# Responses to Comments on the Draft IS-MND

This section includes comments received during the circulation of the Draft Initial Study-Mitigated Negative Declaration (IS-MND) prepared for the Reservoir 2B Replacement Project (Project).

The Draft IS-MND was circulated for a 30-day public review period that began on May 16, 2022 and ended on June 15, 2022. The South Coast Water District received one comment letter on the Draft IS-MND. The commenters and the page number on which each commenter's letter appear are listed below.

| Letter No. and Commenter | | Page No. |
|---|---|---|
| 1 | David Mayer, Environmental Program Manager, California Department of Fish and Wildlife South Coast Region | 2 |

The comment letter and responses follow. The comment letter has been numbered sequentially and each separate issue raised by the commenter, if more than one, has been assigned a number. The responses to each comment identify first the number of the comment letter, and then the number assigned to each issue (Response 1.1, for example, indicates that the response is for the first issue raised in comment Letter 1).

State of California – Natural Resources Agency
DEPARTMENT OF FISH AND WILDLIFE
South Coast Region
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**Letter 1**

*GAVIN NEWSOM, Governor*
*CHARLTON H. BONHAM, Director*

June 15, 2022

Taryn Kjolsing
Engineering Manager
South Coast Water District
31592 West Street
Laguna Beach, CA 92651
TKjolsing@scwd.org

**Subject: Reservoir 2B Replacement Project (PROJECT), Mitigated Negative Declaration (MND), SCH #2022050294**

Dear Ms. Kjolsing:

The California Department of Fish and Wildlife (CDFW) received a Notice of Intent to Adopt an MND from the South Coast Water District (SCWD) for the Project pursuant the California Environmental Quality Act (CEQA) and CEQA Guidelines.[1]

Thank you for the opportunity to provide comments and recommendations regarding those activities involved in the Project that may affect California fish and wildlife. Likewise, we appreciate the opportunity to provide comments regarding those aspects of the Project that CDFW, by law, may be required to carry out or approve through the exercise of its own regulatory authority under the Fish and Game Code.

**CDFW ROLE** 1.1

CDFW is California's **Trustee Agency** for fish and wildlife resources and holds those resources in trust by statute for all the people of the State. (Fish & G. Code, §§ 711.7, subd. (a) & 1802; Pub. Resources Code, § 21070; CEQA Guidelines § 15386, subd. (a).) CDFW, in its trustee capacity, has jurisdiction over the conservation, protection, and management of fish, wildlife, native plants, and habitat necessary for biologically sustainable populations of those species. (*Id.*, § 1802.) Similarly for purposes of CEQA, CDFW is charged by law to provide, as available, biological expertise during public agency environmental review efforts, focusing specifically on projects and related activities that have the potential to adversely affect fish and wildlife resources.

CDFW is also submitting comments as a **Responsible Agency** under CEQA. (Pub. Resources Code, § 21069; CEQA Guidelines, § 15381.) CDFW expects that it may need to exercise regulatory authority as provided by the Fish and Game Code. As proposed, for example, the Project may be subject to CDFW's lake and streambed alteration regulatory authority. (Fish & G. Code, § 1600 et seq.) Likewise, to the extent implementation of the Project as proposed may result in "take" as defined by State law of any species protected under the California Endangered Species Act (CESA) (Fish & G. Code, § 2050 et seq.), related authorization as provided by the Fish and Game Code will be required.

[1] CEQA is codified in the California Public Resources Code in section 21000 et seq. The "CEQA Guidelines" are found in Title 14 of the California Code of Regulations, commencing with section 15000.

**PROJECT DESCRIPTION SUMMARY**

**Proponent:** South Coast Water District (SCWD)

**Objective:** The objective of the Project is to provide additional capacity for and contribute to providing an additional 0.1 million gallons (MG) of operational, fire, and emergency storage for this zone. Primary Project activities include demolition of the existing aboveground Reservoir 2B, site preparation (including slope stabilization and pad expansion), grading, installation of two new aboveground 0.1 MG aboveground steel reservoirs, installation of electrical and piping infrastructure, testing and disinfection activities, and site restoration. Project construction would also include installing a new drain line from both reservoirs to tie into the existing drain line on the Project site, a retaining wall around the perimeter of the replacement reservoir footprints, a tank inlet/outlet pipe vault, an altitude valve, a manifold, new electrical service, new antenna, and a power and control interface.

**Location:** The Project site is in the city of Laguna Beach in southwestern Orange County at coordinates 33°30'31.2"N 117°44'42.8"W. Access to Reservoir 2B is provided by a steep, winding, unpaved road off Ceanothus Drive, which is also used by members of the public as a connector trail between Ceanothus Drive and Toovet Trail. 1.1 (cont.)

**Biological Setting:** The survey area for the Project is located on a south-facing slope at and around the existing water tank facility and its associated dirt access road. The surrounding area consists primarily of undeveloped land composed of coastal sage scrub (CSS) and chaparral habitats. Surrounding conditions include residential development on hillsides to the south and west, with open habitat areas along the hills that ascend to the east. The area surrounding the Project footprint is co-dominated by moderate quality California sagebrush (*Artemisia californica*) and California buckwheat (*Eriogonum fasciculatum*).

**Timeframe:** The proposed Project would be constructed over the course of approximately ten months during 2022 and 2023.

**COMMENTS AND RECOMMENDATIONS**

CDFW offers the comments and recommendations below to assist SCWD in adequately identifying and/or mitigating the Project's significant, or potentially significant, direct and indirect impacts on fish and wildlife (biological) resources. Editorial comments or other suggestions may also be included to improve the document. Based on the Project's avoidance of significant impacts on biological resources with implementation of mitigation measures, CDFW concludes that a Mitigated Negative Declaration is appropriate for the Project.

**COMMENT #1: Nesting Bird Surveys and Avoidance**

Per California Fish and Game Code sections 3503, 3503.5, and 3513 the proposed Project is required to avoid the incidental loss of fertile eggs or nestlings, or activities that lead to nest abandonment. To avoid impacts to nesting birds, CDFW recommends the following changes (**in bold** and ~~strikethrough~~) to Mitigation Measure BIO-1: 1.2

Project-related activities shall occur outside of the bird breeding season (February 1 to August 31) to the extent practicable. If construction must occur within the bird breeding season, **then no more than 3 days prior** to initiation of ground disturbance and/or vegetation removal, a nesting bird preconstruction survey shall be conducted by a qualified biologist within the disturbance footprint plus a 300-foot buffer, where feasible. **A qualified biological monitor shall be present for the**

**duration of construction.** If the proposed project is phased or construction activities stop for more than two weeks during the bird breeding season, a subsequent pre-construction nesting bird survey shall be completed **within 3 days prior to each phase of construction**.

If nests are found, their locations shall be flagged to facilitate avoidance. An appropriate avoidance buffer of **100 feet for passerines, 300 feet for listed bird species, and 500 feet for raptors**, should be established by a qualified biologist and demarcated with bright orange construction fencing or other suitable flagging. Active nests shall be monitored at a minimum of once per week until it has been determined that the nest is no longer being used by either the young or adults. No ground disturbance shall occur within this buffer until the qualified biologist confirms the breeding/nesting is completed and all the young have fledged. ~~If project activities must occur within the buffer, they shall be conducted at the discretion of the qualified biologist.~~ If no nesting birds are observed during pre-construction survey, no further action would be necessary. 1.2 (cont.)

**COMMENT #2: Potential Impacts to Native Vegetation**

Figure 2 of the MND appears to show native vegetation including CSS within the yellow lines depicting the boundary of the Project site. Thus, CSS could be impacted by Project activities, including, but not limited to, road construction and paving. CSS is a sensitive habitat type that supports a great diversity of wildlife and is covered in the County of Orange Central and Coastal Subregion Natural Community Conservation Plan/Habitat Conservation Plan (NCCP/HCP). The City of Laguna Beach is not a participating jurisdiction in the NCCP/HCP and thus has no programmatic involvement in the conservation of CSS. Therefore, if the Project will result in CSS habitat loss, CDFW recommends mitigating the loss in kind at a 3:1 ratio within close proximity to the Project site. 1.3

**COMMENT 3: Hydrological Impacts**

According to the MND, a potentially jurisdictional unnamed ephemeral drainage is located within the Study Area.

CDFW has regulatory authority over activities in streams and/or lakes that will divert or obstruct the natural flow, or change the bed, channel, or bank (which may include associated riparian resources) of any river, stream, or lake or use material from a river, stream, or lake. For any such activities, the Project applicant (or “entity”) must provide written notification CDFW pursuant to section 1600 *et seq*. of the Fish and Game Code. Based on this notification and other information, CDFW determines whether a Lake and Streambed Alteration Agreement (LSAA) with the applicant is required prior to conducting the proposed activities. CDFW’s issuance of a LSAA for a project that is subject to CEQA will require CEQA compliance actions by CDFW as a Responsible Agency. Whether a LSAA is required to satisfy requirements of FCG section 1600 *et seq*. can only be determined at the time a formal notification package is submitted to CDFW. 1.4

If Project activities will affect the hydrological features of such drainages, an LSAA notification may be appropriate. We encourage SCWD to consult further with CDFW regarding the possible submittal of an LSA Notification package.

**ENVIRONMENTAL DATA**

CEQA requires that information developed in environmental impact reports and negative declarations be incorporated into a database which may be used to make subsequent or supplemental environmental determinations. (Pub. Resources Code, § 21003, subd. (e).) Accordingly, please report any special status species and natural communities detected during 1.5

Project surveys to the California Natural Diversity Database (CNDDB). The CNNDB field survey form can be found at the following link: https://wildlife.ca.gov/Data/CNDDB/Training-Survey. The completed form can be mailed electronically to CNDDB at the following email address: CNDDB@wildlife.ca.gov. The types of information reported to CNDDB can be found at the following link: https://wildlife.ca.gov/Data/CNDDB. 1.5 (cont.)

## FILING FEES

The Project, as proposed, would have an impact on fish and/or wildlife, and assessment of filing fees is necessary. Fees are payable upon filing of the Notice of Determination by the Lead Agency and serve to help defray the cost of environmental review by CDFW. Payment of the fee is required in order for the underlying Project approval to be operative, vested, and final. (Cal. Code Regs, tit. 14, § 753.5; Fish & G. Code, § 711.4; Pub. Resources Code, § 21089.) 1.6

## CONCLUSION

CDFW appreciates the opportunity to comment on the MND to assist the SCWD in identifying and mitigating Project impacts on biological resources. 1.7

Questions regarding this letter or further coordination should be directed to Alex Troeller, Environmental Scientist, at Alexandra.Troeller@wildlife.ca.gov.

Sincerely,

DocuSigned by:
David Mayer
D700B4520375406...

David Mayer
Environmental Program Manager
South Coast Region

ec: CDFW
David Mayer, San Diego – David.Mayer@wildlife.ca.gov
Simona Altman, San Diego – Simona.Altman@wildlife.ca.gov
Cindy Hailey, San Diego – Cindy.Hailey@wildlife.ca.gov
State Clearinghouse, Office of Planning and Research – State.Clearinghouse@opr.ca.gov

## REFERENCES

California Department of Fish and Wildlife. 2021. California Natural Diversity Database (CNDDB) – Plants and Animals. Available from: https://wildlife.ca.gov/Data/CNDDB.

