# Appendix D

## Geotechnical Evaluation

# Geotechnical Evaluation

# Treated Water Facilities and Redundant Pipeline Project

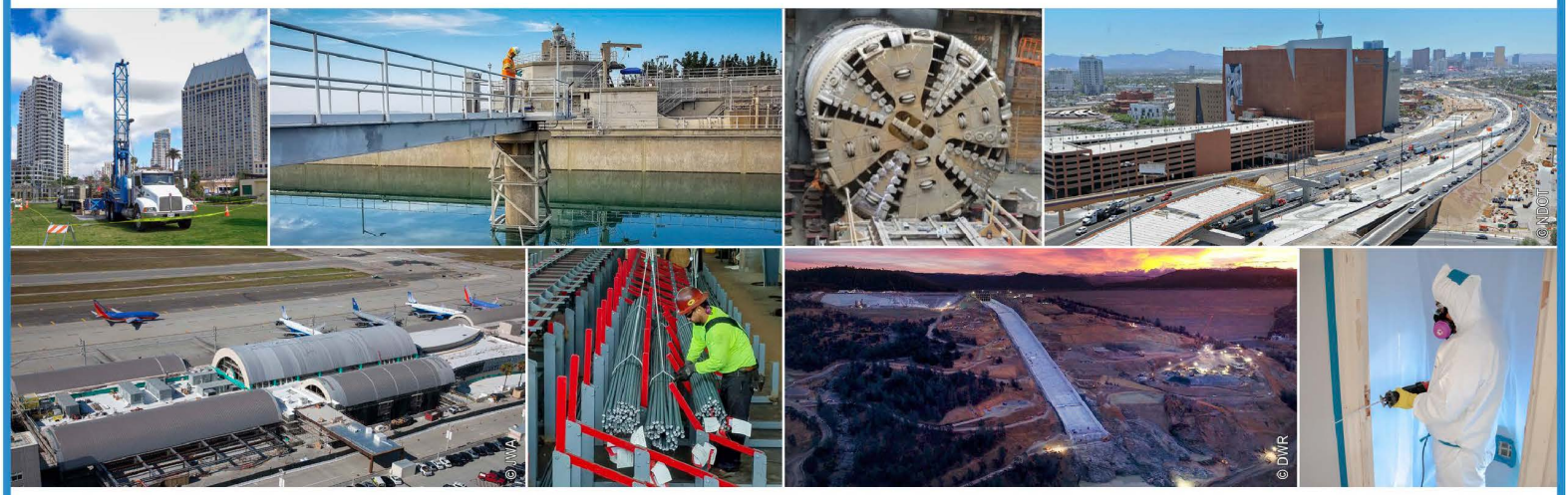
## Espola Road and Lake Poway Road

## Poway, California

Black & Veatch

300 Rancheros Drive, Suite 250 | San Marcos, California 92069

December 12, 2022 | Project No. 109471001

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# Geotechnical Evaluation
# Treated Water Facilities and Redundant Pipeline Project
## Espola Road and Lake Poway Road
## Poway, California

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# 1 INTRODUCTION

In accordance with your authorization, we have performed a geotechnical evaluation for the City of Poway Treated Water Facilities and Redundant Pipeline project located at the Berglund Water Treatment Plant and along Espola Road and Lake Poway Road in Poway, California (Figure 1). The objectives of this study were to assess the soil conditions at the site, evaluate the engineering properties of the soils encountered, and provide recommendations relative to the geotechnical aspects of the proposed project. This report presents the results of our field explorations and laboratory testing, as well as our conclusions regarding the geotechnical conditions at the site and our recommendations for the design and construction of this project.

# 2 SCOPE OF SERVICES

The scope of services for this evaluation included the following:

- Reviewing background information, including geologic and fault maps, topographic maps, groundwater data, and available published and in-house geotechnical literature pertaining to the project area.
- Obtaining a Right-Of-Way permit from the City of Poway, as well as a boring permit from the County of San Diego Department of Environmental Health & Quality (DEHQ).
- Performing a geologic reconnaissance of the site to observe the general site conditions, as well as marking the field explorations for utility clearance by Underground Service Alert (USA).
- Performing a seismic refraction survey consisting of one seismic refraction line at the site in the area of the proposed pump station and forebay tank to evaluate the rippability (excavatability) of bedrock materials.
- Performing a subsurface exploration consisting of the following:
    - Drilling, logging, and sampling of three small diameter soil borings in the areas of the proposed improvements. The soil borings were drilled to depths of up to approximately 19.3 feet using a truck-mounted drill rig equipped with hollow-stem augers.
    - Drilling, logging, and sampling of two HQ (2½-inch diameter) rock core borings in the areas of the proposed improvements. The rock core borings were advanced to depths of approximately 30 feet using a truck-mounted drill rig.
    - Excavating, logging, and sampling of one test pit excavation in the area of the proposed pump station and forebay tank. The test pit was excavated utilizing a rubber-tired backhoe to a depth of approximately 17 feet.
    - Logging of the explorations was performed by a representative from Ninyo & Moore. Relatively undisturbed and bulk soil samples were obtained at selected intervals from the explorations. The collected samples were transported to our in-house geotechnical laboratory for testing.

- Performing geotechnical laboratory testing on representative samples to evaluate their pertinent soil characteristics. Additionally, we utilized a third-party laboratory testing firm to perform laboratory compression and splitting tensile testing of rock cores obtained during our subsurface exploration.

- Compiling and performing an engineering analysis of the information obtained from our background review, subsurface exploration, and laboratory testing.

- Preparing this report presenting our findings, conclusions, and recommendations regarding the geotechnical aspects of the design and construction of the project.

# 3 SITE AND PROJECT DESCRIPTION

The proposed improvements and pipeline alignments are located at the Berglund Water Treatment Plant and along Espola Road and Lake Poway Road in Poway, California (Figures 1 and 2). The project consists of construction of a new booster pump station and forebay tank, and installation of a new 36-inch, treated water pipeline. The proposed pipeline alignment is along Espola Road, Lake Poway Road, and on the City of Poway water treatment plant (Figure 2). Surface topography in the project area range from approximately 703 feet above mean sea level (MSL) at the northern end of the proposed alignment to approximately 843 feet above MSL at the eastern end of the proposed alignment.

Based on our review of the Request for Proposal (RFP No. 22-006) dated September 2021, we understand that the City of Poway plans to construct a new pump station building, a new 323,000-gallon forebay tank, and 5,150 linear feet (LF) of new 36-inch diameter pipeline. The existing pump station facility will be demolished as part of the project, and the proposed pump station and forebay tank will be constructed in the lower parking lot west of the water treatment plant. Approximately 850 LF of pipeline will be installed on the water treatment plant property to connect the Clearwell Bypass Pipeline to the new pump station/forebay tank, and 4,300 LF of pipeline will be installed along Lake Poway Road and Espola Road to connect the new pump station and forebay tank to the San Diego County Water Authority (SDCWA) POW-5 Flow Control Facility. A geotechnical evaluation for the flow control facility is being concurrently performed under separate cover by others. The proposed pipeline alignment will cross the SDCWA aqueduct at two locations, one crossing on Lake Poway Road and the other crossing on Espola Road.

Ninyo & Moore has performed numerous geotechnical evaluations at the Berglund Water Treatment Plant pertaining to the proposed new Clearwell, the Clearwell Bypass Pipeline, and other water treatment pipelines and facilities. One previous geotechnical evaluation (Ninyo & Moore, 2010) was performed for the proposed Clearwell Redundant Pipeline project

and included the performance of five soil borings and five seismic refraction lines. The portion of the current proposed pipeline alignment on the water treatment plant property and along Lake Poway Road is similar to the alignment evaluated in our report. The previous evaluation included drilling three borings within the water treatment property and two borings along the south side of Lake Poway Road. Additionally, five seismic refraction surveys were located along the south side of the Lake Poway Road in the vicinity of the SDCWA aqueduct. The results of our previous evaluations are incorporated into this evaluation, as appropriate.

# 4 FIELD EXPLORATIONS

Our field exploration activities for this evaluation included performance of a subsurface exploration program and seismic refraction survey. Descriptions of these tasks are provided in the following sections.

## 4.1 Subsurface Evaluation and Laboratory Testing

Our subsurface exploration was performed between August 11 and August 22, 2022 and consisted of the exploration, logging, and sampling of three small-diameter soil borings (B-1 through B-3), two HQ (2½-inch diameter) rock core borings (RC-1 and RC-2), and one test pit excavation (TP-1). Prior to commencing the subsurface exploration, USA was notified to markout the existing utilities. The purpose of the explorations was to evaluate subsurface conditions and to collect samples for geotechnical laboratory testing.

The soil borings were drilled to depths of up to approximately 19.3 feet using a truck-mounted drill rig equipped with hollow-stem augers. The rock core borings were drilled to depths of approximately 30 feet using a truck-mounted drill rig. The test pit was excavated utilizing a rubber-tired backhoe to a depth of approximately 17 feet. The explorations were logged and sampled by Ninyo & Moore personnel. Representative bulk and relatively undisturbed soil samples were obtained at selected depths within the explorations and were then transported to our in-house geotechnical laboratory for analysis. The excavation logs are presented in Appendix A. The exploration locations are presented on Figure 2. As part of this evaluation we have reviewed our previous geotechnical evaluation for the Clearwell Redundant Pipeline project (Ninyo & Moore, 2010). Information from this evaluation has been incorporated where appropriate. Borings logs from the previous evaluation are presented in Appendix B.

Geotechnical laboratory testing was performed on representative soil samples collected from our subsurface exploration. Testing included an evaluation of in-situ dry density and moisture content, sieve (gradation) analysis, shear strength, expansion index, soil corrosivity, and

R-value. Additionally, we utilized a third-party laboratory testing subcontractor to perform laboratory compression and tensile testing of rock cores obtained during our subsurface exploration. The results of the in-situ dry density and moisture content tests are presented at the corresponding depths on the excavation logs presented in Appendix A. The results of the other laboratory tests and a description of the test methods used are presented in Appendix C.

## 4.2 Seismic Refraction Survey

As part of our scope of services, we performed a geophysical evaluation, which consisted of performing one seismic refraction survey line that was approximately 161 feet in length. The seismic refraction survey line was conducted to evaluate the approximate depth to, and rippability (excavatability) of, the granitic bedrock in the general in the vicinity of the proposed pump station and forebay tank. The location of the seismic refraction line is shown on Figure 2. Further description of the methodology and the results of our geophysical survey are presented in the following sections.

### 4.2.1 Methods and Equipment

Ninyo & Moore personnel collected seismic refraction data along one survey line, denoted as SL-1 (Figure 2). The location of the seismic refraction survey was dependent on the field conditions encountered. The seismic data was collected using a 24-channel Geode exploration seismograph coupled with 24 vertical component, 14 Hertz geophones. A 16-pound hammer and aluminum plate were used as the energy source for the seismic refraction survey. Field data acquisition included stacking multiple shots at each shot location in order to increase the quality of the data and reduce noise. The seismic refraction method uses recognition of first-arrival times of refracted seismic waves in units of milliseconds to evaluate the thickness and seismic velocities of subsurface layers. Seismic waves generated by the hammer impacting the ground surface at a given “shot” point are refracted at boundaries separating materials of contrasting material velocities. These refracted seismic waves are then detected by a series of surface geophones and recorded with a seismograph. Each hammer shot is recorded as time zero. The elapsed time, in milliseconds, that the seismic compressional wave (P-wave) signals take to travel to each geophone is recorded through the record length. This information is used in conjunction with the known shot-to-geophone horizontal distances to obtain the approximate thickness and velocity information about the subsurface materials.

The refraction method generally necessitates that subsurface velocities (and therefore material densities and modulus of rigidity) increase with depth. A layer having a velocity lower than that

of the layer above it will not be detectable by the seismic refraction method and, therefore, could lead to errors in the depth calculations of subsequently deeper layers. This is known as a "velocity inversion" problem. In addition, relatively significant lateral variations in velocity, such as those which occur at open, or tightly spaced jointed or fractured rock, discontinuous caliche deposits, and nested subsurface cobbles and boulders, can also result in misinterpretation of the subsurface conditions when using this method.

### 4.2.2 Velocities and Rippability

In general, seismic wave velocities can be correlated to material density and/or rock hardness. The relationship between rippability and seismic velocity is empirical and assumes a homogenous mass for each detected layer. Possible areas of differing composition, texture, or structure may affect both the measured data and the actual rippability of the mass. The rippability of a mass is also dependent on the excavation equipment used and the skill and experience of the equipment operator. The rippability characteristics in Table 1 are based on our experience with similar materials. The rippability of a particular material 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 soil characteristics can play a significant role in estimating excavation rates and rippability. In addition, where excavations encounter or penetrate weathered, fresh bedrock or cemented bedrock, the degree of weathering, degree of cementation (if any), or the presence or absence of fractures and/or joints, and fracture/joint spacing and orientation, also play a significant role in evaluating rock rippability. These soil and rock characteristics may also vary with location and depth. Our evaluation did not characterize the presence and nature of bedrock fractures, joints, or bedding planes which can have an effect on rippability rate and excavation efforts.

**Table 1 – Qualitative Rippability Classification**

| P-Wave Velocity Range (Feet/Second) | Qualitative Rippability |
|---|---|
| 0 to 2000 ft/s | Easy Ripping |
| 2000 to 4000 ft/s | Moderate Ripping |
| 4000 to 5500 ft/s | Difficult Ripping, Possible Blasting |
| 5500 to 7000 ft/s | Very Difficult Ripping, Probable Blasting |
| Greater than 7000 ft/s | Blasting Generally Needed |

It should be noted that the rippability estimates presented in Table 1 are slightly more conservative than those published in Edition 49 of the Caterpillar Performance Handbook (Caterpillar, 2019). 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 onsite materials prior to submitting their bids. It should also be noted that, as a general rule of thumb, the effective depth of evaluation for a seismic refraction traverse is approximately one-third to one-fifth the length of the refraction line.

### 4.2.3 Data Analysis

The obtained refraction data were processed using SeisImager processing software. Initially, data were grouped as all shot points in each line and first arrival picks were made manually in PickwinTM v. 7.1.0.1. Once data were grouped as first arrival picks, a travel time curve was constructed and calculations were performed to derive approximate minimum and maximum velocities. Relative topography data collected in the field were incorporated with travel time data to account for topography effects. An initial velocity model of the surface was developed for each profile using a delay-time technique and velocity inversion in PlotrefaTM v. 5.0.0.4. These models were used for each profile to develop a more detailed tomographic profile depicting approximate lateral and vertical changes in P-wave velocity across each seismic line.

### 4.2.4 Seismic Results and Conclusions

The tomographic models of our profile generally indicate surficial fill soils and colluvium underlain by granitic bedrock. The profiles indicate some amount of undulations in the interpreted layers. This could be the result of varying amounts of fill soils, colluvium, and varying degrees of granitic bedrock weathering. The velocity profile of seismic line SL-1 is presented in Appendix D. Table 2 summarizes the findings of the seismic velocity survey.

**Table 2 – Refraction (P) Wave and Shear (S) Wave Velocity Survey Results**

| Location | Depth (ft) | Approximate P-wave Velocity Range (ft/s) | Comments |
|---|---|---|---|
| SL-1 | 0 – 5 | 2,000-4,250 | Easy Ripping to Difficult Ripping, Possible Blasting |
| | 5 – 10 | 2,000-4,500 | Easy Ripping to Difficult Ripping, Possible Blasting |
| | 10 – 15 | 2,250-5,000 | Moderate Ripping to Difficult Ripping, Possible Blasting |
| | 15 – 20 | 3,750-6,250 | Moderate Ripping to Very Difficult Ripping, Probable Blasting |
| | 20+ | 5,250+ | Very Difficult Ripping, Probable Blasting to Blasting Generally Needed |

In general, the seismic refraction surveys performed in the lower parking lot (SL-1) indicate that the easy ripping conditions in the upper 3 feet with moderate ripping conditions expected between 3 to 10 feet. Areas of difficult to very difficult ripping, with a possible/probable need for rock breaking and/or coring, are anticipated to be encountered in excavations extending below 10 feet. Variability in rippability of subsurface materials situated at lateral distances away from seismic traverse, including within the proposed pump station and forebay tank footprints, can be expected.

As noted, our previous evaluation at the site included the performance of five seismic refraction lines in the vicinity of the SDCWA aqueduct on Lake Poway Road. The results of our previous seismic refraction survey are included in Appendix E. Additionally, two seismic refraction surveys were performed by others (Atlas, 2022) as part of the geotechnical evaluation of the proposed flow control facility at the northern end of the redundant pipeline alignment. The results of the seismic refraction survey for the proposed flow control facility are included in Appendix F.

# 5 GEOLOGY

Our findings regarding regional and site geology at the project location are provided in the following sections.

## 5.1 Regional Geology

The project area is situated in the western portion of the Peninsular Ranges Geomorphic Province. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California (Norris and Webb 1990; Harden, 2004). The province varies in width from approximately 30 to 100 miles and generally consists of rugged mountains underlain by Jurassic metavolcanic and metasedimentary rocks, and Cretaceous igneous rocks of the southern California batholith (Figure 3). A geologic cross section of the pump station and forebay tank site is shown on Figure 4.

The Peninsular Ranges Province is traversed by a group of sub-parallel faults and fault zones trending roughly northwest (Jennings, 2010). Several of these faults are considered to be active. The Elsinore, San Jacinto, and San Andreas faults are active fault systems located northeast of the project area and the Rose Canyon, Coronado Bank, San Diego Trough, and San Clemente faults are active faults located west of the project area (Figure 5). Major tectonic activity associated with these and other faults within this regional tectonic framework consists primarily

of right-lateral, strike-slip movement. The Rose Canyon Fault Zone, the nearest active fault system, has been mapped approximately 16.5 miles west of the project site.

## 5.2 Site Geology

The project site is mapped as being underlain by young colluvium, old alluvium, and Cretaceous-aged granite (Kennedy and Tan, 2007; Figure 3). Geologic units encountered during our subsurface exploration, as well as our previous geotechnical exploration at the site (Ninyo & Moore, 2010), included fill, colluvium, old alluvium, and granitic rock. Generalized descriptions of the encountered pavement sections and earth units encountered during our field reconnaissance and subsurface exploration are provided in the subsequent sections. Additional descriptions of the subsurface units are provided on the logs of the excavations in Appendix A.

### 5.2.1 Encountered Pavements

Pavement sections consisting of asphalt concrete (AC) underlain by aggregate base materials were encountered in each exploration location during our subsurface exploration. The aggregate base materials encountered underlying the AC generally consisted of brown and light brown, dry to moist, medium dense, silty gravel. AC was not encountered in boring RC-1. Table 3 summarizes the pavement sections encountered during our subsurface exploration.

**Table 3 – Encountered Pavement Section Thicknesses**

| Exploration Location | Encountered AC Thickness (inches) | Encountered Aggregate Base Thickness (inches) |
|---|---|---|
| B-1 | 6 | 12 |
| B-2 | 5 | 13 |
| B-3 | 3½ | 8½ |
| RC-1 | 4 | 5 |
| RC-2 | -- | 6 |
| TP-1 | 3½ | 3 |

**Note:**
AC = asphalt concrete
-- = not encountered

### 5.2.2 Fill

Fill material was encountered underlying the aggregate base materials in each exploration location (except boring B-3) during our subsurface exploration. The fill soils extended to depths of up to approximately 5½ feet. As encountered, the fill material consisted of various shades of brown, moist, medium dense, clayey and silty sand. The previous evaluation (Ninyo & Moore, 2010) encountered fill materials extending to depths up to 10 feet below

the ground surface which generally consisted of moist, loose to dense, silty sand to poorly graded sand with silt. Scattered amounts of gravel were encountered in the fill soils. Documentation regarding fill placement was not available for our review.

### 5.2.3 Young Colluvium

Young colluvium was encountered underlying the aggregate base material in boring B-3, and underlying the fill soils in excavation TP-1, and extended to depths of up to approximately 7½ feet. As encountered, materials of the young colluvium generally consisted of gray, moist, medium dense, clayey and silty sand. Scattered amounts of gravel were encountered in the colluvium. Colluvium was not encountered in our previous evaluation (Ninyo & Moore, 2010).

### 5.2.4 Old Alluvium

Materials described as old alluvium was encountered in our previous evaluation (Ninyo & Moore, 2010) underlying the fill material in boring B-3 (2010) and extended to the total depth explored of 16½. As encountered, these materials generally consisted of reddish brown and grayish olive, medium dense to very dense, silty to clayey sand. Scattered amounts of gravel and cobbles were encountered in the old alluvium. Additionally, the portion of the project near the intersection of Espola Road and Lake Poway Road is mapped as being underlain by older alluvium deposits (Kennedy and Tan, 2007; Figure 3). While not encountered in borings along the proposed pipeline alignment, the old alluvium should be anticipated to be encountered during construction.

### 5.2.5 Granitic Rock

Granitic rock was encountered underlying the fill materials and colluvium at each exploration location and at the surface or underlying the fill soils or pavement section in previous borings performed at the site (Ninyo & Moore, 2010). In general, granitic rock was encountered at a depth of approximately 5½ feet in borings along Espola Road, depths between approximately 0 to 5 feet along Lake Poway Road, and depths between approximately 1½ to 10 feet in the lower parking lot and water treatment plant property. Drilling and/or excavating refusal was encountered on granitic rock at depths ranging between approximately 8 to 17 feet. Rock coring was performed at the proposed SDCWA aqueduct crossings and advanced to depths of up to 30 feet below the ground surface. Where encountered, the materials generally consisted of light gray and light brown, dry to moist, decomposed to fresh, very soft to very hard, coarse-grained granitic rock. As encountered in the previous evaluation (Ninyo & Moore, 2010)

the granitic rock generally consisted of various shades of brown and gray, damp to saturated, weathered to very hard granitic rock.

## 5.3 Groundwater

Groundwater seepage was encountered during our subsurface evaluation at depths of approximately 15¼ (TP-1) and 18½ (B-3) feet. Groundwater levels can fluctuate due to seasonal variations, groundwater withdrawal or injection, and other factors. Additionally, perched water conditions may be present at the site due to the geologic contact with the granitic rock, and the presence of trench backfill and bedding materials for underground utilities, as these materials tend to act as a conduit for perched water conditions.

A temporary groundwater monitoring well was installed in our boring RC-2 for monitoring of groundwater levels to aid in the design of the project. The construction of the well consisted of the installation of a 2-inch diameter PVC pipe within the borehole and a concrete well mount constructed at the surface. RC-2 was constructed with 25 feet of perforated well pipe at the bottom with sand to a foot above the perforation, and hydrated bentonite to a foot below the surface. The RC-2 location included fill soils overlying granitic rock and was drilled to a depth of 11 feet using hollow stem augers with no indication of groundwater seepage. Upon switching over to HQ rock coring the boring was filled with drilling fluid and depth to groundwater and/or seepage was not able to be determined during the drilling process. Based on ongoing monitoring performed between 9/1/22 and 11/1/22, groundwater at this location has been measured at depths ranging between 13.5 and 14.7 feet.

Potential sources for the encountered seepage may include irrigation from landscaped areas at Lake Poway Park and/or nearby residential properties, lateral seepage from Lake Poway, leaking conduits in the vicinity of the site, and other sources. General groundwater conditions will be further evaluated following review of the temporary monitoring well data.

## 5.4 Faulting and Seismicity

Based on our review of the referenced geologic maps and stereoscopic aerial photographs, the site is not underlain by known active or potentially active faults (i.e., faults that exhibit evidence of ground displacement in the last 11,000 years and 2,000,000 years, respectively). The site is not located within a State of California Earthquake Fault Zone (EFZ) (formerly known as an Alquist-Priolo Special Studies Zone) (Hart and Bryant, 2007). However, like the majority of Southern California, the site is located in a seismically active area and the potential for strong ground motion is considered significant during the design life of the proposed improvements.

Figure 4 shows the approximate site location relative to the major faults in the region. The nearest known active fault is the Rose Canyon fault, located approximately 16.5 miles west of the site (USGS, 2022).

### 5.4.1 Surface Ground Rupture

Based on our review of the referenced literature and our site reconnaissance, active faults are not known to cross the project vicinity. Therefore, the potential for ground surface rupture due to faulting at the site is considered low. However, lurching or cracking of the ground surface as a result of nearby seismic events is possible.

### 5.4.2 Strong Ground Motion

The 2019 California Building Code (CBC) specifies that the potential for liquefaction and soil strength loss be evaluated, where applicable, for the Maximum Considered Earthquake Geometric Mean ($MCE_G$) peak ground acceleration with adjustment for site class effects in accordance with the American Society of Civil Engineers (ASCE) 7-16 Standard. The $MCE_G$ peak ground acceleration is based on the geometric mean peak ground acceleration with a 2 percent probability of exceedance in 50 years. The MCEG peak ground acceleration with adjustment for site class effects ($PGA_M$) was calculated as 0.42g using web-based seismic design tool (SEAOC/OSHPD, 2022) that yielded a mapped $MCE_G$ peak ground acceleration of 0.35g for the site and a site coefficient ($F_{PGA}$) of 1.2 for a Site Class C.

### 5.4.3 Liquefaction and Seismically Induced Settlement

Liquefaction of cohesionless soils can be caused by strong vibratory motion due to earthquakes. Research and historical data indicate that loose granular soils and non-plastic silts that are saturated by a relatively shallow groundwater table are susceptible to liquefaction. Based on the dense nature of the underlying granitic rock materials, along with the observed lack of a static groundwater surface, the potential for liquefaction and seismically induced settlement is not a design consideration for the project.

### 5.4.4 Landslides

Landslides, slope failures, and mudflows of earth materials generally occur where slopes are steep and/or the earth materials are too weak to support themselves. Based on the relatively level topography of the site, the potential for landslides or slope failure to affect the project site is considered low.

### 5.4.5 Tsunamis

Tsunamis are long wavelength seismic sea waves (long compared to the ocean depth) generated by sudden movements of the ocean bottom during submarine earthquakes, landslides, or volcanic activity. Based on the inland location and elevation of the site, the potential for a tsunami to affect the site is not a design consideration.

# 6 CONCLUSIONS

Based on the results of our geotechnical evaluation, the following preliminary conclusions are provided for the proposed project:

- The areas of the proposed improvements are generally underlain by fill soils, colluvium, old alluvium and granitic rock. Fill was encountered in our borings (and previous evaluations) to depths up to approximately 10 feet and colluvium was encountered to depths up to approximately 7½ feet.

- Groundwater seepage was encountered during our subsurface exploration in our excavations in the lower parking lot in the vicinity of the proposed pump station and forebay tank at depths of approximately 15¼ to 18½ feet. Additionally, a groundwater monitoring well installed on the south side of Lake Poway Road in the vicinity of the SDCWA aqueduct crossing encountered groundwater at depths ranging between approximately 13.5 and 14.7 feet.

- The existing fill, colluvium, and alluvium encountered onsite should be generally excavatable with heavy-duty earth moving equipment in good working condition. However, scattered amounts of gravel and cobbles were encountered in our excavations. Due to the presence of gravel and cobbles within the fill, colluvium, and alluvium soils onsite, the contractor should anticipate encountered difficulties when performing onsite excavations.

- Excavations that extend into the granitic rock will encounter very difficult excavation characteristics and additional efforts including heavy ripping and/or rock breaking should be anticipated. Based on the presence of unweathered rock/corestones, as well as gravel and cobble, processing of the onsite soils (including screening and/or crushing) should be anticipated.

- Soils derived from onsite excavations are generally considered suitable for reuse as trench zone backfill, provided they are processed to meet the recommendations of this report. However, excavations into the granitic rock will likely generate oversized materials that are greater than 4 inches in diameter and not suitable for reuse in engineered fills unless crushing to less than 4-inch diameter is performed.

- There are no known active faults or potentially active faults crossing at the site, and the potential for surface ground rupture is considered low.

- Based on the results of our geotechnical laboratory testing presented in Appendix C, as compared to the Caltrans corrosion criteria (Caltrans, 2021), the onsite soils would be considered non-corrosive.

# 7 RECOMMENDATIONS

Based on our understanding of the project, the following recommendations are provided for the design and construction of the project. The proposed site improvements should be constructed in accordance with the requirements of the applicable governing agencies.

## 7.1 General

It is our understanding that the majority of the proposed pipeline will be installed using conventional cut-and-cover methods. We also understand that portions of the pipeline will be installed below the SDCWA aqueduct using jack-and-bore or other trenchless methods. As noted above, there is a potential for less weathered, resistant “corestones” to be present within the granitic rock at the site. These “corestones” can affect trenchless operations in that they can deflect the trenchless drill bit and casing during the drilling of the pilot hole. This can result in the trenchless portion of the pipeline to be installed at an orientation/inclination that can deviate from that which is planned. It has been our experience that microtunneling methods, which utilize the advancement of a small-scale tunnel boring machine with a cutting bit, can be advanced through such obstructions as “corestones”, thus reducing the potential for deviation from the pipeline’s design orientation.

It is anticipated that the existing underground utilities in the project area are sensitive to excessive external pressures. Because trenchless methods may incorporate the drilling of a pilot hole and the subsequent jacking of pipe (larger in diameter than the pilot hole) through the hole, this method could impose additional earth pressures on nearby buried structures. Microtunneling methods, which utilize the advancement of a tunnel boring machine and the subsequent installation of casing, can be expected to exert less pressure on nearby structures during pipeline installation.

We consider both jack-and-bore and microtunneling to be viable methods to install the City of Poway treated water pipelines under the SDCWA aqueduct. However, considerations associated with the possible presence of “corestones” and the possibility of inducing added external pressures on the existing utilities suggest that jack-and-bore methods may be a more complicated procedure. Consequently, we recommend that consideration be given to the use of microtunneling over the use of jack-and-bore methods to install the pipeline crossings, particularly if portions of the pipeline may be installed within granitic rock.

We recommend that the selected method (i.e. jack-and-bore or microtunneling) be performed by a specialty contractor experienced in conditions similar to those that exist at the site, and that he/she is aware of the above considerations. In light of the above information, the following sections provide geotechnical recommendations for the conventional cut-and-cover pipeline

installation, along with those relating to design and construction of the pipeline crossings beneath the SDCWA aqueduct using both jack-and-bore and microtunneling methods.

## 7.2 Earthwork

In general, earthwork should be performed in accordance with the recommendations presented in this report. Ninyo & Moore should be contacted for questions regarding the recommendations or guidelines presented herein.

### 7.2.1 Site Preparation

Prior to performing site excavations, the project alignment should be cleared of vegetation, surface obstructions, rubble and debris, abandoned utilities and foundations, and other deleterious materials. Existing utilities within the project limits, if any, should be re-routed or protected from damage by construction activities. Obstructions that extend below finish grade, if any, should be removed and the resulting holes filled with compacted soils. Materials generated from the clearing operations should be removed from the project site and disposed of at a legal dumpsite.

### 7.2.2 Excavation Characteristics

The result of our field exploration program indicates that the project site is underlain by fill soils, colluvium, alluvium and granitic rock. Excavations of the fill soils, colluvium, and alluvium should be generally excavatable with heavy-duty earth moving equipment in good working condition. Excavations within upper portions of granitic rock can generally be expected to be accomplished with heavy-duty excavation equipment and drilling equipment in good operating condition. However, zones containing more resistant, less weathered rock and “corestones” should be anticipated in the granitic rock. Excavation in such materials may necessitate heavy ripping, rock breaking, or coring. Consequently, oversize cobbles and boulders could be generated during excavation within the fill, colluvium, old alluvium, and granitic rock. Disposal and/or crushing of the resulting oversized material should be anticipated. Section 4.2.4 of this report provides further discussion of anticipated rippability of the subsurface materials in the vicinity of the proposed improvements. Additionally, due to the granular nature of the fill soils, colluvium, and old alluvium, and potential for groundwater seepage, the following ground conditions should be anticipated:

- Caving soils;
- Flowing sands; and
- Sloughing of excavations;

Recommendations for shoring and dewatering are presented in the following sections.

### 7.2.3 Temporary Excavations and Shoring

For temporary excavations, we recommend that the following Occupational Safety and Health Administration (OSHA) soil classifications be used:

| | |
|---|---|
| *Fill, colluvium, and old alluvium* | *Type C* |
| *Granitic Rock* | *Type A* |

Upon making the excavations, the soil classifications and excavation performance should be evaluated in the field by the geotechnical consultant in accordance with the OSHA regulations. Temporary excavations should be constructed in accordance with OSHA recommendations. For trenches or other excavations, OSHA requirements regarding personnel safety should be met using appropriate shoring (including trench boxes) or by laying back the slopes to no steeper than 1.5:1 (horizontal to vertical) in the fill, colluvium, and old alluvium. A ¾:1 temporary slope may be constructed in granitic rock. Temporary excavations that encounter seepage may be shored or stabilized by placing sandbags or gravel along the base of the seepage zone. Excavations encountering seepage should be evaluated on a case-by-case basis. On-site safety of personnel is the responsibility of the contractor.

