August 2022

Basemap: USGS Topo Map Service from Esri, 2021
Reference: USGS Geologic Map of the Sotuthern California GIS Compilation by CGS

REGIONAL GEOLOGY MAP

Rainbow Municipal Water District
Dentro De Lomas Pump Station
Bonsall, California

FIGURE 2

Leighton

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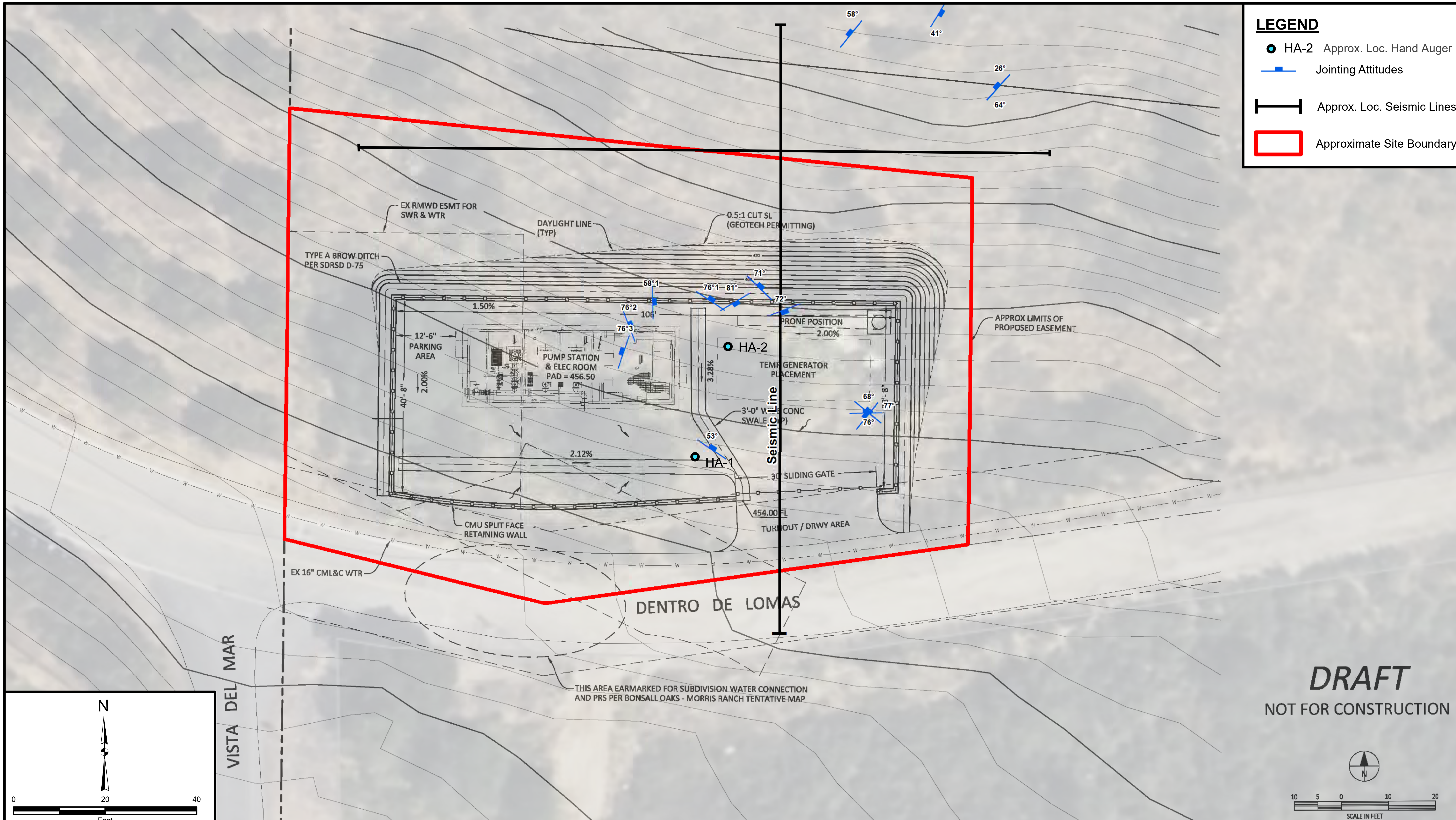

Project: 12600.004	Eng/Geol: SIS/MSB
Scale: 1 " = 20 '	Date: August 2022

Reference:
Dentro Pump Station Drainage and Grading Plan
Sheet 17 of 28, by Hoch Consulting

BORING LOCATION MAP

Rainbow Municipal Water District
Dentro De Lomas Pump Station
Bonsall, California

FIGURE 3

Leighton

Map Saved as V:\Drafting\12600\004\Maps\12600-004_F03_BLM_2022-08-08.mxd on 8/8/2022 10:34:42 AM Author: KVM (btran)

APPENDIX A

FIELD EXPLORATION / LOGS OF EXPLORATORY BORINGS

Our field exploration consisted of a site reconnaissance, geologic mapping and a subsurface exploration program consisting of hand auger soil borings. Our field exploration was performed on August 5, 2022. Approximate locations of the borings are depicted on the Boring Location Plan (*Figure 2*). Encountered soils were logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D 2488). Logs of these subsurface explorations, as well as a key to the classification of the soil, are included as part of this appendix.

Disturbed bag (or bulk) samples were obtained from soil auger cuttings. Types of samples obtained from each location are shown on the boring logs at corresponding depths. Our borings were backfilled with soil cuttings obtained during the excavation. Representative earth-material samples obtained from these subsurface explorations were transported to our Temecula geotechnical laboratory for evaluation and appropriate testing.

The attached subsurface exploration logs and related information depict subsurface conditions only at the locations indicated and at the particular date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

GEOTECHNICAL BORING LOG HA-1

Project No.	12600.004	**Date Drilled**	8-5-22
Project	Dentro De Lomas Pump Station	**Logged By**	MSB
Drilling Co.	Leighton Staff	**Hole Diameter**	3"
Drilling Method	Hand Auger	**Ground Elevation**	'
Location	See Boring Location Map	**Sampled By**	MSB

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0			B1				SM	**Quaternary Colluvium (Qcol)** SILTY SAND, loose, light brown, dry, fine sand, fine roots MD = 121.5 @ 9.5, EI = 0, FINES 24% GRAVEL 8%	MD, SA, EI, CR
									slightly moist, with slightly more cohesion	
									Bedrock (Kgr) Highly weathered, dense, medium sand	
									Total Depth 4' No Groundwater Encountered Backfilled 8/5/2022	
	5									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG HA-2

Project No.	12600.004	**Date Drilled**	8-5-22
Project	Dentro De Lomas Pump Station	**Logged By**	MSB
Drilling Co.	Leighton Staff	**Hole Diameter**	3"
Drilling Method	Hand Auger	**Ground Elevation**	'
Location	See Boring Location Map	**Sampled By**	MSB

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
	0							SM	**Quaternary Colluvium (Qcol)** SILTY SAND, loose, light brown, slightly moist, fine sand SE = 31, FINES 25% GRAVEL 8%	SA, SE
									coarse cobble, gravel at contact	
									Bedrock (Kgr) Highly weathered, grayish brown, jointed ~ 0.5-2.0' spacing	
									Total Depth 2.5' No Groundwater Encountered Backfilled 8/5/2022	
	5									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

Leighton

APPENDIX B

RESULTS OF LABORATORY TESTING

MODIFIED PROCTOR COMPACTION TEST
ASTM D 1557

Project Name:	RMWD Dentro De Lomas PS	Tested By:	F. Mina	Date:	08/31/22
Project No.:	12600.001	Input By:	M. Vinet	Date:	09/01/22
Boring No.:	HA-1	Depth (ft.):	0 - 4.0		
Sample No.:	B-1				
Soil Identification:	Silty Sand (SM), Dark Yellowish Brown.				