California Department of Fish and Wildlife. 2021. Lake and Streambed Alteration Program. Available from: https://wildlife.ca.gov/Conservation/LSA.

South Coast Water District. 2022. Reservoir 2B Replacement Project Draft Initial Study – Mitigated Negative Declaration. Available from: https://files.ceqanet.opr.ca.gov/278574-1/attachment/E7tZJO010CifzxLhAPySuAO7dp7z-A3-HLL7hRanMRdIswh2d0ffq-i4Qy8zpXBEv9B8mNI56BqtWgc60

# Letter 1

**COMMENTER:** David Mayer, Environmental Program Manager, California Department of Fish and Wildlife South Coast Region

**DATE:** June 15, 2022

## Response 1.1

The commenter states the California Department of Fish and Wildlife's (CDFW) role as a responsible and trustee agency under CEQA and provides a summary of the project description and biological setting. The commenter also provides a summary of the proposed project and states the letter includes comments and recommendations to address potential project impacts to biological resources. The commenter concurs with SCWD's determination that an MND is appropriate for the project.

The commenter's role as a trustee agency under CEQA and concurrence with SCWD's preparation of an MND for the project are noted. As stated in Section 4, *Biological Resources*, and Appendix B of the Draft IS-MND, an unnamed ephemeral drainage on site was determined to be potentially subject to the jurisdiction of CDFW. However, as discussed further under Response 1.4, following completion of a site visit with a Rincon Senior Biologist on April 29, 2022, CDFW Environmental Scientist Jennifer Blackhall issued a determination the same day, both verbally and via email, that the project would not impact this drainage, and that the project, as it was described to CDFW, would not require an Lake or Streambed Alteration Agreement (LSAA) notification.[1] As such, SCWD would not be required to obtain an LSAA or other discretionary authorization from CDFW under the California Fish and Game Code. Therefore, CDFW would not have regulatory authority over the project and is not expected to be a responsible agency for the project under CEQA. Individual responses regarding the commenter's concerns on environmental impacts are addressed below in Responses 1.2 through 1.7.

## Response 1.2

The commenter provides recommended revisions to Mitigation Measure BIO-1 for nesting birds.

Mitigation Measure BIO-1 as presented in Section 4, *Biological Resources*, of the Draft IS-MND is sufficient to maintain compliance with the Migratory Bird Treaty Act and California Fish and Game Code. Nevertheless, to clarify this measure, some of the commenter's recommended revisions have been incorporated into the text of the mitigation measure as shown below:

*BIO-1 Nesting Birds*

Project-related activities shall occur outside of the bird breeding season (February 1 to August 31) to the extent practicable. If construction must occur within the bird breeding season, then no more than ~~14~~ three days prior to initiation of ground disturbance and/or vegetation removal, a nesting bird pre-construction survey shall be conducted by a qualified biologist within the disturbance footprint plus a 300-foot buffer, where feasible. If the proposed project is phased or construction activities stop for more than two weeks during

[1] Blackhall, Jennifer. 2022. Environmental Scientist, Lake and Streambed Alteration Agreement Program, California Department of Fish and Wildlife South Coast Region 5. Personal communication via email regarding whether the project would require a Lake or Streambed Alteration Agreement notification with Jared Reed, Senior Biologist, Rincon Consultants, Inc. April 29, 2022.

the bird breeding season, a subsequent pre-construction nesting bird survey shall be completed within three days prior to each phase of construction.

Pre-construction nesting bird surveys shall be conducted during the time of day when birds are active and shall factor in sufficient time to perform this survey adequately and completely. A report of the nesting bird survey results, if applicable, shall be submitted to SCWD for review and approval prior to ground and/or vegetation disturbance activities.

If nests are found, their locations shall be flagged to facilitate avoidance. An appropriate avoidance buffer of ~~150~~ 100 feet for passerines, 300 feet for listed bird species, and 500 feet for raptors ~~and up to 300 feet for raptors, depending on the proposed work activity, shall be determined~~ shall be established by a qualified biologist and demarcated with bright orange construction fencing or other suitable flagging. Active nests shall be monitored at a minimum of once per week until it has been determined that the nest is no longer being used by either the young or adults. No ground disturbance shall occur within this buffer until the qualified biologist confirms the breeding/nesting is completed and all the young have fledged. If project activities must occur within the buffer, they shall be conducted at the discretion of the qualified biologist. If no nesting birds are observed during pre-construction survey, no further action would be necessary.

## Response 1.3

The commenter expresses a concern that the project may impact coastal sage scrub, which is a sensitive habitat type that is covered in the County of Orange Central and Coastal Subregion Natural Community Conservation Plan/ Habitat Conservation Plan. The commenter recommends mitigating any loss of coastal sage scrub habitat in kind at a 3:1 ratio in close proximity to the project site.

The project site boundary shown in Figure 2 of the Draft IS-MND represents an approximation of the actual footprint of project disturbance. The yellow lines depicting the project boundary are not accurate of the design-level detail boundary, which would more tightly follow along the existing road. As indicated under *Description of Project*, of the Draft IS-MND, the project includes installation of a new electrical service feeder under the existing access road and asphalt paving of this road within its existing footprint but would not result in widening of the road such that habitat along the road would be impacted.

Therefore, potential impacts to native vegetation communities would be limited to ground-disturbing activities during installation of the proposed stormwater control improvements associated with the project, the locations of which are shown in more detail in Figure 3 of the project's Hydrology Study (Attachment 1).[2] These activities may result in direct impacts (removal) to a small number of buckwheat – California sagebrush and buckwheat – sumac individuals during installation of two energy dissipation structures adjacent to the access road within the buckwheat – California sagebrush and buckwheat – sumac mapped associations. These two associations are within the larger California buckwheat scrub alliance, an alliance type that is found within the broader coastal sage scrub vegetation community category and that is not considered sensitive on the California Sensitive Natural Communities list.[3] Furthermore, the area of disturbance within this alliance type would total less than 400 square feet. Given the limited project impacts, existing disturbance associated with the access road and existing Reservoir 2B, and prevalence of coastal

[2] MKN & Associates, Inc. 2022. Technical Memorandum – SCWD – Reservoir 2B Design Services, Hydrology Study. January 31, 2022.

[3] California Department of Fish and Wildlife.2021. California Sensitive Natural Communities. August 18, 2021. Available at: https://nrm.dfg.ca.gov/FileHandler.ashx?DocumentID=153398&inline

sage scrub habitat in the surrounding area, no impacts to sensitive natural communities would occur, as concluded in Section 4, *Biological Resources*, of the Draft IS-MND, and no mitigation is required.

## Response 1.4

The commenter states that an LSAA notification may be required for project impacts to the potentially jurisdictional unnamed ephemeral drainage located within the Study Area and encourages further consultation with CDFW regarding the potential submittal of an LSAA notification package.

A Rincon Senior Biologist met with CDFW Environmental Scientist Jennifer Blackhall on the project site on April 29, 2022. Rincon and CDFW reviewed the project site, and Rincon verbally described the project components and indicated the estimated areas of impact. CDFW subsequently issued a determination the same day, both verbally and via email, that the project would not impact the surveyed drainage, and that the project, as it was described to CDFW, would not require an LSAA notification.[4] Therefore, SCWD does not anticipate submittal of an LSAA notification package will be necessary for the proposed project.

## Response 1.5

The commenter states the requirements for reporting observations of special status species, requests submittal of observation data to the California Natural Diversity Database should any special status species be detected, and provides guidance for submittal.

All detected special status species will be reported in accordance with the requirements of Public Resources Code Section 21003(e).

## Response 1.6

The commenter states CDFW's filing fee requirements.

This comment is noted. SCWD would be required by law to pay all appropriate CDFW filing fees.

## Response 1.7

The commenter provides a concluding statement and their contact information.

The comment is noted.

[4] Blackhall, Jennifer. 2022. Environmental Scientist, Lake and Streambed Alteration Agreement Program, California Department of Fish and Wildlife South Coast Region 5. Personal communication via email regarding whether the project would require a Lake or Streambed Alteration Agreement notification with Jared Reed, Senior Biologist, Rincon Consultants, Inc. April 29, 2022.

# Attachment 1

Hydrology Report

TECHNICAL MEMORANDUM

**To:** Taryn Kjolsing, PE | South Coast Water District

**From:** Oscar Daza, PhD, PE | MKN & Associates
Becca Bugielski, PE | MKN & Associates
Tessa Gallagher, EIT | MKN & Associates

**Date:** March 11, 2022

**Re:** SCWD – Reservoir 2B Design Services, Hydrology Study

1 Project Background

The Reservoir 2B Replacement Project for the South Coast Water District (the District) is located in the City of Laguna Beach near Aliso Peak. The project drainage area covers 6.75 acres. A hydrologic study is being performed to determine the appropriate stormwater infrastructure based on existing conditions at the project site. The project improvements are zoned by the City of Laguna Beach as drainage ways (Figure 1).

Figure 1: City of Laguna Beach Drainage Ways Map

The project includes demolition and replacement of an existing tank and the addition of one new above ground tank that will not significantly impact the current stormwater runoff quantities. Currently, the road in the project area leading to the tank sites has no stormwater infrastructure. Flows during large events drain along the roadway in natural ditches and overtop the road at low points, including the hairpin turn and the end of the roadway at Ceanothus Drive. Due to current drainage patterns, the District has experienced erosion on this site during past storm events. The District is proposing to pave the existing access roadway and tank site to improve access, especially for emergency maintenance needs and wet weather accessibility. In an effort to minimize future road maintenance and reduce potential erosion of the proposed roadway base, the District is proposing the addition of stormwater culverts at low points along the access road. This Technical Memorandum (TM) recommends the preliminary stormwater infrastructure with required sizing.

Three main watersheds and six sub-watersheds have been initially defined for the existing condition (Figure 2). These subwatersheds contribute to concentrated runoff at the three indicated points where there are existing topographic depressions (Figure 2). A site investigation was completed on February 25, 2022. During this investigation, it was found that the existing gravel roadway showed evidence of once being paved with asphalt. The site has since been naturally pulverized and topped with gravel. Photos of the site visit and existing roadway conditions can be found in **Appendix A**. The infiltration rate of the existing surface is considered the same as the proposed asphalt surface, impervious. The design of this recommended infrastructure is intended to keep the existing drainage patterns onsite. For this reason, the calculations shown in this TM represent the existing and proposed runoff conditions. In keeping the existing drainage patterns in the proposed condition, the District will not impair the function, scenic or ecological purpose of the watershed.

## 2 Design Parameters

For the hydraulic study, soil types, topographic information, and rainfall data were required. Soil types, taken from the geotechnical report (Appendix B), and hydrologic soil groups of the subwatersheds have been determined along with the tributary areas of each sub-watershed (Figure 2). Six areas were initially defined as sub-watersheds that contribute to runoff based on available topography. A design storm with a return period of 5 years was selected. Using the 5-year return period is a common design standard. The Caltrans Highway Design Manual Section 818.2 instructs a designer to choose an appropriate design flood frequency based on associated risks because "accommodating the worst possible event that could happen is usually so costly that it may not be justified" (Caltrans). The Caltrans Stormwater Quality Handbooks Project Planning and Design Guide recommends sizing systems for "the 2-year, 5-year, or 10-year 24-hour return period" noting that oversizing your system may cause unneeded environmental impact with more area of disturbed soil. The same document goes on to add that even designing for a 2-year storm may oversize a system (Caltrans, 2017). The return period of 5 years was also selected to limit fit improvements to the roadway within the existing site constraints and limit the costs and the impacts of construction.