We understand a shoring system may be incorporated to support the excavation sidewalls and protect existing utilities during construction. Shoring design parameters are presented below. Particular caution should be used near existing underground utilities. Existing utilities that are located within a zone based on a 1:1 plane projected up from the bottom outside edge of the new trench should be considered at risk. The type of shoring and sequence of shoring installation should be designed to protect utilities and surface improvements.

Lateral earth pressures exerted on restrained shoring systems are indicated on Figure 6. The recommended design earth pressures are based on the assumptions that: (a) the shoring system is constructed without raising the ground surface elevation behind the shoring, (b) there are no surcharge loads, such as soil stockpiles, construction materials, or vehicular traffic, and (c) no loads act above a 1:1 plane extending up and back from the base of the shoring system. For shoring subjected to the above-mentioned surcharge loads, the contractor should include the effect of these loads on lateral earth pressures acting on the shoring wall.

Settlement of the ground surface may occur behind the shoring wall system during excavation. The amount of settlement depends on the type of shoring system, the quality of contractor's workmanship, and soil conditions. Settlement may cause distress to adjacent

structures, if present. To reduce the potential for distress to adjacent structures, we recommend that the shoring system be designed to limit the ground settlement behind the shoring to ½ inch or less. Possible causes of settlement that should be addressed include vibration during installation of the sheet piling, excavation for construction, construction vibrations, dewatering, and removal of the support system. We recommend that the potential settlement distress be evaluated carefully by the contractor prior to construction.

The contractor should retain a qualified and experienced engineer to design the shoring system. The shoring parameters presented in this report are for preliminary design purposes and the contractor should evaluate the adequacy of these parameters and make appropriate modifications for their design. We recommend that the contractor take appropriate measures to protect workers. OSHA requirements pertaining to worker safety should be observed. We further recommend that the construction methods provided herein be carefully evaluated by a qualified specialty contractor prior to commencement of the construction.

### 7.2.4 Remedial Grading – Pump Station

We anticipate that shallow foundations will be used to support the proposed pump station. Due to the presence of existing fills, we recommend that the foundation subgrade soils for the pump station be removed down to a depth of 2 feet below the bottom of footings or to competent colluvial materials, whichever is deeper. This overexcavation should extend to the horizontal limits of the structural footprint (including foundations for attached overhangs, canopies, and other building appurtenances) plus a horizontal distance of 5 feet, where feasible. The extent and depths of removals and overexcavations should be evaluated by Ninyo & Moore's representative in the field based on the materials exposed. The resultant overexcavation surface should be scarified to a depth of approximately 8 inches, moisture conditioned and recompacted to a relative compaction of 90 percent as evaluated by the ASTM International (ASTM) Test Method D 1557 prior to placing new fill. The resulting excavation should then be backfilled with granular soils with a very low to low expansion potential (i.e., an expansion index [EI] of 50 or less that are placed at a relative compaction of 90 percent as evaluated by ASTM D 1557.

### 7.2.5 Remedial Grading – Forebay Tank

We recommend that the existing fill and colluvial soils within the forebay tank building pad be overexcavated down to competent bedrock materials consisting of granitic rock. In general, the depth of existing fill and colluvium materials encountered during our subsurface exploration was on the order of 8 feet below ground surface. The contractor should also be

prepared to remediate deeper fills where encountered as a result of past construction and grading activities not disclosed by our explorations. This overexcavation should extend to the horizontal limits of the structural footprint (including foundations) plus a horizontal distance of 5 feet, where feasible. The extent and depths of removals and overexcavations should be evaluated by Ninyo & Moore's representative in the field based on the materials exposed. The resulting overexcavation bottom removal surface that exposes competent granitic rock materials should be left generally undisturbed prior to placing new fill. The resulting excavation should then be backfilled with granular soils with a very low to low expansion potential (i.e., an expansion index [EI] of 50 or less) that are placed at a relative compaction of 95 percent as evaluated by ASTM D 1557.

### 7.2.6 Remedial Grading – Pavements

In proposed pavement areas, we recommend that the onsite soils be overexcavated to a depth of 1 foot below the planned subgrade elevation for the pavement. The proposed overexcavations should extend outward horizontally 2 feet from the exterior limits of the improvement. The extent and depth of removals should be evaluated by Ninyo & Moore's representative in the field based on the material exposed. The resulting surface should be scarified 8 inches, moisture conditioned, and recompacted to a relative compaction of 90 percent as evaluated by ASTM D 1557. The removals should then be filled with materials suitable for reuse as compacted fill. The compacted fill should be moisture conditioned to near optimum moisture content and compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557. In proposed vehicular pavement areas, the compacted fill should be moisture conditioned to near optimum moisture content and compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557.

### 7.2.7 Construction Dewatering

As noted previously, seepage was encountered in our borings at depths ranging of approximately 15¼ and 18½ feet. A monitoring well was installed at boring RC-2 (Figure 2) to further evaluate groundwater conditions prior to construction. Where groundwater, seepage, and/or perched water conditions are encountered, dewatering measures during excavation operations should be prepared by the contractor's engineer and reviewed by the design engineer. The appropriate dewatering method(s) should be evaluated by the contractor; however, we anticipate potential dewatering methods could include (but are not limited to) well points, as well as sumps and trenches. Considerations for construction dewatering should include anticipated drawdown, piping of soils, volume of pumping,

potential for settlement, and groundwater discharge. Evaluation of whether permanent dewatering systems will be need should be made based on ongoing readings at monitoring well RC-2. Disposal of groundwater should be performed in accordance with guidelines of the Regional Water Quality Control Board (RWQCB).

### 7.2.8 Excavation Bottom Stability

Excavation bottoms that expose granitic rock are anticipated to be generally stable and should provide suitable support to the proposed pipeline. However, if unstable excavation bottom conditions are exposed, they may be mitigated by overexcavating the excavation bottom to suitable depths and replacing with a layer of compacted ¾- to 1½-inch crushed gravel encased in a non-woven geotextile (e.g., Mirafi® 140N geotextile or an approved equivalent). Recommendations for stabilizing excavation bottoms should be based on evaluation in the field by the geotechnical consultant at the time of construction.

During excavation of granitic rock to the design bottom elevations, removal of larger rock fragments could result in an irregular surface along the excavation bottom that could contain voids and fractures. In these areas, the contractor can consider the placement of a 3-sack sand-cement slurry upon the bottom of the excavation in order to provide a suitable working surface for proposed improvements and associated earthwork.

### 7.2.9 Pipe Bedding and Modulus of Soil Reaction (*E'*)

We recommend that new pipelines, where constructed in an open excavation, be supported on 6 or more inches of granular bedding material. Granular pipe bedding should be provided to distribute vertical loads around the pipe. Bedding material should conform to the most recent edition of the Standard Specifications for Public Works Construction (Building News, 2018), and compaction requirements should be in accordance with the recommendations contained in this report. Pipe bedding typically consists of graded aggregate with a coefficient of uniformity of three or greater.

The modulus of soil reaction (E') is used to characterize the stiffness of soil backfill placed at the sides of buried flexible pipes for the purpose of evaluating deflection caused by the weight of the backfill over the pipe (Hartley and Duncan, 1987). A soil reaction modulus of 1,600 pounds per square inch (psi) may be used for an excavation depth of up to approximately 5 feet when backfilled with granular soil compacted to a relative compaction of 90 percent as evaluated by the ASTM D 1557. A soil reaction modulus of 2,200 psi may be used for trenches deeper than 5 feet.

### 7.2.10 Pipe Zone Backfill

The pipe zone backfill extends from the top of the pipe bedding material and continues to extend to 1 foot or more above the top of the pipe in accordance with the recent edition of the Standard Specifications for the Public Works Construction ("Greenbook"). Pipe zone backfill should have a Sand Equivalent (SE) of 30 or greater, and be placed around the sides and top of the pipe. Special care should be taken not to allow voids beneath and around the pipe. Compaction of the pipe zone backfill should proceed up both sides of the pipe.

It has been our experience that the voids within a crushed rock material are sufficiently large to allow fines to migrate into the voids, thereby creating the potential for sinkholes and depressions to develop at the ground surface. If open-graded gravel is utilized as pipe zone backfill, this material should be separated from the adjacent trench sidewalls and overlying trench backfill with a geosynthetic filter fabric.

### 7.2.11 Trench Zone Backfill

Based on our subsurface evaluation, granular onsite materials should be generally suitable for reuse as trench zone backfill, provided they do not contain rocks or lumps over 3 inches, and not more than 30 percent larger than ¾ inch. Larger chunks, if generated during excavation, may be broken into acceptably sized pieces or disposed of offsite. Trench zone backfill should be moisture-conditioned to generally at or slightly above the laboratory optimum. Trench zone backfill should be compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557, except for the upper 12 inches of the backfill beneath vehicular pavements that should be compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557. Lift thickness for backfill will depend on the type of compaction equipment utilized, but fill should generally be placed in lifts not exceeding 8 inches in loose thickness. Special care should be exercised to avoid damaging the pipe during compaction of the backfill.

### 7.2.12 Materials for Fill

In general, the existing onsite materials are considered suitable for reuse as fill, provided that the oversize materials (i.e., materials with dimensions in excess of those outlined herein) are removed from the soil mass prior to reuse. Fill material should be free of trash, debris or other deleterious materials. Material for use as fill should not contain rocks or lumps greater than approximately 4 inches in size.

Imported fill material, if used, should generally be granular soils with a very low expansion potential (i.e., an expansion index [EI] of 20 or less evaluated in accordance with ASTM D 4829. Import material should also be non-corrosive in accordance with the California amended (Caltrans, 2021) corrosion criteria. Ninyo & Moore should evaluate materials for use as fill prior to filling or importing.

### 7.2.13 Compacted Fill

Prior to placement of compacted fill, the contractor should request an evaluation of the exposed ground surface by the project geotechnical consultant. Unless otherwise recommended, the exposed ground surface should then be scarified to a depth of approximately 8 inches and watered or dried, as needed, to achieve moisture contents generally at or slightly above the optimum moisture content. The scarified materials should then be compacted to a relative compaction of 90 percent as evaluated in accordance with ASTM D 1557. The evaluation of compaction by the geotechnical consultant should not be considered to preclude any requirements for observation or approval by governing agencies. It is the contractor's responsibility to notify the geotechnical consultant and the appropriate governing agency when project areas are ready for observation, and to provide reasonable time for that review.

Fill materials should be moisture conditioned to generally at or slightly above the laboratory optimum moisture content prior to placement. The optimum moisture content will vary with material type and other factors. Moisture conditioning of fill soils should be generally consistent within the soil mass.

Prior to placement of additional compacted fill material following a delay in the grading operations, the exposed surface of previously compacted fill should be prepared to receive fill. Preparation may include scarification, moisture conditioning, and recompaction.

Compacted fill should be placed in horizontal lifts of approximately 8 inches in loose thickness. Prior to compaction, each lift should be watered or dried as needed to achieve a moisture content generally at or slightly above the laboratory optimum, mixed, and then compacted by mechanical methods, to a relative compaction of 90 percent as evaluated by ASTM D 1557. The upper 12 inches of street subgrade and aggregate materials beneath pavement areas should be compacted to a relative compaction of 95 Percent as evaluated by ASTM D 1557. Successive lifts should be treated in a like manner until the desired finished grades are achieved.

### 7.2.14 Trenchless Construction

We understand that trenchless construction methods are anticipated for the installation of the pipeline below the SDCWA aqueduct. Based on our understanding of the project and the materials encountered in our subsurface exploration, we anticipate that granitic rock may be encountered at the anticipated elevations of the proposed trenchless segments of the pipeline. The contractor should take appropriate measures to reduce the loss of material at the casing head. Depending on the depth of the access pits and seasonal variations in the groundwater elevation, the access pits may require dewatering during excavation. It should be anticipated that more resistant, unweathered zones may be present within the granitic rock and that excavation in such zones would necessitate heavy ripping, rock breaking, or coring. In addition, cemented zones could affect the installation of the jacked portions of the pipelines by deflecting the bore-and-jack equipment away from its design alignment. We recommend that an experienced specialty contractor be used for the trenchless construction operations.

Minor ground surface settlements may occur from the trenchless construction operations. However, these settlements are not anticipated to impact surface improvements and underground utilities, provided an experienced contractor performs the work. In order to evaluate the load factors on 36-inch and 48-inch sleeves for the proposed alignment, the loading presented in the following table should be used.

**Table 4 – Loading on Trenchless Segments of Pipeline**

| Approximate Depth from Existing Ground Surface to Top of Pipeline (feet) | Load on Sleeve (pounds/lineal foot of pipe) | |
|---|---|---|
| | 36” Diameter | 48” Diameter |
| 5 | 900 | 1,500 |
| 10 | 1,400 | 2,450 |
| 15 | 1,650 | 3,050 |
| 20 | 1,800 | 3,450 |
| 25 | 1,900 | 3,700 |
| 30 | 1,950 | 3,850 |

**Note:**
Linear interpolation may be used to obtain loading between the depths shown. Loading may need to be modified for different sleeve sizes.
Reference: McCarthy, David F., 2002, Essentials of Soils Mechanics and Foundations: Basic Geotechnics, Prentice Hall, Sixth Edition.

### 7.2.15 Lateral Pressures for Thrust Blocks and Jacking

Thrust restraint for buried pipelines and lateral pressures for jacking may be achieved by transferring the thrust force to the soil outside the pipe through a thrust block. Thrust blocks

may be designed using the lateral passive earth pressures presented on Figure 7. Thrust blocks should be backfilled with granular backfill material and compacted in accordance with recommendations presented in this report.

## 7.3 Seismic Design Considerations

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 5 presents the site-specific spectral response acceleration parameters in accordance with the CBC (2019) guidelines.

**Table 5 – 2019 California Building Code Seismic Design Criteria**

| Site Coefficients and Spectral Response Acceleration Parameters | Values |
|---|---|
| Site Class | C |
| Mapped Spectral Response Acceleration at 0.2-second Period, $S_s$ | 0.823g |
| Mapped Spectral Response Acceleration at 1.0-second Period, $S_1$ | 0.303g |
| Site-Specific Spectral Response Acceleration at 0.2-second Period, $S_{MS}$ | 0.988g |
| Site-Specific Spectral Response Acceleration at 1.0-second Period, $S_{M1}$ | 0.454g |
| Site-Specific Design Spectral Response Acceleration at 0.2-second Period, $S_{DS}$ | 0.659g |
| Site-Specific Design Spectral Response Acceleration at 1.0-second Period, $S_{D1}$ | 0.303g |
| Site-Specific Maximum Considered Earthquake Geometric Mean ($MCE_G$) Peak Ground Acceleration, $PGA_M$ | 0.424g |

## 7.4 Foundations

We anticipate that the new pump station will be founded on shallow foundations bearing on compacted fills and the new forebay tank will be founded on a mat foundation bearing on compacted fill or granitic rock. Foundations should be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the appropriate governing jurisdictions and applicable building codes should be considered in the design of the structures.

### 7.4.1 Conventional Shallow/Ring Wall Foundations

Continuous footings founded on competent colluvium materials or compacted fills may be designed using a net allowable bearing capacity of 3,000 pounds per square foot (psf). The net allowable bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic forces. Continuous footings should be founded 18 inches below the lowest adjacent grade. Continuous footings should have a width of

18 inches and column footings should have a width 24 inches. Footings should be reinforced in accordance with the recommendations of the project structural engineer.

### 7.4.2 Mat Foundations

A reinforced concrete mat foundation bearing upon compacted fill or granitic rock may be used to support the proposed forebay tank. From a geotechnical engineering standpoint, we recommend that the mat foundation have a thickness of 12 inches or more and be reinforced with No. 4 steel reinforcing bars placed 18 inches on-center (each way) in the middle one-third of the slab height.

The allowable contact pressure(s) beneath the base of the mat foundation will vary with its size and other factors. A net allowable bearing pressure of 3,000 psf may be assumed for the mat foundation supporting the tank and bearing upon weathered granitic rock in accordance with the above recommendations.

Mat foundations will typically experience some deflection due to loads placed on the foundation and the reaction of the soils underlying the mat. We recommend that a design coefficient of subgrade reaction, K, of 250 pounds per cubic inch (pci) be used to evaluate anticipated deflections. This value is based on a unit square food area and should be adjusted for the planned pad size. The coefficient of subgrade reaction $K_b$ for a pad of a specific width, may be evaluated using the following equation:

$$K_b=K[(b+1)/2b]^2$$

Where b is the width of the mat foundation.

We estimate that the deflections of the mat foundation will be on the order of ¾-inch or less, when designed and constructed with the recommendations herein.

### 7.4.3 Lateral Resistance

For resistance of footings to lateral loads, we recommend an allowable passive pressure of 450 psf per foot of depth be used with a value of up to 4,500 psf. This value assumes that the ground is horizontal for a distance of 10 feet, or three multiplied by the height generating the passive pressure, whichever is more. We recommend that the upper 1 foot of soil not protected by pavement or a concrete slab be neglected when calculating passive resistance.

For frictional resistance to lateral loads, a coefficient of friction of 0.35 can be used between soil-concrete interface. The allowable lateral resistance can be taken as the sum of the frictional

resistance and passive resistance provided the passive resistance does not exceed one-half of the total allowable resistance. The passive resistance values may be increased by one-third when considering loads of short duration such as wind or seismic forces.

### 7.4.4 Static Settlement

We estimate that the proposed structures, designed and constructed as recommended herein, will undergo total static settlement of less than approximately 1 inch and differential static settlement of approximately 1/2–inch over a horizontal distance of 40 feet. Note, this does not include the seismically-induced settlements resulting from liquefaction during the design seismic event.

## 7.5 Retaining Walls

In the event that the forebay tank will be constructed as a below-ground or partially below-ground structure, we anticipate that the perimeter walls of the forebay tank would act as retaining walls. For the design of a site yielding retaining wall that is not restrained against movement by rigid corners or structural connections, lateral pressures are presented on Figure 8. For the design of non-yielding site retaining was that are restrained, lateral pressures are presented on Figure 9. These pressures assume select backfill materials are used and free draining conditions. Wall backfill should be moisture conditioned and compacted to a relative compaction of 90 percent at near-optimum moisture content as evaluated by ASTM D 1557. Measures should be taken to reduce the potential for build-up of moisture behind the retaining walls. A drain should be provided behind the retaining wall as shown on Figure 10. The drain should be connected to an appropriate outlet.

## 7.6 Preliminary Flexible Pavement Design

We understand that the project will include the construction of new pavements. Our laboratory testing of near surface soil samples at the project site indicated R-values of 69 and 71. We have used an R-value of 50 for preliminary design of the pavement. This R-value, along with estimated design Traffic Indices (TI) of 5.0, 6.9, and 9.6 (City of Poway, 2018), has been the basis of our preliminary flexible pavement design. Actual pavement recommendations should be based on R-value tests performed on bulk samples of the soils that are exposed at the finished subgrade elevations across the site at the completion of the grading operations. The preliminary recommended flexible pavement sections are presented in Table 6.

| Table 6 – Recommended Preliminary Flexible Pavement Sections | | | |
|---|---|---|---|
| Traffic Index (Pavement Usage) | Design R-Value | Asphalt Concrete Thickness (inches) | Aggregate Base Thickness (inches) |
| 5.0 (Parking) | 50 | 3 | 4 |
| 6.9 (Lake Poway Rd) | 50 | 4 | 5 |
| 9.6 (Espola Road) | 50 | 6 | 7 |

As indicated, these values assume TIs of 9.6 or less for site pavements. If traffic loads are different from those assumed, the pavement design should be re-evaluated. We anticipate that trench excavations in existing pavement areas will involve resurfacing along the trench alignment. In general, trench resurfacing be performed in accordance with the 2018 City of Poway Supplemental Engineering Standards. We recommended that the upper 12 inches of the subgrade and aggregate base materials be compacted to a relative compaction of 95 percent as evaluated by the current version of ASTM D 1557. Additionally, we recommend that AC materials be compacted to 95 percent relative compaction as compared to the laboratory evaluated Hveem density. We recommend that the geotechnical consultant re-evaluate the pavement design, based on the R-value of the subgrade material exposed at the time of construction.

## 7.7 Soil Corrosivity

Laboratory testing was performed on a representative sample of the onsite earth materials to evaluate pH and electrical resistivity, as well as chloride and sulfate contents. The pH and electrical resistivity tests were performed in accordance with California Test (CT) 643 and the sulfate and chloride content tests were performed in accordance with CT 417 and CT 422, respectively. These laboratory test results are presented in Appendix C.

The results of our corrosivity testing indicated an electrical resistivities of 4,100 and 2,800 ohm-cm, soil pH values of 8.1 and 7.9, chloride contents of 85 and 145 parts per million (ppm), and sulfate contents of 0.003 and 0.001 percent (i.e., 30 and 10 ppm). Based on the laboratory test results, ACI 318, and Caltrans (2021) corrosion criteria, the project site would not be classified as corrosive. A corrosive soil environment defined as having earth materials with more than 500 ppm chlorides, more than 0.15 percent sulfates (i.e., 1,500 ppm), an electrical resistivity of 1,100 ohm-cm or less, or a pH of 5.5 or less.

## 7.8 Concrete Placement

Concrete in contact with soil or water that contains high concentrations of water-soluble sulfates can be subject to premature chemical and/or physical deterioration. As noted above, the soil samples tested during this evaluation indicated water-soluble sulfate contents of 0.003 and 0.001 percent by weight (i.e., about 30 and 10 ppm). Based on the American Concrete Institute (ACI) 318 criteria, the site soils would correspond to exposure class S0. For this exposure class, ACI 318 recommends that normal weight concrete in contact with soil possess a compressive strength of 2,500 pounds per square inch (psi) or more. Due to the potential for variability of site soils, we recommend that normal weight concrete in contact with soil use Type II, II/V, or V cement.

## 7.9 Pre-Construction Conference

We recommend that a pre-construction meeting be held prior to commencement of grading. The owner or their representative, the agency representatives, the civil engineer, Ninyo & Moore, and the contractor should attend to discuss the plans, the project, and the proposed construction schedule.

## 7.10 Plan Review and Construction Observation

The conclusions and recommendations presented in this report are based on analysis of observed conditions in widely spaced exploration locations. If conditions are found to vary from those described in this report, Ninyo & Moore should be notified, and additional recommendations will be provided upon request. Ninyo & Moore should review the final project drawings and specifications prior to the commencement of construction. Ninyo & Moore should perform the needed observation and testing services during construction operations.

The recommendations provided in this report are based on the assumption that Ninyo & Moore will provide geotechnical observation and testing services during construction. In the event that it is decided not to utilize the services of Ninyo & Moore during construction, we request that the selected consultant provide the client with a letter (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations, and that they are in full agreement with the design parameters and recommendations contained in this report. Construction of proposed improvements should be performed by qualified subcontractors utilizing appropriate techniques and construction materials.

# 8 LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed 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 encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

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.

This report is intended to present characterization of the anticipated soil, rock, and groundwater conditions at the time of evaluation. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

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.

# 9 REFERENCES


American Concrete Institute (ACI), 2019, ACI 318 Building Code Requirements for Structural Concrete and Commentary.

American Society of Civil Engineers (ASCE), 2017, Minimum Design Loads for Buildings and Other Structures, ASCE 7-16.

ASTM International (ASTM), Annual Book of ASTM Standards.

Atlas, 2022, Seismic Refraction Study, Poway Flow Control Facility, Poway, CA, Atlas No. 122152.P6SWG Report No. 1 Rev: dated July 14.

Benton Engineering, Inc., 1975, Soils Investigation, Water Treatment Plant Expansion, Lake Poway Road, San Diego County, California, Project No. 75-3-3A: dated April 11.

Broad, Michael C., 1996, Calculations for Post Tensioned Slab on Grade by Pit & Compressible Soils Method, Administration Building, City of Poway, JDL Project No A0165: dated December 27.

Building News, 2021, "Greenbook", Standard Specification for Public Works Construction: BNI Publications.

California Building Standards Commission, 2019, California Building Code (CBC), Title 24, Part 2, Volumes 1 and 2.

California Department of Transportation (Caltrans), 2021, Corrosion Guidelines (Version 3.2), Division of Engineering and Testing Services, Corrosion Technology Branch: dated August.

California Department of Water Resources (CDWR), 2022, California Dam Breach Inundation Maps, Division of Safety of Dams, fmds.water.ca.gov/maps/damim/: accessed in August.

California Geological Survey, 2022, BETA Landslide Inventory and Deep Landslide Susceptibility Map, https://maps.conservation.ca.gov/cgs/lsi/app/: accessed in August.

Caterpillar, 2019, Caterpillar Performance Handbook Edition 49.

City of Poway, 2018, Supplemental Engineering Standards.

County of San Diego, 1958, Topographic Survey, San Diego County, California, Sheet 306-1761: Scale 1:2,400.

County of San Diego, 1974, Orthotopographic Survey, San Diego County, California, Sheet 306-1761: Scale 1:2,400.

Geotracker website, 2022, www.geotracker.waterboards,ca.gov: access in August.

Google Earth, 2022, https://www.google.com/earth/: accessed in August.

Group Delta Consultants Inc., 1993, Geotechnical Investigation, Poway Water Treatment Plant, Poway, California, Project No. 1514-SI01: dated August 23.

Group Delta Consultants Inc., 1994, Addendum to Geotechnical Investigation, Poway Water Treatment Plant, Poway, California, Project No. 1514-SI01: dated September 12.

Group Delta Consultants Inc., 1995, Limited Geotechnical Investigation, Proposed Public Services Administration Building, Water Treatment Plant, City of Poway, California, Project No. 1467-24: dated June 19.

Group Delta Consultants Inc., 1995, Addendum No. 1 Limited Geotechnical Investigation, Proposed Public Services Administration Building, Water Treatment Plant, City of Poway, California, Project No. 1467-24: dated September 15.

Group Delta Consultants Inc., 1996, Addendum No. 2 Limited Geotechnical Investigation, Proposed Public Services Administration Building, Water Treatment Plant, City of Poway, California, Project No. 1467-24: dated May 6.

Group Delta Consultants Inc., 1996, Geotechnical Input for Design of Post-Tensioned Slab-on-Ground, Public Services Administration Building, Water Treatment Plant, City of Poway, California, Project No. 1467-24: dated December 20.

Group Delta Consultants Inc., 1997, Perimeter Footings, Public Administration Building, Poway, California, Project No. 1467-24: dated January 28.

Harden, D.R., 2004, California Geology, 2nd ed.: Prentice Hall, Inc.

Hart, E.W., and Bryant, W.A., 2007, Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps: California Department of Conservation, Division of Mines and Geology, Special Publication 42, with Supplements 1 and 2 added in 1999.

Hartley and Duncan, 1987, E' and Its Variation with Depth, Journal of Transportation Engineering, Volume 113, Issue 5: dated September.

Historic Aerials website, 2022, www.historicaerials.com: accessed in August.

Jennings, C.W. and Bryant, W.A., 2010, Fault Activity Map of California and Adjacent Areas: California Division of Mines and Geology, California Geologic Data Map Series, Map No. 6, Scale 1:750,000.

Kennedy, M.P., Tan, S.S., 2007, Geologic Map of the Oceanside 30' x 60' Quadrangle, California, California Geological Survey, Scale 1:100,000.

Ninyo & Moore, In-house Proprietary Data.

Ninyo & Moore, 2010, Geotechnical Evaluation, CIP 1757 – Clearwell Redundant Pipeline Project, Poway California, Project No. 106729001: revised November 29.

Ninyo & Moore, 2020, Geotechnical Evaluation, Clearwell Bypass Pipeline Project, Poway, California, Project No. 108978001: dated March 31.

Ninyo & Moore, 2022a, Geotechnical Evaluation, Clearwell Replacement Project - Phase I, Poway, California, Project No. 108978005: dated March 18.

Ninyo & Moore, 2022b, Proposal for Geotechnical Evaluation, Treated Water Facilities, Lake Poway Road and Espola Road, Poway, California, Project No. WTR0028, Proposal No. 02-02561: dated January 13.

Norris, R. M. and Webb, R. W., 1990, Geology of California, Second Edition: John Wiley & Sons, Inc.

Structural Engineers Association of California/Office of Statewide Health Planning and Development (SEAOC/OSHPD), 2022, Seismic Design Maps, https://seismicmaps.org/.

Tan, S.S., 1995, Landslide Hazards in the Northern Part of the San Diego Metropolitan Area, San Diego County, California, Plate F, Escondido Quadrangle: California Geological Survey, Landslide Hazard Identification Map No. 33, Open-File Report 95-03.

United States Department of Agriculture (USDA), Aerial Photograph, Date January 1, 1953, Flight AXN-10M, Numbers 26 and 27, Scale 1:20,000.

United States Department of the Interior, Bureau of Reclamation, 1989, Engineering Geology Field Manual.

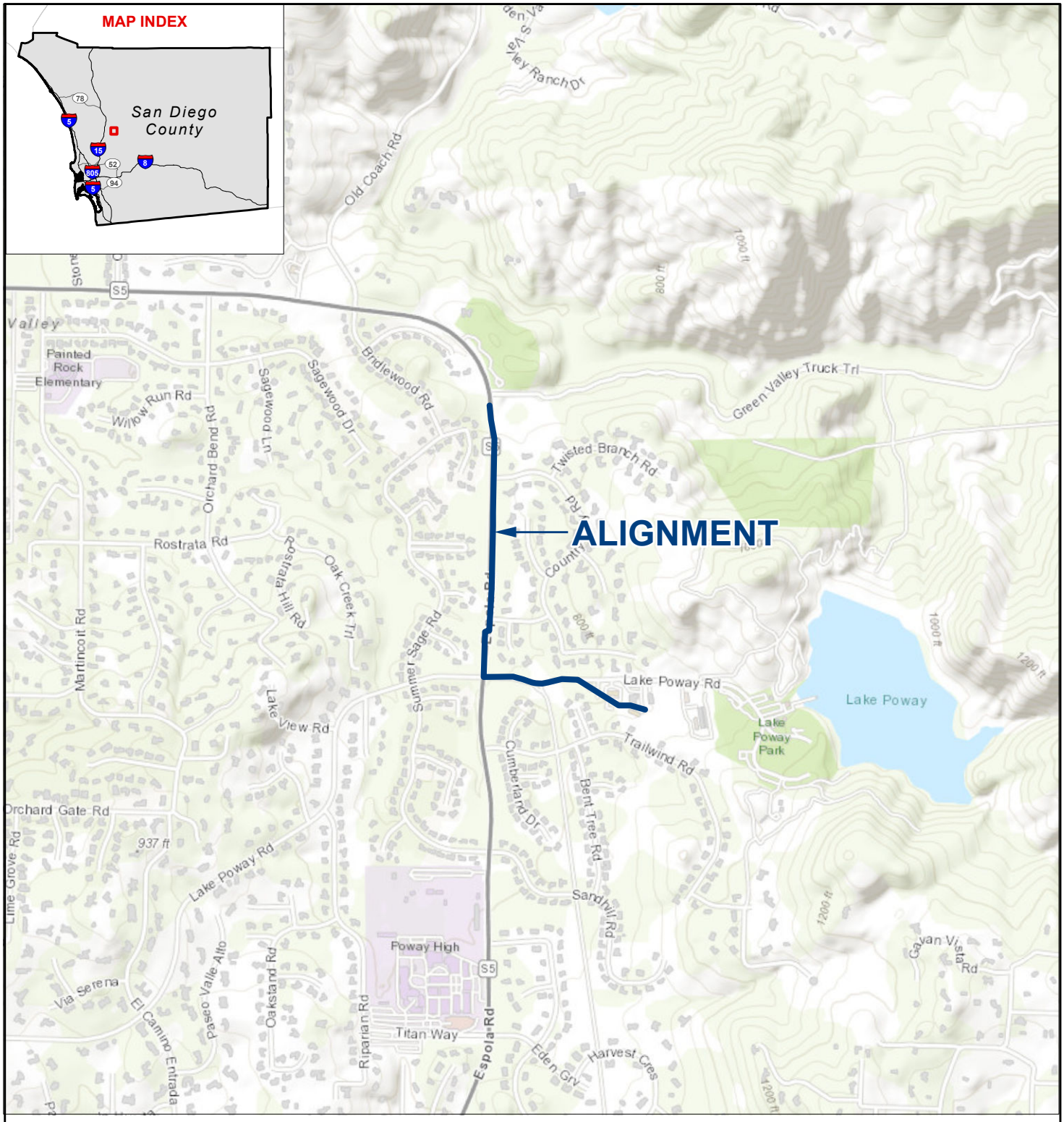
United States Federal Emergency Management Agency (FEMA), 2022, FEMA Mapping Information Platform, World Wide Web, https://hazards.fema.gov/femaportal/wps/portal/: accessed in August.