Note: Corrected dry density calculation assumes specific gravity of 2.70 and moisture content of 1.0% for oversize particles

Preparation Method:	X	Moist
		Dry
Compaction Method	X	Mechanical Ram
		Manual Ram

Scalp Fraction (%)	
#3/4	
#3/8	6.9
#4	

Rammer Weight (lb.) = 10.0
Height of Drop (in.) = 18.0
Mold Volume (ft^3) 0.03340

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	5419	5475	5522	5495		
Weight of Mold (g)	3531	3531	3531	3531		
Net Weight of Soil (g)	1888	1944	1991	1964		
Wet Weight of Soil + Cont. (g)	1412.3	1365.2	1402.8	1532.2		
Dry Weight of Soil + Cont. (g)	1342.1	1278.2	1294.2	1390.8		
Weight of Container (g)	280.4	278.2	276.8	277.5		
Moisture Content (%)	6.6	8.7	10.7	12.7		
Wet Density (pcf)	124.6	128.3	131.4	129.6		
Dry Density (pcf)	116.9	118.0	118.7	115.0		

Maximum Dry Density (pcf)	**118.9**	**Optimum Moisture Content (%)**	**10.0**
Corrected Dry Density (pcf)	**121.5**	**Corrected Moisture Content (%)**	**9.5**

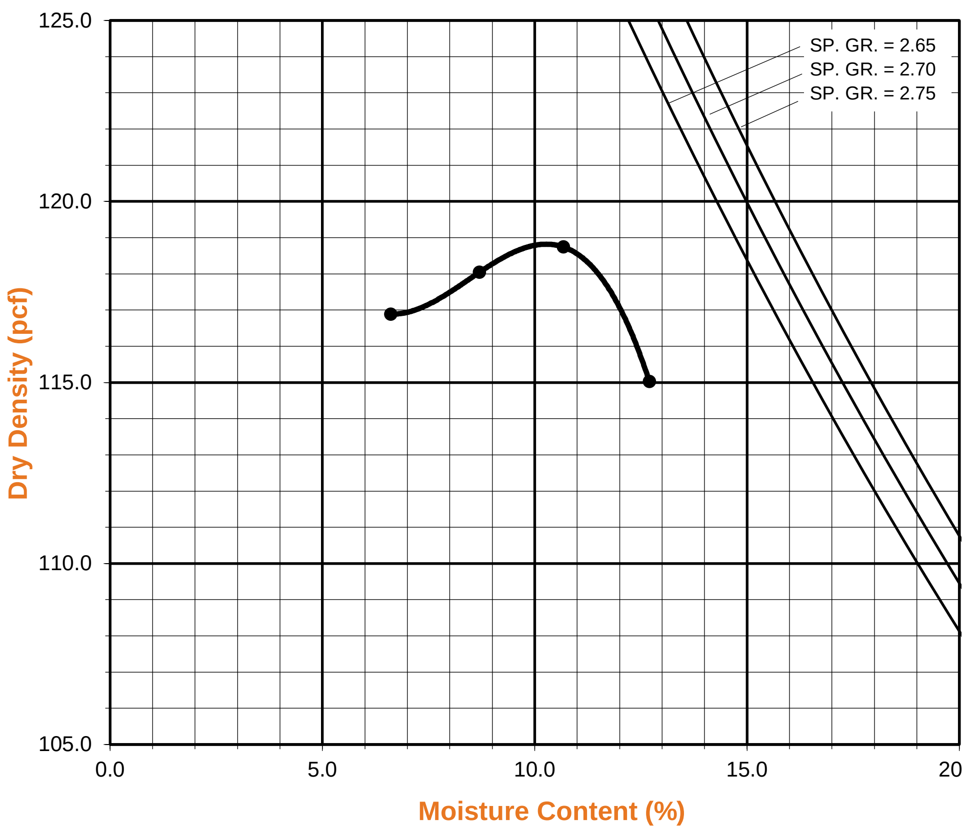
☐ **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

☒ **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

☐ **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:
8:68:24
GR:SA:FI
Atterberg Limits:

LL,PL,PI



TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name: RMWD Dentro De Lomas PS

Project No. : 12600.004

Tested By : F. Mina Date: 09/01/22

Data Input By: M. Vinet Date: 09/01/22

Boring No.	HA-1			
Sample No.	B-1			
Sample Depth (ft)	0 - 4.0			
Soil Identification:	Silty Sand (SM)			
Wet Weight of Soil + Container (g)	100.00			
Dry Weight of Soil + Container (g)	100.00			
Weight of Container (g)	0.00			
Moisture Content (%)	0.00			
Weight of Soaked Soil (g)	100.00			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	1			
Crucible No.	1			
Furnace Temperature (°C)	850			
Time In / Time Out	Timer			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	23.0398			
Wt. of Crucible (g)	23.0361			
Wt. of Residue (g) (A)	0.0037			
PPM of Sulfate (A) x 41150	152.25			
PPM of Sulfate, Dry Weight Basis	**152**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	1.3			
PPM of Chloride (C -0.2) * 100 * 30 / B	110			
PPM of Chloride, Dry Wt. Basis	**110**			

pH TEST, DOT California Test 643

pH Value	**6.50**			
Temperature °C	21.0			

SOIL RESISTIVITY TEST
DOT CA TEST 643

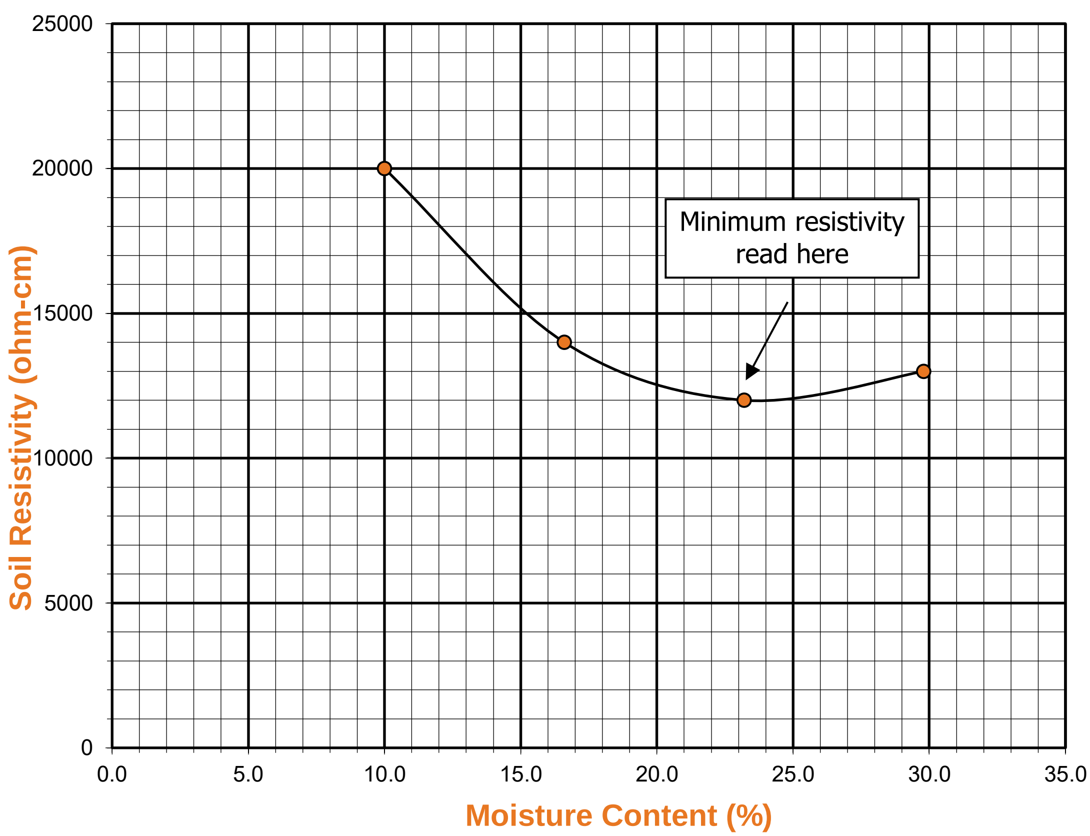
Project Name:	RMWD Dentro De Lomas PS	Tested By :	F. Mina	Date:	09/01/22
Project No. :	12600.004	Data Input By:	M. Vinet	Date:	09/01/22
Boring No.:	HA-1	Depth (ft.) :	0 - 4.0		
Sample No. :	B-1				
Soil Identification:*	Silty Sand (SM)				