Based on the 5-year storm, the precipitation values were found using the Intensity-Duration-Frequency (IDF) curves provided in the Orange County Hydrology Manual (Table 1). The topographical information was used to determine the distance and slope of each basin (Table 2). These values along with the time of concentration were used to estimate the peak discharges at the points of interest.

Table 1: Rainfall data from the Orange County Hydrology Manual.

| | Rainfall Intensity i (in/hr) | | | |
|---|---|---|---|---|
| $t_c$ | $T_r$ (years) | | | |
| min | 10 | 25 | 50 | 100 |
| 5 | 6.30 | 7.50 | 9.00 | 10.00 |
| 15 | 2.20 | 3.40 | 4.00 | 4.50 |
| 30 | 1.70 | 2.10 | 2.40 | 2.70 |
| 60 | 1.20 | 1.50 | 1.70 | 1.95 |
| 180 | 0.80 | 1.30 | 1.20 | 1.35 |
| 360 | 0.65 | 0.80 | 0.90 | 1.05 |
| 1440 | 0.29 | 0.36 | 0.42 | 0.47 |

Table 2: Slope and distance of each basin.

| **Basin ID** | **Area A** | | **Elevation** | | **Elevation Difference** | **Distance L (from $z_1$ to $z_2$)** | | | | **Slope $S_o$** |
|---|---|---|---|---|---|---|---|---|---|---|
| | **$ft^2$** | **Acres** | **$z_1$** | **$z_2$** | **ft** | **ft** | **in** | **ft** | **m** | **ft/ft** |
| 1 | 43583.31 | 1.00 | 523 | 380 | 143 | 293.0 | 4.50 | 293.4 | 89.4 | 0.487 |
| 2 | 50634.30 | 1.16 | 574 | 430 | 144 | 298.0 | 10.25 | 298.9 | 91.1 | 0.482 |
| 3 | 71432.95 | 1.64 | 623 | 472 | 151 | 346.0 | 7.00 | 346.6 | 105.6 | 0.436 |
| 4 | 30936.98 | 0.71 | 428 | 350 | 78 | 140.0 | 2.00 | 140.2 | 42.7 | 0.556 |
| 5 | 72620.13 | 1.67 | 472 | 330 | 142 | 455.0 | 0.00 | 455.0 | 138.7 | 0.312 |
| 6 | 24979.41 | 0.57 | 346 | 285 | 61 | 152.0 | 2.75 | 152.2 | 46.4 | 0.401 |

## 3 Methodology

The rational method, as given in the Orange County Hydrology Manual, was used to determine the runoff for each basin. The rational method can be used for the drainage basins for this project because they are less than 640 acres, and the rainfall intensity can be assumed to be uniformly distributed over the drainage basin (Orange County Environmental Management Agency, 1986). The equation for the rational method is given as

$$Q = CIA$$

where Q = runoff (cubic feet per second), C = runoff coefficient, I = time-averaged rainfall intensity (in/hr), and A = drainage area (acres) (Chin, 2000).

The total runoff for each basin was calculated for a return period of 5 years. The project area was divided into six drainage basins for each of the five culverts. The rational method was then used to determine the runoff that is concentrated at each culvert from the tributary areas. Culverts 2 and 3 are downstream of culvert 1 which is collecting runoff from the tank site drainage area. For these culverts, areas that were upstream were added together to determine the total runoff. An initial runoff coefficient of 0.36 was used for each area based on applied technical references (Mays, 2005). Data for the rainfall intensity was obtained from the Intensity-Duration-Frequency figures found in the Orange County Hydrology Manual and using the regressed equation from the original data. The data for the 5-year storm was graphed, and a power function was used to determine the rainfall intensity as a function of the time of concentration. The regression equation used was


FIGRUE 2- EXISTING DRAINAGE CONDITIONS

$$I = a t_c^b$$

where I = rainfall intensity (in/hr), coefficients a = 10.620 and b = -0.596 (for the 5-year storm), and $t_c$ = time of concentration (min).

Time of concentration was calculated using the longest distance from the topographic top of the drainage area to the culvert and the slope of the drainage area, using the Kirpich equation

$$t_c = 0.019 \frac{L^{0.77}}{S_o^{0.385}}$$

where $t_c$ = time of concentration (min), L = flow length (m), $S_o$ = average slope along the flow path (Chin, 2000).

Preliminary hydraulic calculations of the proposed ditches and culverts were performed using the Bentley FlowMaster software (Bentley Systems, Inc.). The results of the hydrologic analysis show an increase of peak flows in the downstream direction as expected.

## 4 Results

Three culverts were planned at points of interest to allow runoff to ultimately flow toward an existing inlet on Ceanothus Drive (Table 3). Runoff values for each culvert were calculated using the rational method (Table 4).

Table 3: Drainage areas and inlet time

| Catchment | Area A (acres) | Runoff Coefficient C Tr = 5 years | Time of Overland Flow $t_o$ (min) | Time of Channelized Flow $t_{ch}$ |
|---|---|---|---|---|
| 1 | 1.00 | 0.36 | 7.94 | **0.82** |
| 2 | 1.16 | 0.36 | 8.03 | 0.82 |
| 3 | 1.64 | 0.36 | 8.81 | 0.97 |
| 4 | 0.71 | 0.36 | 5.45 | 0.44 |
| 5 | 1.67 | 0.36 | 7.65 | 1.37 |
| 6 | 0.57 | 0.36 | 6.12 | 0.53 |
| Total | 6.75 | | | |

'Time of overland and channelized flow are calculated in SCWD Hydrology

Table 4: Runoff calculations using the rational method.

| Location | Catch't Area No. | Drained Area A | C | Channel | Time of Channelized Flow $t_{ch}$ | Time of Conc'tn $t_c$ | Storm Duration $t_d$ | Rainfall Intensity i | Design Discharge Q | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|
| | | (acres) | | | (min) | (min) | (min) | (in/hr) | ($ft^3/s$) | |
| PT3 | 3 | 1.64 | 0.36 | | 0.00 | 8.81 | | | | |
| | | 1.64 | | | | | 8.81 | 2.9 | 1.7 | Culvert 1 |
| | | | | | | | | | | |
| | | | | G5 | 0.77 | 9.58 | | | | |
| PT5 | 3 | 1.64 | 0.36 | | | | | | | |
| | 5 | 1.67 | 0.36 | | | | | | | |
| | | 3.31 | | | | | 9.58 | 2.8 | 3.3 | |
| | | | | | | | | | | |
| PT2 | 2 | 1.16 | 0.36 | | 0.00 | 8.03 | | | | |
| | | 1.16 | | | | | 8.03 | 3.1 | 1.3 | |
| | | | | | | | | | | |
| | | | | CD1 | 1.05 | 9.08 | | | | |
| PT1 | 2 | 1.16 | 0.36 | | | | | | | |
| | 1 | 1.00 | 0.36 | | | 7.94 | | | | |
| | | 2.16 | | | | | 9.08 | 2.9 | 2.2 | Culvert 2 |
| | | | | | | | | | | |
| | | | | CD4 | 1.50 | 10.58 | | | | |
| PT4 | 2 | 1.16 | 0.36 | | | | | | | |
| | 1 | 1.00 | 0.36 | | | | | | | |
| | 4 | 0.71 | 0.36 | | | 5.45 | | | | |
| | | 2.87 | | | | | 10.58 | 2.6 | 2.7 | |
| | | | | | | | | | | |
| | | | | CD5-1 | 0.91 | 11.49 | | | | |
| | | | | | | 9.58 | | | | |
| PT6 | 2 | 1.16 | 0.36 | | | | | | | |
| | 1 | 1.00 | 0.36 | | | | | | | |
| | 4 | 0.71 | 0.36 | | | | | | | |
| | 3 | 1.64 | 0.36 | | | | | | | |
| | 5 | 1.67 | 0.36 | | | | | | | |
| | | 6.18 | | | | | 11.49 | 2.5 | 5.5 | Culvert 3 |

Using the runoff values from a 5-year storm, the preliminary sizing of culverts was determined using Bentley FlowMaster software (Table 5). The size of each culvert is provided for illustration purposes assuming open channel flow in a circular conduit flowing half full. Hydraulic parameters include slope $S_o$=0.005 and Manning roughness coefficient n=0.013. The flowrates involve velocities that still need to be defined and managed along the main path in each catchment area to prevent erosion and transport of sediments. The hydraulics of each culvert will need to be managed individually depending on the topographic and soil conditions. Flowrate estimates do not include the contribution of runoff and its management along the road. Runoff along the roadway will be managed by maintaining the natural existing drainage ditches and where feasible triangular concrete channels to prevent erosion. Transmission along a catchment area is assumed to be in a triangular ditch with 1:1 side slope and a Manning roughness coefficient n=0.030.

Table 5: Summary of Culvert Capacity

| Culvert | Design discharge Q ($ft^3/s$) | Geometry | Diameter (ft) | Diameter (in) |
|---|---|---|---|---|
| 1 | 2.2 | Circular | 1.25 | 15 |
| 2 | 1.7 | Circular | 1.25 | 15 |
| 3 | 5.5 | Circular | 1.75 | 21 |

The culverts were sized based on the calculated discharge values. If there are profile sizing concerns, dual pipes with smaller diameters may be considered rather than one large pipe. There are a couple of issues that need to be considered as part of the design process. It is expected that these flows will have high velocities that need to be managed to prevent local scouring at the discharge of the culverts. Providing energy dissipation structures and/or rip-rap hard surface downstream from these locations will allow the water to continue flowing downstream along the existing natural drainage system without washouts. The flow estimates do not include the contribution of the tributary area along the road. It is expected that this contribution will not significantly impact the initial flow estimates presented in this TM. What is important is the definition of how the lateral drainage of the road is going to be conformed (e.g., single lateral slope draining towards the toe of the slope, dual lateral slope with a crown). The lateral flows need to be collected and transported using existing or lined ditches along the road to be finally discharged into the corresponding culvert. **Figure 3** shows the proposed stormwater improvements for the site.

## 4 Conclusion

The design for this infrastructure is intended to keep the existing drainage patterns. The existing and proposed drainage patterns follow the same flow paths. Based upon the existing surfaces properties identified in the geotechnical report, no additional impervious area is being proposed on the tank site or roadway. Flows will now be channelized under the proposed asphalt roadway, through culverts, as to not erode the roadway base material. These channelized flows will be offset with the addition of energy dissipation measures. These measures will reduce the flow velocity as well as capture erosive soils before they would otherwise reach Ceanothus Drive. The City of Laguna Beach Drainage Ways will maintain their function, scenic and ecological purposes in the watershed.