United States Geological Survey (USGS), 2018, Escondido Quadrangle, California, 7.5-Minute Series: Scale 1:24,000.

United States Geological Survey (USGS), 2020, Unified Hazard Tool; https://earthquake.usgs.gov/hazards/interactive/.

United States Geological Survey (USGS), 2022, 2008 National Seismic Hazard Maps - Fault Parameters, World Wide Web, http://earthquake.usgs.gov/cfusion/hazfaults_2008_search/query_main.cfm/.

# FIGURES

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NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: ESRI WORLD TOPO, 2017

**FIGURE 1**

**SITE LOCATION**

TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA

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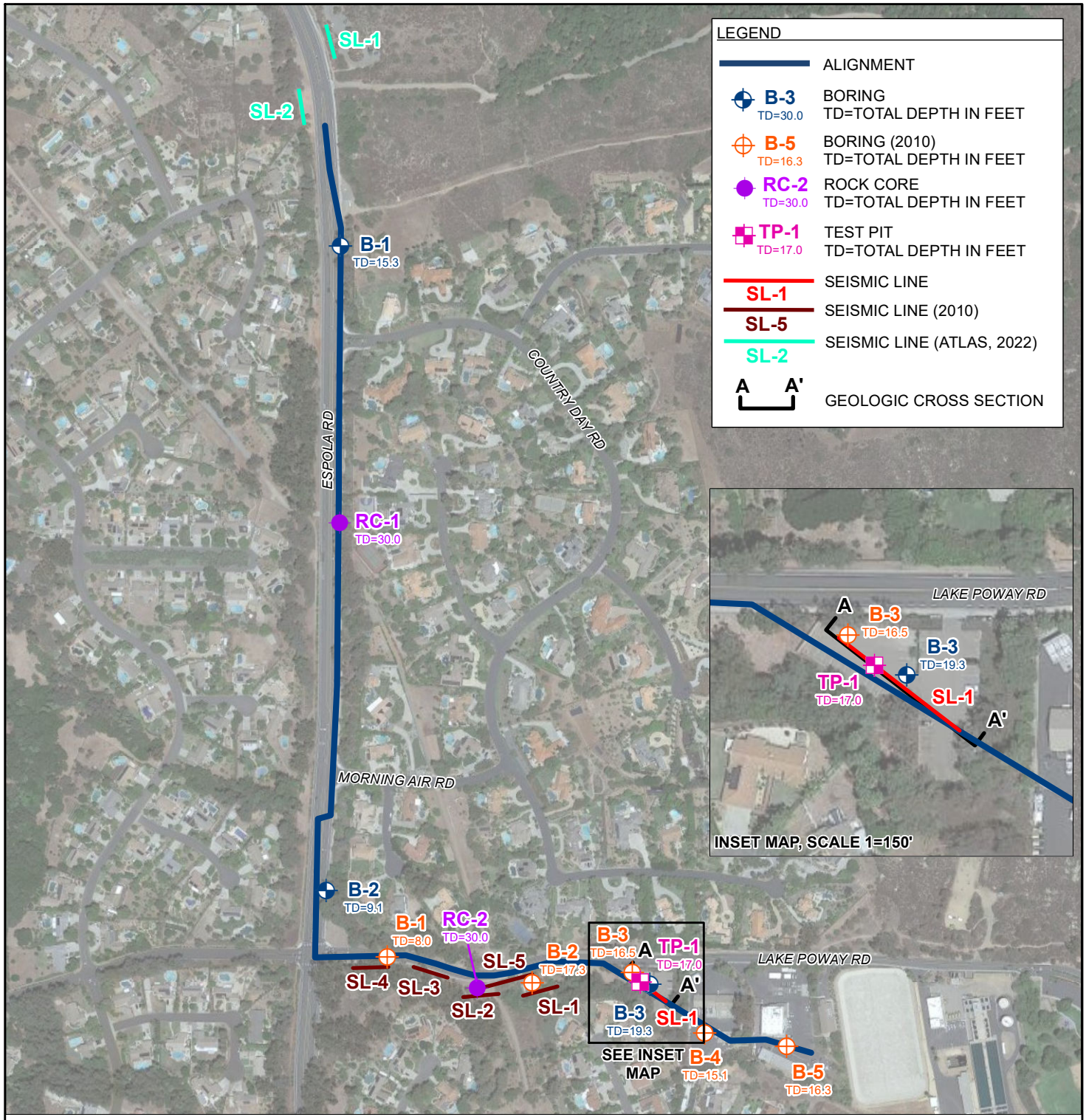


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NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: GOOGLE EARTH, 2022

**FIGURE 2**

## EXPLORATION LOCATIONS

TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA

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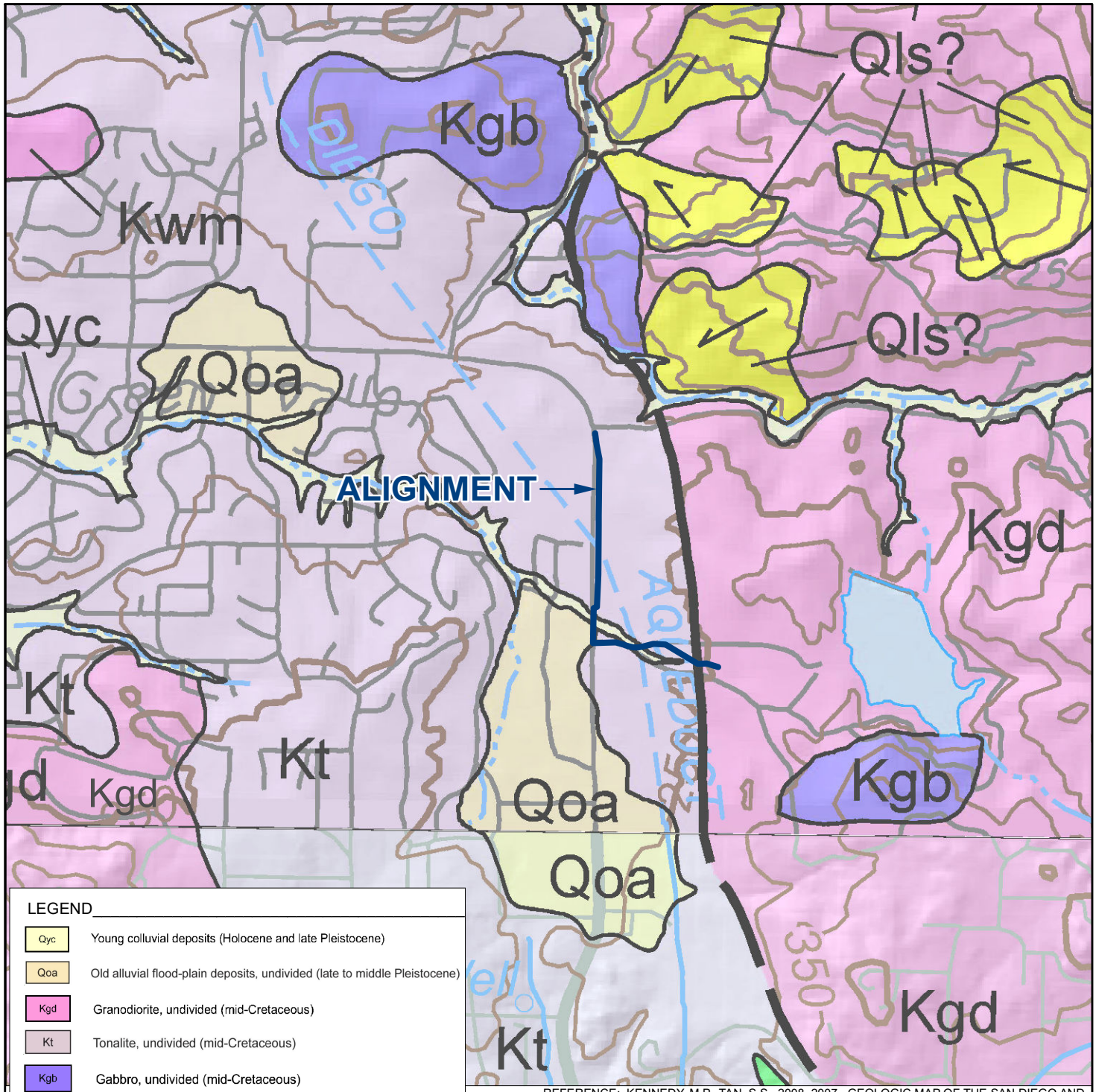


REFERENCE: KENNEDY, M.P., TAN, S.S., 2008, 2007, GEOLOGIC MAP OF THE SAN DIEGO AND OCEANSIDE 30 X 60 MINUTE QUADRANGLES, CALIFORNIA

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

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**FIGURE 3**

**GEOLOGY**

TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA

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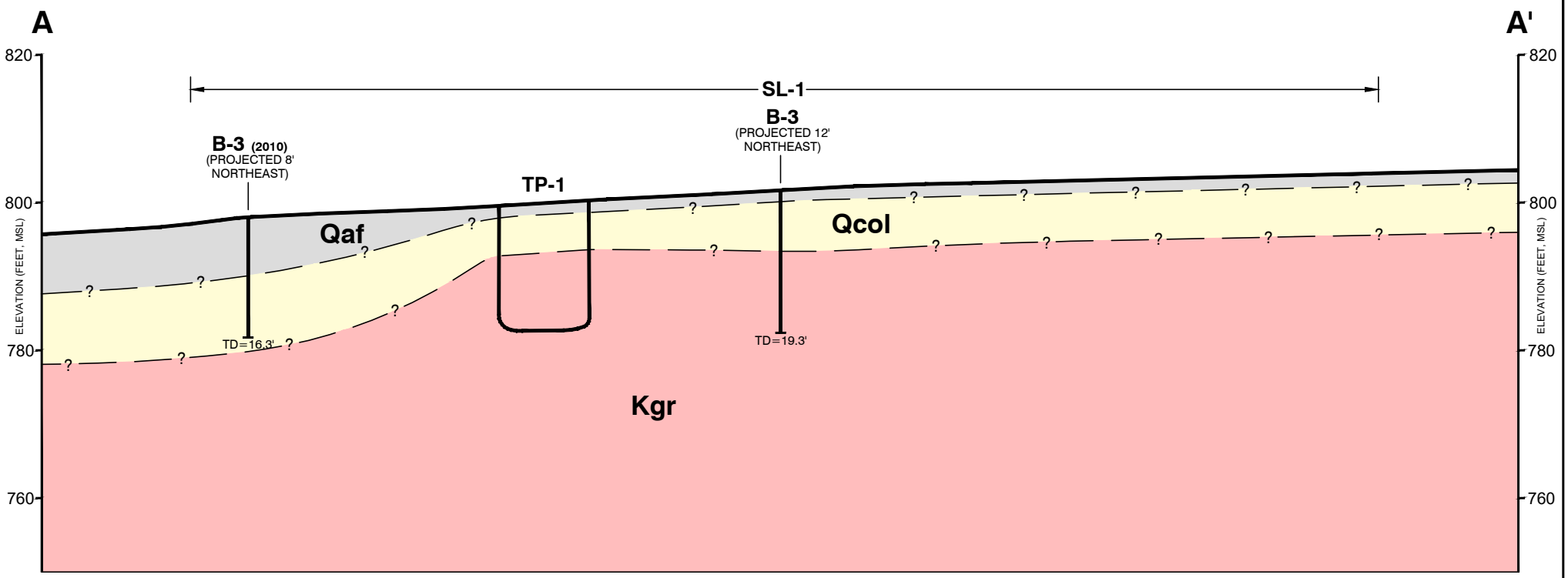


**LEGEND**

| | |
|---|---|
| B-3 (TD=19.3') | BORING TD=TOTAL DEPTH IN FEET |
| B-3 (2010) (TD=16.3') | BORING, 2010 TD=TOTAL DEPTH IN FEET |
| SL-1 | SEISMIC LINE |
| TP-1 | TEST PIT |
| Qaf | FILL |
| Qcol | COLLUVIUM (REFERED TO AS OLD ALLUVIUM [NINYO & MOORE, 2010]) |
| Kgr | GRANITIC ROCK |
| —— ? —— | GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN |

FEET
0 20 40

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

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FIGURE 4

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## GEOLOGIC CROSS SECTION A-A'

TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA

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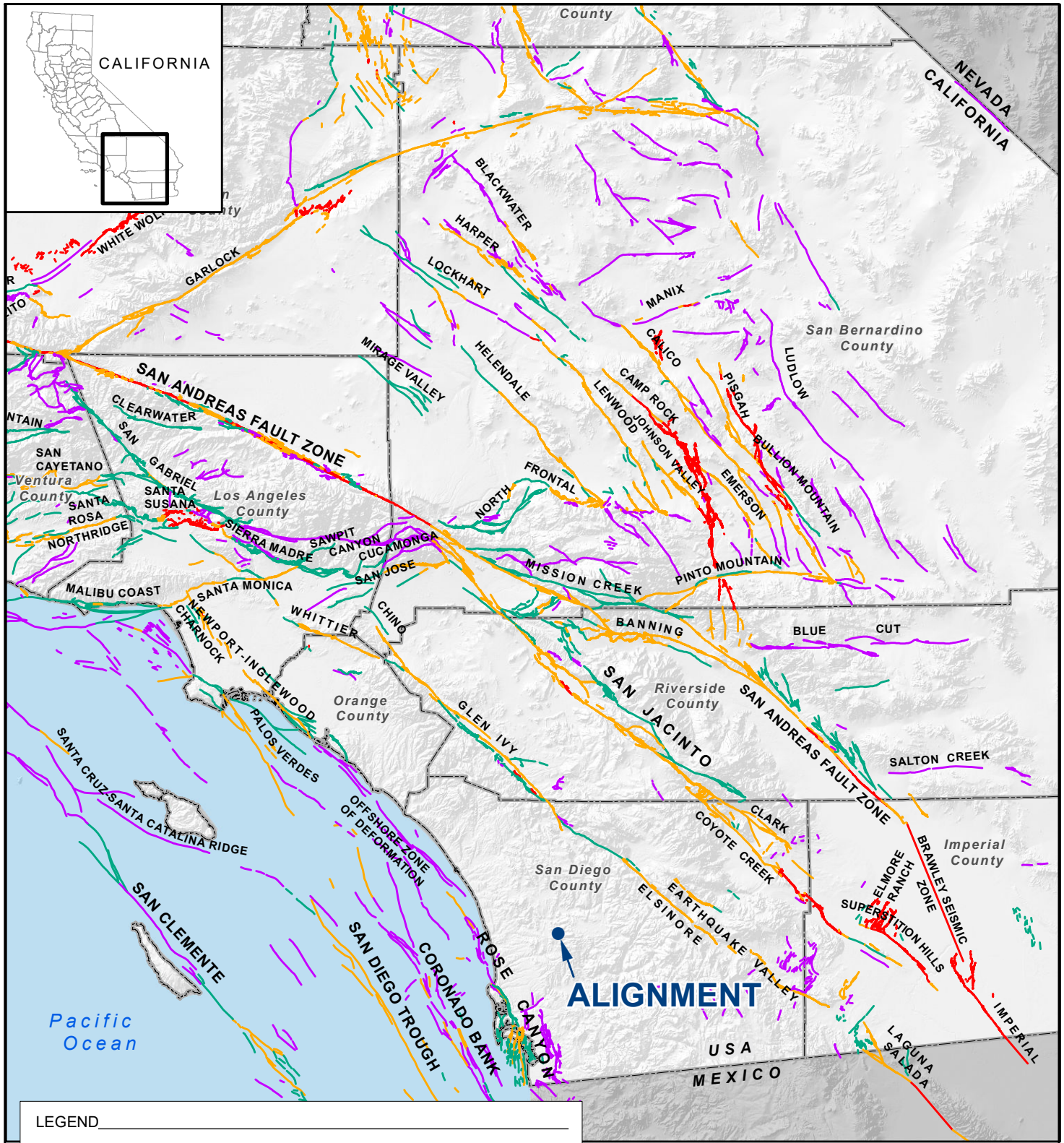


LEGEND

CALIFORNIA FAULT ACTIVITY

- HISTORICALLY ACTIVE
- HOLOCENE ACTIVE
- LATE QUATERNARY (POTENTIALLY ACTIVE)
- QUATERNARY (POTENTIALLY ACTIVE)
- STATE/COUNTY BOUNDARY

SOURCE: U.S. GEOLOGICAL SURVEY AND CALIFORNIA GEOLOGICAL SURVEY, 2006, QUATERNARY FAULT AND FOLD DATABASE FOR THE UNITED STATES.

N

MILES

0 30 60

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

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FIGURE 5

## FAULT LOCATIONS

TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA

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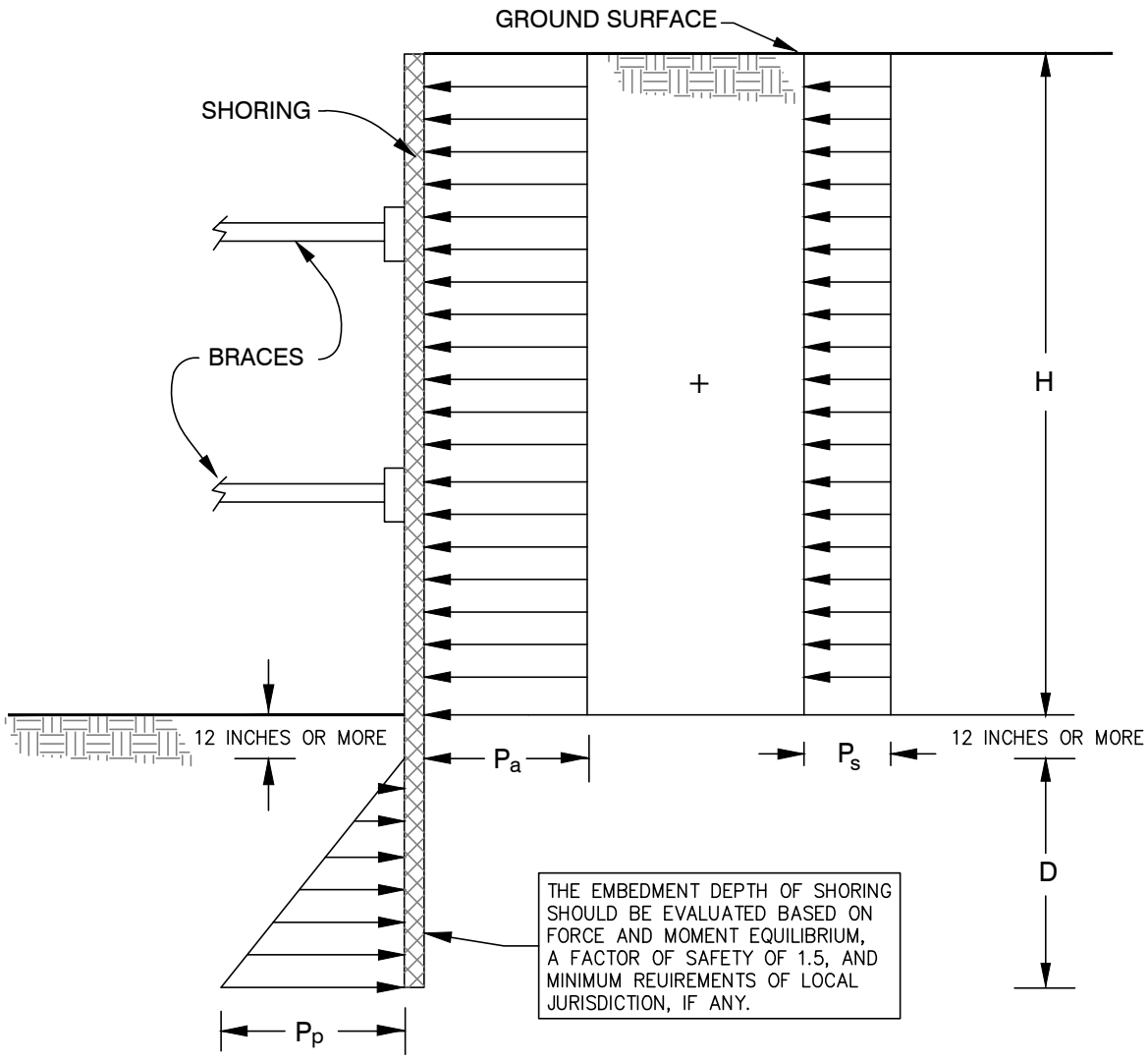


**NOTES:**

1. APPARENT LATERAL EARTH PRESSURE, $P_a$
   $P_a$ = 25 H psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, $P_s$
   $P_s$ = 120 psf
3. PASSIVE LATERAL EARTH PRESSURE, $P_p$
   $P_p$ = 450 D psf
4. ASSUMES GROUNDWATER IS NOT PRESENT
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H AND D ARE IN FEET

NOT TO SCALE

109471001 DETAIL FIGURES.DWG AOB

**FIGURE 6**

# LATERAL EARTH PRESSURES FOR BRACED EXCAVATION (GRANULAR SOIL)

TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA

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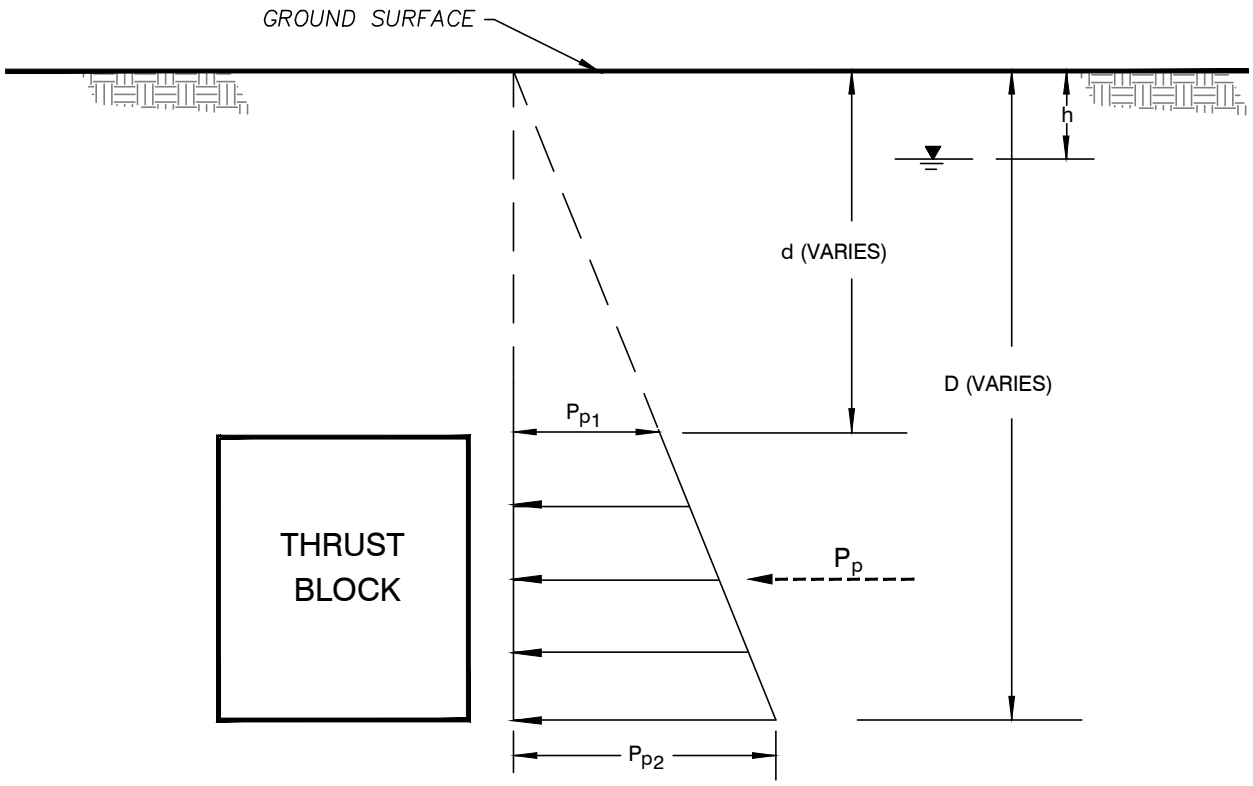


NOTES:

1. GROUNDWATER BELOW BLOCK

   $P_p = 225 (D^2-d^2)$ lb/ft

2. GROUNDWATER ABOVE BLOCK

   $P_p = 1.7 ( D - d )[ 124.8h + 69 ( D+d )]$ lb/ft

3. ASSUMES BACKFILL IS GRANULAR MATERIAL
4. ASSUMES THRUST BLOCK IS ADJACENT TO COMPETENT MATERIAL
5. D, d AND h ARE IN FEET
6. GROUNDWATER TABLE

NOT TO SCALE

109471001 DETAIL FIGURES.DWG AOB

**FIGURE 7**

## THRUST BLOCK LATERAL EARTH PRESSURE DIAGRAM

TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA

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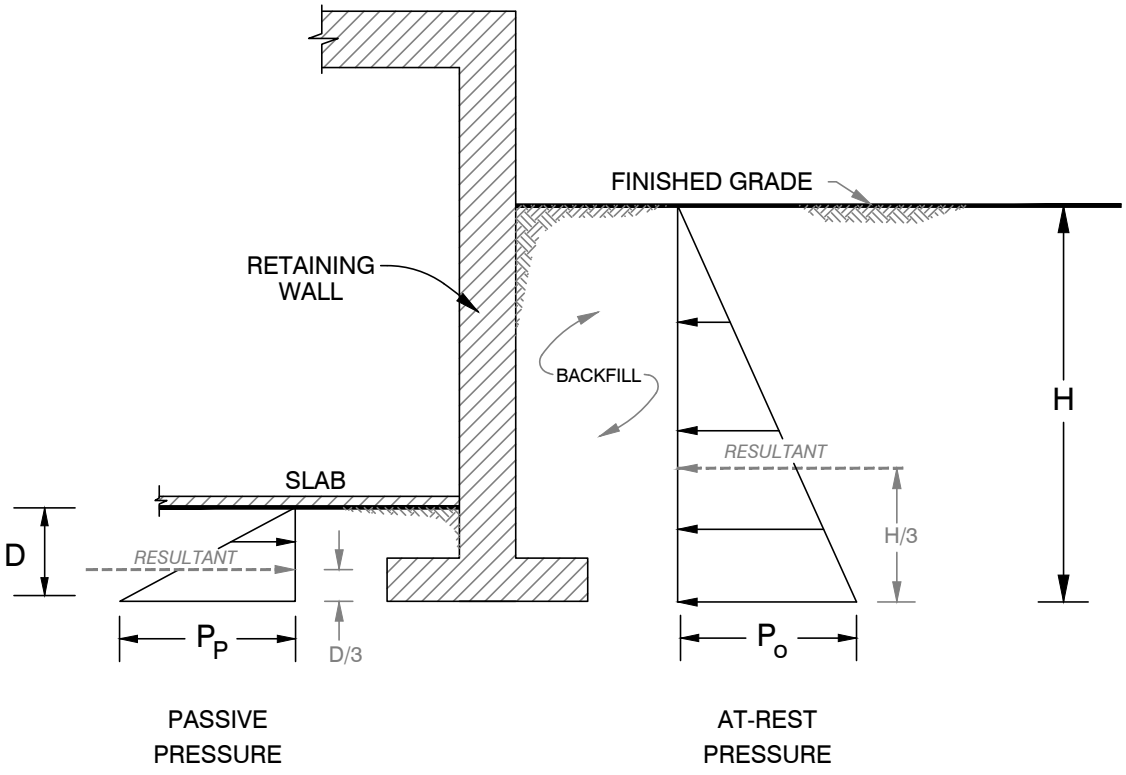


**NOTES:**

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. GRANULAR BACKFILL MATERIALS SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
5. H AND D ARE IN FEET

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

| Lateral Earth Pressure | Equivalent Fluid Pressure ($lb/ft^2/ft$)[1] | |
|---|---|---|
| $P_O$ | Level Backfill with Granular Soils[2] | 2H:1V Sloping Backfill with Granular Soils[2] |
| | 60 H | 87 H |
| $P_P$ | Level Ground | 2H:1V Descending Ground |
| | 450 D | 160 D |

NOT TO SCALE

109471001 DETAIL FIGURES.DWG AOB

**FIGURE 8**

# LATERAL EARTH PRESSURES FOR RESTRAINED RETAINING WALLS

TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA

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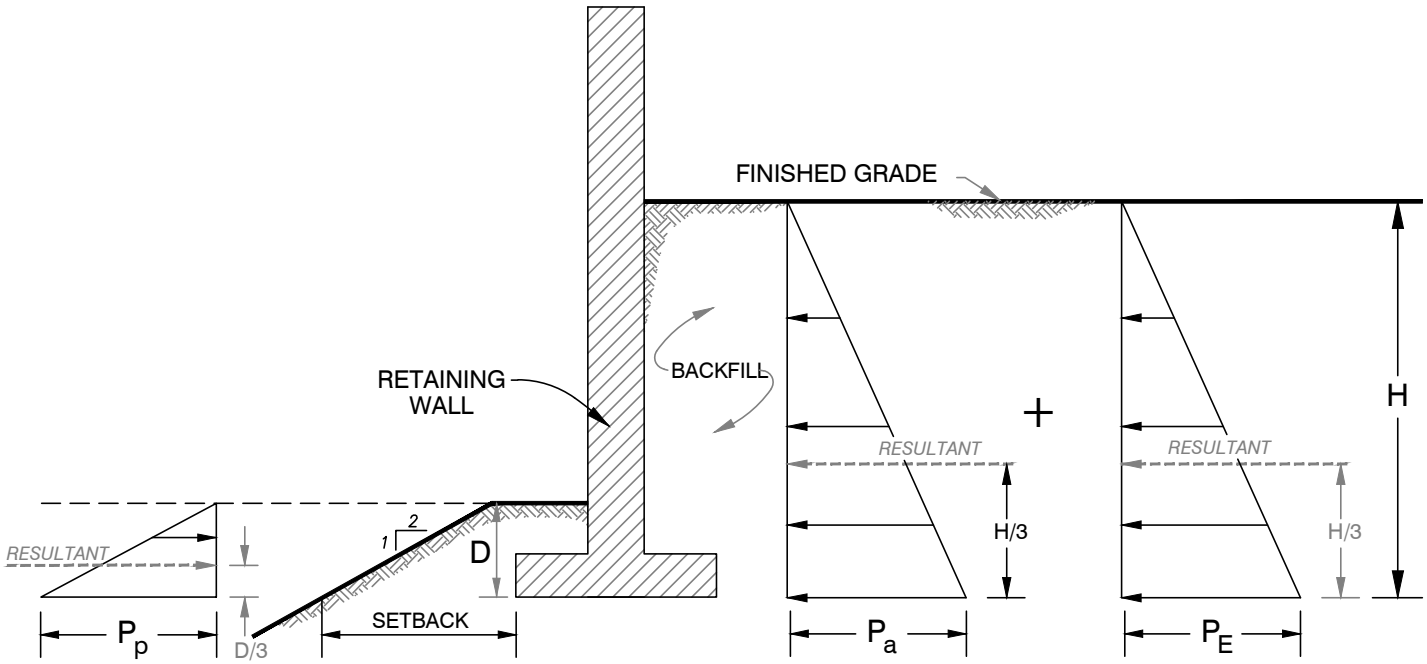


**NOTES:**

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. GRANULAR BACKFILL MATERIALS SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. DYNAMIC LATERAL EARTH PRESSURE IS BASED ON A MAPPED DESIGN PEAK GROUND ACCELERATION OF 0.42 g
5. $P_E$ IS CALCULATED IN ACCORDANCE WITH THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), AND ATIK AND SITAR (2010).
6. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
7. H AND D ARE IN FEET
8. SETBACK SHOULD BE IN ACCORDANCE WITH THE CBC

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

| Lateral Earth Pressure | Equivalent Fluid Pressure ($lb/ft^2/ft$)[1] | |
|---|---|---|
| $P_a$ | Level Backfill with Granular Soils[2] | 2H:1V Sloping Backfill with Granular Soils[2] |
| | 38 H | 60 H |
| $P_E$ | 16 H | |
| $P_p$ | Level Ground | 2H:1V Descending Ground |
| | 450 D | 160 D |

NOT TO SCALE

109471001 DETAIL FIGURES.DWG AOB

FIGURE 9

# LATERAL EARTH PRESSURES FOR YIELDING RETAINING WALLS

TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA

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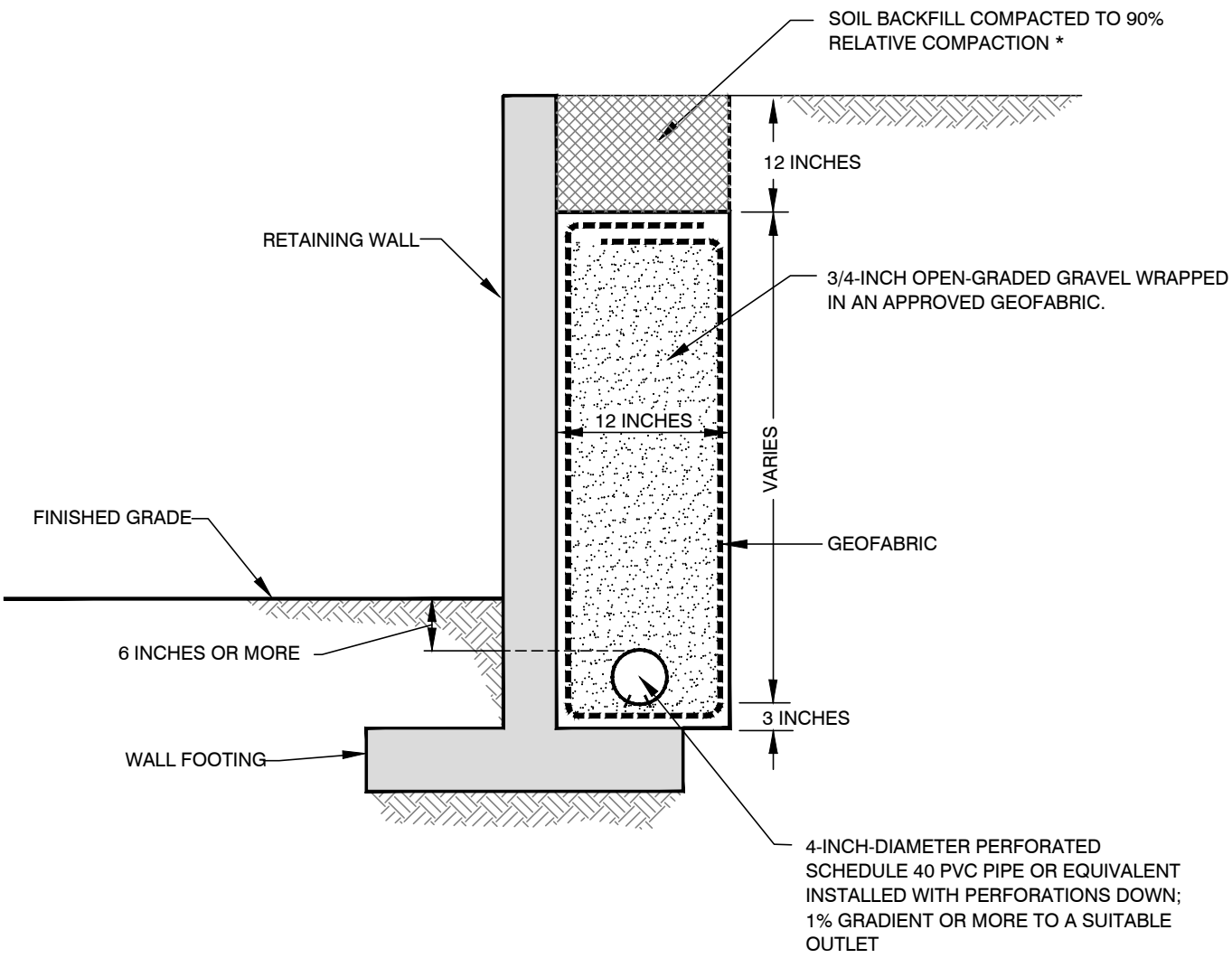


*BASED ON ASTM D1557

NOT TO SCALE

**FIGURE 10**

## RETAINING WALL DRAINAGE DETAIL

TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA

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# APPENDIX A

## Excavation Logs

# APPENDIX A

## EXCAVATION LOGS

### Field Procedure for the Collection of Disturbed Samples

Disturbed soil samples were obtained in the field using the following methods.