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	50	10.00	20000	20000
2	83	16.60	14000	14000
3	116	23.20	12000	12000
4	149	29.80	13000	13000
5				

Moisture Content (%) (MCi)	0.00
Wet Wt. of Soil + Cont. (g)	100.00
Dry Wt. of Soil + Cont. (g)	100.00
Wt. of Container (g)	0.00
Container No.	A
Initial Soil Wt. (g) (Wt)	500.00
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH: pH	Soil pH: Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
12000	**23.2**	**152**	**110**	**6.50**	**21.0**



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EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	RMWD Dentro De Lomas PS	Tested By:	F. Mina	Date:	8/31/22
Project No. :	12600.004	Checked By:	M. Vinet	Date:	9/1/22
Boring No.:	HA-1	Depth:	0 - 4.0		
Sample No. :	B-1	Location:	N/A		
Sample Description:	Silty Sand (SM), Dark Yellowish Brown.				

Dry Wt. of Soil + Cont. (gm.)	1315.1
Wt. of Container No. (gm.)	0.0
Dry Wt. of Soil (gm.)	1315.1
Weight Soil Retained on #4 Sieve	110.5
Percent Passing # 4	91.6

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	0.9998
Wt. Comp. Soil + Mold (gm.)	605.2	631.2
Wt. of Mold (gm.)	188.3	188.3
Specific Gravity (Assumed)	2.70	2.70
Container No.	7	7
Wet Wt. of Soil + Cont. (gm.)	581.2	631.2
Dry Wt. of Soil + Cont. (gm.)	557.7	384.2
Wt. of Container (gm.)	281.2	188.3
Moisture Content (%)	8.5	15.3
Wet Density (pcf)	125.8	133.6
Dry Density (pcf)	115.9	115.9
Void Ratio	0.455	0.454
Total Porosity	0.312	0.312
Pore Volume (cc)	64.7	64.6
Degree of Saturation (%) [S meas]	**50.5**	**90.7**

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h.

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
8/31/22	11:30	1.0	0	0.5000
8/31/22	11:40	1.0	10	0.5000
	Add Distilled Water to the Specimen			
9/1/22	6:00	1.0	1100	0.4998
9/1/22	7:00	1.0	1160	0.4998

Expansion Index (EI meas) =	((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	-0.2
Expansion Index (Report) =	Nearest Whole Number or Zero (0) if Initial Height is > than Final Heigh	**0**

SAND EQUIVALENT TEST

ASTM D 2419 / DOT CA Test 217

Project Name:	RMWD Dentro De Lomas PS	Tested By:	F. Mina	Date:	8/31/22
Project No. :	12600.004	Computed By:	F. Mina	Date:	8/31/22
Client:	Rainbow Municipal Water District	Checked By:	M. Vinet	Date:	9/1/22

Boring No.	Sample No.	Depth (ft.)	Soil Description	T1	T2	T3	T4	R1	R2	SE	Average SE
HA-2	B-1	0 - 2.5	Silty Sand (SM)	14:00	14:10	14:12	14:32	7.3	2.2	31	**31**
				14:02	14:12	14:14	14:34	7.0	2.1	30	

T1 = Starting Time

T2 = (T1 + 10 min) Begin Agitation

T3 = Settlement Starting Time

T4 = (T3 + 20 min) Take Clay Reading (R1)

Sand Equivalent = R2 / R1 * 100

Record SE as Next Higher Integer

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS
ASTM D 6913

Leighton

Project Name:	RMWD Dentro De Lomas PS	Tested By:	FLM	Date:	08/31/22
Project No.:	12600.004	Checked By:	MRV	Date:	09/01/22
Boring No.:	HA-1	Depth (feet):	0 - 4.0		
Sample No.:	B-1				
Soil Identification:	Silty Sand (SM), Dark Yellowish Brown.				

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	P	P	Wt. of Air-Dry Soil + Cont.(g)	2036.6	1026.5
Wt. Air-Dried Soil + Cont.(g)	2036.6	1026.5	Wt. of Dry Soil + Cont. (g)	2031.2	1026.5
Wt. of Container (g)	716.2	716.2	Wt. of Container No._____(g)	716.2	716.2
Dry Wt. of Soil (g)	1315.1	310.3	Moisture Content (%)	0.4	0.0

Passing #4 Material After Wet Sieve		
	Container No.	P
	Wt. of Dry Soil + Container (g)	954.3
	Wt. of Container (g)	716.2
	Dry Wt. of Soil Retained on # 200 Sieve (g)	238.1

U. S. Sieve Size		Cumulative Weight of Dry Soil Retained (g)		Percent Passing (%)
	(mm.)	Whole Sample	Sample Passing #4	
1 1/2"	37.500			100.0
1"	25.000	0.0		100.0
3/4"	19.000	24.0		98.2
1/2"	12.500	58.0		95.6
3/8"	9.500	90.9		93.1
#4	4.750	110.5		91.6
#8	2.360		5.9	89.9
#16	1.180		16.4	86.8
#30	0.600		30.8	82.5
#50	0.300		55.7	75.2
#100	0.150		124.8	54.8
#200	0.075		228.9	24.0
PAN				