FIGURE 3- PROPOSED STORMWATER IMPROVEMENTS

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## 6 Appendices

Appendix A- Site Visit Photos

Appendix B- Geotechnical Engineering Report

APPENDIX A

16310 Bake Parkway
Irvine, CA 92618
714.213.9758 PHONE

# 2/25/2022 Site Visit

Roadway Entrance Ceanothus Drive

Evidence of previous asphalt surface 15 ft beyond front gate

Sandbags at Front Entrance along Ceanothus Drive

Existing drainage path 100 ft from front gate


Existing Drainage way from Tank Site


Existing roadway drains to toe of slope


Existing Drainage way west of the tank site.


Lack of width to install v-ditch

Hair-pin turn including roadway washouts

Existing drainage into hair-pin turn

Hair-pin turn including roadway washouts

Roadway to Tank Site

Catch Basin in Ceanothus Drive

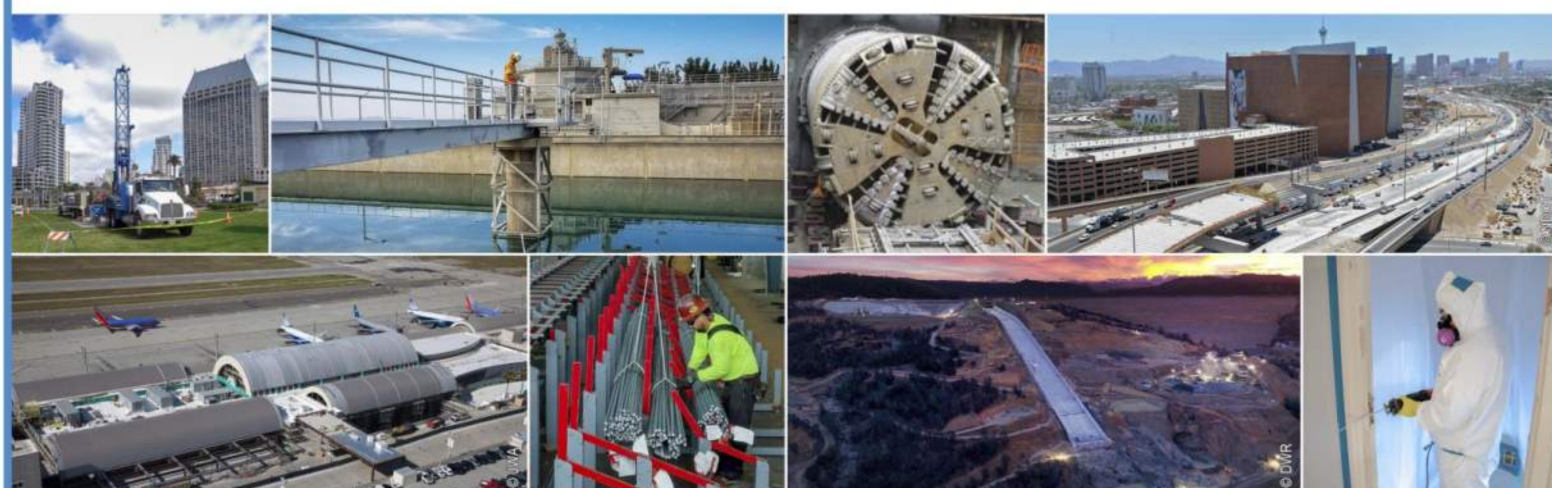
Catch Basin in Ceanothus Drive

APPENDIX B

# Geotechnical Evaluation
# Reservoir 2B Replacement Project
# South Coast Water District
## Laguna Beach, California


Michael K. Nunley & Associates (MKN)
16310 Bake Parkway | Irvine, California 92618


January 28, 2022 | Project No. 211532002




Geotechnical | Environmental | Construction Inspection & Testing | Forensic Engineering & Expert Witness

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Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

CELEBRATING 35 YEARS Ninyo & Moore

# Geotechnical Evaluation
# Reservoir 2B Replacement Project
# South Coast Water District
## Laguna Beach, California

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## TABLES



## FIGURES

## APPENDICES

# 1 INTRODUCTION

In accordance with your request and authorization, we have performed a geotechnical evaluation for the proposed Reservoir 2B Replacement Project located to the east of Ceanothus Drive with the physical address of 31456 Alta Loma Drive in Laguna Beach, California (Figure 1). The site includes a single reservoir: Reservoir 2B. The project includes the demolition of the existing Reservoir 2B and constructing two new reservoirs as a replacement. The purpose of our study was to evaluate the soil and geologic conditions at the site and provide geotechnical recommendations for the design and construction of the proposed improvements. This report presents the findings from our background review and subsurface exploration, results of our laboratory testing, conclusions regarding the subsurface conditions at the site, and geotechnical recommendations for design and construction of this project.

# 2 SCOPE OF SERVICES

Our scope of services included the following:

- Project coordination, planning, and scheduling of the subsurface exploration.
- Review of our previous Technical Memorandum for the Reservoirs 2B and 3B Replacement Project (Ninyo & Moore, 2020) and preliminary plans provided by the client.
- A site reconnaissance, performed on November 1, 2021, to observe the general site conditions, mark-out the proposed boring locations, and coordination with Underground Service Alert for utility clearance.
- Subsurface exploration consisting of the excavation, sampling, and logging of two exploratory large-diameter borings to depths of up to approximately 60 feet. The borings were downhole logged in the field by our representative and relatively undisturbed and bulk samples were collected and returned to our laboratory for evaluation and testing. The large-diameter borings were backfilled with the excavated soil.
- Laboratory testing on selected soil samples including evaluation of in-situ moisture content and dry density, gradation, percent of soil particles finer than the No. 200 sieve, draft shear strength, Proctor density, R-value, and soil corrosivity characteristics (including pH, resistivity, and water soluble sulfates and chlorides).
- Participation in project planning meetings to discuss the project, geotechnical constraints, and alternative reservoir layouts and retaining wall options.
- Data compilation and engineering analysis of the information from our background review, subsurface exploration, and laboratory testing.
- Preparation of this report presenting our findings, conclusions, and recommendations pertaining to the geotechnical aspects of the design and construction of the proposed improvements.

## 3 SITE DESCRIPTION AND BACKGROUND

The Reservoir 2B site is located to the east of existing residential properties on a predominantly natural hillside. The site is accessed by an approximately ¼-mile-long unpaved access road from Ceanothus Drive with a noted physical address of 31456 Alta Loma Drive. Based on our review of historical aerial images, the access road was present prior to construction of the reservoir in 1946 (Historical Aerials, 2020). A relatively steep natural slope ascends from the north side of Reservoir 2B to Aliso Peak to an elevation of approximately 623 feet above mean sea level (MSL) (USGS, 1981). Relatively steep slopes descend to the south and west from the tank site to an elevation of approximately 290 feet above MSL to residential properties and Ceanothus Drive. The slopes have a relatively thick growth of native brush.

Based on our review of undated South Coast County Water District record drawings, construction of Reservoir 2B (previously known as Coast Royal Reservoir) involved cut and fill grading to construct a relatively level pad for the reservoir. The ground surface in the area of Reservoir 2B is approximately 472 feet above MSL (Figure 2). Grading of the reservoir pad involved the excavation of a cut slope on the north and east sides of the reservoir at a relatively steep inclination of approximately 3/4:1, horizontal to vertical, up to approximately 34 feet in height and the partial filling of a southwest-trending drainage swale. The partial filling of the drainage swale resulted in an approximately 50-foot-high slope that was graded at a slope ratio of approximately 1½:1 (horizontal to vertical).

## 4 PROPOSED CONSTRUCTION

Based on our review of the Request for Task Order Proposals (SCWD, 2021) and on our review of concept drawings prepared by MKN (2021), we understand that the existing Reservoir 2B and associated equipment will be demolished and replaced with two new 33-foot-diameter, 100,000-gallon, welded steel tanks (Figure 3). In order to construct the new tanks, new site grading and retaining wall construction will be needed to create a larger pad area for the two tanks. We understand that soldier pile retaining walls are planned for the project. The conceptual drawing indicates that two retaining walls, one up to approximately 16-feet-high and one up to approximately 20-feet-high, are planned to construct the larger pad. The new 16-foot-high retaining wall is planned to enlarge the reservoir pad to the southeast and the 20-foot-high retaining wall is planned to enlarge the reservoir pad to the northwest. Planned reservoir finished floor elevations are approximately four feet below current grade elevations. We understand that the new reservoirs will be supported on a ring foundation, which will be primarily founded on bedrock. In areas where existing fill is present (mainly the southwest sides of the proposed tanks), the foundations will be deepened so that the foundations are supported by the bedrock. New piping, an inlet/outlet pipe vault, and drainage improvements are also planned. The existing site conditions, previous site topography, proposed improvements, and anticipated grading are shown on Figure 3 and in Cross Section A-A’ on Figure 4.

# 5 SUBSURFACE EVALUATION AND LABORATORY TESTING

Our subsurface exploration consisted of the drilling, logging, and sampling of two large diameter borings (B-1 and B-2) on November 8 and 9, 2021. The borings were drilled to depths of approximately 25 feet and 60 feet, respectively, using a truck-mounted drill rig utilizing 24-inch-diameter bucket augers. The borings were logged in the field by a representative of Ninyo & Moore during drilling and subsequently downhole logged by a certified engineering geologist upon completion of drilling to the depths shown on the logs. Representative bulk and relatively undisturbed soil and bedrock samples were collected from the borings at selected depths for laboratory testing and transported to our laboratory. The approximate locations of the borings are presented on Figure 2. Logs of the borings are presented in Appendix A.

Laboratory testing was performed on representative soil samples collected from borings. The laboratory testing included evaluation of in-situ moisture content and dry density, gradation, percent of soil particles finer than the No. 200 sieve, direct shear strength, Proctor density, R-value, and soil corrosivity characteristics (including pH, resistivity, and water soluble sulfates and chlorides). The results of the in-situ moisture content and dry density tests are presented on the boring logs in Appendix A. The remaining laboratory testing results are presented in Appendix B.

As a part of our evaluation, we reviewed our Technical Memorandum for the proposed Reservoirs 2B and 3B dated November 17, 2020. The prior study included seismic refraction profiles to evaluate rippability of the San Onofre bedrock material, discussed in Section 9 of this report. The locations of the seismic refraction profiles, and our previous geologic mapping, are also shown on Figure 2. Copies of the profiles are included in Appendix C.

# 6 GEOLOGY AND SUBSURFACE CONDITIONS

## 6.1 Regional Geology

The project site is located in the Peninsular Ranges Geomorphic Province of Southern California (Norris and Webb, 1990). The province is characterized by northwest to southeast trending mountain ranges and valleys and similarly trending strike-slip faults associated with the boundary between the North American and Pacific tectonic plates. In general, the mountain ranges are underlain by Jurassic-age metavolcanic and metasedimentary rocks and Cretaceous-age igneous rocks of the southern California batholith. Based on our review of the referenced geologic maps, the site is mapped as being underlain by middle Miocene-age San Onofre Breccia (Morton, 2006; Kennedy, 2007). The San Onofre Breccia generally consists of massive to well-bedded, well-indurated breccia with interbedded conglomerate, sandstone, siltstone, and mudstone (Figure 5).