#### Bulk Samples

Bulk samples of representative earth materials were obtained from the exploratory excavations. The samples were bagged and transported to the laboratory for testing.

#### The Standard Penetration Test (SPT) Sampler

Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The sampler was driven into the ground 12 to 18 inches with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

### Field Procedure for the Collection of Relatively Undisturbed Samples

Relatively undisturbed soil samples were obtained in the field using the following method.

#### The Modified Split-Barrel Drive Sampler

The sampler, with an external diameter of 3 inches, was lined with 1-inch-long, thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

### Rock Core Terminology

The following tables present the terminology used, along with definitions and abbreviations of the terminology used on the rock core boring logs presented herein (from the Engineering Geology Field Manual published by the U.S. Department of the Interior Bureau of Reclamation).

**Table A–1 – Description of Rock Fracturing**

| | | |
|---|---|---|
| 1 | Unfractured | No observed fractures |
| 2 | Very slightly fractured | Core length recovery greater than 3' |
| 3 | Slightly to very slightly fractured | * |
| 4 | Slightly fractured | Core length recovery from 1' to 3' |
| 5 | Moderately to slightly fractured | * |
| 6 | Moderately fractured | Core length recovery from 0.33' to 1' |
| 7 | Intensely to moderately fractured | * |
| 8 | Intensely fractured | Core length recovery from 0.1' to 0.33' |
| 9 | Very intensely to intensely fractured | * |
| 10 | Very intensely fractured | Core length recovery mostly chips and fragments |

*Combinations of fracture densities are used where equal distribution of both fracture density characteristics are present over a significant core interval or exposure.

## Table A–2 – Description of Rock Weathering

| | | |
|---|---|---|
| 1 | Fresh | No discoloration, surfaces not oxidized. |
| 2 | Slightly weathered | Discoloration and oxidation limited to surfaces. |
| 3 | Moderately weathered | Discoloration and oxidation extends from fractures, partial chemical alteration of some minerals. |
| 4 | Intensely weathered | Discoloration and oxidation throughout; chemical alteration of most minerals; rock is friable. |
| 5 | Decomposed | Rock decomposed; resembles a soil. |

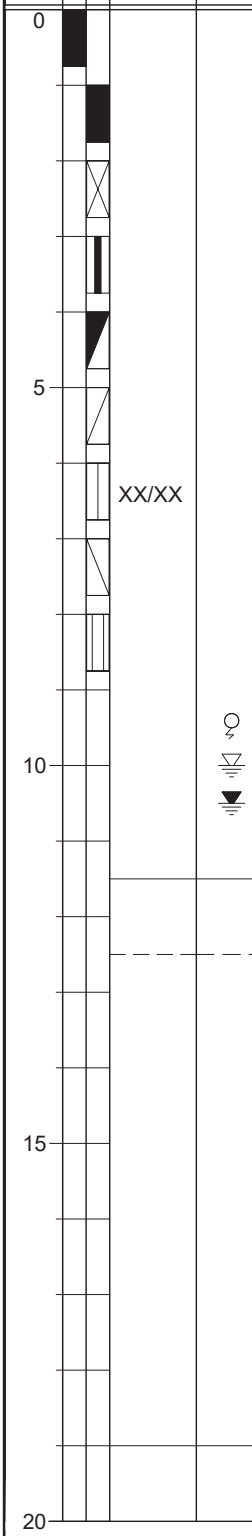
## Table A–3 – Description of Rock Hardness

| | | |
|---|---|---|
| 1 | Extremely hard | Cannot be scratched with knife; can only be chipped with repeated heavy hammer blows. |
| 2 | Very hard | Cannot be scratched with knife; core can only be broken with repeated heavy hammer blows. |
| 3 | Hard | Can be scratched with a knife with difficulty; core can be broken with heavy hammer blow. |
| 4 | Moderately hard | Can be scratched with a knife with light to moderate pressure; core breaks with moderate hammer blow. |
| 5 | Moderately soft | Can be grooved 1/16 in. deep by knife with moderate to heavy pressure; core can be broken with light hammer blow. |
| 6 | Soft | Can be grooved easily with a knife with light pressure; core breaks with light to moderate manual pressure. |
| 7 | Very soft | Can be easily carved with a knife; core breaks with light manual pressure. |

## Table A–4 – Descriptions and Abbreviations for Fractures

| | | |
|---|---|---|
| **Surface Roughness** | | |
| St | Stepped | Near normal steps and ridges occur on the fracture surface. |
| R | Rough | Large, angular asperities can be seen. |
| Mr | Moderately rough | Asperities are clearly visible and fracture surface feels abrasive. |
| Sm | Smooth | No asperities; smooth to the touch. |
| **Surface Shape** | | |
| Pl | Planar | Flat and uniform. |
| Un | Undulatory | Wavy and symmetrical. |
| Irr | Irregular | Non-uniform and symmetrical. |
| **Aperture** | | |
| T | Tight | No visible separation |
| So | Slightly Open | <1/32 inch |
| Mo | Moderately Open | 1/32 inch – 1/8 inch |
| O | Open | 1/8 inch – 3/8 inch |
| Mw | Moderately Wide | 3/8 inch – 1.2 inches |
| W | Wide | >1.2 inches |
| **Type of Infilling** | | |
| Cl | Clay | |
| Ca | Calcite | |
| Ch | Chlorite | |
| Fe | Iron Oxide | |
| G | Gypsum | |
| Mn | Manganese Oxide | |
| Q | Quartz | |
| **Thickness of Infilling** | | |
| Sta | Surface Staining | --- |
| Vth | Very thin | <1/32 inch |
| Mth | Moderately thin | 1/32 inch – 1/8 inch |
| Th | Thin | 1/8 inch – 3/8 inch |
| Mtk | Moderately thick | 3/8 inch – 1.2 inches |
| Tk | Thick | >1.2 inches |

# BORING LOG EXPLANATION SHEET




Bulk sample.

Modified split-barrel drive sampler.

No recovery with modified split-barrel drive sampler.

Sample retained by others.

Standard Penetration Test (SPT).

No recovery with a SPT.

Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.

No recovery with Shelby tube sampler.

Continuous Push Sample.

Seepage.

Groundwater encountered during drilling.

Groundwater measured after drilling.

SM

MAJOR MATERIAL TYPE (SOIL):

Solid line denotes unit change.

CL

Dashed line denotes material change.

Attitudes: Strike/Dip
b: Bedding
c: Contact
j: Joint
f: Fracture
F: Fault
cs: Clay Seam
s: Shear
bss: Basal Slide Surface
sf: Shear Fracture
sz: Shear Zone
sbs: Shear Bedding Surface

The total depth line is a solid line that is drawn at the bottom of the boring.

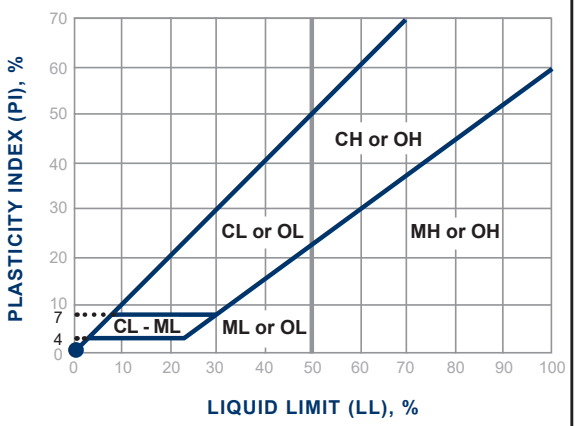
## Soil Classification Chart Per ASTM D 2488

| Primary Divisions | | | Secondary Divisions | |
|---|---|---|---|---|
| | | | Group Symbol | Group Name |
| **COARSE-GRAINED SOILS** more than 50% retained on No. 4 sieve | **GRAVEL** more than 50% of coarse fraction retained on No. 4 sieve | CLEAN GRAVEL less than 5% fines | GW | well-graded GRAVEL |
| | | | GP | poorly graded GRAVEL |
| | | GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines | GW-GM | well-graded GRAVEL with silt |
| | | | GP-GM | poorly graded GRAVEL with silt |
| | | | GW-GC | well-graded GRAVEL with clay |
| | | | GP-GC | poorly graded GRAVEL with clay |
| | | GRAVEL with FINES more than 12% fines | GM | silty GRAVEL |
| | | | GC | clayey GRAVEL |
| | | | GC-GM | silty, clayey GRAVEL |
| | **SAND** 50% or more of coarse fraction passes No. 4 sieve | CLEAN SAND less than 5% fines | SW | well-graded SAND |
| | | | SP | poorly graded SAND |
| | | SAND with DUAL CLASSIFICATIONS 5% to 12% fines | SW-SM | well-graded SAND with silt |
| | | | SP-SM | poorly graded SAND with silt |
| | | | SW-SC | well-graded SAND with clay |
| | | | SP-SC | poorly graded SAND with clay |
| | | SAND with FINES more than 12% fines | SM | silty SAND |
| | | | SC | clayey SAND |
| | | | SC-SM | silty, clayey SAND |
| **FINE-GRAINED SOILS** 50% or more passes No. 200 sieve | **SILT and CLAY** liquid limit less than 50% | INORGANIC | CL | lean CLAY |
| | | | ML | SILT |
| | | | CL-ML | silty CLAY |
| | | ORGANIC | OL (PI > 4) | organic CLAY |
| | | | OL (PI < 4) | organic SILT |
| | **SILT and CLAY** liquid limit 50% or more | INORGANIC | CH | fat CLAY |
| | | | MH | elastic SILT |
| | | ORGANIC | OH (plots on or above "A"-line) | organic CLAY |
| | | | OH (plots below "A"-line) | organic SILT |
| | Highly Organic Soils | | PT | Peat |

## Grain Size

| Description | | Sieve Size | Grain Size | Approximate Size |
|---|---|---|---|---|
| Boulders | | > 12" | > 12" | Larger than basketball-sized |
| Cobbles | | 3 - 12" | 3 - 12" | Fist-sized to basketball-sized |
| Gravel | Coarse | 3/4 - 3" | 3/4 - 3" | Thumb-sized to fist-sized |
| | Fine | #4 - 3/4" | 0.19 - 0.75" | Pea-sized to thumb-sized |
| Sand | Coarse | #10 - #4 | 0.079 - 0.19" | Rock-salt-sized to pea-sized |
| | Medium | #40 - #10 | 0.017 - 0.079" | Sugar-sized to rock-salt-sized |
| | Fine | #200 - #40 | 0.0029 - 0.017" | Flour-sized to sugar-sized |
| Fines | | Passing #200 | < 0.0029" | Flour-sized and smaller |

## Plasticity Chart




## Apparent Density - Coarse-Grained Soil

| Apparent Density | Spooling Cable or Cathead | | Automatic Trip Hammer | |
|---|---|---|---|---|
| | SPT (blows/foot) | Modified Split Barrel (blows/foot) | SPT (blows/foot) | Modified Split Barrel (blows/foot) |
| Very Loose | ≤ 4 | ≤ 8 | ≤ 3 | ≤ 5 |
| Loose | 5 - 10 | 9 - 21 | 4 - 7 | 6 - 14 |
| Medium Dense | 11 - 30 | 22 - 63 | 8 - 20 | 15 - 42 |
| Dense | 31 - 50 | 64 - 105 | 21 - 33 | 43 - 70 |
| Very Dense | > 50 | > 105 | > 33 | > 70 |

## Consistency - Fine-Grained Soil

| Consis-tency | Spooling Cable or Cathead | | Automatic Trip Hammer | |
|---|---|---|---|---|
| | SPT (blows/foot) | Modified Split Barrel (blows/foot) | SPT (blows/foot) | Modified Split Barrel (blows/foot) |
| Very Soft | < 2 | < 3 | < 1 | < 2 |
| Soft | 2 - 4 | 3 - 5 | 1 - 3 | 2 - 3 |
| Firm | 5 - 8 | 6 - 10 | 4 - 5 | 4 - 6 |
| Stiff | 9 - 15 | 11 - 20 | 6 - 10 | 7 - 13 |
| Very Stiff | 16 - 30 | 21 - 39 | 11 - 20 | 14 - 26 |
| Hard | > 30 | > 39 | > 20 | > 26 |

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

**USCS METHOD OF SOIL CLASSIFICATION**

DATE DRILLED 8/15/22 BORING NO. B-1

GROUND ELEVATION 725' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (TriCounty)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY KWK LOGGED BY KWK REVIEWED BY ZH/GTF

| DEPTH (feet) | SAMPLES (Bulk / Driven) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | | ASPHALT CONCRETE: Approximately 6 inches thick. |
| | | | | | | GM | AGGREGATE BASE: Light brown, moist, medium dense, silty GRAVEL; approximately 12 inches thick. |
| | | | | | | SM | FILL: Light brown, moist, medium dense, silty fine to medium SAND. |
| 5 | | 50/5" | 2.8 | 140.6 | | | GRANITIC ROCK: Light brown and light gray, moist, moderately to weakly weathered, coarse- grained GRANITIC ROCK. |
| 10 | | 50/4" | | | | | |
| 15 | | 50/3" | | | | | Total Depth = 15.3 feet. Groundwater not encountered during drilling. Backfilled on 8/15/22, patched with aquaphalt. |
| 20 | | | | | | | Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. |

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

**FIGURE A-1**

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**TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT**
**ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA**

DATE DRILLED 8/15/22 BORING NO. B-2

GROUND ELEVATION 748' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (TriCounty)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY KWK LOGGED BY KWK REVIEWED BY ZH/GTF

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | ASPHALT CONCRETE: Approximately 5 inches thick. |
| | | | | | | | GM | AGGREGATE BASE: Light brown, moist, medium dense, silty GRAVEL; approximately 13 inches thick. |
| | | | | 7.6 | | | SM | FILL: Brown, moist, medium dense, silty fine to coarse SAND; trace roots. |
| 5 | | | 91/10" | 8.8 | 129.3 | | | GRANITIC ROCK: Light brown, moist, strongly weathered, coarse-grained GRANITIC ROCK. |
| | | | 50/1" | | | | | Difficult drilling, auger refusal, weakly weathered. |
| 10 | | | | | | | | Total Depth = 9.1 feet. Groundwater not encountered during drilling. Backfilled on 8/15/22, patched with temporary asphalt patching. |
| 15 | | | | | | | | Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. |
| 20 | | | | | | | | The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents. |

**FIGURE A-2**

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

**TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT**
**ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA**

DATE DRILLED 8/15/22 BORING NO. B-3

GROUND ELEVATION 800' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (TriCity)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY KWK LOGGED BY KWK REVIEWED BY ZH/GTF

| DEPTH (feet) | SAMPLES (Bulk / Driven) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | | ASPHALT CONCRETE: Approximately 3.5 inches thick. |
| | | | | | | GM | AGGREGATE BASE: |
| | | | | | | SM | Light brown, moist, medium dense, silty GRAVEL. |
| | | | | | | | COLLUVIUM: Gray, moist, medium dense, silty fine to medium SAND; scattered gravel; organic odor. |
| 5 | | 31 | 14.2 | 115.3 | | SC | Gray, moist, medium dense, clayey SAND; organic odor. |
| | | | | | | | GRANITIC ROCK: Light brown, moist, decomposed, coarse-grained GRANITIC ROCK; trace clay. |
| 10 | | 29 | | | | | |
| 15 | | 28 | 17.2 | 111.6 | | | Increased clay content; iron oxide staining. |
| | | 50/4" | | | | | Light brown and gray, moderately weathered; iron oxide staining. |
| 20 | | | | | | | Total Depth = 19.3 feet. Groundwater seepage encountered at 18.5 feet. |

FIGURE A-3

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA

109471001 | 12/22

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|
| 20 | | | | | | | | Backfilled on 8/15/22, patched with aquaphalt.<br><br>Note: Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report.<br><br>The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents. |
| 25 | | | | | | | | |
| 30 | | | | | | | | |
| 35 | | | | | | | | |
| 40 | | | | | | | | |

DATE DRILLED 8/15/22 BORING NO. B-3

GROUND ELEVATION 800' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (TriCity)

DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"

SAMPLED BY KWK LOGGED BY KWK REVIEWED BY ZH/GTF

**FIGURE A-4**

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

**TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT**
**ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA**

DATE DRILLED 8/16/2022 & 8/22/2022 CORE NO. RC-1

GROUND ELEVATION ± 744 (MSL) SHEET 1 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger and 2.3 inch HQ Rock

DRIVE WEIGHT 140 lbs. (Auto-Trip 0 to 12 feet) DROP 30" (0 to 12 feet)

SAMPLED BY KWK LOGGED BY KWK REVIEWED BY ZH/GTF

| DEPTH (feet) | DRIVEN SAMPLE | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (ppd) | INTERVAL CORED (feet) | RECOVERY (%) | RQD (%) | FRACTURE FREQ. (ft) | FRACTURE DIP (degrees) | GRAPHIC LOG SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | | | | GM | ASPHALT CONCRETE: Approximately 4 inches thick. |
| | | | | | | | | | | | SM | AGGREGATE BASE: Light brown, moist, medium dense, silty GRAVEL. |
| | | | | | | | | | | | | FILL: Light brown, moist, medium dense, silty fine to medium SAND. |
| 5 | | 50/4" | 3.2 | 123.1 | | | | | | | | |
| | | | | | | | | | | | | GRANITIC ROCK: Light brown and gray, moist, decomposed, coarse-grained GRANITIC ROCK. |
| 10 | | 95/9" | | | | | | | | | | Moderately weathered. |
| | | | | | | | | | | | | @ 12 feet: Switch to HQ rock coring. |
| | | | | | | | | | | | | Brown, moist, strongly weathered to decomposed, very soft, iron oxide staining, micaceous. |
| | | | | | 3 | 100 | 0 | | | | | |
| 15 | | | | | | | | | | | | |
| | | | | | 3.5 | 100 | 0 | | | | | |
| | | | | | | | | | 10 | | | Gray, dry, very slightly fractured, fresh, very hard, Mr, Un, So. |
| | | | | | 1.5 | 92 | 100 | | | | | |
| 20 | | | | | | | | | | | | |

FIGURE A-5

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA

DATE DRILLED 8/16/2022 & 8/22/2022 CORE NO. RC-1

GROUND ELEVATION ± 744 (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger and 2.3 inch HQ Rock

DRIVE WEIGHT 140 lbs. (Auto-Trip 0 to 12 feet) DROP 30" (0 to 12 feet)

SAMPLED BY KWK LOGGED BY KWK REVIEWED BY ZH/GTF

| DEPTH (feet) | DRIVEN SAMPLE | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (pcf) | INTERVAL CORED (feet) | RECOVERY (%) | RQD (%) | FRACTURE FREQ. (ft) | FRACTURE DIP (degrees) | GRAPHIC LOG | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 20 | | | | | | | | | | | | | GRANITIC ROCK: (CONTINUED)<br>Gray, dry, very slightly fractured, fresh, very hard, GRANITIC ROCK.<br>St-R, Un, Mo. |
| | | | | | | | | | 23 | | | | |
| | | | | | | | | | 25 | | | | |
| | | | | | 5 | 50 | 43 | 1 | | | | | Brown, decomposed, very soft. |
| 25 | | | | | | | | | | | | | |
| | | | | | | | | | | | | | Gray, unfractured, fresh, very hard. |
| | | | | | 2.5 | 100 | 66 | | | | | | |
| | | | | | | | | | | | | | Brown, decomposed, very soft. |
| | | | | | 2.5 | 100 | 66 | 1 | 28 | | | | Gray, unfractured, fresh, very hard. |
| 30 | | | | | | | | | | | | | Total Depth = 30 feet.<br>Groundwater not encountered during drilling.<br>Backfilled with approximately 4.5 cubic feet of cement-bentonite grout on 8/22/22. |
| | | | | | | | | | | | | | Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. |
| | | | | | | | | | | | | | Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report. |
| | | | | | | | | | | | | | The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents. |
| 35 | | | | | | | | | | | | | |
| 40 | | | | | | | | | | | | | |

DATE DRILLED 8/15/2022 & 8/22/2022 CORE NO. RC-2

GROUND ELEVATION ± 772 (MSL) SHEET 1 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger and 2.3 inch HQ Rock

DRIVE WEIGHT 140 lbs. (Auto-Trip 0 to 12 feet) DROP 30" (0 to 12 feet)

SAMPLED BY KWK LOGGED BY KWK REVIEWED BY ZH/GTF

| DEPTH (feet) | DRIVEN SAMPLE | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (ppd) | INTERVAL CORED (feet) | RECOVERY (%) | RQD (%) | FRACTURE FREQ. (ft) | FRACTURE DIP (degrees) | GRAPHIC LOG SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | | | | GM | AGGREGATE BASE:<br>Light brown, dry to moist, medium dense, silty GRAVEL, approximately 6 inches thick. |
| | | | | | | | | | | | SM | FILL:<br>Light reddish brown, moist, medium dense, silty fine to medium SAND. |
| | | | | | | | | | | | | Light brown. |
| 5 | | 50/3" | 3.4 | 120.4 | | | | | | | | GRANITIC ROCK:<br>Light gray to light brown, moist, moderately weathered, coarse-grained GRANITIC ROCK. |
| 10 | | 86/11" | | | | | | | | | | @ 11 feet: Switch to HQ rock coring. |
| | | | | | | | | | 70 | | | Gray, dry, slightly fractured, fresh, very hard, Mr, Irr, T, CL, Sta. |
| | | | | | 2.5 | 72 | 38 | 2 | | | | |
| | | | | | | | | | 40 | | | Brown, decomposed, very soft, Mr, Irr, T, CL, Sta. |
| | | | | | | | | | | | | Gray, slightly fractured, fresh, very hard. |
| | | | | | 1.5 | 100 | 92 | 1 | | | | Mr, Irr, T, Cl, Sta. |
| 15 | | | | | | | | | | | | Unfractured. |
| | | | | | 5 | 95 | 95 | | | | | |
| 20 | | | | | | | | | | | | |

FIGURE A-7

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TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA

DATE DRILLED 8/15/2022 & 8/22/2022 CORE NO. RC-2

GROUND ELEVATION ± 772 (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger and 2.3 inch HQ Rock

DRIVE WEIGHT 140 lbs. (Auto-Trip 0 to 12 feet) DROP 30" (0 to 12 feet)

SAMPLED BY KWK LOGGED BY KWK REVIEWED BY ZH/GTF

| DEPTH (feet) | DRIVEN SAMPLE | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (ppd) | INTERVAL CORED (feet) | RECOVERY (%) | RQD (%) | FRACTURE FREQ. (ft) | FRACTURE DIP (degrees) | GRAPHIC LOG | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 20 | | | | | 5 | 100 | 100 | | | | | | GRANITIC ROCK: (CONTINUED)<br>Gray, dry, unfractured, fresh, very hard, GRANITIC ROCK. |
| 25 | | | | | 5 | 100 | 100 | 1 | 12 | | | | Very slightly fractured, Mr, Un, Mo, CL, Sta.<br>Very slightly fractured, Mr, Un, Mo, CL, Sta. |
| 30 | | | | | | | | | | | | | Total Depth = 30 feet.<br>Groundwater not encountered during drilling.<br>Converted to a temporary piezometer following drilling on 8/22/2022.<br>Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.<br><br>Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report.<br><br>The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents. |
| 35 | | | | | | | | | | | | | |
| 40 | | | | | | | | | | | | | |

Ninyo & Moore

# TEST PIT LOG

TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA

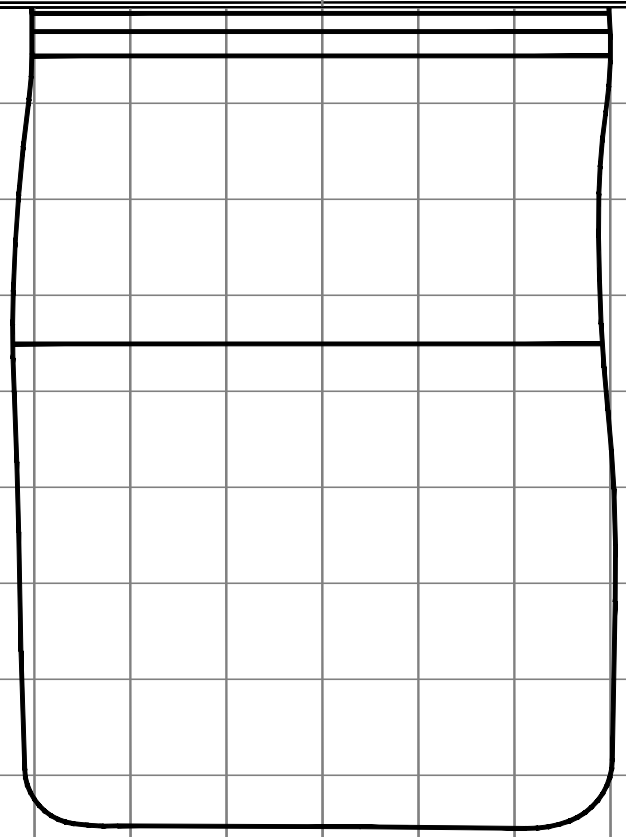
| PROJECT NO. | DATE |
|---|---|
| 109471001 | 12/22 |

DATE EXCAVATED 8/12/22 TEST PIT NO. TP-1

GROUND ELEVATION 799' ± (MSL) LOGGED BY KWK

METHOD OF EXCAVATION Excavator

LOCATION Employee Parking Lot



| DEPTH (FEET) | SAMPLES: Bulk | SAMPLES: Driven | SAMPLES: Sand Cone | MOISTURE (%) | DRY DENSITY (PCF) | CLASSIFICATION U.S.C.S. | DESCRIPTION |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | GM | ASPHALT CONCRETE: Approximately 3.5 inches thick. |
| | | | | | | SC | AGGREGATE BASE: Brown, moist, medium dense, silty GRAVEL approximately 3 inches thick. |
| | | | | | | SM | FILL: Light brown, moist, medium dense, clayey fine to medium SAND. |
| 4 | | | | | | | COLLUVIUM: Gray, moist, medium dense, silty fine to medium SAND; trace rounded gravel; organic odor. |
| 8 | | | | 5.0 | | | GRANITIC ROCK: Light brown, moist, decomposed, coarse-grained GRANITIC ROCK. |
| | | | | | | | Difficult excavating, laminated yellowish-red, increased clay content. |
| 12 | | | | | | | |
| | | | | | | | Seepage @ 15.3 feet. |
| 16 | | | | | | | Excavating refusal on granitic rock. |
| | | | | | | | Total Depth = 17 feet. (Refusal) Groundwater seepage encountered at 15 feet. Backfilled on 8/12/22, patched with temporary asphalt patching. |
| 20 | | | | | | | Note: Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report. |
| | | | | | | | The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents. |
| 24 | | | | | | | |

FIGURE A-9

SCALE = 1 in./4 ft.