GRAVEL: **8 %**

SAND: **68 %**

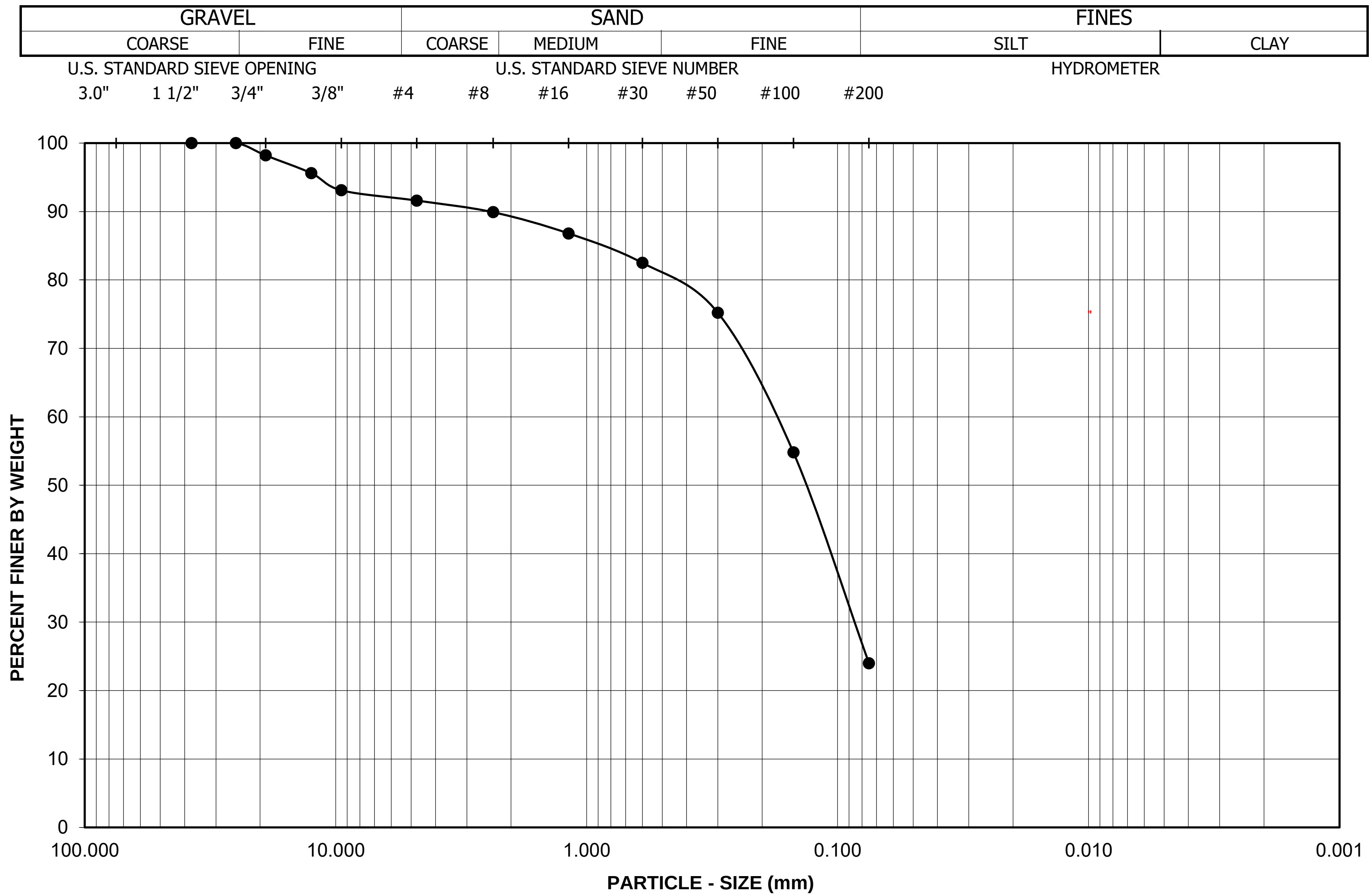
FINES: **24 %**

GROUP SYMBOL: **SM**

$Cu = D60/D10 =$ N/A

$Cc = (D30)^2/(D60*D10) =$ N/A

Remarks:



Project Name: RMWD Dentro De Lomas PS

Project No.: 12600.004

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Boring No.: HA-1

Sample No.: B-1

Depth (feet): 0 - 4.0

Soil Type : SM

Soil Identification: Silty Sand (SM), Dark Yellowish Brown.

GR:SA:FI : (%) 8 : 68 : 24

Sep-22

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS
ASTM D 6913

Leighton

Project Name:	RMWD Dentro De Lomas PS	Tested By:	FLM	Date:	08/31/22
Project No.:	12600.004	Checked By:	MRV	Date:	09/01/22
Boring No.:	HA-2	Depth (feet):	0 - 2.5		
Sample No.:	B-1				
Soil Identification:	Silty Sand (SM), Dark Yellowish Brown.				

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	A	A	Wt. of Air-Dry Soil + Cont.(g)	2512.2	622.2
Wt. Air-Dried Soil + Cont.(g)	2512.2	622.2	Wt. of Dry Soil + Cont. (g)	2450.6	622.2
Wt. of Container (g)	277.8	277.8	Wt. of Container No._____(g)	277.8	277.8
Dry Wt. of Soil (g)	2173.5	344.4	Moisture Content (%)	2.8	0.0

Passing #4 Material After Wet Sieve		
	Container No.	A
	Wt. of Dry Soil + Container (g)	527.8
	Wt. of Container (g)	277.8
	Dry Wt. of Soil Retained on # 200 Sieve (g)	250.0

U. S. Sieve Size		Cumulative Weight of Dry Soil Retained (g)		Percent Passing (%)
	(mm.)	Whole Sample	Sample Passing #4	
1 1/2"	37.500			100.0
1"	25.000	0.0		100.0
3/4"	19.000	51.0		97.7
1/2"	12.500	90.8		95.8
3/8"	9.500	125.6		94.2
#4	4.750	182.3		91.6
#8	2.360		10.2	88.9
#16	1.180		20.4	86.2
#30	0.600		35.6	82.1
#50	0.300		60.4	75.5
#100	0.150		180.2	43.7
#200	0.075		250.0	25.1
PAN				