## 6.2 Site Geology

Earth materials observed during our site visit and subsurface exploration consisted of artificial fill, slope wash and bedrock materials of the San Onofre Breccia. A general description of the soil and bedrock materials that we observed is provided below. More detailed descriptions of the subsurface materials encountered during our subsurface exploration are presented on the boring logs in Appendix A, and our interpretation of the subsurface conditions is depicted on Cross Section A-A' (Figure 4).

### 6.2.1 Artificial Fill (Qaf)

Artificial fill was observed adjacent to the descending slope of the reservoir pad and adjacent to the descending slopes along the access road. The fill material is presumably derived from the San Onofre Breccia and generally consisted of silty sand with gravel and cobbles. Fill in Boring B-2 was observed to a depth of approximately 14 feet. Based on our borings and a review of the original construction plans provided by SCWD, fills approximately 20 feet in thickness are anticipated at Reservoir 2B to the southwest edge of the reservoir pad. Figure 3 includes the previous site topography shown on the original construction plans.

### 6.2.2 Slopewash (Qsw)

Slopewash was observed on the slopes and drainage gullies adjacent to the site and within Boring B-2 at a depth of between approximately 14 feet and 16 feet. The slopewash generally consisted of silty sand with gravel and cobbles and is anticipated to be relatively thin based on Boring B-2 and nearby road cut exposures. However, thicker accumulations of slope wash are anticipated in drainage swales.

### 6.2.3 Bedrock – San Onofre Breccia (Tsob)

Bedrock materials of the San Onofre Breccia were observed in both borings B-1 and B-2, and is exposed in road cuts and other cut slopes beneath the relatively thin mantle of slopewash. The San Onofre Breccia generally consisted of blueish green to blueish gray, moderately hard to hard, well-cemented, massive to thickly bedded, fine- to coarse-grained sandstone, sandstone with angular gravel (breccia), and conglomerate (breccia). The bedrock was observed to be intensely weathered near the contact with the overlying slopewash and became less weathered, and harder, with depth.

#### 6.2.4 Geologic Structure

The San Onofre bedrock materials were generally observed to be massive to indistinctly-bedded. In the borings, the bedrock was generally massive and significant fractures were not observed. Imbricated boulders were observed at a depth of approximately 8 feet in boring B-1 with an inferred bedding orientation of N53°W, 40°S. The geologic structure of the bedrock observed in the access roadcut exposures in the vicinity of Reservoir 2B was observed to dip at approximately 27 to 52 degrees to the southeast to southwest, which is generally consistent with the bedding orientation shown on the regional geologic map (Figure 4). The apparent dips of bedding observed was approximately 37 to 40 degrees to the south. Joints observed in the access roadcut exposures and cut slopes adjacent to the reservoir pad were undulatory and discontinuous, and were observed to dip at approximately 10 degrees top vertical in varying directions. The geologic structure is shown on Figure 2 and Cross Section A-A’, Figure 4.

### 6.3 Groundwater

Groundwater was not encountered at the time of drilling our exploratory borings. However, seepage along the contacts between slopewash and bedrock could develop during the rainy season and could be encountered during future earthwork at the site. Fluctuations in groundwater levels will occur due to variations in precipitation, ground surface topography, subsurface stratification, irrigation, groundwater pumping, and other factors that may not have been evident at the time of our field evaluation.

## 7 FAULTING AND SEISMICITY

The project site is located in a seismically active area, as is the majority of southern California. The numerous faults in southern California include active, potentially active, and inactive faults. As defined by the California Geological Survey (CGS), active faults are faults that have ruptured within Holocene time, or within approximately the last 11,000 years. Potentially active faults are those that show evidence of movement during Quaternary time (approximately the last 1.6 million years) but for which evidence of Holocene movement has not been established. Inactive faults have not ruptured in the last approximately 1.6 million years. The approximate locations of major active faults in the region and their geographic relationship to the project site is shown on Figure 6.

The site is not located within a State of California Earthquake Fault Zone (formerly known as Alquist-Priolo Special Studies Zone). Based on our review of seismic hazard maps, geologic literature, and geologic maps, no active faults are known to cross the subject sites. The reservoir site is located about 2½ miles northeast of the offshore segment of the Newport Inglewood fault zone (USGS, 2008). The principal seismic hazards at the subject site are surface fault rupture, strong ground motion and earthquake-induced landslides and slope stability. Liquefaction is not a consideration for the project due to the shallow depth of bedrock at the site. A brief description of these hazards and the potential for their occurrences on site are discussed below.

## 7.1 Surface Fault Rupture

There are no known active faults crossing the subject site, and the potential for ground rupture due to faulting is considered low. Surface ground cracking related to shaking from distant events is not considered a significant hazard, although it is a possibility.

## 7.2 Seismic Ground Motion

The 2019 California Building Code (CBC) specifies that the Risk-Targeted, Maximum Considered Earthquake ($MCE_R$) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. The $MCE_R$ ground motion response accelerations are based on the spectral response accelerations for 5 percent damping in the direction of Maximum horizontal response and incorporate a target risk for structural collapse equivalent to 1 percent in 50 years with deterministic limits for near-source effects. Based on our review of California Geological Society's on-line Data Viewer Application (CGS, 2020), the inferred site shear wave velocity ($V_{S-30}$) is approximately 387 meters per second. Accordingly, the site is considered to be Site Class is C. The horizontal peak ground acceleration (PGA) that corresponds to the $MCE_R$ for the site was calculated as 0.58g using the Applied Technology Council (ATC) seismic design tool (ATC, 2020 [web-based]).

The 2019 CBC specifies that the potential for liquefaction and soil strength loss be evaluated, where applicable, for the mapped Maximum Considered Earthquake Geometric Mean ($MCE_G$) PGA ($PGA_M$) with adjustment for site class effects in accordance with the American Society of Civil Engineers 7-16 Standard. The $MCE_G$ PGA is based on the geometric mean PGA with a 2 percent probability of exceedance in 50 years. The site modified $PGA_M$ was calculated as 0.70g using the ATC seismic design tool (ATC, 2021 [web-based]).

## 7.3 Liquefaction Potential

Liquefaction is the phenomenon in which loosely deposited granular soils and non-plastic silts located below the water table undergo rapid loss of shear strength when subjected to strong earthquake-induced ground shaking. Ground shaking of sufficient duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure, and causes the soil to behave as a fluid for a short period of time. Liquefaction is known generally to occur in saturated or near-saturated cohesionless soils at depths shallower than 50 feet below the ground surface. Liquefaction is also known to occur in relatively fine-grained soils (i.e., sandy silt and clayey silt) with a plasticity index (PI) of less than 12 and an in-place moisture content more than 85 percent of the liquid limit (LL) and sensitive silts and clays with a PI more than 18. Factors known to influence liquefaction potential include composition and thickness of soil layers, grain size, relative density, groundwater level, degree of saturation, and both intensity and duration of ground shaking.

Review of the State of California Seismic Hazards Zones map (CDMG, 1999) indicates that the site is not located in an area mapped as a potential liquefaction hazard zone (Figure 7). Additionally, the historic high groundwater at the site is more than approximately 100 feet below the ground surface and the site is generally underlain by bedrock. Accordingly, it is our opinion that liquefaction and liquefaction-related seismic hazards (e.g., dynamic settlement, ground subsidence, and/or lateral spreading) are not design considerations for the project.

## 7.4 Earthquake-Induced Landslides and Slope Stability

The State of California Seismic Hazards Zones Maps indicates that the site is located within areas that are mapped as susceptible to seismically induced landslides (Figures 7). Landslides may be induced by strong vibratory motion produced by earthquakes. Research and historical data indicate that seismically induced landslides tend to occur in weak soil and rock on sloping terrain. The process for zoning earthquake-induced landslides incorporates expected future earthquake shaking, existing landslide features, slope gradient and strength of earth materials on the slope.

Earthquake-induced landslide failures tend to occur along weak bedding planes that are adversely oriented with respect to the exposed slope face. Regional geologic maps, our field mapping, and our subsurface exploration indicates that the formational materials consists of San Onofre Breccia. Where observed in cut slopes and in our borings, the bedrock consisted of relatively strong, well cemented sandstone, conglomerate and breccia. We did not observe siltstone or mudstone exposed in the large-diameter borings or the existing cut slopes. Bedding in the vicinity of Reservoir 2B was observed to dip moderately to steeply to the southeast to southwest with discontinuous joints observed to from 10 degrees to vertical in varying directions. The bedding orientations have an out-of-slope component for the southwest-facing slopes at the site, and may be out-of-slope for the south-facing slope behind the existing reservoir. However, bedding behind the reservoir was thickly bedded and not readily discernable. During our site exploration and geologic mapping, we did not observe evidence of significant slope instability such as existing landslides, scarps or tension cracks.

# 8 SLOPE STABILITY ANALYSIS

Slope stability analyses were performed to evaluate the stability of the slopes above and below the new reservoirs (Cross Section A-A'). As described in Section 6.2.4, the bedding orientation documented during our geologic mapping was utilized in our analyses and the apparent dips were considered at the cross section A-A'. Since the apparent dips are steeper than the slope angle, the slope instability model for along out-of-slope bedding was not considered in our slope stability analysis.

The shear strength parameters used in our stability analyses for artificial fill and bedrock materials were selected based on laboratory direct shear tests performed during this study and our experience with these geologic units. The material properties used in our stability analyses are presented in Table 1.

**Table 1 –Material Properties Used in Slope Stability Evaluation**

| Earth Material | Unit Weight (pcf) | Cohesion (psf) | Friction Angle (degree) |
|---|---|---|---|
| Artificial Fill | 120 | 300 | 33 |
| San Onofre Formation | 130 | 300 | 42 |

**Notes:**
psf – pounds per square feet

Our slope stability analyses were performed on the basis of the Modified Bishop method using a two-dimensional stability analysis program, GSTABL7 (Geo-Slope International Ltd., 2012), for static and pseudo-static conditions. Failure surfaces were generated using the "CIRCLE" and "RANDOM" search algorithms. Iterations using these subroutines yield what we consider to be critical failure surfaces. Pseudo-static analyses were performed using a coefficient of horizontal ground acceleration of 0.15 as recommended in Special Publication (SP) 117A (CGS, 2008). As discussed in SP 117A, slopes with factors of safety of 1.5 and 1.1 for the static and pseudo-static conditions, respectively, are considered adequately stable.

The results of our global stability evaluation indicate that the slopes adjacent to the site will have adequate factors of safety against global instability under static and pseudo-static loading conditions. The results of our slope stability analysis are presented in Appendix C.

# 9 RIPPABILITY

As a part of our previous study completed in 2020, Ninyo & Moore evaluated the rippability of the bedrock materials expected to be encountered during grading for the proposed Reservoir 2B. We performed three seismic refraction profiles (Appendix C) at the locations shown on Figure 2. The profiles were performed using a 24-channel, digital seismograph with a 12-pound hammer impacting a steel plate as the energy source. A real-time noise monitor showing the geophones was checked during the survey to monitor noise levels from nearby traffic and other sources.

The modeled bedrock pressure-wave (P-wave) velocities indicated by our seismic refraction profiles generally indicate that P-wave velocities are less than 4,600 feet per second. Based on our experience with rock materials and based on the ripper performance charts provided in the Caterpillar Performance Handbook (Caterpillar, 2018), bedrock materials with seismic P-wave velocities of less than approximately 6,000 feet per second are generally rippable by a Caterpillar D-8 dozer, or equivalent, with a single-shank ripper. However, it should be noted that rock characteristics, such as fracture spacing and orientation, play a significant role in rock rippability. Rippability will also be dependent on the excavation equipment used and the skill and experience of the equipment operator.