# APPENDIX B

## Previous Boring Logs
## (Ninyo & Moore, 2010)

DATE DRILLED 8/11/10 BORING NO. B-1

GROUND ELEVATION 758' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-75) (Baja Exploration)

DRIVE WEIGHT 140 lbs. (Auto-Trip Hammer) DROP 30"

SAMPLED BY MJB LOGGED BY MJB REVIEWED BY JG

| DEPTH (feet) | SAMPLES (Bulk / Driven) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | | ASPHALT CONCRETE:<br>Approximately 4 inches thick. |
| | | | | | | GM | BASE MATERIAL:<br>Light brown, damp, medium dense, silty GRAVEL; approximately 11 inches thick. |
| | | | | | | SM | FILL:<br>Pale reddish brown, damp, loose to medium dense, silty fine to coarse SAND; scattered fine to coarse gravel and cobbles. |
| 5 | | 50/4" | | | | | GRANITIC ROCK:<br>Pale reddish brown, damp, GRANITIC ROCK; soft; intensely weathered.<br>@ 3': Large cobble/hard granite.<br>@ 5': Refusal on hard granite. Moved boring over 5 feet. Redrilled to 8 feet.<br>@ 7': Granite; very hard drilling.<br>Refusal on very hard granite. |
| 10 | | | | | | | Total Depth = 8 feet.<br>Groundwater not encountered during drilling.<br>Backfilled and patched with black-dyed concrete shortly after drilling on 8/11/10.<br><br>Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. |
| 15 | | | | | | | |
| 20 | | | | | | | |

Ninyo & Moore

**BORING LOG**

CIP 1757 - CLEARWELL REDUNDANT PIPELINE PROJECT
POWAY, CALIFORNIA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 106729001 | 11/10 | A-1 |

DATE DRILLED 8/11/10 BORING NO. B-2

GROUND ELEVATION 776' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-75) (Baja Exploration)

DRIVE WEIGHT 140 lbs. (Auto-Trip Hammer) DROP 30"

SAMPLED BY MJB LOGGED BY MJB REVIEWED BY JG

| DEPTH (feet) | SAMPLES (Bulk / Driven) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | | GRANITIC ROCK:<br>Pale reddish brown to reddish brown, damp, soft, GRANITIC ROCK; weathered. |
| 5 | | 100/4" | 2.0 | 113.3 | | | Dry. |
| 10 | | 50/4" | 19.7 | 107.4 | | | Moist to wet. |
| 15 | | 100/1" | | | | | Increase in drilling difficulty. |
| | | | ▽ | | | | Saturated; very hard drilling. Refusal on hard rock. |
| | | | | | | | Total Depth = 17.3 feet.<br>Groundwater encountered at approximately 17 feet during drilling.<br>Backfilled with approximately 4.5 cubic feet of bentonite grout shortly after drilling on 8/11/10.<br>Note: Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report. |
| 20 | | | | | | | |

Ninyo & Moore

**BORING LOG**

CIP 1757 - CLEARWELL REDUNDANT PIPELINE PROJECT
POWAY, CALIFORNIA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 106729001 | 11/10 | A-2 |

DATE DRILLED 8/11/10 BORING NO. B-3

GROUND ELEVATION 795' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-75) (Baja Exploration)

DRIVE WEIGHT 140 lbs. (Auto-Trip Hammer) DROP 30"

SAMPLED BY MJB LOGGED BY MJB REVIEWED BY JG

| DEPTH (feet) | SAMPLES (Bulk / Driven) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | GP | ASPHALT CONCRETE:<br>Approximately 3 inches thick. |
| | | | | | | SP | BASE MATERIAL:<br>Brown, damp, medium dense, poorly graded sandy GRAVEL; approximately 4 inches thick. |
| | | | | | | | FILL:<br>Brown to reddish brown, damp to moist, dense, poorly graded SAND; trace silt and clay; scattered fine to coarse gravel. |
| | | | | | | SM | Dark gray, damp, dense, silty fine to medium SAND; scattered coarse gravel and cobbles. |
| 5 | | 46 | 7.4 | 121.7 | | | |
| | | | | | | SM | OLD ALLUVIUM:<br>Pale reddish brown, moist, very dense, silty, SAND; scattered fine to coarse gravel and cobbles. |
| 10 | | 73 | 21.0 | 110.0 | | | Wet. |
| | | | | | | SC | Grayish olive, moist to wet, medium dense, clayey fine to coarse SAND; calcium carbonate nodules. |
| 15 | | 35 | 14.8 | 118.2 | | | |
| | | | | | | | Total Depth = 16.5 feet.<br>Groundwater not encountered during drilling.<br>Backfilled with bentonite and patched with asphalt concrete shortly after drilling on 8/11/10.<br><br>Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. |
| 20 | | | | | | | |

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**BORING LOG**

CIP 1757 - CLEARWELL REDUNDANT PIPELINE PROJECT
POWAY, CALIFORNIA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 106729001 | 11/10 | A-3 |

DATE DRILLED 8/11/10 BORING NO. B-4

GROUND ELEVATION 815' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-75) (Baja Exploration)

DRIVE WEIGHT 140 lbs. (Auto-Trip Hammer) DROP 30"

SAMPLED BY MJB LOGGED BY MJB REVIEWED BY JG

| DEPTH (feet) | SAMPLES (Bulk / Driven) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | GP | ASPHALT CONCRETE:<br>Approximately 3 inches thick.<br>BASE MATERIAL:<br>Brown, damp, medium dense, poorly graded GRAVEL; approximately 4 inches thick.<br>GRANITIC ROCK:<br>Reddish brown, damp, GRANITC ROCK; soft; weathered; contains gravel- and cobble-size pieces of less weathered granitic rock; micaceous; (recovered as dense, silty fine to coarse sand). |
| 5 | | 60 | 5.1 | 129.9 | | | Dark gray. |
| 10 | | 22 | 5.5 | 123.4 | | | Reddish brown; micaceous.<br>Difficult drilling. |
| 15 | | 50/1" | | | | | Refusal to further sampling.<br>Total Depth = 15.1 feet.<br>Groundwater not encountered during drilling.<br>Backfilled with bentonite and patched with black-dyed concrete shortly after drilling on 8/11/10.<br><br>Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. |
| 20 | | | | | | | |

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**BORING LOG**

CIP 1757 - CLEARWELL REDUNDANT PIPELINE PROJECT
POWAY, CALIFORNIA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 106729001 | 11/10 | A-4 |

DATE DRILLED 8/11/10 BORING NO. B-5

GROUND ELEVATION 840' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-75) (Baja Exploration)

DRIVE WEIGHT 140 lbs. (Auto-Trip Hammer) DROP 30"

SAMPLED BY MJB LOGGED BY MJB REVIEWED BY JG

| DEPTH (feet) | SAMPLES (Bulk / Driven) | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | GP | ASPHALT CONCRETE:<br>Approximately 2 inches thick. |
| | | | | | | SP-SM | BASE MATERIAL:<br>Brown, damp, medium dense, poorly graded GRAVEL; approximately 5 inches thick. |
| | | | | | | | FILL:<br>Brown to reddish brown, damp to moist, dense, poorly graded SAND with silt; few coarse sand; scattered fine to coarse gravel. |
| 5 | | 54 | 8.5 | 134.7 | | | Dark grayish and reddish brown. |
| 10 | | 85/11" | 12.3 | 123.5 | | | GRANITIC ROCK:<br>Reddish brown, moist, GRANITIC ROCK; sift; weathered; (recovered as very dense, silty fine to coarse sand). |
| 15 | | 85/10" | 7.4 | 129.5 | | | Total Depth = 16.3 feet.<br>Groundwater not encountered during drilling.<br>Backfilled with bentonite and patched with black-dyed concrete shortly after drilling on 8/11/10.<br><br>Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. |
| 20 | | | | | | | |

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**BORING LOG**

CIP 1757 - CLEARWELL REDUNDANT PIPELINE PROJECT
POWAY, CALIFORNIA

| PROJECT NO. | DATE | FIGURE |
|---|---|---|
| 106729001 | 11/10 | A-5 |

# APPENDIX C

## Geotechnical Laboratory Testing

# APPENDIX C

## GEOTECHNICAL LABORATORY TESTING

**Classification**
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory excavations in Appendix A.

**In-Place Moisture and Density Tests**
The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory excavations in Appendix A.

**Gradation Analysis**
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 422. The grain-size distribution curves are shown on Figures C-1 and C-2. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

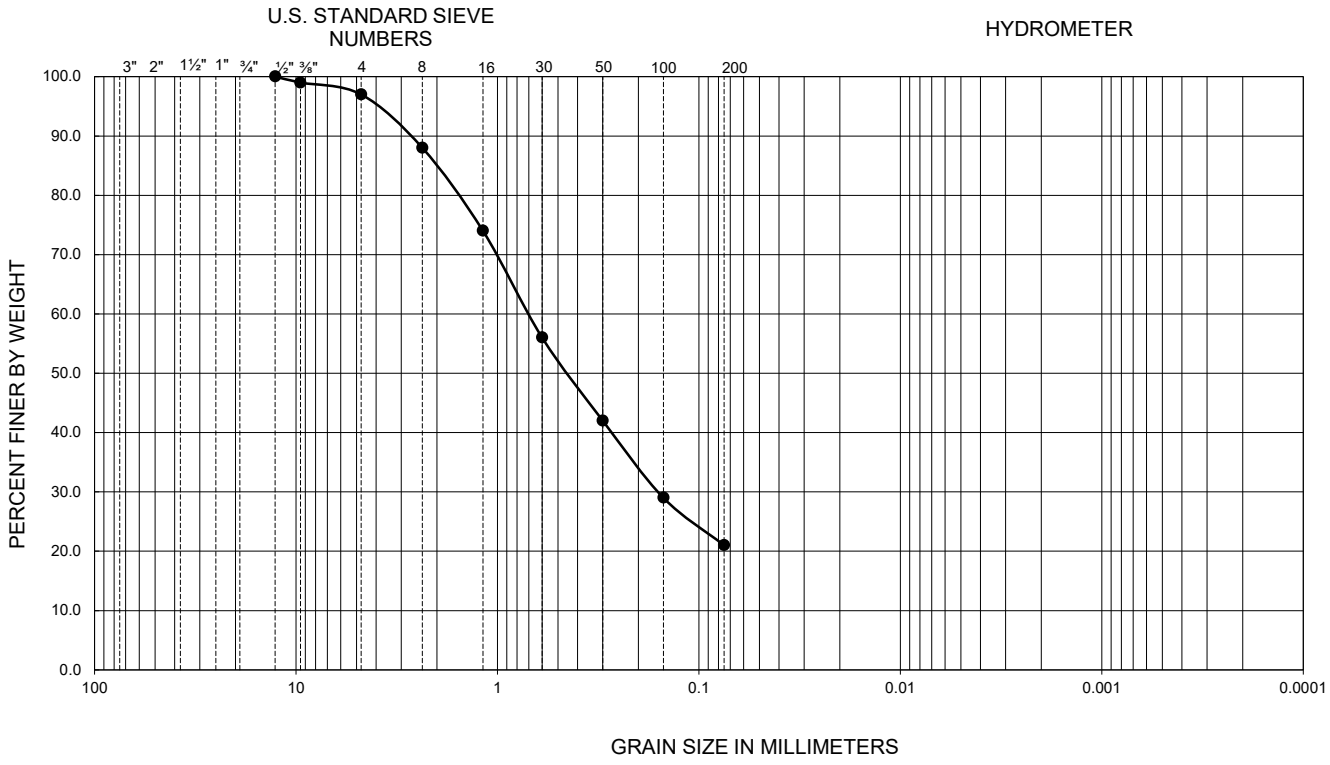
**Direct Shear Test**
Direct shear tests were performed on relatively undisturbed samples in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected material. The samples were inundated during shearing to represent adverse field conditions. The results are shown on Figures C-3 and C-4.

**Expansion Index Test**
The expansion index of selected materials was evaluated in general accordance with ASTM D 4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimen was loaded with a surcharge of 144 pounds per square foot and was inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of this test are presented on Figure C-5.

**Soil Corrosivity Tests**
Soil pH and electrical resistivity tests were performed on a representative sample in general accordance with CT 643. The sulfate and chloride contents of the selected sample were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure C-6.

**R-Value**
The resistance value, or R-value, for site soils was evaluated in general accordance with California Test (CT) 301. Samples were prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test results are shown on Figure C-7.

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | SILT | CLAY |




| Symbol | Sample Location | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (percent) | USCS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | B-2 | 1.5-5.0 | -- | -- | -- | -- | -- | -- | -- | -- | 21 | SM |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

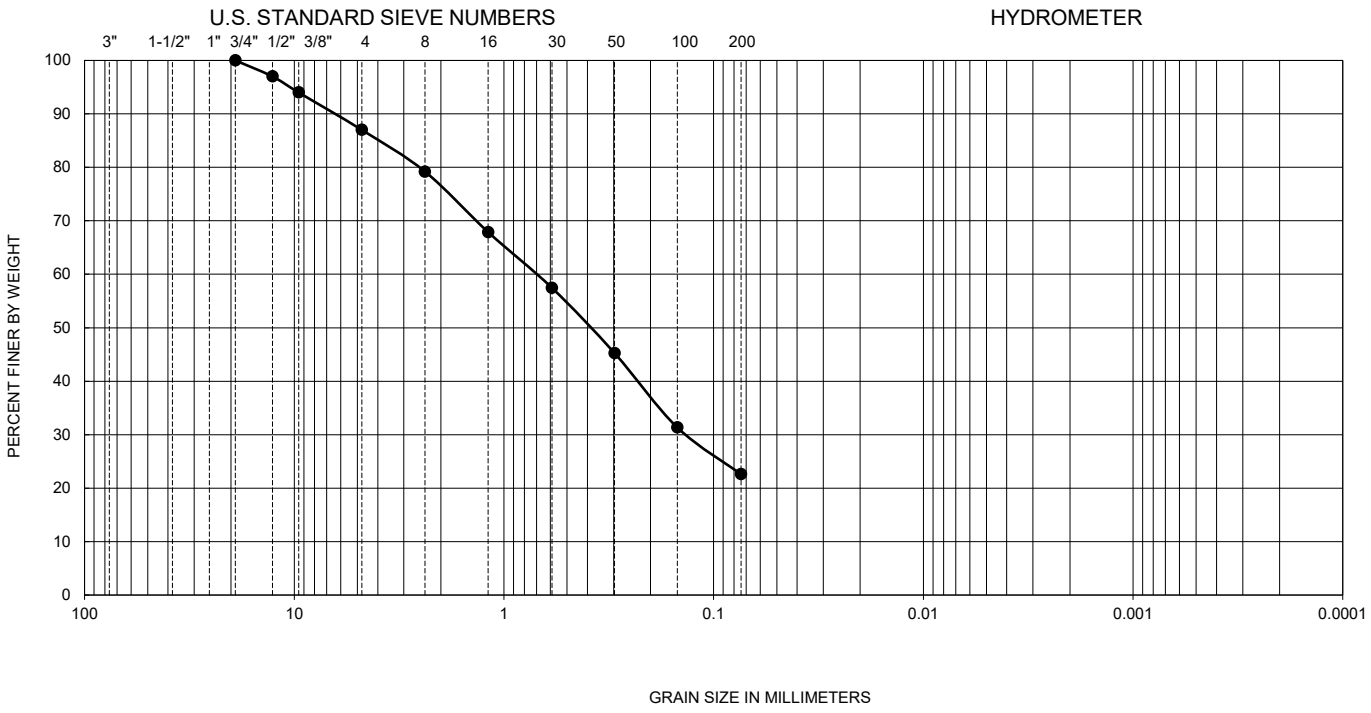
**FIGURE C-1**

## GRADATION TEST RESULTS

TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

| GRAVEL | | SAND | | | FINES | |
|---|---|---|---|---|---|---|
| Coarse | Fine | Coarse | Medium | Fine | SILT | CLAY |




| Symbol | Sample Location | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (percent) | USCS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | TP-1 | 6.5-11.0 | -- | -- | -- | -- | -- | -- | -- | -- | 23 | SM |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

**FIGURE C-2**

**GRADATION TEST RESULTS**

TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA

109471001 | 12/22

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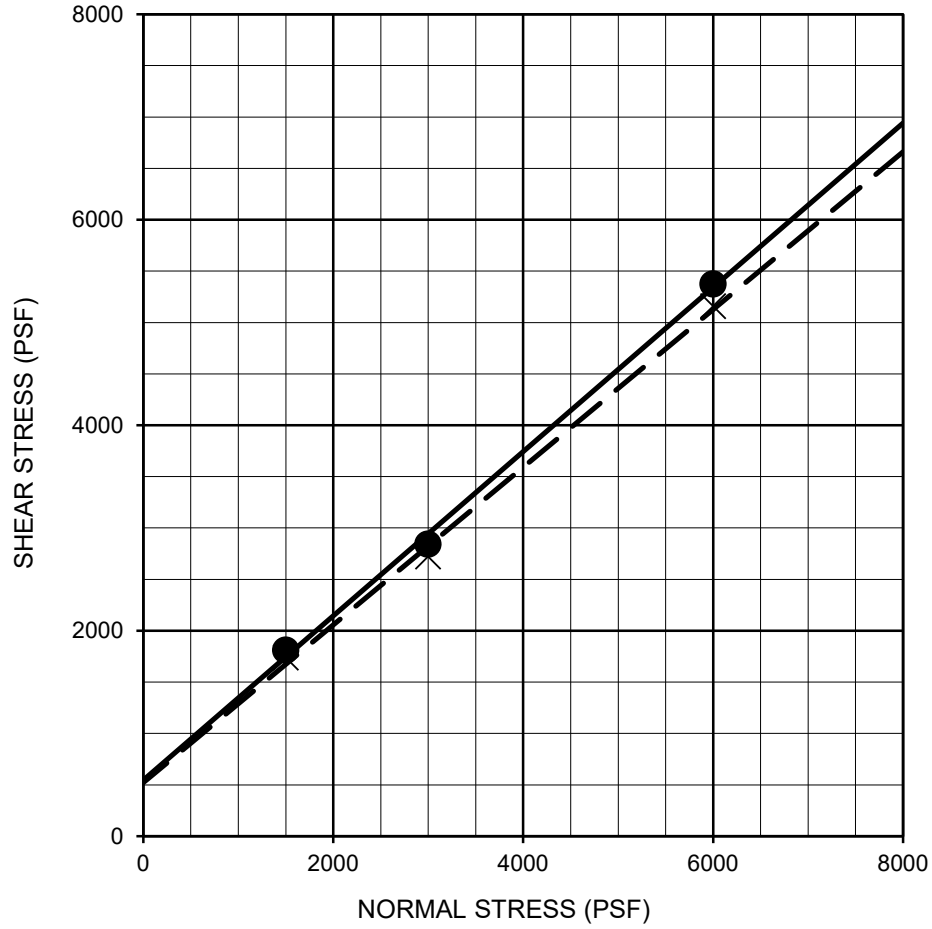
109471001_SPLITSIEVE TP-1 @ 6.5-11.0




| Description | Symbol | Sample Location | Depth (ft) | Shear Strength | Cohesion (psf) | Friction Angle (degrees) | Soil Type |
|---|---|---|---|---|---|---|---|
| Granitic ROCK | —●— | B-1 | 5.0-5.9 | Peak | 550 | 39 | Bedrock |
| Granitic ROCK | – – X – – | B-1 | 5.0-5.9 | Ultimate | 520 | 38 | Bedrock |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

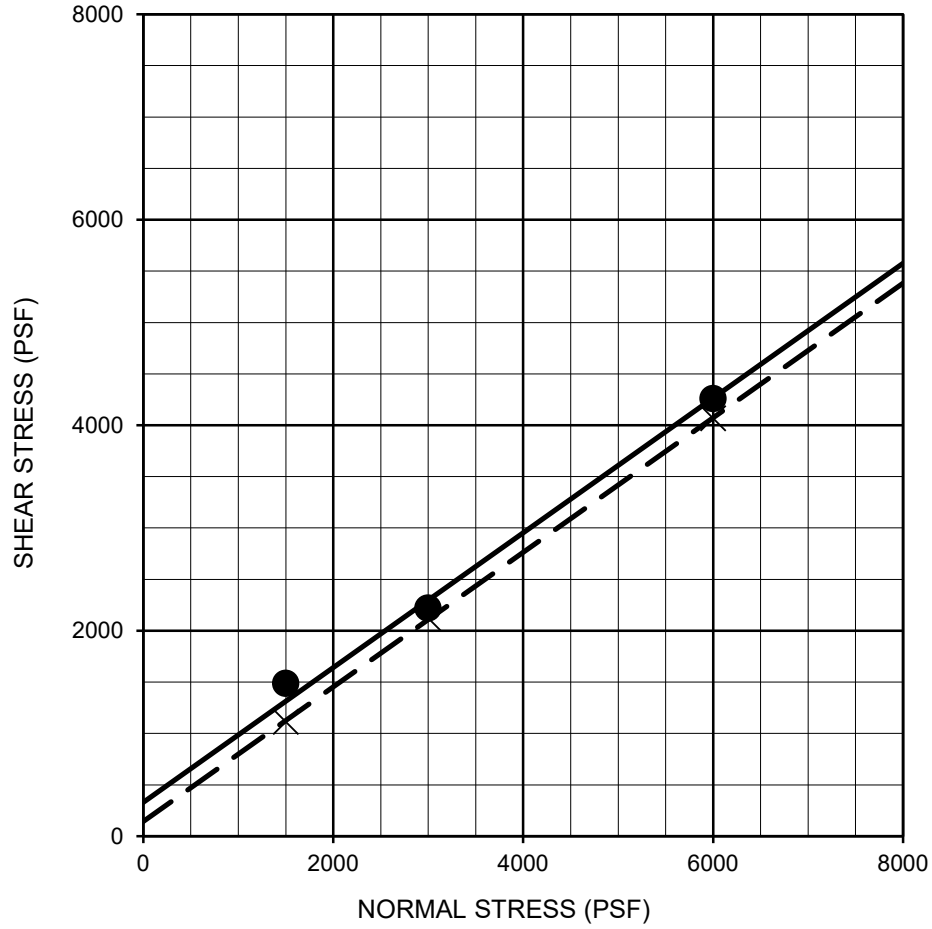
**FIGURE C-3**

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

**DIRECT SHEAR TEST RESULTS**

TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA

109471001 | 12/22




| Description | Symbol | Sample Location | Depth (ft) | Shear Strength | Cohesion (psf) | Friction Angle (degrees) | Soil Type |
|---|---|---|---|---|---|---|---|
| Clayey SAND | —●— | B-3 | 5.0-6.5 | Peak | 330 | 33 | SC |
| Clayey SAND | – – X – – | B-3 | 5.0-6.5 | Ultimate | 140 | 33 | SC |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

**FIGURE C-4**

Ninyo & Moore
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**DIRECT SHEAR TEST RESULTS**

TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA

109471001 | 12/22

| SAMPLE LOCATION | SAMPLE DEPTH (ft) | INITIAL MOISTURE (percent) | COMPACTED DRY DENSITY (pcf) | FINAL MOISTURE (percent) | VOLUMETRIC SWELL (in) | EXPANSION INDEX | POTENTIAL EXPANSION |
|---|---|---|---|---|---|---|---|
| B-3 | 1.0-5.0 | 7.5 | 121.2 | 12.2 | 0.001 | 1 | Very Low |

PERFORMED IN GENERAL ACCORDANCE WITH ☐ UBC STANDARD 18-2 ☑ ASTM D 4829

**FIGURE C-5**

**EXPANSION INDEX TEST RESULTS**

TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA

109471001 | 12/22

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| SAMPLE LOCATION | SAMPLE DEPTH (ft) | pH [1] | RESISTIVITY [1] (ohm-cm) | SULFATE CONTENT [2] | | CHLORIDE CONTENT [3] (ppm) |
|---|---|---|---|---|---|---|
| | | | | (ppm) | (%) | |
| TP-1 | 1.5-6.5 | 8.1 | 4,100 | 30 | 0.003 | 85 |
| B-3 | 1.0-5.0 | 7.9 | 2,800 | 10 | 0.001 | 145 |

[1] PERFORMED IN ACCORDANCE WITH CALIFORNIA TEST METHOD 643
[2] PERFORMED IN ACCORDANCE WITH CALIFORNIA TEST METHOD 417
[3] PERFORMED IN ACCORDANCE WITH CALIFORNIA TEST METHOD 422

**FIGURE C-6**

**CORROSIVITY TEST RESULTS**

TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY RODA, POWAY, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

109471001 | 12/22

| SAMPLE LOCATION | SAMPLE DEPTH (ft) | SOIL TYPE | R-VALUE |
|---|---|---|---|
| TP-1 | 1.5-6.5 | Silty SAND (SM) | 70 |
| B-3 | 1.0-5.0 | Silty SAND (SM) | 69 |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

**FIGURE C-7**

**R-VALUE TEST RESULTS**

TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA

109471001 | 12/22

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

| | | | | | |
|---|---|---|---|---|---|
| Client: | Ninyo & Moore | Sampled By: | Client | Date: | 8/11 & 8/15/2022 |
| Project Name: | Clearwell (109471001) | Submitted By: | Client | Date: | 9/7/2022 |
| Project No.: | 5015-19-0022.04 | Tested By: | MG/RS | Date: | 9/16/2022 |
| Project Location: | Poway Treated Water Facility | Reviewed By: | MF | Date: | 9/19/2022 |

## ASTM D 7012 Method C/D - Compressive Strength and Bulk Density of Rock Core Specimens/Young's Modulus

| Lab No. | Boring ID | Sample Depth (ft) | Bulk Density (lb./ft$^3$) | Compressive Strength (psi) | Young's Modulus (psi) | Sample Description | Comments |
|---|---|---|---|---|---|---|---|
| 34780 | RC-1 | 18.5-20.0 | 178.1 | 24,360 | 14,450,000 | Granitic rock | |
| 34782 | RC-1 | 25.9-27.5 | 178.2 | 23,970 | 15,100,000 | Granitic rock | |
| 34784 | RC-2 | 13.5-15.0 | 178.8 | 23,220 | 15,500,000 | Granitic rock | |
| 34786 | RC-2 | 20.0-25.0 | 177.9 | 22,920 | 15,750,000 | Granitic rock | |
| 34788 | RC-2 | 27.5-30.0 | 178.0 | 25,060 | 15,700,000 | Granitic rock | |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |

**Notes:**

1.) See attached sheets for Young's Modulus stress/strain graphs and conformance to ASTM D4543. All samples met ASTM D4543 criteria before compressive strength tested.

2.) All samples were tested at approximate as-received moisture content and standard laboratory room temperature.

3.) Bulk density based upon measured weight and dimensions.

5.) Axial deformation/strain data was obtained by compressometer. Correction factor for compressometer was applied per Section 4.2.1 of ASTM C 469-14e1.

6.) Young's Modulus was calculated by average slope of the more-or-less straight-line portion of the stress-strain curve per Section 11.2.5.2 and Fig. 2 (b) of ASTM D7012-14e1.

Submitted by:

Michael Farr, CEG 1938
Associate Engineering Geologist

| | | | | | |
|---|---|---|---|---|---|
| Client: | Ninyo & Moore | Sampled By: | Client | Date: | 8/11 & 8/15/2022 |
| Project Name: | Clearwell (109471001) | Submitted By: | Client | Date: | 9/7/2022 |
| Project No.: | 5015-19-0022.04 | Tested By: | MG/RS/RM | Date: | 9/16/2022 |
| Project Location: | Poway Treated Water Facility | Reviewed By: | MF | Date: | 9/19/2022 |
| Lab No.: | 34780 | Boring ID: | RC-1 | Depth: | 18.5-20.0 |
| **Description:** | Granitic rock | Dial Reading (lbs) : | 110,340 | Wt(g): | 1230.8 |

## ASTM D 4543-19: DETERMINATION OF DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS

### LENGTH/DIAMETER RATIO (ASTM D 4543 Section 7.2)

| | 1 | 2 | Average | |
|---|---|---|---|---|
| Specimen Length (in): | 5.81 | 5.81 | 5.81 | |
| Specimen Diameter (in): | 2.40 | 2.40 | 2.40 | |
| Length to Diameter Ratio: | 2.4 | **L/D Tolerance Met?** | | **YES** |

### DEVIATION FROM STRAIGHTNESS (ASTM D 4543 Section 9.1.1) PROCEDURE S1

| | Max | Min | Difference | |
|---|---|---|---|---|
| Measurement at 0˚ (in): | 0.088 | 0.073 | 0.015 | Maximum difference must be < 0.020 in. |
| Measurement at 120˚ (in): | 0.089 | 0.075 | 0.014 | **Straightness Tolerance Met?** **YES** |
| Measurement at 240˚ (in): | 0.088 | 0.073 | 0.015 | |

### END FLATNESS AND PARALLELISM (ASTM D 4543 Section 9.2.1): PROCEDURE FP1

| END 1 | -1.000 | -0.875 | -0.750 | -0.625 | -0.500 | -0.375 | -0.250 | -0.125 | 0.000 | 0.125 | 0.250 | 0.375 | 0.500 | 0.625 | 0.750 | 0.875 | 1.000 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Diameter 1 at 0˚ (in): | -0.0030 | -0.0025 | -0.0022 | -0.0019 | -0.0013 | -0.0009 | -0.0004 | -0.0002 | 0.0000 | 0.0002 | 0.0002 | 0.0004 | 0.0005 | 0.0004 | 0.0006 | 0.0004 | 0.0003 |
| Best-Fit Line: | -0.0023 | -0.0021 | -0.0019 | -0.0017 | -0.0015 | -0.0012 | -0.0010 | -0.0008 | -0.0006 | -0.0004 | -0.0002 | 0.0000 | 0.0003 | 0.0005 | 0.0007 | 0.0009 | 0.0011 |
| Difference: | 0.0007 | 0.0004 | 0.0003 | 0.0002 | 0.0002 | 0.0003 | 0.0006 | 0.0006 | 0.0006 | 0.0006 | 0.0004 | 0.0004 | 0.0003 | 0.0001 | 0.0001 | 0.0005 | 0.0008 |
| Diameter 2 at 90˚ (in): | -0.0012 | -0.0010 | -0.0008 | -0.0005 | -0.0002 | -0.0001 | 0.0000 | 0.0001 | 0.0000 | -0.0001 | -0.0003 | -0.0007 | -0.0010 | -0.0011 | -0.0017 | -0.0019 | -0.0023 |
| Best-Fit Line: | -0.0002 | -0.0003 | -0.0004 | -0.0004 | -0.0005 | -0.0006 | -0.0007 | -0.0007 | -0.0008 | -0.0009 | -0.0010 | -0.0010 | -0.0011 | -0.0012 | -0.0013 | -0.0013 | -0.0014 |
| Difference: | 0.0010 | 0.0007 | 0.0005 | 0.0001 | 0.0003 | 0.0005 | 0.0007 | 0.0008 | 0.0008 | 0.0008 | 0.0007 | 0.0003 | 0.0001 | 0.0001 | 0.0005 | 0.0006 | 0.0009 |
| END 2 | -1.000 | -0.875 | -0.750 | -0.625 | -0.500 | -0.375 | -0.250 | -0.125 | 0.000 | 0.125 | 0.250 | 0.375 | 0.500 | 0.625 | 0.750 | 0.875 | 1.000 |
| Diameter 1 at 0˚ (in): | -0.0014 | -0.0011 | -0.0009 | -0.0008 | -0.0006 | -0.0004 | -0.0001 | 0.0000 | 0.0001 | -0.0001 | 0.0000 | 0.0001 | 0.0000 | 0.0001 | 0.0000 | -0.0001 | -0.0001 |
| Best-Fit Line: | -0.0009 | -0.0008 | -0.0008 | -0.0007 | -0.0006 | -0.0005 | -0.0005 | -0.0004 | -0.0003 | -0.0002 | -0.0002 | -0.0001 | 0.0000 | 0.0001 | 0.0002 | 0.0002 | 0.0003 |
| Difference: | 0.0005 | 0.0003 | 0.0002 | 0.0001 | 0.0000 | 0.0001 | 0.0004 | 0.0004 | 0.0004 | 0.0001 | 0.0002 | 0.0002 | 0.0000 | 0.0000 | 0.0002 | 0.0003 | 0.0004 |
| Diameter 2 at 90˚ (in): | -0.0031 | -0.0026 | -0.0024 | -0.0018 | -0.0014 | -0.0010 | -0.0006 | -0.0003 | 0.0001 | 0.0002 | 0.0001 | 0.0004 | 0.0004 | 0.0004 | 0.0003 | 0.0003 | 0.0001 |
| Best-Fit Line: | -0.0023 | -0.0021 | -0.0019 | -0.0017 | -0.0015 | -0.0012 | -0.0010 | -0.0008 | -0.0006 | -0.0004 | -0.0002 | 0.0000 | 0.0003 | 0.0005 | 0.0007 | 0.0009 | 0.0011 |
| Difference: | 0.0008 | 0.0005 | 0.0005 | 0.0001 | 0.0000 | 0.0002 | 0.0004 | 0.0005 | 0.0007 | 0.0006 | 0.0003 | 0.0004 | 0.0002 | 0.0001 | 0.0004 | 0.0006 | 0.0010 |

Maximum difference between smooth curve and best-fit line must not be > 0.001 in. **Flatness Tolerance Met?** **YES**

DIAMETER 1

| | |
|---|---|
| End 1: Slope of Best Fit Line: | 0.0017 |
| Angle of Best Fit Line (degree): | 0.097 |
| End 2: Slope of Best Fit Line: | 0.0006 |
| Angle of Best Fit Line (degree): | 0.034 |
| Maximum Angular Difference (degree): | 0.063 |

Maxiumum angular difference must be ≤0.25˚ for spherically seated

**Parellelism Tolerance Met?** **YES**

DIAMETER 2

| | |
|---|---|
| End 1: Slope of Best Fit Line: | -0.0006 |
| Angle of Best Fit Line (degree): | -0.034 |
| End 2: Slope of Best Fit Line: | 0.0017 |
| Angle of Best Fit Line (degree): | 0.097 |
| Maximum Angular Difference (degree): | 0.132 |

Maxiumum angular difference must be ≤0.25˚ for spherically seated

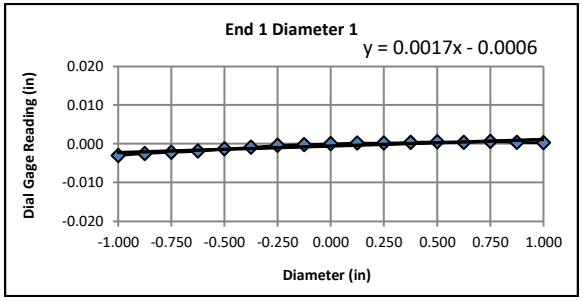
**Parellelism Tolerance Met?** **YES**

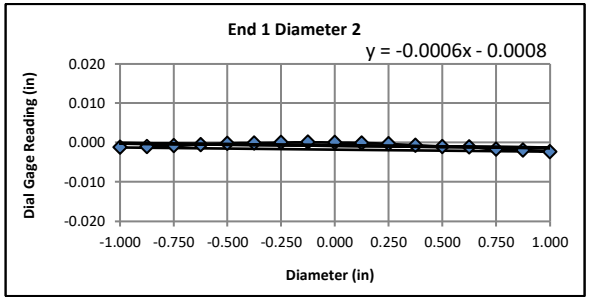
### PERPENDICULARITY (ASTM D 4543 Section 9.3.2): PROCEDURE P1