GRAVEL: **8 %**

SAND: **67 %**

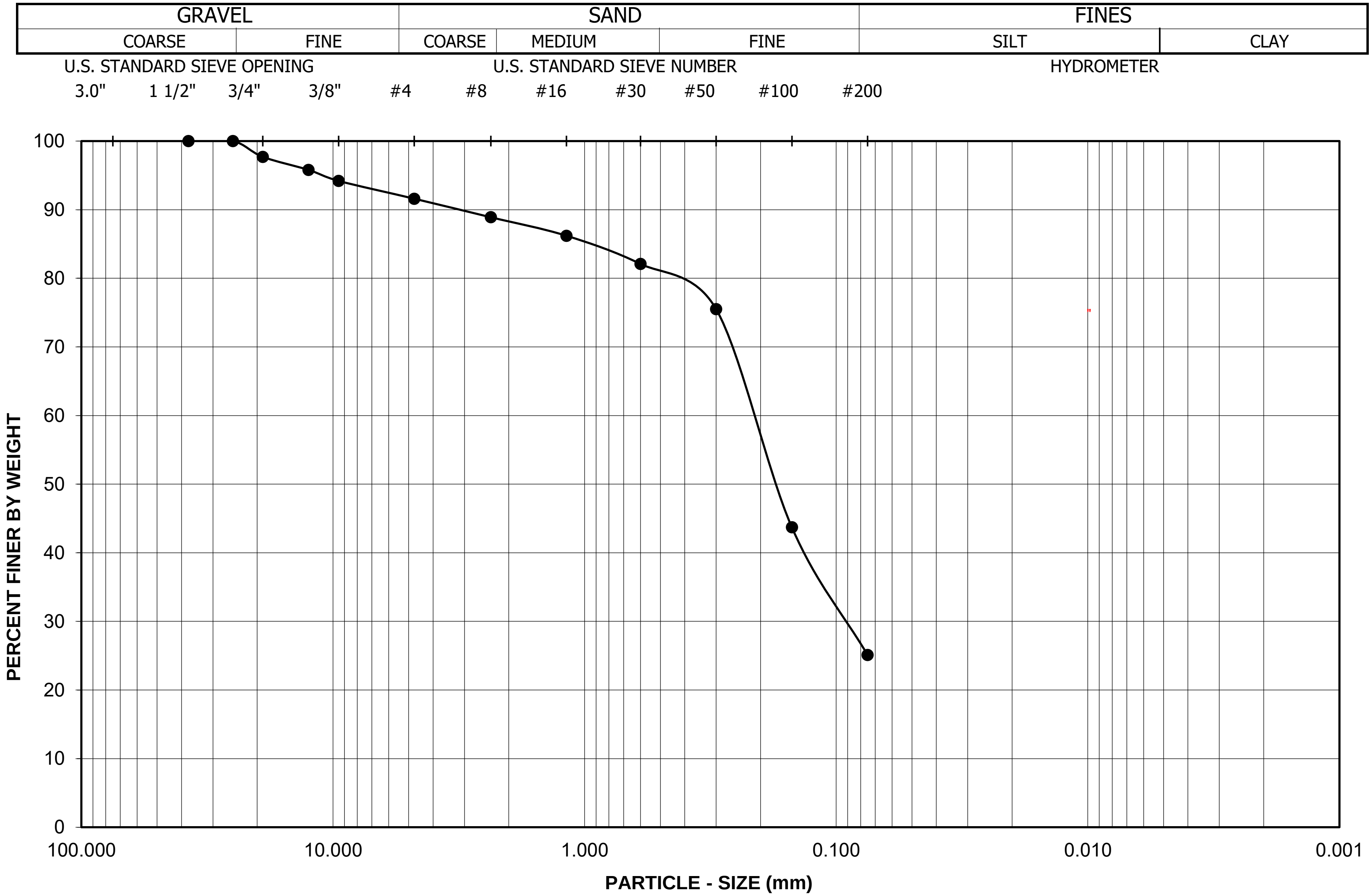
FINES: **25 %**

GROUP SYMBOL: **SM**

$Cu = D60/D10 =$ N/A

$Cc = (D30)^2/(D60*D10) =$ N/A

Remarks:



Project Name: RMWD Dentro De Lomas PS

Project No.: 12600.004

Boring No.: HA-2 Sample No.: B-1

Depth (feet): 0 - 2.5 Soil Type : SM

Soil Identification: Silty Sand (SM), Dark Yellowish Brown.

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 8 : 67 : 25

Sep-22

APPENDIX C

SEISMIC REFRACTION STUDY

BY ATLAS TECHNICAL CONSULTANTS LLC

SEISMIC REFRACTION STUDY

RAINBOW MUNICIPAL WATER DISTRICT
BONSALL PUMP STATION SITE

Bonsall, California

PREPARED FOR:

Mr. Mitch Bornyasz
Leighton
17781 Cowan
Irvine, CA 92614

PREPARED BY:

Atlas Technical Consultants LLC
6280 Riverdale Street
San Diego, CA 92120

August 31, 2022

6280 Riverdale Street
San Diego, CA 92120
(877) 215-4321 | oneatlas.com

August 31, 2022

Atlas No. 122230.P6SWG
Report No. 1

MR. MITCH BORNYASZ
LEIGHTON
17781 COWAN
IRVINE, CA 92614

Subject: Rainbow Municipal Water District
Bonsall Pump Station Site
Bonsall, California

Dear Mr. Bornyasz:

In accordance with your authorization, Atlas Technical Consultants has performed a seismic refraction study pertaining to the Rainbow Municipal Water District, Bonsall Pump Station project located off Dentro De Lomas in Bonsall, California. Specifically, our evaluation consisted of performing two seismic P-wave refraction traverses at the site. The purpose of our evaluation was to develop subsurface velocity profiles of the study areas in order to assess the depth to bedrock and apparent rippability of the subsurface materials. Our field services were conducted on August 1st, 2022. This data report presents our methodology, equipment used, analysis, and results.

If you have any questions, please call us at (619) 280-4321.

Respectfully submitted,
Atlas Technical Consultants LLC

REGISTERED PROFESSIONAL GEOPHYSICIST
PATRICK F. LEHRMANN
No. 1043
Exp. 1/31/2024
STATE OF CALIFORNIA

Paul W. Gresoro
Senior Staff Geophysicist

Patrick F. Lehrmann, P.G., P.Gp.
Principal Geologist/Geophysicist

PWG:OAA:PFL:ds

Distribution: mbornyasz@leightongroup.com

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FIGURES

1. INTRODUCTION

In accordance with your authorization, Atlas Technical Consultants has performed a seismic refraction study pertaining to the Rainbow Municipal Water District, Bonsall Pump Station project located off Dentro De Lomas in Bonsall, California. Specifically, our evaluation consisted of performing two seismic P-wave refraction traverses at the site. The purpose of our evaluation was to develop subsurface velocity profiles of the study areas in order to assess the depth to bedrock and apparent rippability of the subsurface materials. Our field services were conducted on August 1st, 2022. This data report presents our methodology, equipment used, analysis, and results.

2. SCOPE OF SERVICES

Our scope of services included:

- Performance of two seismic P-wave refraction traverse at the project site.
- Compilation and analysis of the data collected.
- Preparation of this data report presenting our results and conclusions.

3. SITE AND PROJECT DESCRIPTION

The project site is located within a plot of undeveloped land, west of Highway 76 off Old River Road and Dentro De Lomas in Bonsall, California (Figure 1). Two seismic traverses were conducted in the study area in locations selected by a representative from your office. The site has a steep slope upwards toward the north with granitic boulder outcrops covered by patches of sage brush and seasonal grasses. Figures 2 and 3 show the seismic line locations and depict the general site condition in the area of the seismic traverses.

Based on our discussions with you, it is our understanding that your office requested this study in advance of construction activities for the subject project. We also understand that the results of our study may be used in the formulation of design and construction parameters for the project.

4. STUDY METHODOLOGY

Three seismic P-wave (compression wave) refraction studies were conducted at the project to develop subsurface velocity profiles of the areas studied, and to assess the depth to bedrock and apparent rippability of the subsurface materials. The seismic refraction method uses first-arrival times of refracted seismic waves to estimate the thicknesses and seismic velocities of subsurface layers. Seismic P-waves generated at the surface, using a hammer and plate, are refracted at boundaries separating materials of contrasting velocities. These refracted seismic waves are then detected by a series of surface vertical component 14-Hz geophones and recorded with a 24-channel Geometrics Geode seismograph. The travel times of the seismic P-waves are used in conjunction with the shot-to-geophone distances to obtain thickness and velocity information on the subsurface materials.

Two 135-foot seismic traverses (SL-1 and SL-2) were conducted in the study area. The general locations and lengths of the line were determined by surface conditions, site access, and depth of investigation, as determined by you. Shot points (signal generation locations) were conducted along the lines at the ends, midpoint, and intermediate points between the ends and the midpoint.

In general, classical seismic refraction theory requires that subsurface velocities increase with depth (generalized reciprocal method (GRM) and time-intercept modeling). In classical analysis methods a layer having a velocity lower than that of the layer above will not generally be detectable by the seismic refraction method and, therefore, could lead to errors in the depth calculations of subsequent layers. In addition, lateral variations in velocity such as those caused by core stones, intrusions, or boulders can also result in the misinterpretation of the subsurface conditions. However, application of seismic tomography methods, as was performed for this project by Atlas, produce velocity models which, in general, are not subject to this limitation. However, even the application of seismic tomography analysis does have certain limitations regarding vertical and horizontal resolution. When a velocity anomaly target is of similar scale length to the seismic wavelet (or smaller), then diffraction behavior dominates because scattering is governing the loci of the wavefronts. For travel time analysis a target feature must be at a scale vs. its depth that is detectable relative to the scale length of the seismic wavelet we produce and receive. There is therefore a general limit to what scale of feature seismic tomography methods can detect regarding relatively small velocity anomaly features, related to both source and to medium velocities, and travel time uncertainties. In effect, some relatively smaller scale features including "thin" velocity inversion layers or voids, and some types of lateral and vertical velocity variations caused by core stones and intrusions might not be detected in our results. In general, the effective depth of evaluation for a seismic refraction traverse is approximately one third to one-fifth of the length of the spread.