# 10 CONCLUSIONS

Based on the results of our evaluation, it is our opinion that replacement of Reservoir 2B with two 100,000-gallon reservoirs is feasible from a geotechnical perspective, provided that the following recommendations are incorporated into the design and construction of the project. The primary geotechnical considerations at the Reservoir 2B site include the excavatability/rippability of the bedrock materials, differential settlement of the tank foundations due to transitions between bedrock and fill, and handling of oversize rock generated during earthwork.

The findings from our subsurface exploration and engineering analysis were discussed with the design team while alternative tank configurations were being considered for presentation to the South Coast Water District. Several alternative concepts were evaluated that included reservoirs of various diameters and locations that would meet the District's needs, and their respective foundation-bearing depths.

Due to the presence of potentially compressible undocumented fill and porous slope wash materials that may be sensitive to future settlement, it was preferred from a geotechnical perspective to construct a reservoir that has a deeper foundation bearing level, as this depth of excavation for the new reservoir would remove the majority of the settlement-sensitive soils. However, the planned excavations may not remove all of the settlement-sensitive soils beneath portions of the reservoir foundations on the southwest side of the new reservoirs. This condition could be mitigated by either overexcavating the bedrock areas to provide a more consistent blanket of compacted fill below the foundations, or by extending the foundations so that they bear on bedrock materials and not on fill. However, overexcavating the bedrock areas would create additional oversize material that would need to be disposed of in order to accomplish the bedrock overexcavation. Therefore, in consultation with the design team, it was decided that in lieu of overexcavating the bedrock below the planned foundation level, it was preferred to have the foundations bear directly on the bedrock. In order to transfer the foundation loads to the bedrock on the southwest side of the reservoirs, the design team decided to deepen the foundations to the bedrock.

In general, the following conclusions were made:

- The site is underlain by fill, slope wash deposits, and San Onofre Breccia Formational materials. The fill material generally consisted of moist, medium dense, silty sand with gravel, cobbles, and boulders. The slope wash deposits generally consist of moist, medium dense, silty sand with gravel and cobbles. The bedrock at the site can overall be described as moist, moderately hard to hard, weathered, breccia.

- Excavations in the on-site soils and bedrock should be feasible with earthmoving equipment in good working condition. Difficult excavating conditions should be anticipated in the bedrock and significant amounts of cobbles and boulders will be encountered during site grading, utility installation and backfill, and retaining wall construction. During drilling for soldier pile walls, the contractor should anticipate coring to penetrate through very hard boulders to achieve design depths.

- We understand that the drain line may be replaced as part of the project. Installing a new drain pipeline that will descend down the 1.5:1 (horizontal to vertical) slope is feasible utilizing earthmoving equipment that is capable of traversing relatively steep terrain. Due to the steepness of the slope, the pipe should be designed with pipe anchors such that the bottom of the anchor is founded at a depth that will provide a horizontal distance of 7 feet from the outside, bottom edge of the anchor to the slope face.

- The on-site sandy soils and bedrock should be suitable for re-use as backfill once moisture-conditioned to near the optimum moisture content. Oversize materials with a diameter of 4 inches or more should be anticipated and should be removed before use as fill. The contractor should anticipate handling oversize materials during grading, utility installation and backfill, and construction.

- Groundwater was not encountered during our subsurface exploration and groundwater is not anticipated to be a design consideration for the project. Fluctuations in groundwater levels may occur due to variations in precipitation, ground surface topography, subsurface stratification, irrigation, groundwater pumping, and other factors which may not have been evident at the time of our field evaluation.

- The bedrock materials are not subject to dynamic settlement due to earthquake-induced liquefaction or dynamic compaction of dry soils.

- The site is not located within a State of California Earthquake Fault Zone (formerly known as an Alquist-Priolo Special Studies Zone). Based on our review of published geologic maps, there are no known active faults underlying the site. Therefore, the potential for surface fault rupture at the site is considered to be low.

- Our laboratory corrosivity testing indicates that the on-site materials can be classified as non-corrosive based on the California Department of Transportation (Caltrans, 2021) corrosion guidelines.

# 11 RECOMMENDATIONS

The recommendations presented in the following sections provide geotechnical criteria regarding the design and construction of the proposed site improvements. The recommendations are based on the results of our subsurface evaluation, geotechnical analysis, and our project understanding. Detailed construction drawings were not available at the time this report was prepared. We recommend that the final construction drawings be submitted to Ninyo & Moore for review to evaluate conformance to the geotechnical recommendations provided in this report. Additional or revised recommendations may be appropriate. The proposed work should be performed in conformance with the recommendations presented in this report, project specifications, and appropriate agency standards.

## 11.1 Earthwork

Based on our understanding of the project, earthwork at the site is anticipated to consist of site clearing, drilling of soldier piles for the retaining walls, excavations of up to approximately 20 feet deep for the construction of the reservoir pad and retaining walls, remedial grading associated with the preparation of equipment pads, excavations for tank foundations including deepened foundations, trenching and backfilling associated with underground utility installation, and finished grading for establishment of site drainage. Earthwork operations at the site should be performed in accordance with the recommendations provided in the following sections of this report and applicable governing agencies.

### 11.1.1 Pre-Construction Conference

We recommend that a pre-construction conference be held. The owner and/or their representative, the governing agencies' representatives, the civil engineer, the geotechnical engineer, and the contractor should attend to discuss the work plan, project schedule, and earthwork requirements.

### 11.1.2 Demolition, Clearing, and Grubbing

Prior to performing excavations or other earthwork, the area should be cleared of existing structures, reservoir improvements, AC pavements where present, rubble and debris, abandoned utilities, surface obstructions, and other deleterious materials. Existing utilities within the project limits should be re-routed or protected from damage by construction activities. Materials generated from the clearing operations should be removed from the project site and disposed of at a legal dumpsite.

### 11.1.3 Excavation Characteristics

Based on our field exploration, we anticipate that excavations at the site may be accomplished with conventional earthmoving equipment in good working condition. The fill and slope wash materials generally consisted of medium dense, silty sand with gravel, cobbles, and boulders. The bedrock at the site can generally be described as moderately hard to hard, weathered, breccia and will involve difficult drilling and excavating conditions. Difficult drilling conditions were encountered in our exploratory borings, and the difficulty generally increased with depth. Based on the results of our seismic refraction survey, the bedrock materials have P-wave wave velocities that are less than 6,000 feet per second to the depths surveyed, as such, the bedrock materials are considered to be generally rippable by a Caterpillar D8 dozer in good working order. However, zones of well cemented, non-rippable bedrock could be encountered and cannot be ruled out. Difficult excavating conditions should be anticipated during earthwork at the site and using specialized equipment such as a rock breaker may be needed.

Cobbles and boulders should be anticipated during construction. Oversized material is not suitable for fill or backfill and should be broken into smaller pieces or disposed off-site. Contractors should make their own independent evaluation of the excavatability of the on-site materials prior to submitting their bids.

### 11.1.4 Temporary Excavations

Given the relatively hard and massive nature of bedrock, the bedrock can be considered stable rock, per Occupational Safety and Health Administration (OSHA) standards. However, the existing fill and slopewash materials are granular and may be prone to caving. The fill and slopewash materials are granular and may be prone to caving. Temporary excavations in these materials be laid back to slope inclinations of approximately 1½:1 (horizontal to vertical) or flatter and should conform to the Occupational Safety and Health Administration (OSHA) standards for Type C soils. Based on the site conditions, we anticipate that there will be sufficient space to layback the temporary excavations, however, shoring recommendations can be provided, if needed. Onsite safety of personnel is the responsibility of the contractor.

### 11.1.5 Foundation Subgrade Preparation for New Structures

Preliminary construction drawings indicate that the existing pad area will be lowered approximately 4 feet to reach the new pad grade for the reservoirs. Competent bedrock is anticipated to be exposed in the majority of the excavated pad. However older fill materials associated with the original site grading are anticipated to be exposed on the south and southwest sides of the pad. The following sections provide our recommendations for the new reservoirs and other minor structures.

#### 11.1.5.1 Foundations for New Reservoirs

As described in the previous sections, the foundations for the new reservoirs are planned to be founded on the bedrock materials. In areas where the depth to bedrock is deeper than the bottom of foundations, the footing excavation should be deepened so that it is extended 2 feet or more into competent bedrock. However, in lieu of deepening the reinforced concrete footing, two-sack cement slurry can be used to transfer the foundation load through the existing fill and slope wash to the bedrock formation.

In order to estimate the depths to competent bedrock for project planning purposes, the approximate elevations of the site topography prior to construction of the reservoir in 1946 are shown on Figure 3. Boring B-2 was drilled in the area where fill was placed to construct the existing pad. Boring B-2 encountered approximately 14 feet of fill, underlain by approximately 2 feet of slopewash materials, and then bedrock. The depth of fill encountered in boring B-2 generally correlates with the depth of fill that was anticipated based on the previous site topography. For estimating the depths to competent bedrock for foundation construction, the previous site topography can be used. However, an additional excavation depth of approximately 2 to 4 feet should be included beyond the previous topographic elevation so that the foundation excavation extends through the unsuitable native slopewash materials and highly weathered bedrock, and approximately 2 feet into competent bedrock. A Ninyo & Moore representative should observe the foundation excavations during construction to evaluate the depth to competent bedrock; the depths to competent bedrock may exceed the depths described above and should be planned for by the contractor.

#### 11.1.5.2 Foundations for Minor Structures

Where minor structure equipment pads are planned in areas where competent bedrock is exposed at the foundation subgrade, additional remedial grading should not be needed. However, where minor structures are located in areas of older fill, or where they span transitions between bedrock and older fill, remedial grading will be appropriate in order to provide suitable support and reduce the potential for differential settlement. In these areas, we recommend that soils beneath the proposed structure footprints be overexcavated and replaced with 2 feet of compacted fill. The limits of the excavation should extend laterally so that the bottom of the excavation is approximately 2 feet beyond the outside edge of the structure's footprint, or a distance corresponding to the depth of the overexcavation, whichever is farther. The excavation bottom should be evaluated by our representative during the excavation work. Additional overexcavation of loose, soft, and/or wet areas may be appropriate, depending on our observations during construction. Prior to placing newly compacted fill in areas that are overexcavated and/or in areas where the existing subgrade will be raised with new fill, the exposed bottom should be scarified, moisture-conditioned, and recompacted to a depth of approximately 8 inches.

### 11.1.6 Fill Material

In general, the on-site soils should be suitable for re-use as fill, structural fill, and trench backfill, provided they are free of trash, debris, roots, vegetation, or other deleterious materials. The contractor should anticipate using rock screens so that fill will generally be free of rocks or lumps of material in excess of 4 inches in diameter. Rocks or hard lumps larger than approximately 4 inches in diameter should be broken into smaller pieces or should be removed from the site. On-site soils used as fill will involve moisture-conditioning to achieve appropriate moisture content for compaction.