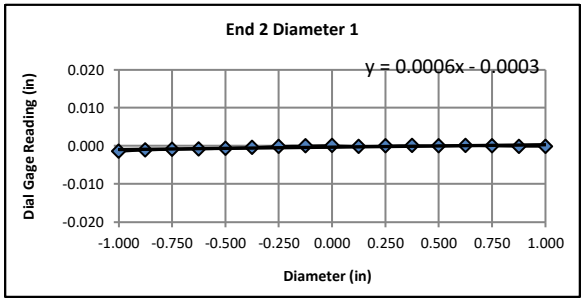
| | |
|---|---|
| Max Gap Between Square and Top of Specimen (in): | 0.005 |
| Gap / Length: | 0.0009 |

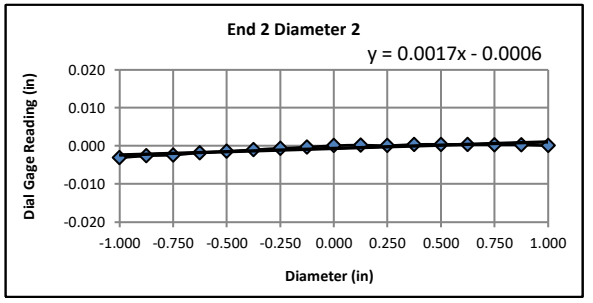
Gap / Length must be ≤ 0.0043

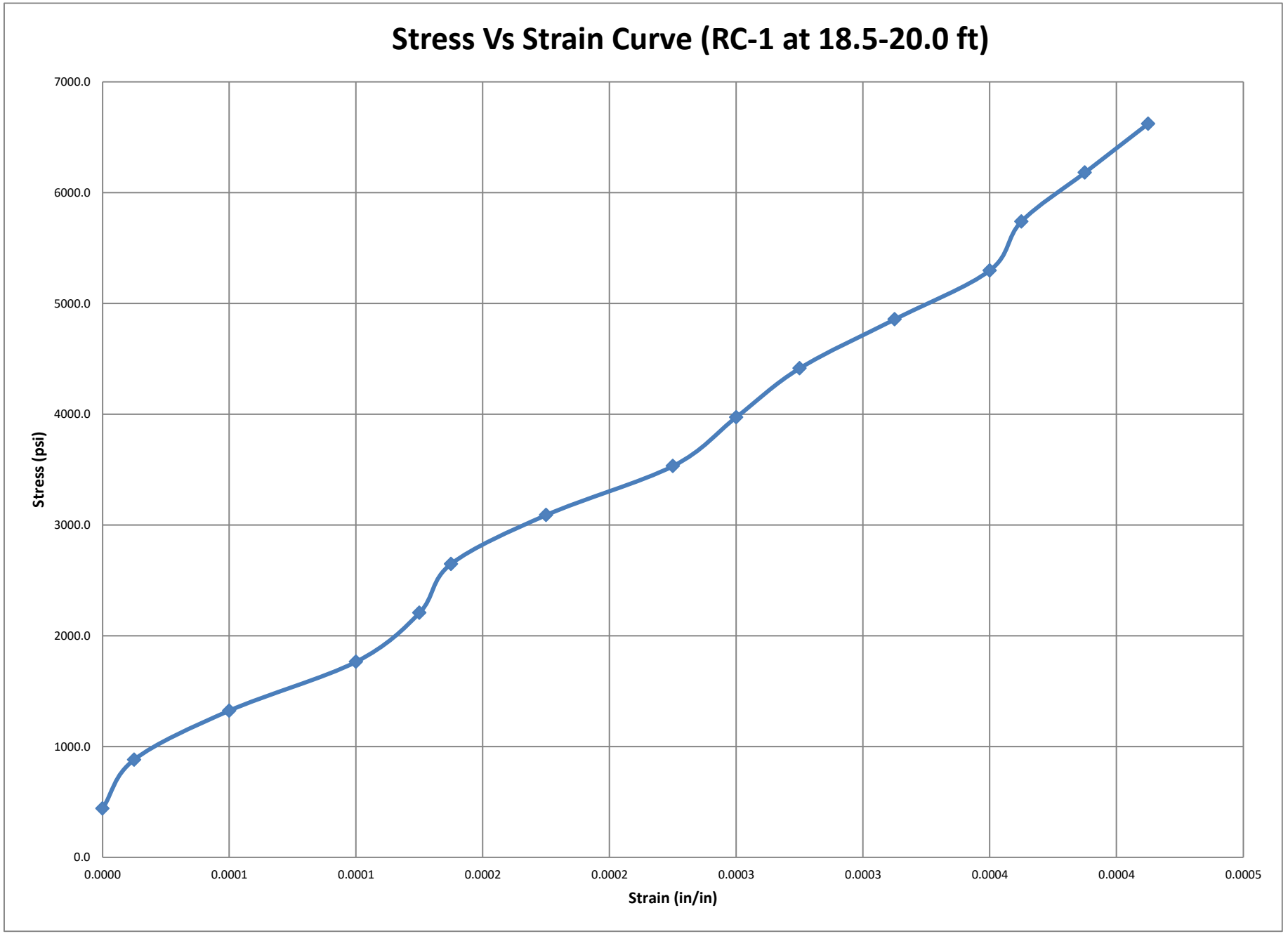
**Perpendicularity Tolerance Met?** **YES**




Stress Vs Strain Curve (RC-1 at 18.5-20.0 ft)
7000.0
6000.0
5000.0
4000.0
3000.0
2000.0
1000.0
0.0
Stress (psi)
0.0000
0.0001
0.0001
0.0002
0.0002
0.0003
0.0003
0.0004
0.0004
0.0005
Strain (in/in)

| | | | | | |
|---|---|---|---|---|---|
| Client: | Ninyo & Moore | Sampled By: | Client | Date: | 8/11 & 8/15/2022 |
| Project Name: | Clearwell (109471001) | Submitted By: | Client | Date: | 9/7/2022 |
| Project No.: | 5015-19-0022.04 | Tested By: | MG/RS/RM | Date: | 9/16/2022 |
| Project Location: | Poway Treated Water Facility | Reviewed By: | MF | Date: | 9/19/2022 |
| Lab No.: | 34782 | Boring ID: | RC-1 | Depth: | 25.9-27.5 |
| **Description:** | Granitic rock | Dial Reading (lbs) : | 108,575 | Wt(g): | 1231.5 |

## ASTM D 4543-19: DETERMINATION OF DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS

### LENGTH/DIAMETER RATIO (ASTM D 4543 Section 7.2)

| | 1 | 2 | Average | |
|---|---|---|---|---|
| Specimen Length (in): | 5.81 | 5.81 | 5.81 | |
| Specimen Diameter (in): | 2.40 | 2.40 | 2.40 | |
| Length to Diameter Ratio: | 2.4 | **L/D Tolerance Met?** | | **YES** |

### DEVIATION FROM STRAIGHTNESS (ASTM D 4543 Section 9.1.1) PROCEDURE S1

| | Max | Min | Difference |
|---|---|---|---|
| Measurement at 0˚ (in): | 0.087 | 0.072 | 0.015 |
| Measurement at 120˚ (in): | 0.084 | 0.079 | 0.005 |
| Measurement at 240˚ (in): | 0.090 | 0.079 | 0.012 |

Maximum difference must be < 0.020 in.

**Straightness Tolerance Met?** **YES**

### END FLATNESS AND PARALLELISM (ASTM D 4543 Section 9.2.1): PROCEDURE FP1

| END 1 | -1.000 | -0.875 | -0.750 | -0.625 | -0.500 | -0.375 | -0.250 | -0.125 | 0.000 | 0.125 | 0.250 | 0.375 | 0.500 | 0.625 | 0.750 | 0.875 | 1.000 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Diameter 1 at 0˚ (in): | -0.0012 | -0.0009 | -0.0007 | -0.0006 | -0.0005 | -0.0003 | -0.0002 | -0.0001 | -0.0001 | -0.0002 | -0.0004 | -0.0003 | -0.0004 | -0.0005 | -0.0007 | -0.0008 | -0.0010 |
| Best-Fit Line: | -0.0006 | -0.0006 | -0.0006 | -0.0006 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 |
| Difference: | 0.0006 | 0.0003 | 0.0001 | 0.0000 | 0.0000 | 0.0002 | 0.0003 | 0.0004 | 0.0004 | 0.0003 | 0.0001 | 0.0002 | 0.0001 | 0.0000 | 0.0002 | 0.0003 | 0.0005 |
| Diameter 2 at 90˚ (in): | -0.0011 | -0.0010 | -0.0008 | -0.0006 | -0.0004 | -0.0003 | -0.0001 | 0.0000 | 0.0001 | 0.0000 | 0.0000 | -0.0001 | -0.0003 | -0.0006 | -0.0008 | -0.0010 | -0.0013 |
| Best-Fit Line: | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 |
| Difference: | 0.0006 | 0.0005 | 0.0003 | 0.0001 | 0.0001 | 0.0002 | 0.0004 | 0.0005 | 0.0006 | 0.0005 | 0.0005 | 0.0004 | 0.0002 | 0.0001 | 0.0003 | 0.0005 | 0.0008 |
| END 2 | -1.000 | -0.875 | -0.750 | -0.625 | -0.500 | -0.375 | -0.250 | -0.125 | 0.000 | 0.125 | 0.250 | 0.375 | 0.500 | 0.625 | 0.750 | 0.875 | 1.000 |
| Diameter 1 at 0˚ (in): | -0.0022 | -0.0020 | -0.0017 | -0.0015 | -0.0009 | -0.0007 | -0.0006 | -0.0003 | -0.0002 | -0.0001 | -0.0004 | -0.0007 | -0.0010 | -0.0013 | -0.0015 | -0.0018 | -0.0020 |
| Best-Fit Line: | -0.0012 | -0.0012 | -0.0012 | -0.0012 | -0.0012 | -0.0012 | -0.0011 | -0.0011 | -0.0011 | -0.0011 | -0.0011 | -0.0011 | -0.0011 | -0.0010 | -0.0010 | -0.0010 | -0.0010 |
| Difference: | 0.0010 | 0.0008 | 0.0005 | 0.0003 | 0.0003 | 0.0005 | 0.0005 | 0.0008 | 0.0009 | 0.0010 | 0.0007 | 0.0004 | 0.0001 | 0.0003 | 0.0005 | 0.0008 | 0.0010 |
| Diameter 2 at 90˚ (in): | -0.0023 | -0.0020 | -0.0018 | -0.0017 | -0.0012 | -0.0008 | -0.0005 | -0.0002 | 0.0000 | 0.0000 | 0.0000 | -0.0002 | -0.0008 | -0.0012 | -0.0015 | -0.0017 | -0.0019 |
| Best-Fit Line: | -0.0013 | -0.0013 | -0.0013 | -0.0012 | -0.0012 | -0.0011 | -0.0011 | -0.0011 | -0.0010 | -0.0010 | -0.0010 | -0.0009 | -0.0009 | -0.0009 | -0.0008 | -0.0008 | -0.0008 |
| Difference: | 0.0010 | 0.0007 | 0.0006 | 0.0005 | 0.0000 | 0.0003 | 0.0006 | 0.0009 | 0.0010 | 0.0010 | 0.0010 | 0.0007 | 0.0001 | 0.0003 | 0.0007 | 0.0009 | 0.0011 |

Maximum difference between smooth curve and best-fit line must not be > 0.001 in.

**Flatness Tolerance Met?** **YES**

DIAMETER 1

| | |
|---|---|
| End 1: Slope of Best Fit Line: | 0.0001 |
| Angle of Best Fit Line (degree): | 0.003 |
| End 2: Slope of Best Fit Line: | 0.0001 |
| Angle of Best Fit Line (degree): | 0.006 |
| Maximum Angular Difference (degree): | 0.003 |

Maxiumum angular difference must be ≤0.25˚ for spherically seated

**Parellelism Tolerance Met?** **YES**

DIAMETER 2

| | |
|---|---|
| End 1: Slope of Best Fit Line: | 0.0000 |
| Angle of Best Fit Line (degree): | -0.001 |
| End 2: Slope of Best Fit Line: | 0.0003 |
| Angle of Best Fit Line (degree): | 0.016 |
| Maximum Angular Difference (degree): | 0.017 |

Maxiumum angular difference must be ≤0.25˚ for spherically seated

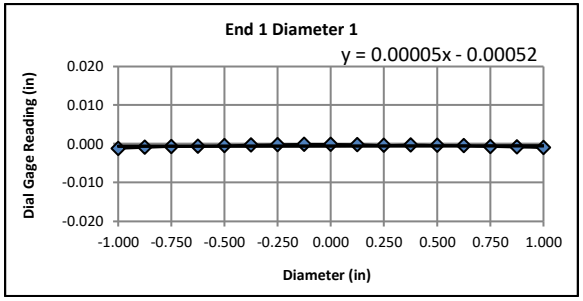
**Parellelism Tolerance Met?** **YES**

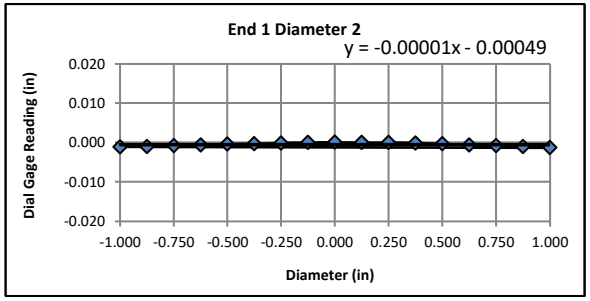
### PERPENDICULARITY (ASTM D 4543 Section 9.3.2): PROCEDURE P1

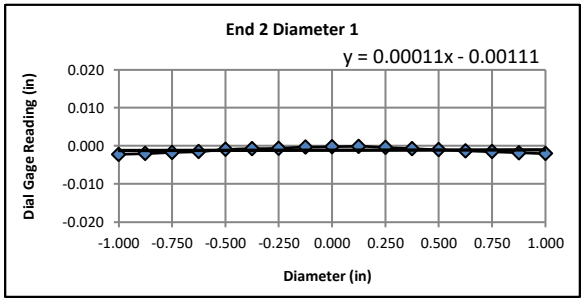
| | |
|---|---|
| Max Gap Between Square and Top of Specimen (in): | 0.005 |
| Gap / Length: | 0.0009 |

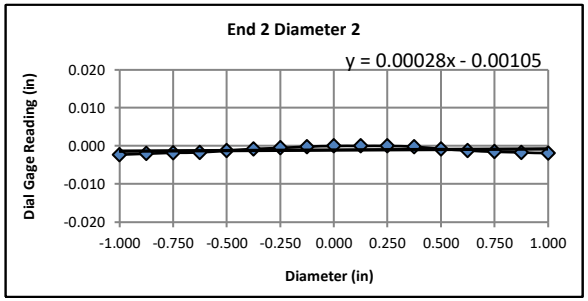
Gap / Length must be ≤ 0.0043

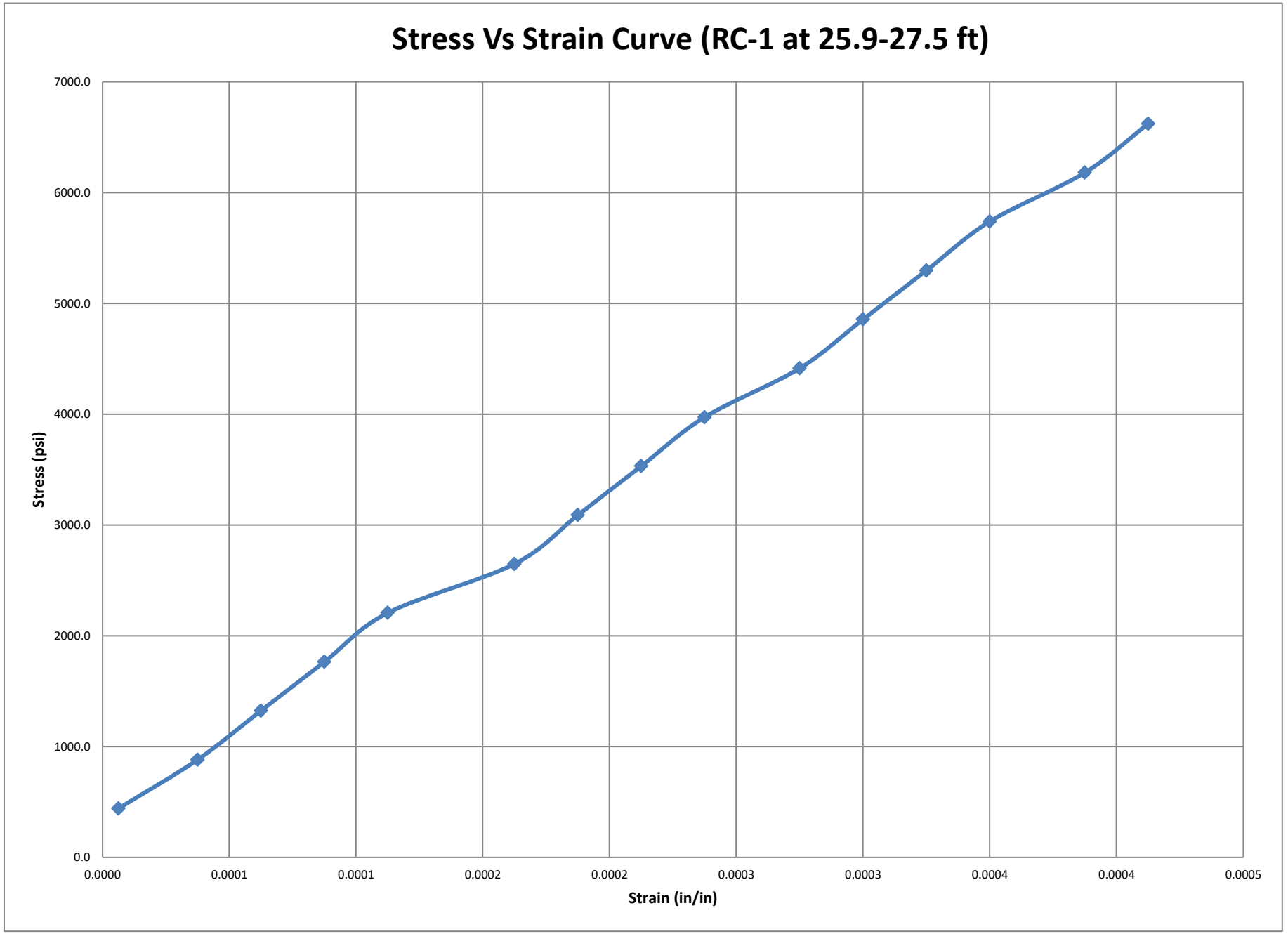
**Perpendicularity Tolerance Met?** **YES**




Stress Vs Strain Curve (RC-1 at 25.9-27.5 ft)
7000.0
6000.0
5000.0
4000.0
3000.0
2000.0
1000.0
0.0
Stress (psi)
0.0000
0.0001
0.0001
0.0002
0.0002
0.0003
0.0003
0.0004
0.0004
0.0005
Strain (in/in)

| | | | | | |
|---|---|---|---|---|---|
| Client: | Ninyo & Moore | Sampled By: | Client | Date: | 8/11 & 8/15/2022 |
| Project Name: | Clearwell (109471001) | Submitted By: | Client | Date: | 9/7/2022 |
| Project No.: | 5015-19-0022.04 | Tested By: | MG/RS/RM | Date: | 9/16/2022 |
| Project Location: | Poway Treated Water Facility | Reviewed By: | MF | Date: | 9/19/2022 |
| Lab No.: | 34784 | Boring ID: | RC-2 | Depth: | 13.5-15.0 |
| **Description:** | Granitic rock | Dial Reading (lbs) : | 105,170 | Wt(g): | 1231.2 |

## ASTM D 4543-19: DETERMINATION OF DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS

### LENGTH/DIAMETER RATIO (ASTM D 4543 Section 7.2)

| | 1 | 2 | Average | |
|---|---|---|---|---|
| Specimen Length (in): | 5.79 | 5.79 | 5.79 | |
| Specimen Diameter (in): | 2.40 | 2.40 | 2.40 | |
| Length to Diameter Ratio: | 2.4 | **L/D Tolerance Met?** | | **YES** |

### DEVIATION FROM STRAIGHTNESS (ASTM D 4543 Section 9.1.1) PROCEDURE S1

| | Max | Min | Difference |
|---|---|---|---|
| Measurement at 0˚ (in): | 0.081 | 0.076 | 0.005 |
| Measurement at 120˚ (in): | 0.086 | 0.075 | 0.011 |
| Measurement at 240˚ (in): | 0.083 | 0.073 | 0.010 |

Maximum difference must be < 0.020 in.

**Straightness Tolerance Met?** **YES**

### END FLATNESS AND PARALLELISM (ASTM D 4543 Section 9.2.1): PROCEDURE FP1

| END 1 | -1.000 | -0.875 | -0.750 | -0.625 | -0.500 | -0.375 | -0.250 | -0.125 | 0.000 | 0.125 | 0.250 | 0.375 | 0.500 | 0.625 | 0.750 | 0.875 | 1.000 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Diameter 1 at 0˚ (in): | -0.0009 | -0.0007 | -0.0003 | -0.0002 | -0.0001 | 0.0001 | 0.0003 | 0.0002 | 0.0002 | 0.0000 | -0.0003 | -0.0005 | -0.0009 | -0.0012 | -0.0015 | -0.0019 | -0.0022 |
| Best-Fit Line: | 0.0001 | 0.0000 | -0.0001 | -0.0002 | -0.0003 | -0.0003 | -0.0004 | -0.0005 | -0.0006 | -0.0007 | -0.0008 | -0.0009 | -0.0010 | -0.0010 | -0.0011 | -0.0012 | -0.0013 |
| Difference: | 0.0010 | 0.0007 | 0.0002 | 0.0000 | 0.0002 | 0.0004 | 0.0007 | 0.0007 | 0.0008 | 0.0007 | 0.0005 | 0.0004 | 0.0000 | 0.0002 | 0.0004 | 0.0007 | 0.0009 |
| Diameter 2 at 90˚ (in): | -0.0009 | -0.0007 | -0.0005 | -0.0003 | -0.0001 | 0.0004 | 0.0005 | 0.0006 | 0.0006 | 0.0005 | 0.0002 | -0.0003 | -0.0006 | -0.0009 | -0.0011 | -0.0015 | -0.0019 |
| Best-Fit Line: | 0.0001 | 0.0001 | 0.0000 | 0.0000 | -0.0001 | -0.0002 | -0.0002 | -0.0003 | -0.0004 | -0.0004 | -0.0005 | -0.0005 | -0.0006 | -0.0007 | -0.0007 | -0.0008 | -0.0008 |
| Difference: | 0.0010 | 0.0008 | 0.0005 | 0.0003 | 0.0000 | 0.0006 | 0.0007 | 0.0009 | 0.0010 | 0.0009 | 0.0007 | 0.0002 | 0.0000 | 0.0002 | 0.0004 | 0.0007 | 0.0011 |
| END 2 | -1.000 | -0.875 | -0.750 | -0.625 | -0.500 | -0.375 | -0.250 | -0.125 | 0.000 | 0.125 | 0.250 | 0.375 | 0.500 | 0.625 | 0.750 | 0.875 | 1.000 |
| Diameter 1 at 0˚ (in): | -0.0007 | -0.0006 | -0.0006 | -0.0003 | -0.0001 | 0.0000 | 0.0001 | 0.0001 | 0.0002 | 0.0003 | 0.0002 | 0.0002 | 0.0003 | 0.0002 | 0.0001 | -0.0002 | -0.0007 |
| Best-Fit Line: | -0.0003 | -0.0003 | -0.0003 | -0.0002 | -0.0002 | -0.0002 | -0.0002 | -0.0001 | -0.0001 | -0.0001 | 0.0000 | 0.0000 | 0.0000 | 0.0001 | 0.0001 | 0.0001 | 0.0002 |
| Difference: | 0.0004 | 0.0003 | 0.0003 | 0.0001 | 0.0001 | 0.0002 | 0.0003 | 0.0002 | 0.0003 | 0.0004 | 0.0002 | 0.0002 | 0.0003 | 0.0001 | 0.0000 | 0.0003 | 0.0009 |
| Diameter 2 at 90˚ (in): | -0.0009 | -0.0007 | -0.0006 | -0.0005 | -0.0004 | -0.0002 | -0.0002 | -0.0001 | 0.0000 | -0.0002 | -0.0002 | -0.0003 | -0.0003 | -0.0004 | -0.0005 | -0.0007 | -0.0011 |
| Best-Fit Line: | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 |
| Difference: | 0.0005 | 0.0003 | 0.0002 | 0.0001 | 0.0000 | 0.0002 | 0.0002 | 0.0003 | 0.0004 | 0.0002 | 0.0002 | 0.0001 | 0.0001 | 0.0000 | 0.0001 | 0.0003 | 0.0007 |

Maximum difference between smooth curve and best-fit line must not be > 0.001 in.

**Flatness Tolerance Met?** **YES**

DIAMETER 1

| | |
|---|---|
| End 1: Slope of Best Fit Line: | -0.0007 |
| Angle of Best Fit Line (degree): | -0.040 |
| End 2: Slope of Best Fit Line: | 0.0002 |
| Angle of Best Fit Line (degree): | 0.014 |
| Maximum Angular Difference (degree): | 0.054 |

Maxiumum angular difference must be ≤0.25˚ for spherically seated

**Parellelism Tolerance Met?** **YES**

DIAMETER 2

| | |
|---|---|
| End 1: Slope of Best Fit Line: | -0.0005 |
| Angle of Best Fit Line (degree): | -0.028 |
| End 2: Slope of Best Fit Line: | 0.0000 |
| Angle of Best Fit Line (degree): | -0.001 |
| Maximum Angular Difference (degree): | 0.028 |

Maxiumum angular difference must be ≤0.25˚ for spherically seated

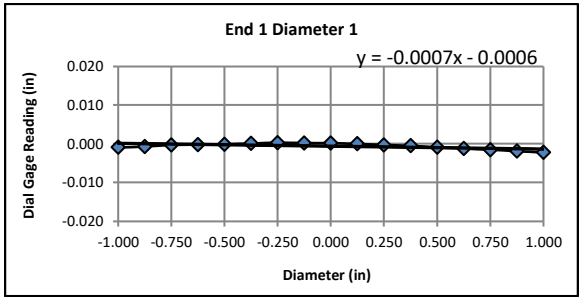
**Parellelism Tolerance Met?** **YES**

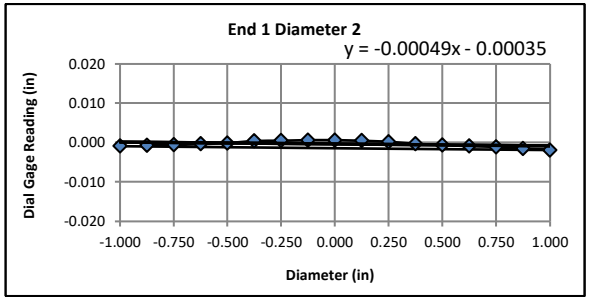
### PERPENDICULARITY (ASTM D 4543 Section 9.3.2): PROCEDURE P1

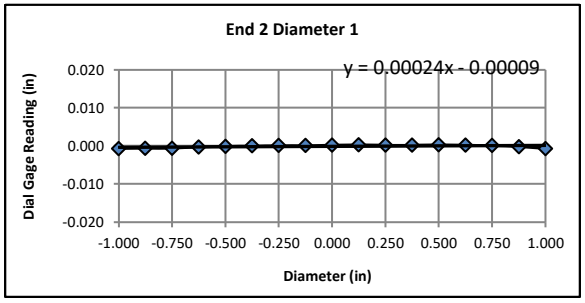
| | |
|---|---|
| Max Gap Between Square and Top of Specimen (in): | 0.003 |
| Gap / Length: | 0.0004 |

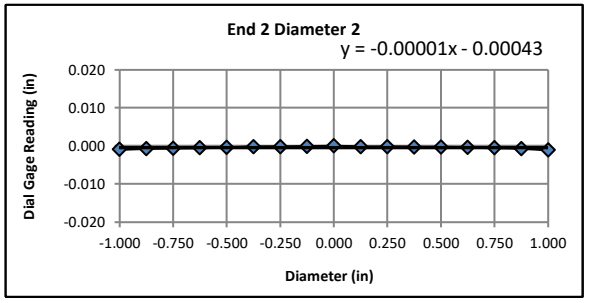
Gap / Length must be ≤ 0.0043

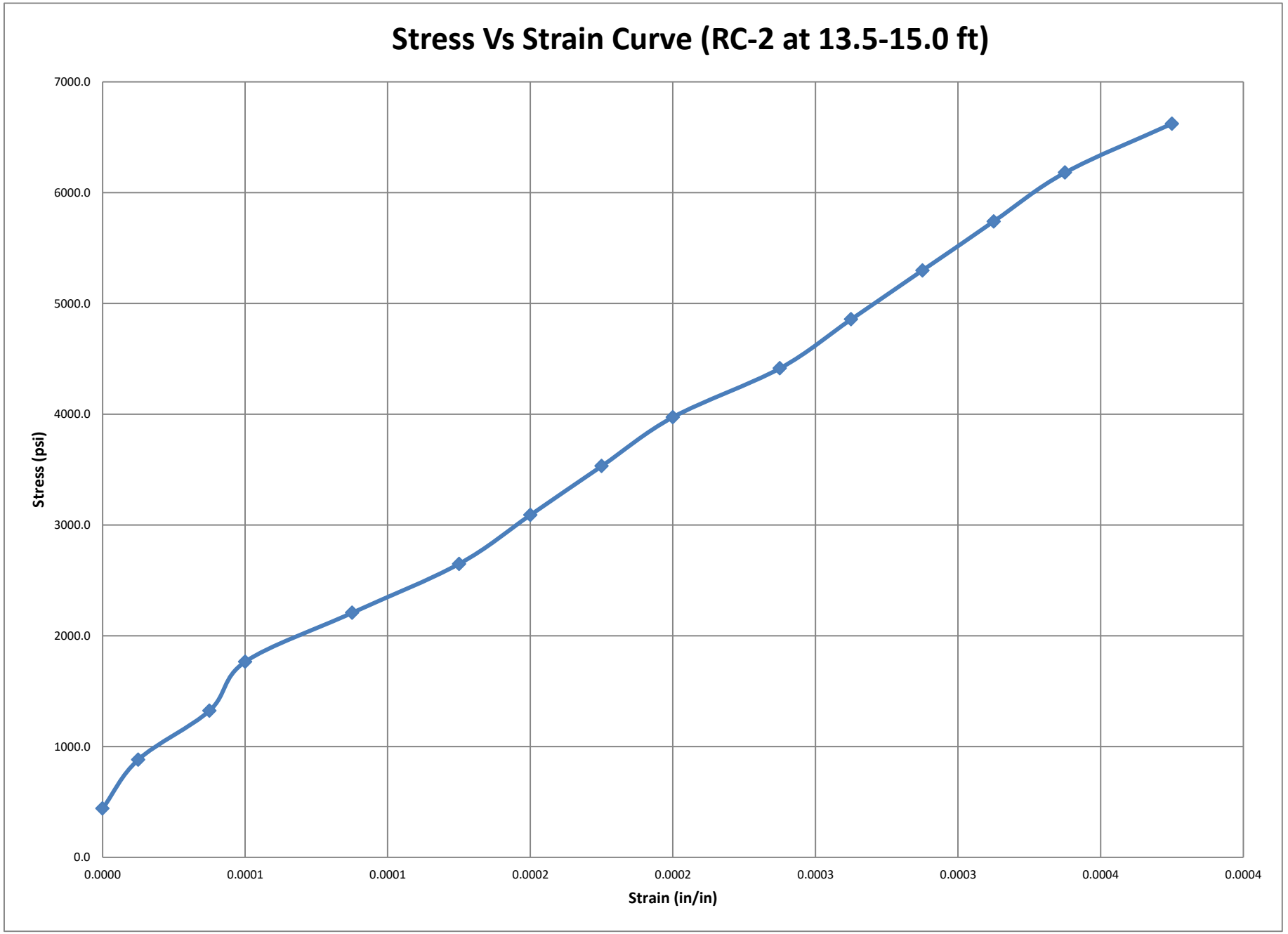
**Perpendicularity Tolerance Met?** **YES**




Stress Vs Strain Curve (RC-2 at 13.5-15.0 ft)
7000.0
6000.0
5000.0
4000.0
3000.0
2000.0
1000.0
0.0
Stress (psi)
0.0000
0.0001
0.0001
0.0002
0.0002
0.0003
0.0003
0.0004
0.0004
Strain (in/in)

| | | | | | |
|---|---|---|---|---|---|
| Client: | Ninyo & Moore | Sampled By: | Client | Date: | 8/11 & 8/15/2022 |
| Project Name: | Clearwell (109471001) | Submitted By: | Client | Date: | 9/7/2022 |
| Project No.: | 5015-19-0022.04 | Tested By: | MG/RS/RM | Date: | 9/16/2022 |
| Project Location: | Poway Treated Water Facility | Reviewed By: | MF | Date: | 9/19/2022 |
| Lab No.: | 34786 | Boring ID: | RC-2 | Depth: | 20.0-25.0 |
| **Description:** | Granitic rock | Dial Reading (lbs) : | 103,765 | Wt(g): | 1230.2 |

## ASTM D 4543-19: DETERMINATION OF DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS

### LENGTH/DIAMETER RATIO (ASTM D 4543 Section 7.2)

| | 1 | 2 | Average | |
|---|---|---|---|---|
| Specimen Length (in): | 5.82 | 5.82 | 5.82 | |
| Specimen Diameter (in): | 2.40 | 2.40 | 2.40 | |
| Length to Diameter Ratio: | 2.4 | **L/D Tolerance Met?** | | **YES** |

### DEVIATION FROM STRAIGHTNESS (ASTM D 4543 Section 9.1.1) PROCEDURE S1

| | Max | Min | Difference |
|---|---|---|---|
| Measurement at 0˚ (in): | 0.074 | 0.073 | 0.002 |
| Measurement at 120˚ (in): | 0.075 | 0.072 | 0.003 |
| Measurement at 240˚ (in): | 0.073 | 0.071 | 0.002 |

Maximum difference must be < 0.020 in.