In general, the seismic P-wave velocity of a material can be correlated to rippability (see Table 1 below), or to some degree "hardness." Table 1 is based on published information from the Caterpillar Performance Handbook (Caterpillar, 2018), as well as our experience with similar materials, and assumes that a Caterpillar D-9 dozer ripping with a single shank is used. We emphasize that the cutoffs in this classification scheme are approximate and that rock characteristic, such as fracture spacing and orientation, play a significant role in determining rock quality or rippability. The rippability of a mass is also dependent on the excavation equipment used and the skill and experience of the equipment operator.

For trenching operations, the rippability values should be scaled downward. For example, velocities as low as 3,500 feet/second may indicate difficult ripping during trenching operations. In addition, the presence of boulders, which can be troublesome in narrow trenching operations, should be anticipated.

Table 1 – Rippability Classification

Seismic P-wave Velocity	Rippability
0 to 2,000 feet/second	Easy
2,000 to 4,000 feet/second	Moderate
4,000 to 5,500 feet/second	Difficult, Possible Blasting
5,500 to 7,000 feet/second	Very Difficult, Probable Blasting
Greater than 7,000 feet/second	Blasting Generally Required

It should be noted that the rippability cutoffs presented in Table 1 are slightly more conservative than those published in the Caterpillar Performance Handbook. Accordingly, the above classification scheme should be used with discretion, and contractors should not be relieved of making their own independent evaluation of the rippability of the on-site materials prior to submitting their bids.

5. DATA ANALYSIS

The collected data were processed and analyzed using Rayfract® Version 4.02 (Intelligent Resources Inc., 2021) which employs wave path analysis. RAYFRACT first provides forward modeling of refraction, transmission, and diffraction and then back-projects travel-time residuals along wave paths also known as Fresnel volumes instead of conventional analysis by rays. This increases the numerical robustness of the inversion. A smooth minimum-structure 1-D starting velocity-depth profile model is determined automatically directly from the seismic travel-time data first arrival picks and elevation data to produce subsurface velocities by horizontally averaging via the Delta t-V method. The Delta t-V method is based on common mid-point (CMP) sorted travel times and assumes multiple horizontal layers with constant interior velocity gradients (Rohdewald 2007, and Gebrande 1985). Modeled seismic rays follow circular arcs inside each modeled layer. The Delta t-V starting model is then refined with 2D Wavepath Eikonal Traveltime (WET) inversion method (Schuster, 1993). The resulting 2-D WET velocity model provides a 2-D tomographic image of the P-wave velocities which can be used to estimate subsurface geologic conditions. Both vertical and lateral velocity information is contained in the tomography model. Changes in layer velocity are generally revealed as gradients rather than discrete contacts, which typically are more representative of actual conditions.

6. RESULTS AND CONCLUSIONS

As previously indicated, two seismic traverses were conducted as part of our study and Figures 4a and 4b present the velocity models generated from our analysis. Based on the results it appears that the study areas are underlain by low velocity materials (e.g., colluvium and topsoil) in the near surface and granitic rock at depth. Distinct vertical and lateral velocity variations are evident in the models. Moreover, the degree of bedrock weathering and the depth to bedrock appears to be highly variable across the study areas. In addition, remnant boulders appear to be present in the subsurface in some areas.

Based on the refraction results, variability in the excavatability (including depth of rippability) of the subsurface materials may be expected across the project area. Furthermore, blasting may be required depending on the excavation, depth, location, equipment used, and desired rate of production. In addition, oversized materials should be expected. A contractor with excavation experience in similarly difficult conditions should be consulted for expert advice on excavation methodology, equipment, and production rate.

7. LIMITATIONS

The field evaluation and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, express or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be present. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface surveying will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Atlas should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

8. SELECTED REFERENCES

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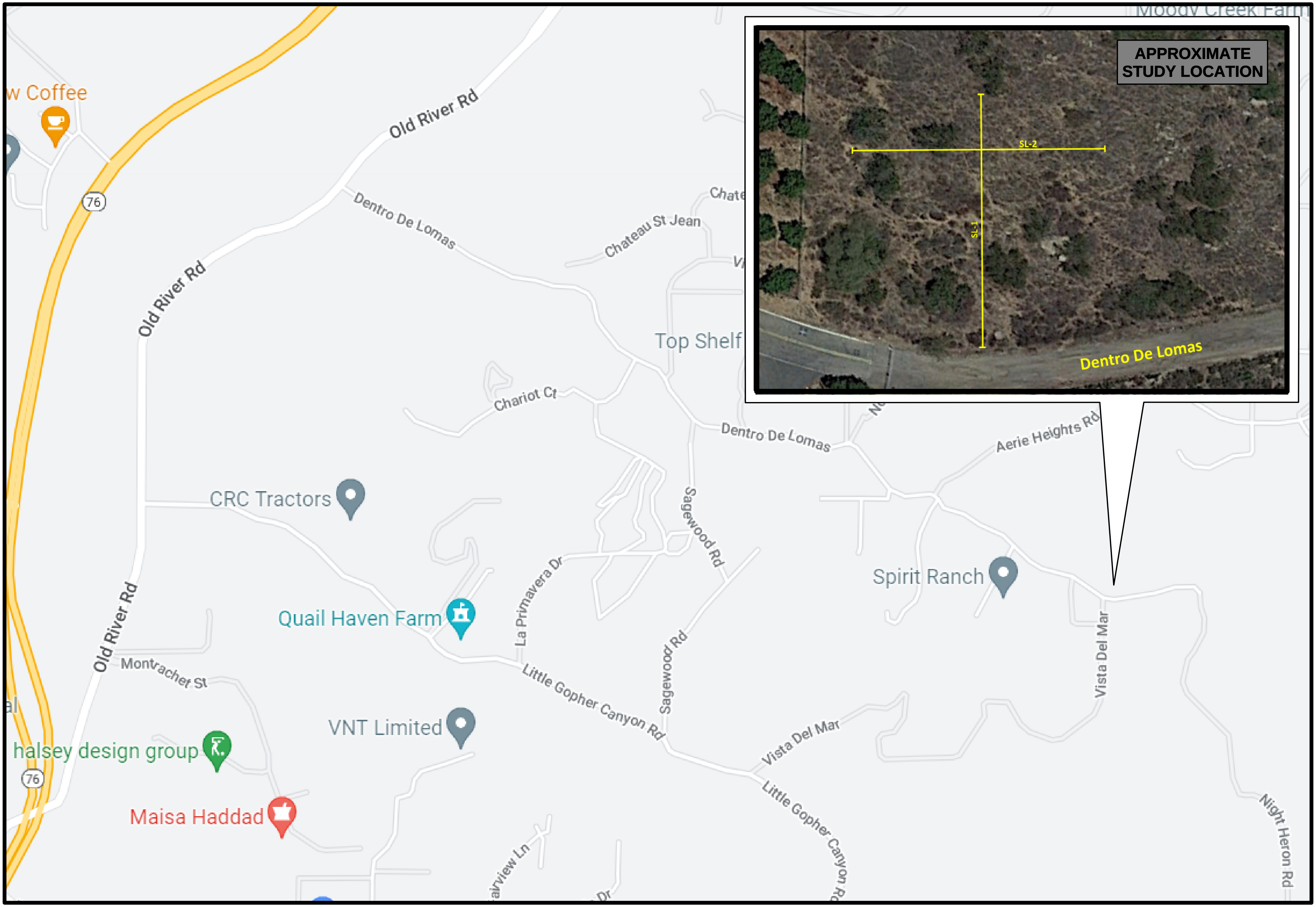