Fill used as backfill behind retaining walls and vaults should consist of free-draining, granular, non-expansive soil that conforms with the latest edition of "Greenbook" Standard Specifications for Public Works Construction for structure backfill. "Non-expansive" can be defined as soil having an EI of 20 or less in accordance with ASTM D 4829 (CBC, 2019).

Imported materials should consist of clean, non-expansive, granular material, which conforms to the latest edition of "Greenbook" Standard Specifications for Public Works Construction for structure backfill in accordance with ASTM D 4829 (CBC, 2019). Soil should also be tested for corrosive properties prior to importing. We recommend that the imported materials comply with the Caltrans (2021) criteria for non-corrosive soils (i.e., soils having a chloride concentration of 500 ppm or less, a soluble sulfate content of approximately 0.15 percent [1,500 ppm] or less, a pH value of 5.5 or higher, and a resistivity of 1,500 ohm-centimeters [ohm-cm] or more). Materials for use as fill should be evaluated by the geotechnical consultant prior to importing. The contractor should be responsible for the uniformity of import material brought to the site.

### 11.1.7 Fill Placement and Compaction

Fill placed for support of the new reservoir (above its foundation level) or beneath other site improvements such as drain vault and trench backfill should be compacted in horizontal lifts to a relative compaction of 90 percent or more as evaluated by ASTM D 1557. Fill soils should be placed at slightly above the optimum moisture content as evaluated by ASTM D 1557. The optimum lift thickness of fill will depend on the type of compaction equipment used but generally should not exceed 8 inches in loose thickness. Placement and compaction of the fill soils should be in general accordance with appropriate governing agency grading ordinances and good construction practice.

## 11.2 Underground Utilities

Where new utilities are proposed in the pad area, as well as new storm drain pipes along the access road, remedial excavations are not anticipated where bedrock is exposed at the pipeline subgrade. Where utility trenches expose existing older fill materials associated with previous site grading, the subgrade soils should be scarified, moisture conditioned, and recompacted to 90 percent relative compaction, as evaluated by ASTM D 1557.

Utility trenches should not be excavated parallel to structure footings. If needed, trenches can be excavated adjacent to a continuous footing, provided that the bottom of the trench is located above a 1:1 (horizontal to vertical) plane projected downward from a point 6 inches above the bottom of the adjacent footing. Utility lines that cross beneath footings should be encased in concrete below the footing.

### 11.2.1 Pipe Bedding

We recommend that pipelines be supported on 6 inches or more of granular bedding material such as sand with a sand equivalent value of 30 or more. Bedding material should be placed and compacted around the pipe, and 12 inches or more above the top of the pipe in accordance with the current "Greenbook" Standard Specifications for Public Works. We do not recommend the use of crushed rock for bedding material. It has been our experience that the voids within a crushed rock material are sufficiently large enough to allow fines to migrate into the voids, thereby creating the potential for sinkholes and depressions to develop at the ground surface.

Special care should be taken not to allow voids beneath and around the pipe. Bedding material and compaction requirements should be in accordance with the recommendations of this report, the project specifications, and applicable requirements of the appropriate agencies. Compaction of the bedding material and backfill should proceed along both sides of the pipe concurrently and be compacted to 90 percent or more relative compaction as evaluated by ASTM D 1557.

#### 11.2.2 Modulus of Soil Reaction

The modulus of soil reaction is used to characterize the stiffness of soil backfill placed on the sides of buried flexible pipelines for the purpose of evaluating lateral deflection caused by the weight of the backfill above the pipe. We recommend that a modulus of soil reaction of 2,000 pounds per square inch (psi) be used for design, provided that relatively granular bedding material is placed adjacent to the pipe, as recommended in this report.

#### 11.2.3 Lateral Pressures for Thrust Blocks

Thrust restraint for buried pipelines 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 passive lateral earth pressures presented on Figure 8. Excavations for construction of thrust blocks should be backfilled with granular backfill material and compacted following the recommendations presented in this report.

### 11.3 Site-Specific Seismic Design Considerations

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 2 presents the site-specific spectral response acceleration parameters in accordance with the CBC (2019) guidelines.

**Table 2 – 2019 California Building Code Seismic Design Criteria**

| Site Coefficients and Spectral Response Acceleration Parameters | Value |
|---|---|
| Site Class | C |
| Site Amplification Factor, Fa | 1.2 |
| Site Amplification Factor, $F_v$ | 1.5 |
| Mapped Spectral Response Acceleration at 0.2-second Period, $S_s$ | 1.326g |
| Mapped Spectral Response Acceleration at 1.0-second Period, $S_1$ | 0.471g |
| Site-Modified Spectral Response Acceleration at 0.2-second Period, $S_{MS}$ | 1.591g |
| Site-Modified Spectral Response Acceleration at 1.0-second Period, $S_{M1}$ | 0.707g |
| Design Spectral Response Acceleration at 0.2-second Period, $S_{DS}$ | 1.061g |

| Table 2 – 2019 California Building Code Seismic Design Criteria | |
|---|---|
| **Site Coefficients and Spectral Response Acceleration Parameters** | **Value** |
| Design Spectral Response Acceleration at 1.0-second Period, $S_{D1}$ | 0.471g |
| Mapped Maximum Considered Earthquake Geometric Mean ($MCE_G$) Peak Ground Acceleration, $PGA_M$ | 0.699g |

## 11.4 Foundations

The proposed new reservoir may be supported on a ring wall foundation bearing on the bedrock material. Other site improvements may be supported on shallow spread footings or mat foundations. 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.

### 11.4.1 Ring Wall Foundation

The footing design recommendations provided below are based on the assumption that the footing for the new reservoirs will extend 2 feet or more below the lowest adjacent finished grade with a width of 2 feet or more. These recommendations are also based on the assumption that the footings will bear on competent bedrock materials. Spread footings should be reinforced in accordance with the recommendations of the structural engineer. In addition, requirements of the governing jurisdictions and applicable building codes should be considered in the design.

Footings, as described above, may be designed using an allowable bearing capacity of 8,000 pounds per square foot (psf). Total and differential settlements for the new reservoir footings designed and constructed in accordance with the above recommendations are estimated to be on the order of ½ inch and ¼ inch over a horizontal span of approximately 40 feet, respectively.

Footings bearing on competent bedrock or two-sack cement slurry may be designed using a coefficient of friction of 0.40, where the total frictional resistance equals the coefficient of friction times the dead load. Footings may be designed using a passive resistance of 400 psf per foot of depth for level ground condition up to a value of 4,000 psf. 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 may be increased by one-third when considering loads of short duration such as wind or seismic forces.

To reduce the potential for pipe-to-tank differential settlement, which could cause pipe shearing, we recommend that a flexible pipe joint be located close to the exterior of the tank. The type of joint should be such that minor relative movement can be accommodated without distress. The pipe connections should be sufficiently flexible to withstand differential settlement of up to approximately 1 inch.

### 11.4.2 Spread Footings

The drain vault walls, retaining walls, and other miscellaneous at-grade equipment pads may be supported on shallow spread footings bearing on compacted fill prepared in accordance with the earthwork recommendations of this report. Footings should extend 24 inches or more below the lowest adjacent finished grade. Continuous footings should have a width of 24 inches or more. Isolated pad footings should have a width of 24 inches or more. Spread footings should be reinforced with a minimum of two No. 4 steel reinforcing bars, one placed near the top and one placed near the bottom of the footings, and further detailed in accordance with the recommendations of the structural engineer.

Footings, as described above and bearing on compacted fill soils with low expansion potential, may be designed using a net allowable bearing capacity of 3,000 psf. The allowable bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic forces. Total and differential settlements for footings designed and constructed in accordance with the above recommendations are estimated to be less than approximately 1 inch and ½ inch over a horizontal span of 40 feet, respectively.

Footings bearing on compacted fill may be designed using a coefficient of friction of 0.35, where the total frictional resistance equals the coefficient of friction times the dead load. Footings may be designed using a passive resistance of 350 psf per foot of depth for level ground condition up to a value of 3,500 psf. 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. In the event that the passive resistance is greater than one-half of the total allowable resistance, the passive resistance should be reduced to be the same value as the frictional resistance. The passive resistance may be increased by one-third when considering loads of short duration such as wind or seismic forces.

## 11.5 Lateral Earth Pressures for Retaining Walls

Lateral earth pressures recommended for design of soldier-beam-lagging retaining walls without and with tiebacks are provided on Figures 9 and 10, respectively. Passive pressures may be increased by one-third when considering loads of short duration, including wind and seismic loads. Measures should be taken to reduce the potential for build-up of hydrostatic pressure behind the retaining walls.

Below-grade walls of the drain vault structures may be considered to be restrained from lateral displacement under static loading conditions. Lateral earth pressures for precast vaults are typically provided with the precast structure specifications. In the event that a cast-in-place vault is used for the project, vault walls subjected to lateral earth pressures should be designed using the parameters presented on Figure 11.

## 11.6 Drilled Shaft Construction Considerations for Soldier Piles and Tiebacks

We understand that the new retaining walls will be designed as soldier pile retaining walls. The soldier pile wall contractor should mobilize equipment of sufficient size and operating capability to achieve the structural engineer’s recommended embedment length in the bedrock. The excavation technique chosen by the contractor should not adversely affect the quality or strength of the shaft side or end bearing materials. If refusal is encountered in these materials during actual installation, the geotechnical engineer should evaluate the subsurface condition to establish that true refusal has been met with adequate drilling equipment.

Groundwater is not anticipated to be encountered in the drilled holes for the piles or tiebacks. However, the drilled shafts are anticipated to be irregular due to the presence of cobbles and boulders in the bedrock. Coring may be needed where large boulders are encountered. We recommend that concrete be placed by tremie method so that the aggregate and cement do not segregate during concrete placement. Concrete utilized in the drilled shafts should be a fluid mix with sufficient slump so that it will fill the void between the rebar cage, steel beam, tieback tendon, and the drill-hole wall. The contractor should take care to reduce enlargement of the excavation at the tops of drilled shafts, which could result in mushrooming of the drilled shaft top.

Drilled shaft holes should be cleaned prior to placement of concrete. Care should be taken to check that the bedrock at the drilled shaft bottom has not been disturbed. The successful advancement of drill-holes for the construction of drilled shafts will depend largely on the suitability of the drilling equipment and the skill of the operator. The drilled foundation contractor should try to reduce the time during which the excavation remains open. The contractor should schedule the sequence of operations so that each excavation can be finished, the rebar cage, steel beam , or tieback tendon placed, and the concrete poured within the same work-day. Drilled shaft excavations should not be left open overnight. In case of delay in placing concrete within the drill hole due to equipment breakdown or other unforeseen circumstances, casing may be used to protect the integrity of the hole. While pouring concrete, the casing should be withdrawn gradually.