**Straightness Tolerance Met?** **YES**

### END FLATNESS AND PARALLELISM (ASTM D 4543 Section 9.2.1): PROCEDURE FP1

| END 1 | -1.000 | -0.875 | -0.750 | -0.625 | -0.500 | -0.375 | -0.250 | -0.125 | 0.000 | 0.125 | 0.250 | 0.375 | 0.500 | 0.625 | 0.750 | 0.875 | 1.000 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Diameter 1 at 0˚ (in): | -0.0017 | -0.0016 | -0.0014 | -0.0011 | -0.0008 | -0.0006 | -0.0003 | -0.0001 | -0.0001 | -0.0001 | -0.0005 | -0.0007 | -0.0010 | -0.0015 | -0.0018 | -0.0021 | -0.0023 |
| Best-Fit Line: | -0.0007 | -0.0007 | -0.0008 | -0.0008 | -0.0009 | -0.0009 | -0.0009 | -0.0010 | -0.0010 | -0.0010 | -0.0011 | -0.0011 | -0.0012 | -0.0012 | -0.0012 | -0.0013 | -0.0013 |
| Difference: | 0.0010 | 0.0009 | 0.0006 | 0.0003 | 0.0001 | 0.0003 | 0.0006 | 0.0009 | 0.0009 | 0.0009 | 0.0006 | 0.0004 | 0.0002 | 0.0003 | 0.0006 | 0.0008 | 0.0010 |
| Diameter 2 at 90˚ (in): | -0.0016 | -0.0011 | -0.0009 | -0.0007 | -0.0005 | -0.0003 | 0.0000 | 0.0000 | -0.0002 | 0.0004 | 0.0008 | 0.0012 | 0.0016 | 0.0014 | 0.0010 | 0.0006 | 0.0003 |
| Best-Fit Line: | -0.0011 | -0.0010 | -0.0008 | -0.0007 | -0.0005 | -0.0004 | -0.0002 | -0.0001 | 0.0001 | 0.0003 | 0.0004 | 0.0006 | 0.0007 | 0.0009 | 0.0010 | 0.0012 | 0.0013 |
| Difference: | 0.0005 | 0.0002 | 0.0001 | 0.0001 | 0.0000 | 0.0001 | 0.0002 | 0.0001 | 0.0003 | 0.0002 | 0.0004 | 0.0007 | 0.0009 | 0.0006 | 0.0000 | 0.0006 | 0.0010 |
| END 2 | -1.000 | -0.875 | -0.750 | -0.625 | -0.500 | -0.375 | -0.250 | -0.125 | 0.000 | 0.125 | 0.250 | 0.375 | 0.500 | 0.625 | 0.750 | 0.875 | 1.000 |
| Diameter 1 at 0˚ (in): | -0.0018 | -0.0016 | -0.0014 | -0.0011 | -0.0007 | -0.0001 | -0.0002 | 0.0000 | 0.0000 | -0.0005 | -0.0008 | -0.0005 | -0.0007 | -0.0012 | -0.0015 | -0.0018 | -0.0022 |
| Best-Fit Line: | -0.0008 | -0.0008 | -0.0009 | -0.0009 | -0.0009 | -0.0009 | -0.0010 | -0.0010 | -0.0010 | -0.0010 | -0.0011 | -0.0011 | -0.0011 | -0.0011 | -0.0012 | -0.0012 | -0.0012 |
| Difference: | 0.0010 | 0.0008 | 0.0006 | 0.0002 | 0.0002 | 0.0008 | 0.0008 | 0.0010 | 0.0010 | 0.0005 | 0.0003 | 0.0006 | 0.0004 | 0.0001 | 0.0004 | 0.0006 | 0.0010 |
| Diameter 2 at 90˚ (in): | -0.0018 | -0.0014 | -0.0011 | -0.0009 | -0.0007 | -0.0006 | -0.0005 | -0.0004 | -0.0002 | 0.0000 | 0.0001 | 0.0000 | -0.0005 | -0.0006 | -0.0008 | -0.0009 | -0.0010 |
| Best-Fit Line: | -0.0010 | -0.0010 | -0.0009 | -0.0009 | -0.0009 | -0.0008 | -0.0008 | -0.0007 | -0.0007 | -0.0007 | -0.0006 | -0.0006 | -0.0006 | -0.0005 | -0.0005 | -0.0004 | -0.0004 |
| Difference: | 0.0008 | 0.0004 | 0.0002 | 0.0000 | 0.0002 | 0.0002 | 0.0003 | 0.0003 | 0.0005 | 0.0007 | 0.0007 | 0.0006 | 0.0001 | 0.0001 | 0.0003 | 0.0005 | 0.0006 |

Maximum difference between smooth curve and best-fit line must not be > 0.001 in.

**Flatness Tolerance Met?** **YES**

DIAMETER 1

| | |
|---|---|
| End 1: Slope of Best Fit Line: | -0.0003 |
| Angle of Best Fit Line (degree): | -0.017 |
| End 2: Slope of Best Fit Line: | -0.0002 |
| Angle of Best Fit Line (degree): | -0.011 |
| Maximum Angular Difference (degree): | 0.006 |

Maxiumum angular difference must be ≤0.25˚ for spherically seated

**Parellelism Tolerance Met?** **YES**

DIAMETER 2

| | |
|---|---|
| End 1: Slope of Best Fit Line: | 0.0012 |
| Angle of Best Fit Line (degree): | 0.069 |
| End 2: Slope of Best Fit Line: | 0.0003 |
| Angle of Best Fit Line (degree): | 0.017 |
| Maximum Angular Difference (degree): | 0.052 |

Maxiumum angular difference must be ≤0.25˚ for spherically seated

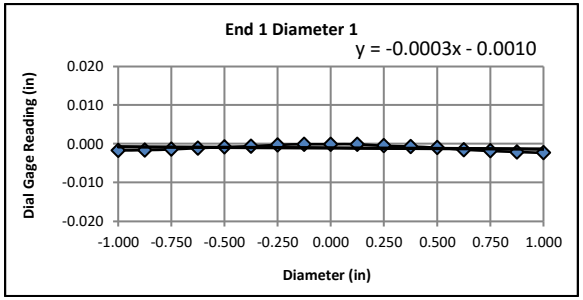
**Parellelism Tolerance Met?** **YES**

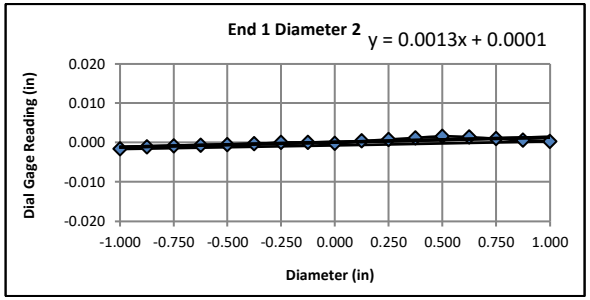
### PERPENDICULARITY (ASTM D 4543 Section 9.3.2): PROCEDURE P1

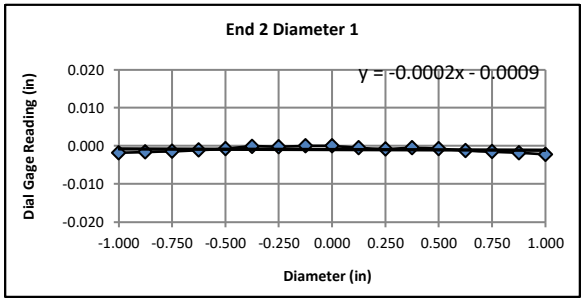
| | |
|---|---|
| Max Gap Between Square and Top of Specimen (in): | 0.003 |
| Gap / Length: | 0.0004 |

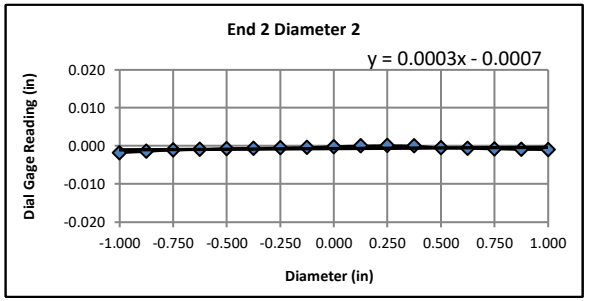
Gap / Length must be ≤ 0.0043

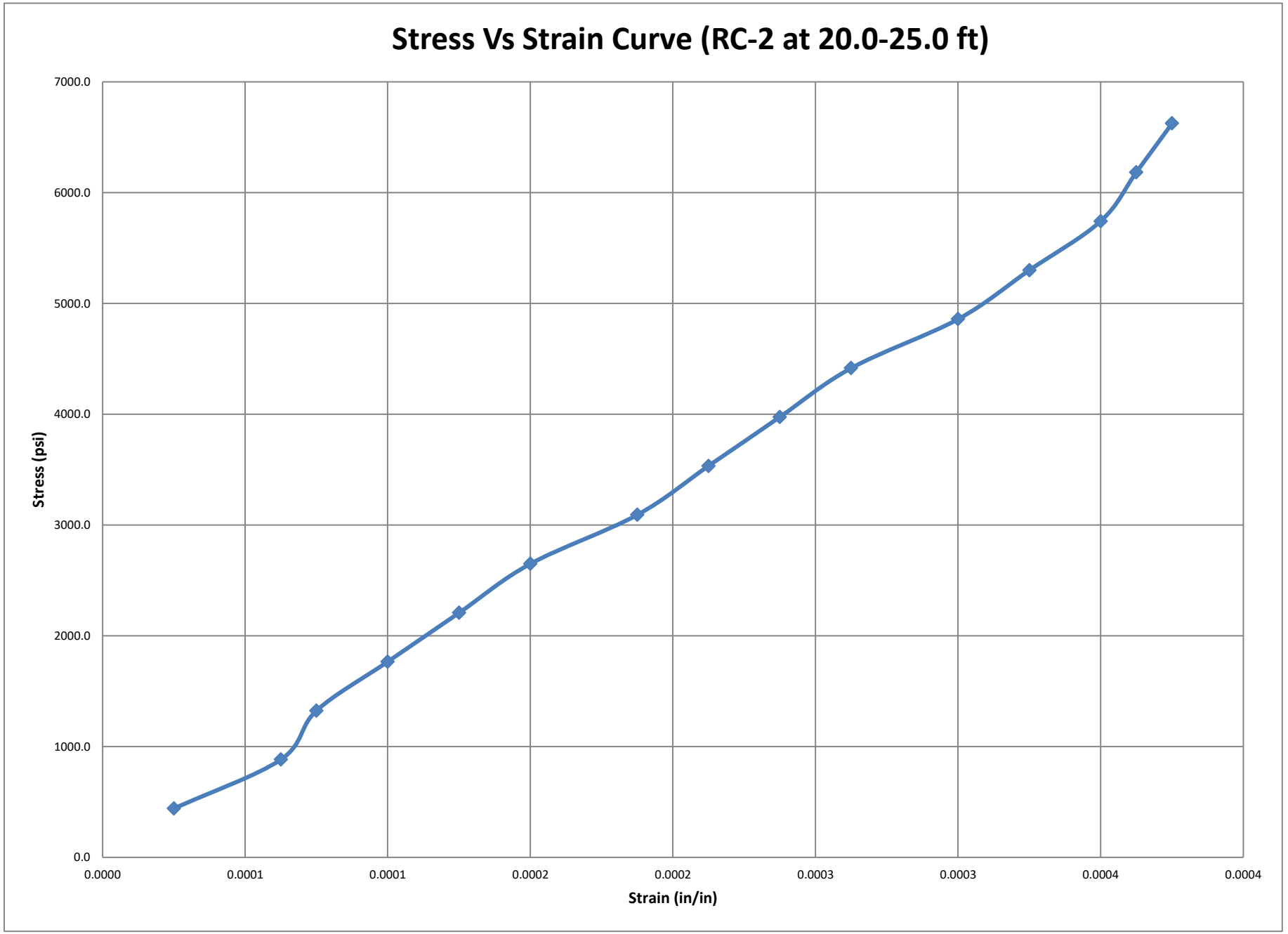
**Perpendicularity Tolerance Met?** **YES**




Stress Vs Strain Curve (RC-2 at 20.0-25.0 ft)
7000.0
6000.0
5000.0
4000.0
3000.0
2000.0
1000.0
0.0
Stress (psi)
0.0000
0.0001
0.0001
0.0002
0.0002
0.0003
0.0003
0.0004
0.0004
Strain (in/in)

| | | | | | |
|---|---|---|---|---|---|
| Client: | Ninyo & Moore | Sampled By: | Client | Date: | 8/11 & 8/15/2022 |
| Project Name: | Clearwell (109471001) | Submitted By: | Client | Date: | 9/7/2022 |
| Project No.: | 5015-19-0022.04 | Tested By: | MG/RS/RM | Date: | 9/16/2022 |
| Project Location: | Poway Treated Water Facility | Reviewed By: | MF | Date: | 9/19/2022 |
| Lab No.: | 34788 | Boring ID: | RC-2 | Depth: | 27.5-30.0 |
| **Description:** | Granitic rock | Dial Reading (lbs) : | 113,505 | Wt(g): | 1219.8 |

## ASTM D 4543-19: DETERMINATION OF DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS

### LENGTH/DIAMETER RATIO (ASTM D 4543 Section 7.2)

| | 1 | 2 | Average | |
|---|---|---|---|---|
| Specimen Length (in): | 5.76 | 5.76 | 5.76 | |
| Specimen Diameter (in): | 2.40 | 2.40 | 2.40 | |
| Length to Diameter Ratio: | 2.4 | **L/D Tolerance Met?** | | **YES** |

### DEVIATION FROM STRAIGHTNESS (ASTM D 4543 Section 9.1.1) PROCEDURE S1

| | Max | Min | Difference |
|---|---|---|---|
| Measurement at 0˚ (in): | 0.075 | 0.074 | 0.001 |
| Measurement at 120˚ (in): | 0.075 | 0.074 | 0.001 |
| Measurement at 240˚ (in): | 0.074 | 0.073 | 0.001 |

Maximum difference must be < 0.020 in.

**Straightness Tolerance Met?** **YES**

### END FLATNESS AND PARALLELISM (ASTM D 4543 Section 9.2.1): PROCEDURE FP1

| END 1 | -1.000 | -0.875 | -0.750 | -0.625 | -0.500 | -0.375 | -0.250 | -0.125 | 0.000 | 0.125 | 0.250 | 0.375 | 0.500 | 0.625 | 0.750 | 0.875 | 1.000 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Diameter 1 at 0˚ (in): | -0.0010 | -0.0008 | -0.0007 | -0.0006 | -0.0005 | -0.0004 | -0.0002 | -0.0002 | 0.0000 | -0.0001 | -0.0005 | -0.0007 | -0.0010 | -0.0012 | -0.0015 | -0.0019 | -0.0020 |
| Best-Fit Line: | -0.0003 | -0.0004 | -0.0004 | -0.0005 | -0.0006 | -0.0006 | -0.0007 | -0.0007 | -0.0008 | -0.0009 | -0.0009 | -0.0010 | -0.0011 | -0.0011 | -0.0012 | -0.0012 | -0.0013 |
| Difference: | 0.0007 | 0.0004 | 0.0003 | 0.0001 | 0.0001 | 0.0002 | 0.0005 | 0.0005 | 0.0008 | 0.0008 | 0.0004 | 0.0003 | 0.0001 | 0.0001 | 0.0003 | 0.0007 | 0.0007 |
| Diameter 2 at 90˚ (in): | -0.0012 | -0.0010 | -0.0008 | -0.0004 | -0.0005 | -0.0005 | -0.0003 | -0.0002 | -0.0002 | -0.0002 | -0.0003 | -0.0003 | -0.0005 | -0.0007 | -0.0010 | -0.0014 | -0.0017 |
| Best-Fit Line: | -0.0005 | -0.0005 | -0.0006 | -0.0006 | -0.0006 | -0.0006 | -0.0007 | -0.0007 | -0.0007 | -0.0007 | -0.0008 | -0.0008 | -0.0008 | -0.0008 | -0.0009 | -0.0009 | -0.0009 |
| Difference: | 0.0007 | 0.0005 | 0.0003 | 0.0002 | 0.0001 | 0.0001 | 0.0004 | 0.0005 | 0.0005 | 0.0005 | 0.0005 | 0.0005 | 0.0003 | 0.0001 | 0.0002 | 0.0005 | 0.0008 |
| END 2 | -1.000 | -0.875 | -0.750 | -0.625 | -0.500 | -0.375 | -0.250 | -0.125 | 0.000 | 0.125 | 0.250 | 0.375 | 0.500 | 0.625 | 0.750 | 0.875 | 1.000 |
| Diameter 1 at 0˚ (in): | -0.0011 | -0.0011 | -0.0009 | -0.0008 | -0.0005 | -0.0002 | 0.0001 | 0.0002 | 0.0003 | 0.0002 | 0.0002 | 0.0000 | -0.0002 | -0.0006 | -0.0009 | -0.0011 | -0.0015 |
| Best-Fit Line: | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 | -0.0005 |
| Difference: | 0.0006 | 0.0006 | 0.0004 | 0.0003 | 0.0000 | 0.0003 | 0.0006 | 0.0007 | 0.0008 | 0.0007 | 0.0007 | 0.0005 | 0.0003 | 0.0001 | 0.0004 | 0.0006 | 0.0010 |
| Diameter 2 at 90˚ (in): | -0.0011 | -0.0011 | -0.0008 | -0.0006 | -0.0002 | 0.0000 | 0.0001 | 0.0002 | 0.0004 | 0.0003 | 0.0002 | 0.0001 | -0.0001 | -0.0004 | -0.0008 | -0.0011 | -0.0013 |
| Best-Fit Line: | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 | -0.0004 |
| Difference: | 0.0007 | 0.0007 | 0.0004 | 0.0002 | 0.0002 | 0.0004 | 0.0005 | 0.0006 | 0.0008 | 0.0007 | 0.0006 | 0.0005 | 0.0003 | 0.0000 | 0.0004 | 0.0007 | 0.0009 |

Maximum difference between smooth curve and best-fit line must not be > 0.001 in. **Flatness Tolerance Met?** **YES**

DIAMETER 1

| | |
|---|---|
| End 1: Slope of Best Fit Line: | -0.0005 |
| Angle of Best Fit Line (degree): | -0.029 |
| End 2: Slope of Best Fit Line: | 0.0000 |
| Angle of Best Fit Line (degree): | 0.000 |
| Maximum Angular Difference (degree): | 0.029 |

Maxiumum angular difference must be ≤0.25˚ for spherically seated

**Parellelism Tolerance Met?** **YES**

DIAMETER 2

| | |
|---|---|
| End 1: Slope of Best Fit Line: | -0.0002 |
| Angle of Best Fit Line (degree): | -0.011 |
| End 2: Slope of Best Fit Line: | 0.0000 |
| Angle of Best Fit Line (degree): | 0.000 |
| Maximum Angular Difference (degree): | 0.011 |

Maxiumum angular difference must be ≤0.25˚ for spherically seated

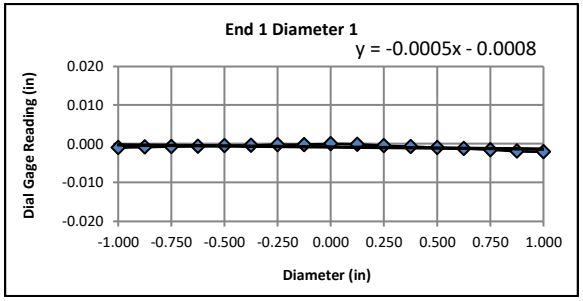
**Parellelism Tolerance Met?** **YES**

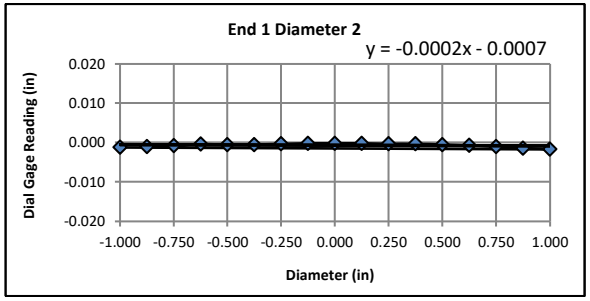
### PERPENDICULARITY (ASTM D 4543 Section 9.3.2): PROCEDURE P1

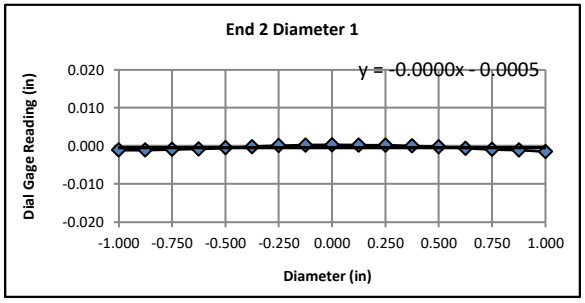
| | |
|---|---|
| Max Gap Between Square and Top of Specimen (in): | 0.005 |
| Gap / Length: | 0.0009 |

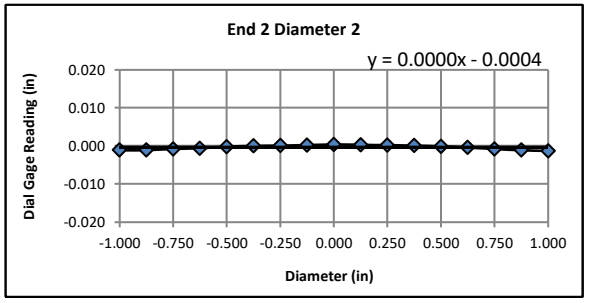
Gap / Length must be ≤ 0.0043

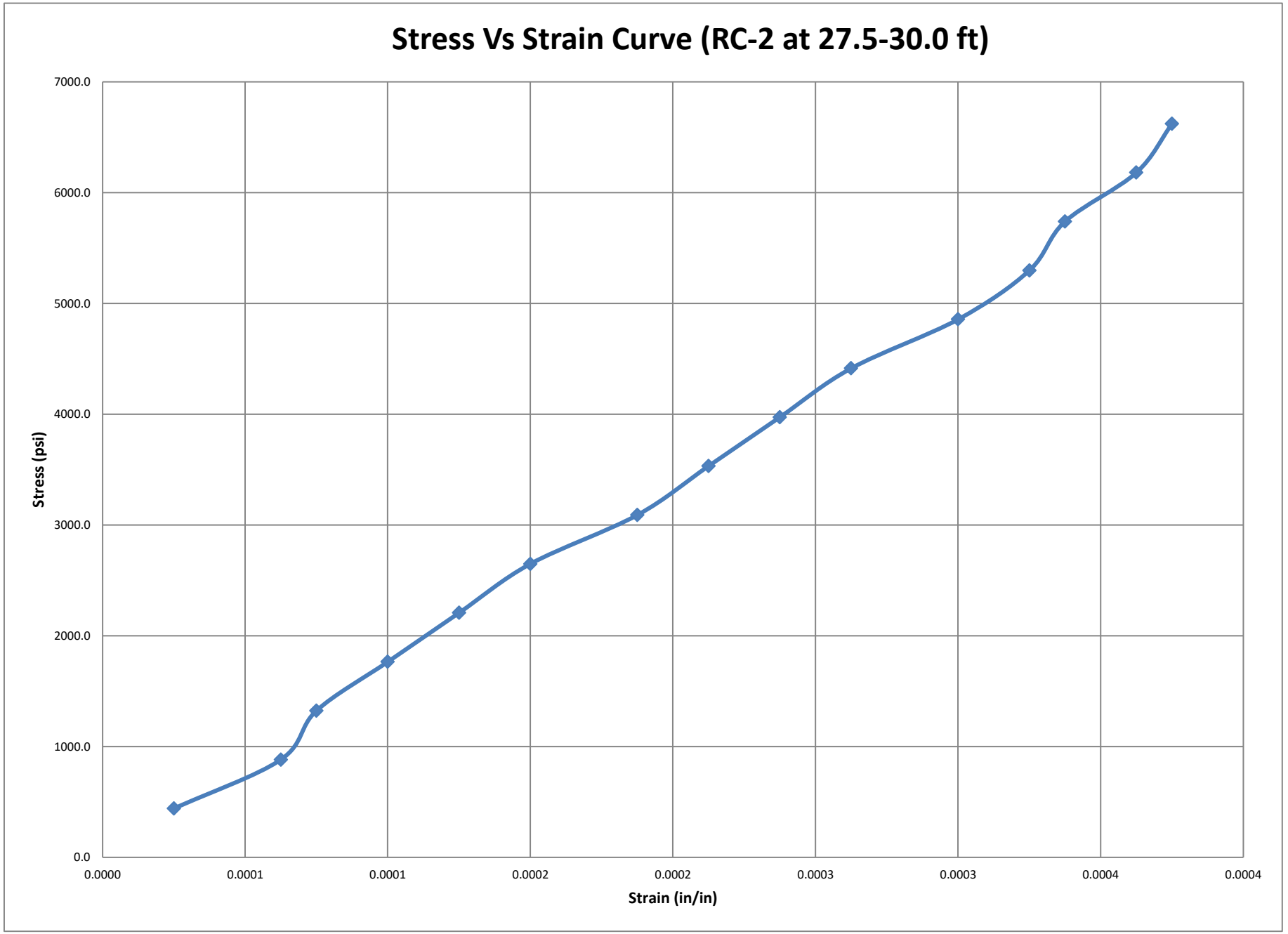
**Perpendicularity Tolerance Met?** **YES**



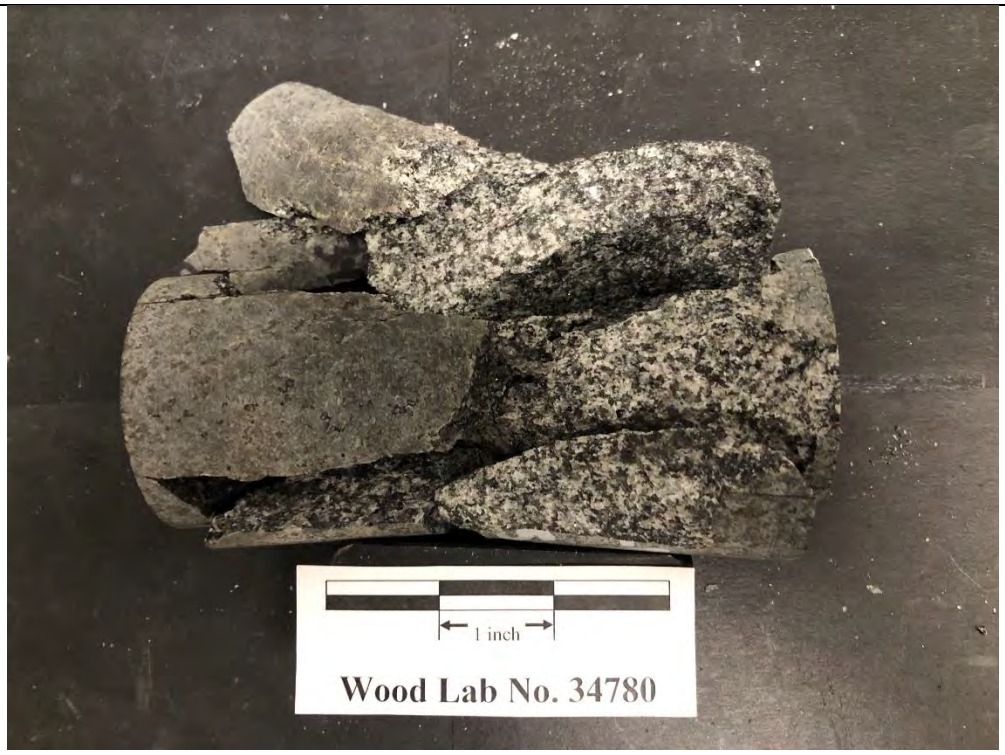




Stress Vs Strain Curve (RC-2 at 27.5-30.0 ft)
7000.0
6000.0
5000.0
4000.0
3000.0
2000.0
1000.0
0.0
Stress (psi)
0.0000
0.0001
0.0001
0.0002
0.0002
0.0003
0.0003
0.0004
0.0004
Strain (in/in)

| | |
|---|---|
| Client: | Ninyo & Moore |
| Project Name: | Clearwell (109471001) |
| Project No.: | 5015-19-0022.04 |
| Project Location: | Poway Treated Water Facility |
| Lab No.: | 34780 |
| Boring ID: | RC-1 |
| Depth (ft.): | 18.5’-20.0’ |

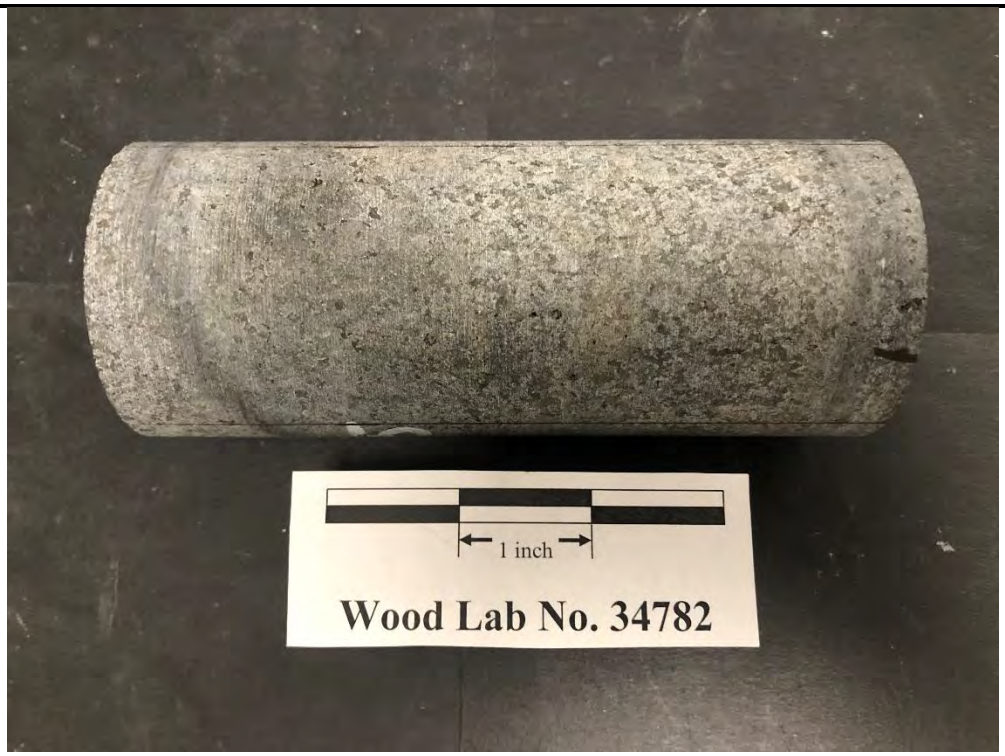



Before testing




After break

| | |
|---|---|
| Client: | Ninyo & Moore |
| Project Name: | Clearwell (109471001) |
| Project No.: | 5015-19-0022.04 |
| Project Location: | Poway Treated Water Facility |
| Lab No.: | 34782 |
| Boring ID: | RC-1 |
| Depth (ft.): | 25.9'-27.5' |