SITE LOCATION MAP

N

Rainbow Municipal Water District
Bonsal Pump Station Site
Bonsal, California

Project No.: 122230P.6SWG | Date: 08/22

ATLAS

Figure 1

SEISMIC LINE LOCATION MAP

N

Rainbow Municipal Water District
Bonsal Pump Station Site
Bonsal, California

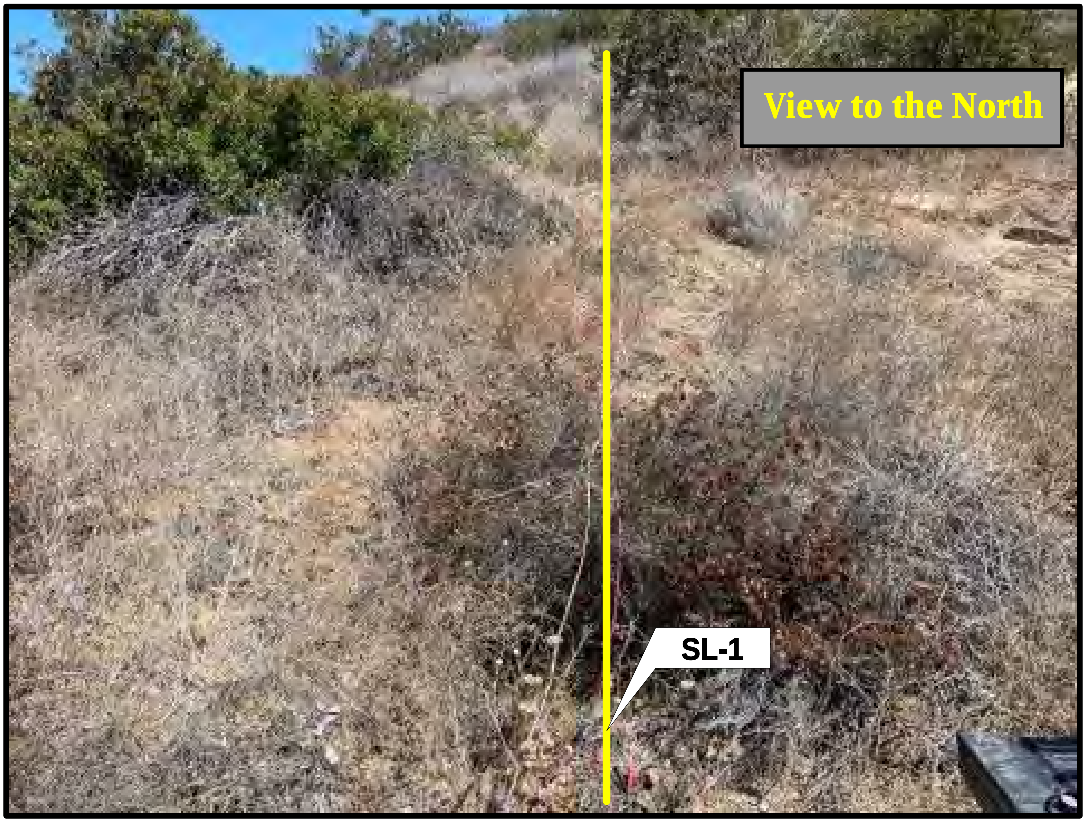
Project No.: 122230P.6SWG

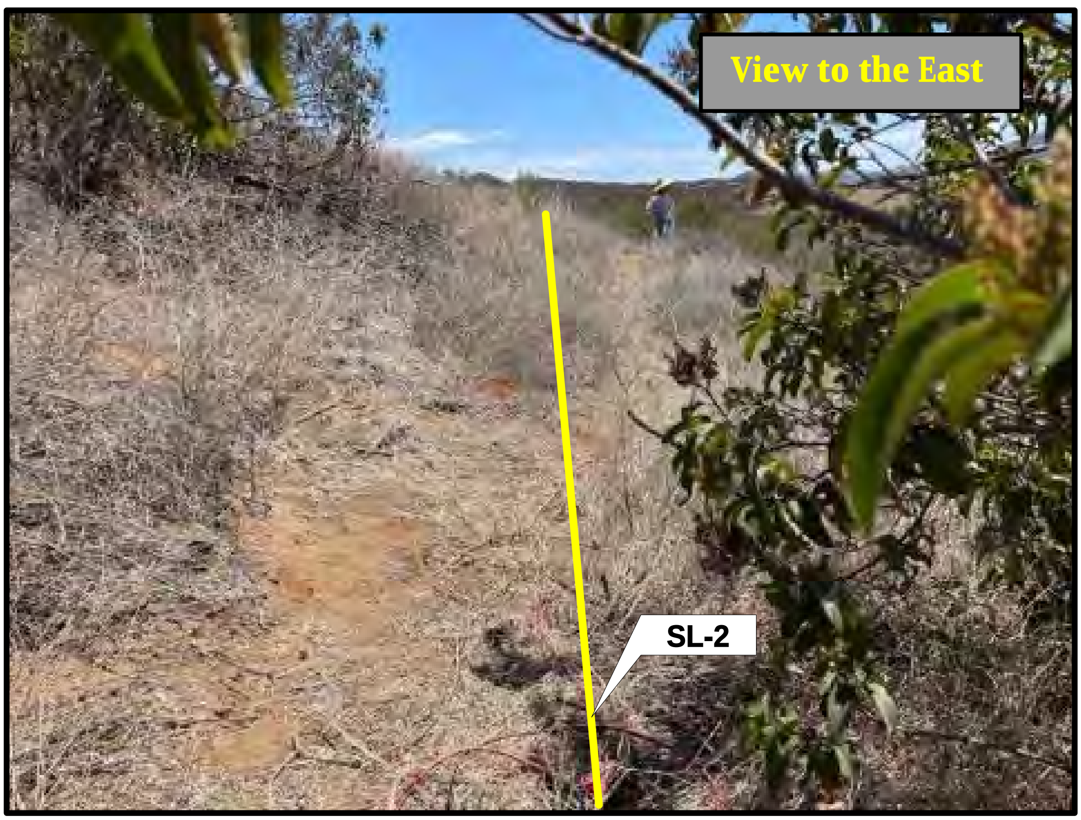
Date: 08/22

ATLAS

Figure 2

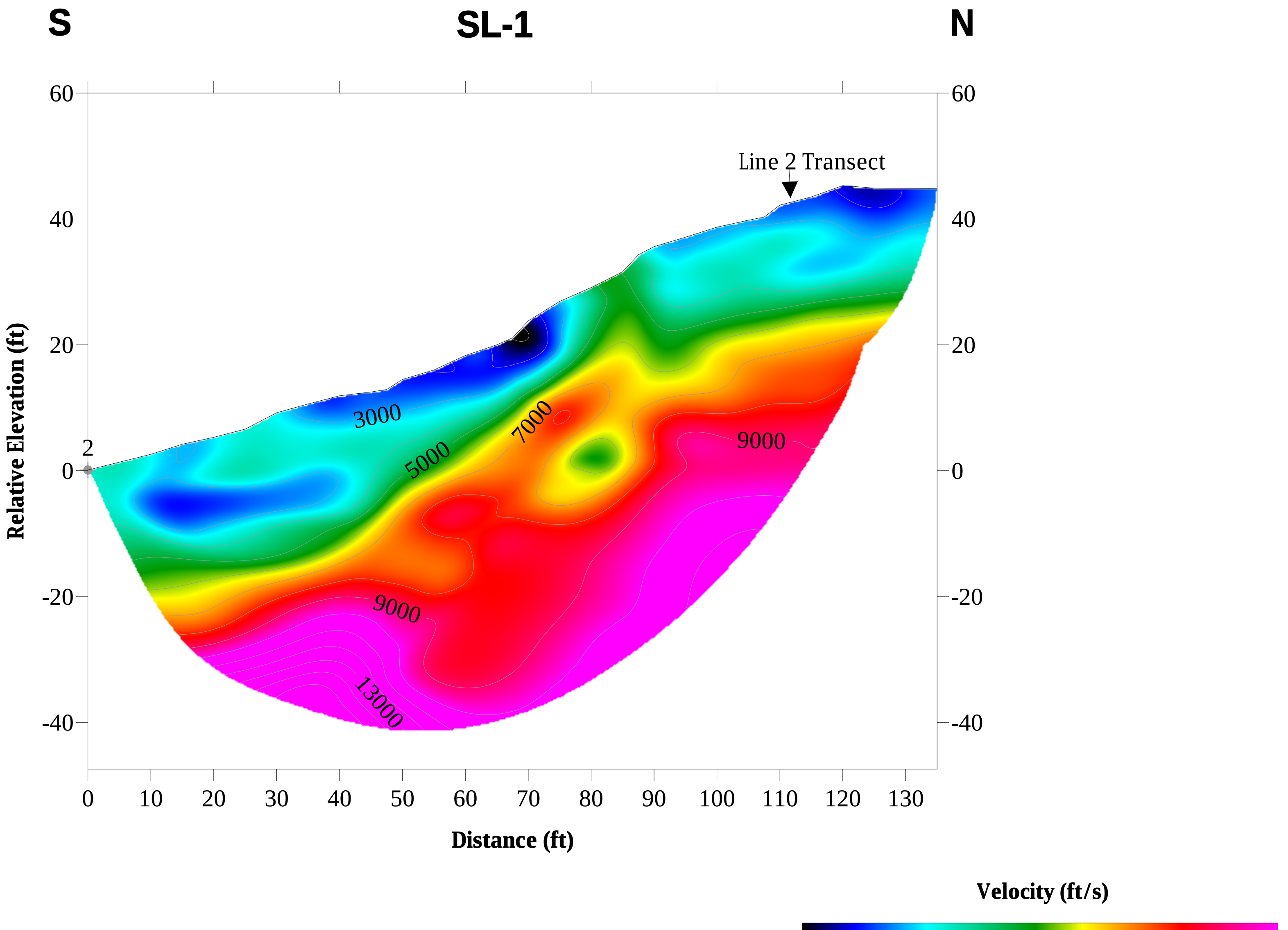
0 25 50
approximate scale in feet





SITE PHOTOGRAPHS	Rainbow Municipal Water District Bonsal Pump Station Site Bonsal, California Project No.: 122230P.6SWG — Date: 08/22	ATLAS Figure 3