Before testing




After break

| | |
|---|---|
| Client: | Ninyo & Moore |
| Project Name: | Clearwell (109471001) |
| Project No.: | 5015-19-0022.04 |
| Project Location: | Poway Treated Water Facility |
| Lab No.: | 34784 |
| Boring ID: | RC-2 |
| Depth (ft.): | 13.5'-15.0' |

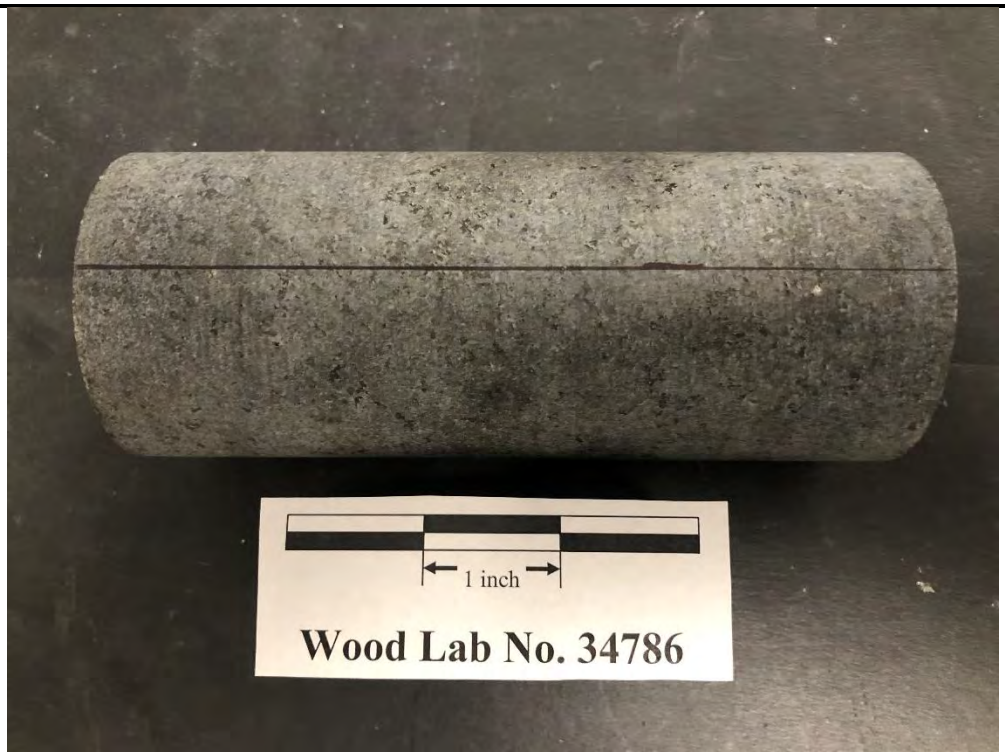



Before testing




After break

| | |
|---|---|
| Client: | Ninyo & Moore |
| Project Name: | Clearwell (109471001) |
| Project No.: | 5015-19-0022.04 |
| Project Location: | Poway Treated Water Facility |
| Lab No.: | 34786 |
| Boring ID: | RC-2 |
| Depth (ft.): | 20.0'-25.0' |




Before testing




After break

| | |
|---|---|
| Client: | Ninyo & Moore |
| Project Name: | Clearwell (109471001) |
| Project No.: | 5015-19-0022.04 |
| Project Location: | Poway Treated Water Facility |
| Lab No.: | 34788 |
| Boring ID: | RC-2 |
| Depth (ft.): | 27.5'-30.0' |




Before testing




After break

| | | | | | |
|---|---|---|---|---|---|
| Client: | Ninyo & Moore | Sampled By: | Client | Date: | 8/11 & 8/15/2022 |
| Project Name: | Clearwell (109471001) | Submitted By: | Client | Date: | 9/7/2022 |
| Project No.: | 5015-19-0022.04 | Tested By: | MG/RM | Date: | 9/14/2022 |
| Project Location: | Poway Treated Water Facility | Reviewed By: | MF | Date: | 9/19/2022 |
| Lab No.: | 34781 | Boring ID: | RC-1 | Depth: | 18.5-20.0 |
| **Description:** | Splitting Tensile Samples | Dial Reading (lbs) : | N/A | Wt(g): | N/A |

**ASTM D 3967-16: SPLITTING TENSILE STRENGTH OF INTACT ROCK CORE SPECIMENS**

| Sample # | Diameter (in) | Thickness (in) | t/D Ratio | Average Load Rate (lbf/sec) | Load at Failure (lbf) | Splitting Tensile Strength (psi) |
|---|---|---|---|---|---|---|
| 34781-1 | 2.40 | 1.24 | 0.51 | 127 | 5,802 | 1240 |
| 34781-2 | 2.40 | 1.25 | 0.52 | 142 | 6,401 | 1360 |
| 34781-3 | 2.40 | 1.26 | 0.52 | 191 | 7,260 | 1530 |
| 34781-4 | 2.40 | 1.23 | 0.51 | 138 | 5,978 | 1290 |
| 34781-5 | 2.40 | 1.21 | 0.50 | 104 | 5,319 | 1170 |
| 34781-6 | 2.40 | 1.23 | 0.51 | 187 | 6,667 | 1430 |
| 34781-7 | 2.40 | 1.24 | 0.52 | 186 | 6,354 | 1360 |
| 34781-8 | 2.40 | 1.24 | 0.51 | 187 | 6,169 | 1320 |
| 34781-9 | 2.40 | 1.24 | 0.52 | 187 | 5,921 | 1270 |
| 34781-10 | 2.40 | 1.22 | 0.51 | 177 | 6,854 | 1490 |

Platen Type Used: Flat

Average Loading Rate: 163 lbf/sec (104-191 lb/sec)

Moisture: As-Received (Air-Dried)

Average Splitting Tensile Strength: 1350

Submitted by:

Michael Farr, CEG 1938

Associate Engineering Geologist

| | | | | | |
|---|---|---|---|---|---|
| Client: | Ninyo & Moore | Sampled By: | Client | Date: | 8/11 & 8/15/2022 |
| Project Name: | Clearwell (109471001) | Submitted By: | Client | Date: | 9/7/2022 |
| Project No.: | 5015-19-0022.04 | Tested By: | MG/RM | Date: | 9/14/2022 |
| Project Location: | Poway Treated Water Facility | Reviewed By: | MF | Date: | 9/19/2022 |
| Lab No.: | 34785 | Boring ID: | RC-2 | Depth: | 15.0-20.0 |
| **Description:** | Splitting Tensile Samples | Dial Reading (lbs) : | N/A | Wt(g): | N/A |

**ASTM D 3967-16: SPLITTING TENSILE STRENGTH OF INTACT ROCK CORE SPECIMENS**

| Sample # | Diameter (in) | Thickness (in) | t/D Ratio | Average Load Rate (lbf/sec) | Load at Failure (lbf) | Splitting Tensile Strength (psi) |
|---|---|---|---|---|---|---|
| 34785-1 | 2.40 | 0.73 | 0.30 | 83 | 3,646 | 1330 |
| 34785-2 | 2.40 | 0.72 | 0.30 | 61 | 3,435 | 1270 |
| 34785-3 | 2.40 | 0.73 | 0.31 | 62 | 3,873 | 1400 |
| 34785-4 | 2.40 | 0.71 | 0.30 | 72 | 4,014 | 1500 |
| 34785-5 | 2.40 | 0.75 | 0.31 | 82 | 3,653 | 1290 |
| 34785-6 | 2.40 | 0.75 | 0.31 | 124 | 3,149 | 1120 |
| 34785-7 | 2.40 | 0.64 | 0.27 | 53 | 3,504 | 1450 |
| 34785-8 | 2.40 | 0.65 | 0.27 | 55 | 3,671 | 1500 |
| 34785-9 | 2.40 | 0.73 | 0.30 | 46 | 3,920 | 1430 |
| 34785-10 | 2.40 | 0.68 | 0.28 | 63 | 3,927 | 1520 |

Platen Type Used: Flat

Average Loading Rate: 70 lbf/sec (46-124 lb/sec)

Moisture: As-Received (Air-Dried)

Average Splitting Tensile Strength: 1380

Submitted by:

Michael Farr, CEG 1938

Associate Engineering Geologist

| | | | | | |
|---|---|---|---|---|---|
| Client: | Ninyo & Moore | Sampled By: | Client | Date: | 8/11 & 8/15/2022 |
| Project Name: | Clearwell (109471001) | Submitted By: | Client | Date: | 9/7/2022 |
| Project No.: | 5015-19-0022.04 | Tested By: | MG/RM | Date: | 9/14/2022 |
| Project Location: | Poway Treated Water Facility | Reviewed By: | MF | Date: | 9/19/2022 |
| Lab No.: | 34787 | Boring ID: | RC-2 | Depth: | 25.0-27.5 |
| **Description:** | Splitting Tensile Samples | Dial Reading (lbs) : | N/A | Wt(g): | N/A |

**ASTM D 3967-16: SPLITTING TENSILE STRENGTH OF INTACT ROCK CORE SPECIMENS**

| Sample # | Diameter (in) | Thickness (in) | t/D Ratio | Average Load Rate (lbf/sec) | Load at Failure (lbf) | Splitting Tensile Strength (psi) |
|---|---|---|---|---|---|---|
| 34787-1 | 2.40 | 1.05 | 0.44 | 126 | 5,120 | 1300 |
| 34787-2 | 2.40 | 1.04 | 0.43 | 136 | 5,125 | 1310 |
| 34787-3 | 2.40 | 1.04 | 0.44 | 143 | 5,311 | 1350 |
| 34787-4 | 2.40 | 1.04 | 0.43 | 154 | 5,393 | 1380 |
| 34787-5 | 2.40 | 1.08 | 0.45 | 149 | 4,836 | 1190 |
| 34787-6 | 2.40 | 1.07 | 0.45 | 164 | 5,608 | 1390 |
| 34787-7 | 2.40 | 1.07 | 0.45 | 136 | 5,425 | 1340 |
| 34787-8 | 2.40 | 1.06 | 0.44 | 111 | 4,243 | 1070 |
| 34787-9 | 2.40 | 1.07 | 0.44 | 142 | 5,981 | 1490 |
| 34787-10 | 2.40 | 1.06 | 0.44 | 133 | 5,515 | 1390 |

Platen Type Used: Flat

Average Loading Rate: 139 lbf/sec (111-164 lb/sec)

Moisture: As-Received (Air-Dried)

Average Splitting Tensile Strength: 1320

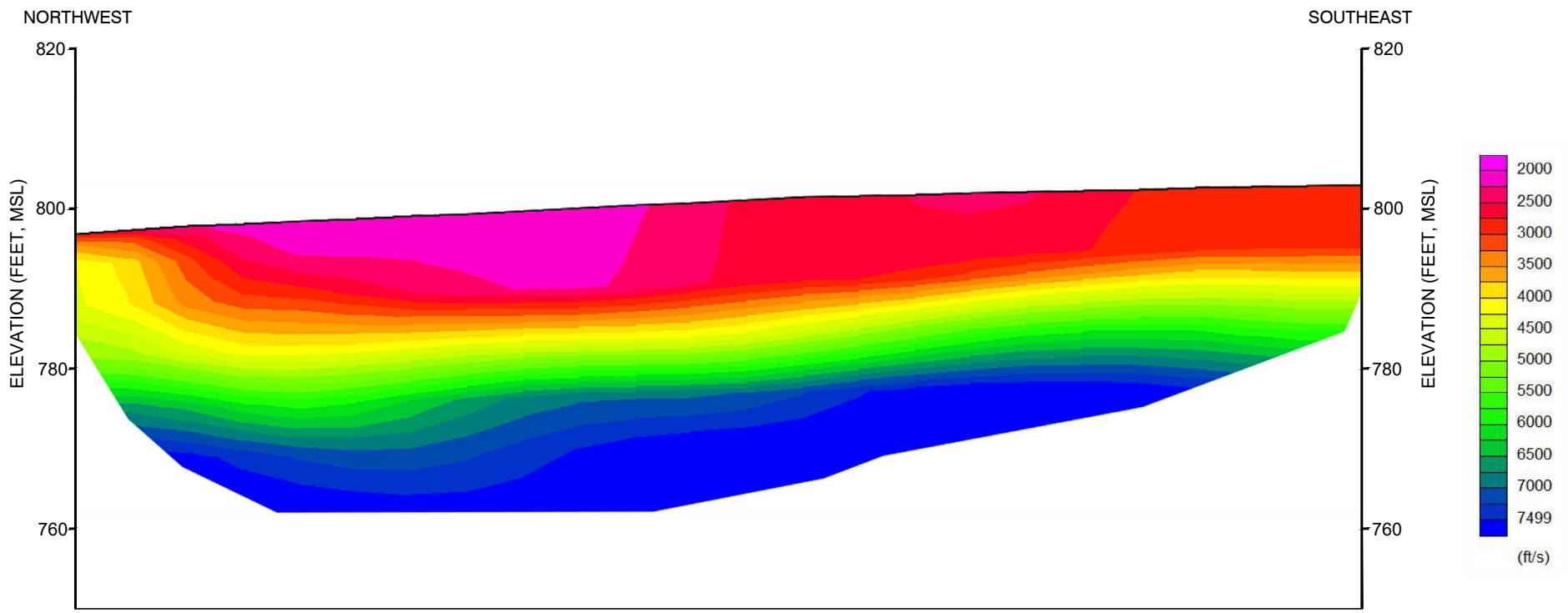
Submitted by:

Michael Farr, CEG 1938

Associate Engineering Geologist

# APPENDIX D

## Seismic Refraction Survey Results




NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

FEET
0 20 40

D1 109471001 SL-1.DWG AOB

**FIGURE D-1**

## SEISMIC LINE SL-1

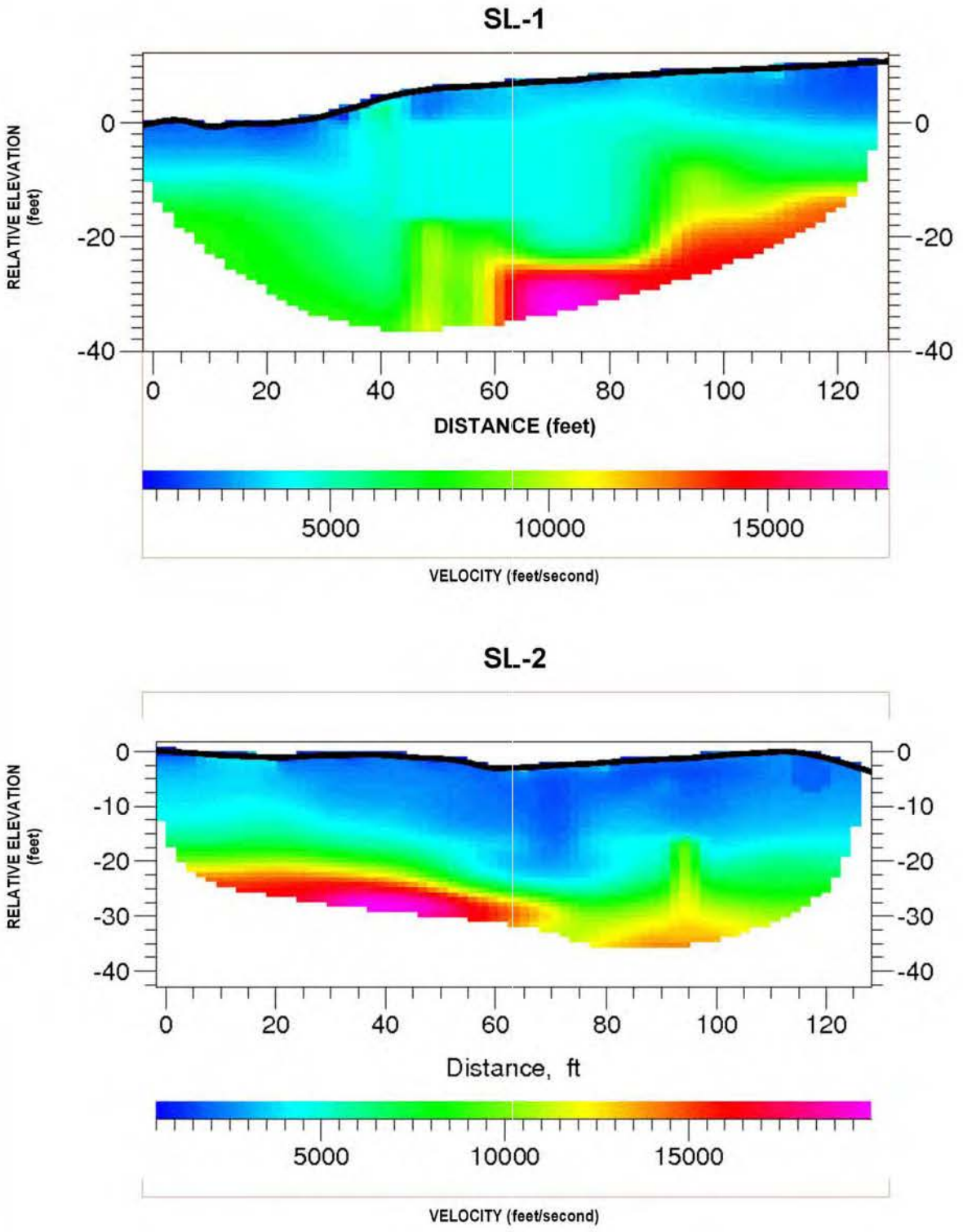
TREATED WATER FACILITIES AND REDUNDANT PIPELINE PROJECT
ESPOLA ROAD AND LAKE POWAY ROAD, POWAY, CALIFORNIA

109471001 I 12/22

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

# APPENDIX E

## Previous Seismic Refraction Survey Results (Ninyo & Moore, 2010)



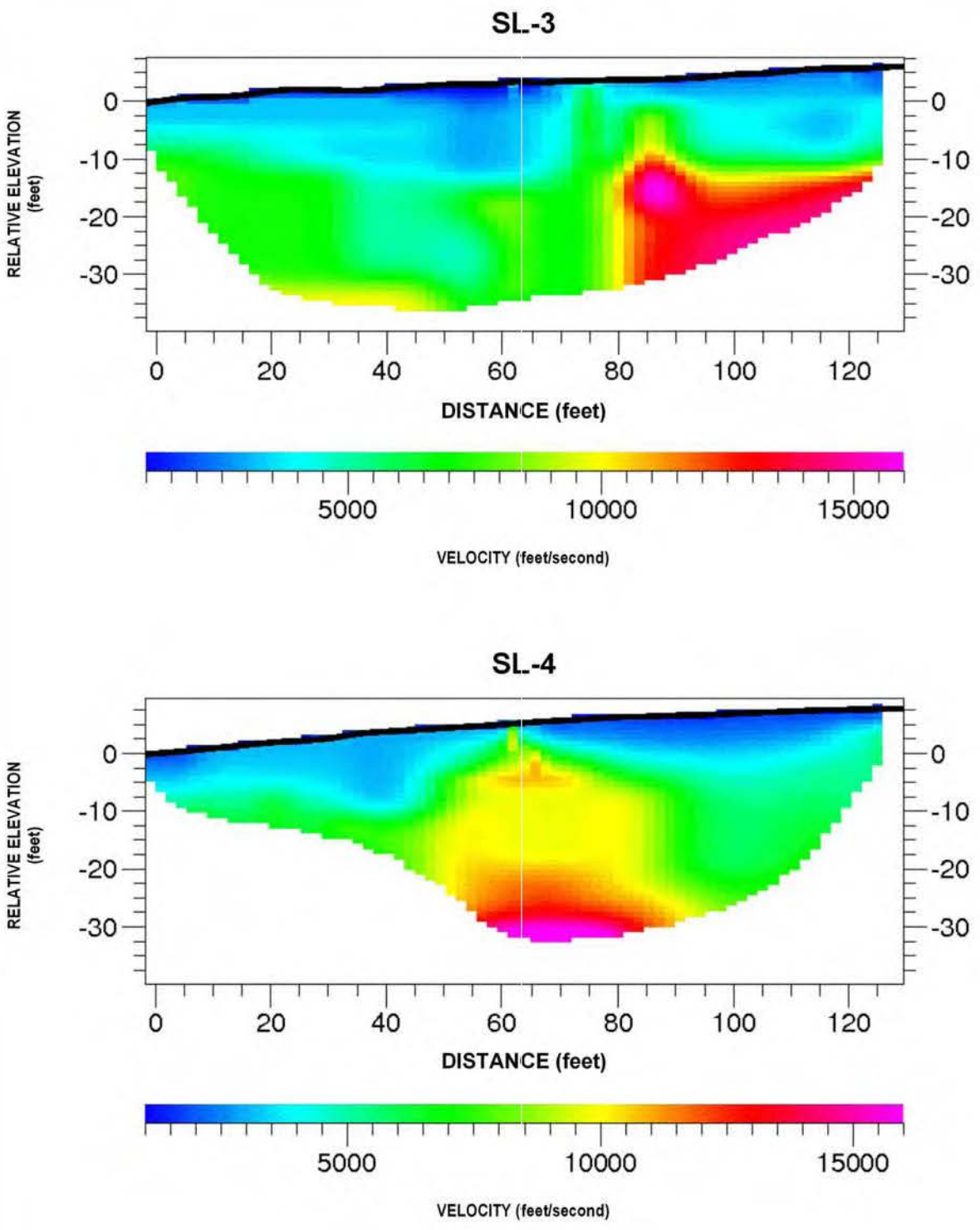

**VELOCITY PROFILES**
**SL-1 & SL-2**

Clearwell Redundant Pipeline Project
Poway, California

Project No.: 110248rev | Date: 11/10

SOUTHWEST GEOPHYSICS INC.

Figure 4a



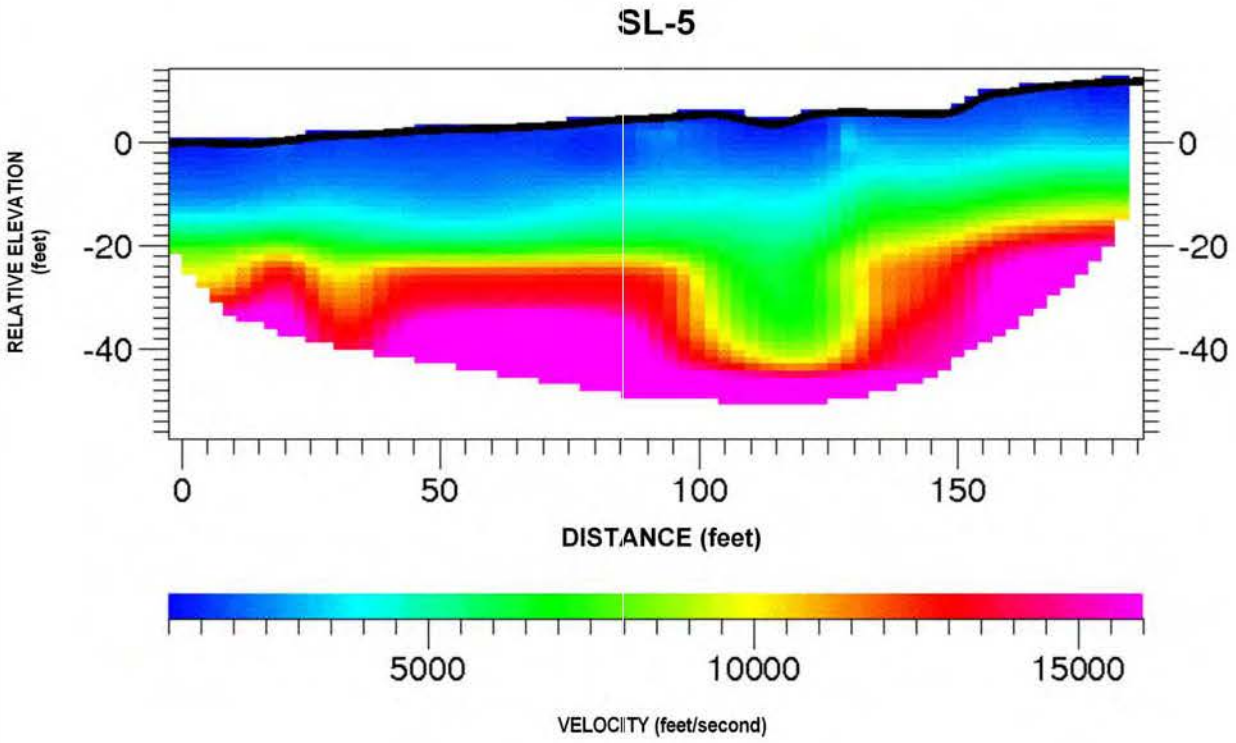

**VELOCITY PROFILES**
**SL-3 & SL-4**

Clearwell Redundant Pipeline Project
Poway, California

Project No.: 110248rev | Date: 11/10

SOUTHWEST GEOPHYSICS INC.

Figure 4b



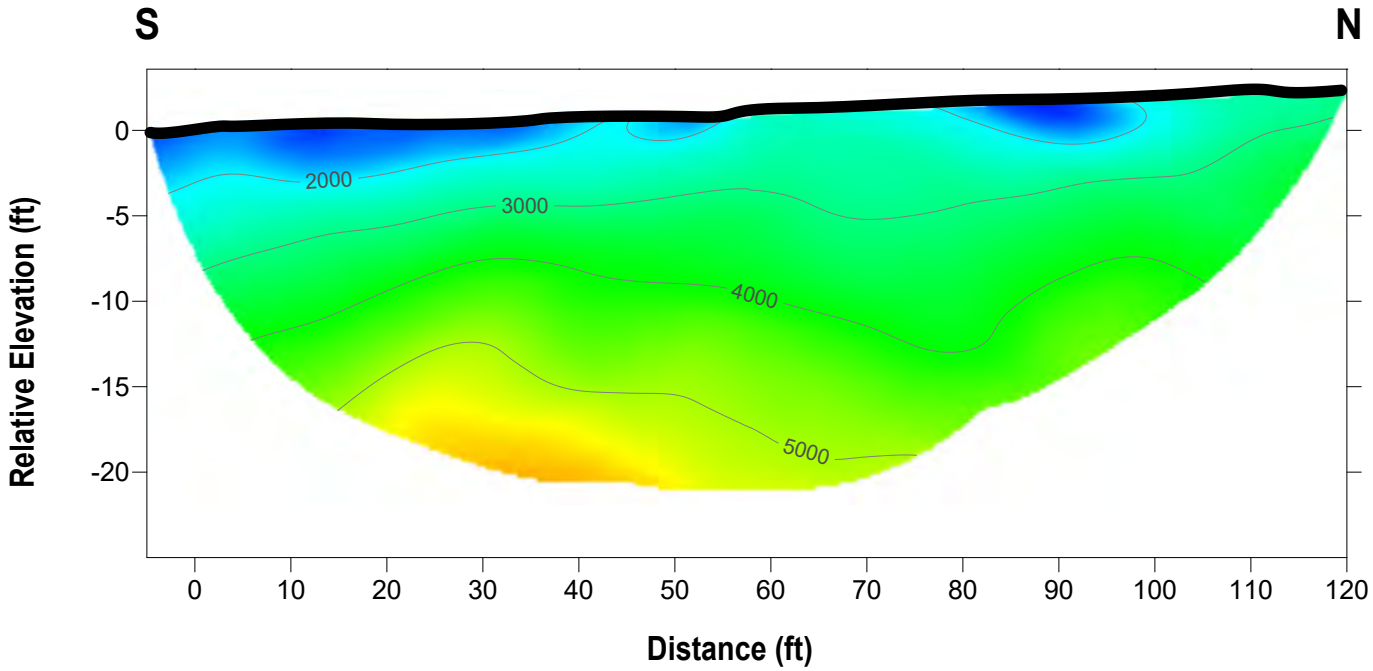

| | | |
|---|---|---|
| **VELOCITY PROFILE SL-5** | Clearwell Redundant Pipeline Project<br>Poway, California | SOUTHWEST GEOPHYSICS INC. |
| | Project No.: 110248rev \| Date: 11/10 | Figure 4c |

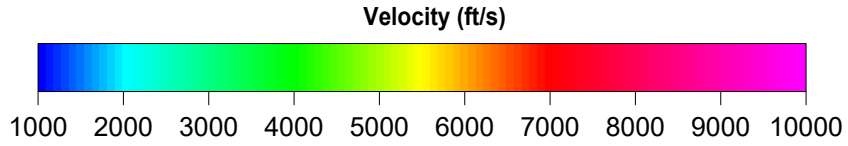
# APPENDIX F

## Flow Control Facility Seismic Refraction Survey Results (Atlas, 2022)

# TOMOGRAPHY MODEL

## SL-1

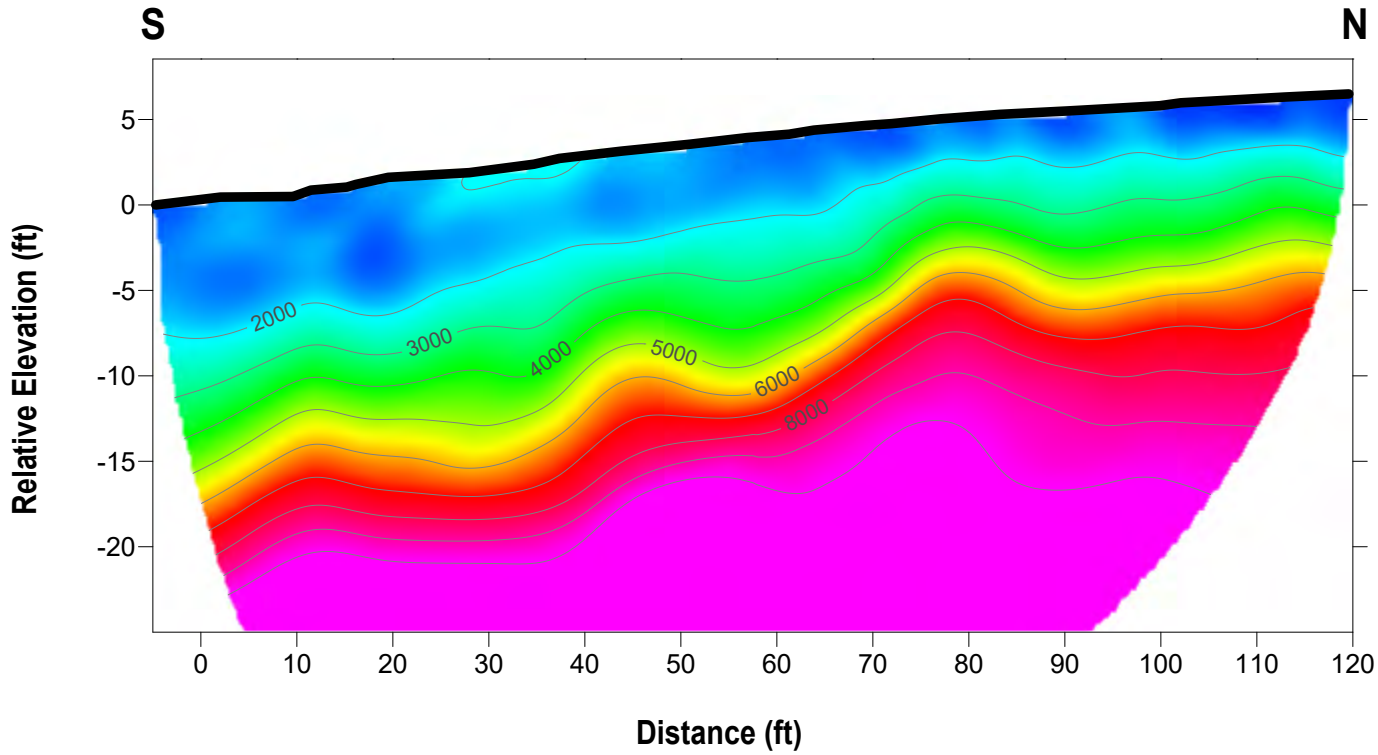


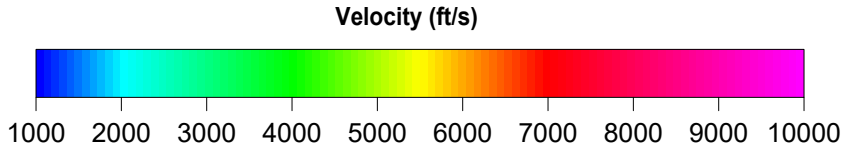




| | | | |
|---|---|---|---|
| **P-WAVE PROFILE SL-1** | Poway Flow Control Facility<br>Poway, California | ATLAS<br>Figure 4a | **Note: Contour Interval = 1,000 feet per second** |
| | Project No.: 122152.P6SWG \| Date: 07/22 | | |

# TOMOGRAPHY MODEL

## SL-2




| | | |
|---|---|---|
| **P-WAVE PROFILE SL-2** | Poway Flow Control Facility<br>Poway, California<br>Project No.: 122152.P6SWG Date: 07/22 | ATLAS<br>Figure 4b |

**Note: Contour Interval = 1,000 feet per second**

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