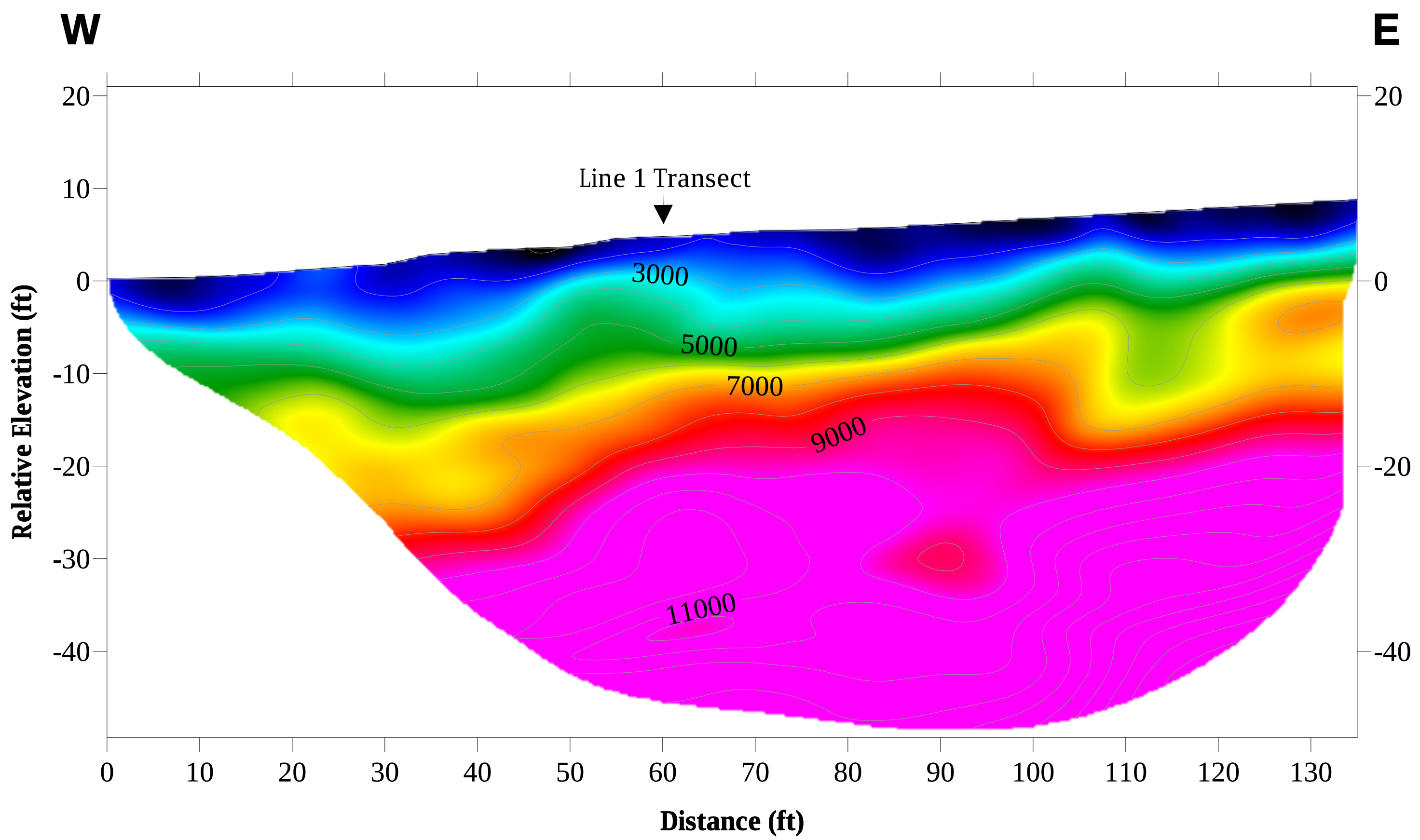
TOMOGRAPHY MODEL

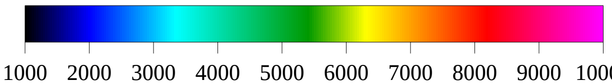


Note: Contour Interval = 1,000 feet per second

SEISMIC PROFILE SL-1	Rainbow Municipal Water District Bonsal Pump Station Site Bonsal, California		ATLAS Figure 4a
	Project No.: 122230.P6SWG	Date: 08/22	

TOMOGRAPHY MODEL
SL-2





Note: Contour Interval = 1,000 feet per second

SEISMIC PROFILE SL-2	Rainbow Municipal Water District Bonsal Pump Station Site Bonsal, California Project No.: 122230.P6SWG · Date: 08/22	ATLAS Figure 4b