The contractor should not place drilled shafts adjacent to each other until the first one is set. The installation of drilled shafts should be scheduled to allow the concrete in adjacent shafts to set before drilling the next shaft. Drilled shafts spaced closer than about three shaft diameters (clear spacing) should be placed on alternate days. The minimum clearance between installing soldier piles adjacent to existing piles should not be less than 3 feet. However, this should be evaluated on a case-by-case basis depending on the type of equipment being used and the sensitivity of the improvements

The drilled shaft installation should be observed by a Ninyo & Moore representative to check that, among other things: 1) subsurface conditions are as anticipated from the boring, 2) the drilled shafts are constructed to the specified size and penetration, 3) drilled shafts are within allowable tolerances for plumbness, and 4) reinforcements are placed per project specifications. These items are fundamental to the installation and behavior of the drilled shafts. Furthermore, we recommend the following for the installation of drilled shafts:

- The clear spacing between the rebar cage or steel beam and the drill-hole surface should be three times the maximum size of the coarse aggregate used in the concrete.
- Centralizers should be installed to keep the rebar cage, steel beam, or tieback tendon positioned per project specifications.
- If casing is used, a sufficient head of concrete that fills the casing should be placed before pulling the casing.

## 11.7 Tiebacks

Tieback design should include review of the soil and geologic conditions and potential conflicts at each planned tie-back location. Tiebacks may consist of either multi-strand steel tendons or high tensile strength steel bars placed in inclined drilled holes and backfilled with low-slump concrete grout. The tiebacks should be designed for an ultimate tensile strength of 100 kips, should be inclined at 15 degrees below horizontal, and should be between 6 and 12 inches in diameter. If caving occurs, casing should be provided. For design purposes we have assumed that tieback anchors will be embedded in well-cemented formational material. Tiebacks anchored into bedrock materials may be designed using an ultimate bond stress of 30 pounds per square inch (PSI). Please note that the preliminary bond stress values provided here are for non-pressurized grouted anchors. An unbonded length equivalent to the distance between the wall face to the line projected

30 degrees from vertical up from the toe of the wall should be maintained for each tieback. Actual unbonded length will be evaluated during the design.

### 11.7.1 Tieback Installation

Tieback anchors should be installed in drilled holes using centering devices to improve anchor uniformity. The anchor holes should be filled with concrete, placed using tremie techniques, to the limit of the bonded length. The unbonded length should remain ungrouted until after testing and lock-off of the anchor. Anchors should be backfilled with lean-mix concrete or sands after testing to provide additional protection to potentially corrosive soils. If caving becomes a problem, the unbonded length should be backfilled with well-compacted sand or be cased prior to and during testing.

### 11.7.2 Tieback Testing

The tie-back anchors should be tested during construction to evaluate the design assumptions and allowable pullout capacities. The contractor should provide equipment and instrumentation to check the adequacy of the tie-backs. A dial gauge capable of measuring displacements to 0.01-inch precision should be used to measure the anchor movement. A hydraulic jack and pump should be used to apply the test load, and the jack and a calibrated pressure gauge should be used to measure the load. The standard testing procedures recommended by the Post-Tensioning Institute (2004) typically consist of the following methods.

- Performance Tests – These tests are performed on a limited number of production anchors to check that 1) the design load may be safely carried, 2) effective bonded length corresponds to the design requirements, and 3) the residual movement is within tolerable range. The performance test consists of incrementally applying cycles of anchor loading and unloading until the reference test load is attained. In order to evaluate the long-term creep potential, each load increment is maintained until the measured deflection is negligible (i.e., displacement rate is smaller than a specified displacement increment per log cycle of load-hold time) and a one-hour creep test is conducted under the reference test load. The reference test load should be 133 percent of the design allowable pullout capacity for permanent anchors. The performance test should be conducted on 5 percent of the tie-back anchors in each row of tie-backs proposed.

- Proof Tests – These tests are performed on each tie-back anchor to check that the load-deflection behavior of the production anchor is consistent with the specified acceptance criteria. The proof test consists of a single cycle of incremental loading to the reference test load (i.e., 133 percent of the design allowable pullout capacity) followed by unloading. Each load increment is maintained until the measured deflection is negligible.

The performance and proof test schedules for the anchors, including the load increments, load hold periods, acceptance criteria, and repair mechanism of failed test anchors, should be developed by the contractor utilizing his experience on similar projects and anchor design/testing recommendations and guidelines presented in this report. In general, the acceptance criteria for the tested anchors should be based on the following aspects.

- In order to allow the load transfer to reach the anchor bond length, the deflection of the anchor head should exceed 80 percent of the calculated elastic elongation of the unbonded tendon length.
- Total anchor deflection measured at the reference test load should not exceed the calculated elastic elongation of the tendon length measured from the anchor head to the center of the bond length.
- Creep displacement should not exceed 0.10 inch during the final log cycle of the load-hold period.

The test schedules of the tie-back anchors and the acceptance criteria should be included in the project plans. The project plans should be signed and stamped by a professional engineer registered in the state of California. Ninyo & Moore should be given the opportunity to review the project plans to check its compliance with design and construction recommendations presented herein

## 11.8 Pavement Design

Pavement design recommendations were prepared for new pavement that may be constructed for the access road to the tank pad. The pavement design was based on our evaluation of the subgrade soil/bedrock conditions and our laboratory testing.

The R-value characteristics of the subgrade soils were evaluated from a representative near-surface soil sample obtained from our exploratory boring B-1. Laboratory R-value testing indicates that the R-value of the materials encountered in our boring was approximately 72. Considering the variation of on-site soils, an R-value of 60 was used for the pavement design. We have prepared pavement structural sections for a Traffic Index of 5.0 and 6.0. Our pavement analysis was performed using the methodology outlined by the Highway Design Manual (Caltrans, 2012). The analysis assumes an approximately 20-year design life for new pavements. Based on the design R-value and TIs, recommendations for new pavement construction are provided in Table 3.

**Table 3 – Structural Pavement Recommendations**

| Traffic Index | Full Depth AC (inches) | AC/AB or AC/CMB (inches) |
|---|---|---|
| 5.0 | 4 | 3 over 4 |
| 6.0 | 4½ | 3 over 5 |

**Notes:**
AC – Asphalt Concrete
AB – Caltrans Class II Aggregate Base
CMB – Crushed Miscellaneous Base

Significant remedial grading is not anticipated for preparing the access road subgrade. Prior to placement of the new structural pavement section present above, the upper approximately 12 inches of the subgrade beneath the new pavements should be scarified, moisture conditioned, and recompacted to a relative compaction of 90 percent, or more, as evaluated by ASTM test method D1557. Base material should be placed at a relative compaction of 95 percent, or more, as evaluated by ASTM D 1557. If a full depth asphalt concrete pavement is selected, the subgrade soil should be compacted to 95 percent relative compaction. The subgrade compaction should also result in a non-yielding condition to allow for pavement construction. If soft subgrade conditions are encountered, overexcavation and recompaction may be needed to achieve a non-yielding subgrade surface suitable for paving.

### 11.8.1 Material Specifications

AC should conform to the latest edition of the "Greenbook," Section 203-6. Class 2 aggregate base and CMB should conform to the latest edition of the "Greenbook," Sections 200-2.2 and 200-2.4, respectively. Hot-mix asphalt materials should conform to the "Greenbook" Section 203-6. Placement and rolling of hot-mix asphalt materials should conform to the "Greenbook" Section 302-5.

## 11.9 Corrosivity

Laboratory testing was performed on one representative soil sample to evaluate pH, electrical resistivity, water-soluble chloride content, and water-soluble sulfate content. The soil pH and electrical resistivity tests were performed in general accordance with California Test Method (CT) 643. Chloride content test was performed in general accordance with CT 422. Sulfate testing was performed in general accordance with CT 417. The laboratory test results are presented in Appendix B.

The results of the corrosivity testing indicated a soil pH of between 7.9 and 8.4. The electrical resistivity was measured 12,956 ohm-cm. The chloride content was measured between 15 and 35 ppm. The sulfate content was measured 0.001 percent (i.e., 10 ppm). Based on the laboratory test results and Caltrans (2021) criteria, the soils at the project site can be classified as non-corrosive, which is defined as having earth materials with less than 500 ppm chlorides, less than 1,500 ppm sulfates, a pH of 5.5 or more, and an electrical resistivity of more than 1,500 ohm-cm.

## 11.10 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. Based on the CBC criteria, the potential for sulfate attack is negligible for water-soluble sulfate contents in soil ranging from

0.00 to 0.10 percent by weight and moderate for water-soluble sulfate contents ranging from 0.10 to 0.20 percent by weight. The potential for sulfate attack is severe for water-soluble sulfate contents ranging from 0.20 to 2.00 percent by weight and very severe for water-soluble sulfate contents over 2.00 percent by weight. The soil sample tested for this evaluation, using Caltrans Test Method 417, indicates a water-soluble sulfate content of 0.001 percent by weight (i.e., 10 ppm). Accordingly, the on-site soils are considered to have a negligible potential for sulfate attack. However, due to the potential variability of the soils on site, consideration should be given to using Type II/V cement for the project.

In order to reduce the potential for shrinkage cracks in the concrete during curing, we recommend that the concrete for the proposed structures be placed with a slump of 4 inches based on ASTM C 143. The slump should be checked periodically at the site prior to concrete placement. We further recommend that concrete cover over reinforcing steel for foundations be provided in accordance with CBC (2019). The structural engineer should be consulted for additional concrete specifications.

### 11.11 Drainage

Positive surface drainage is imperative for performance of site improvements. Positive drainage should be provided and maintained to transport surface water away from foundations and other site improvements. Positive drainage incorporates a slope of 2 percent or more over a distance of 5 feet or more away from structures, pavements, and top of slopes. Surface water should not be allowed to flow over slope faces or pond adjacent to footings.

## 12 CONSTRUCTION OBSERVATION

The recommendations provided in this report are based on our understanding of the proposed project and our evaluation of the data collected based on subsurface conditions observed in our exploratory borings and test pits. It is imperative that the geotechnical consultant checks the subsurface conditions during construction.

During construction, we recommend that the duties of the geotechnical consultant include, but not be limited to:

- Observing clearing, grubbing, and removals.
- Observe soldier pile drilled shafts and retaining wall construction.
- Observe foundation excavations and transitions for deepened footings/stepped foundations for reservoirs and cleaning prior to placement of reinforcing steel or concrete.
- Observe and remedial grading for minor structures.

- Observing excavation, placement, and compaction of fill, including trench backfill.
- Evaluating on-site soil for suitability as use as engineered fill/structural backfill prior to placement.
- Evaluating imported materials prior to their use as fill, if used.
- Performing field tests to evaluate fill compaction.
- Performing material testing services including concrete compressive strength and steel tensile strength tests and inspections.

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 the services of Ninyo & Moore are not utilized during construction, we request that the selected consultant provide the owner 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.

## 13 LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical 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. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified, and additional recommendations, if warranted, will be provided upon request. 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. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. 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.

# 14 REFERENCES

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# FIGURES


NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: USGS, 2018.

N

FEET
0 2,000 4,000

1_211532002_SL.dwg 01/11/2022 GK

**FIGURE 1**

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## SITE LOCATION

RESERVOIR 2B REPLACEMENT PROJECT
LAGUNA BEACH, CALIFORNIA

211532002 I 1/22


LEGEND

Qaf ARTIFICIAL FILL

Tso SAN ONOFRE FORMATION

GEOLOGIC CONTACT, DASHED WHERE INFERRED

TOP OF EXISTING CUT SLOPE

35 STRIKE AND DIP OF BEDDING; DIP IN DEGREES

62 STRIKE AND DIP OF JOINT; DIP IN DEGREES

STRIKE OF VERTICAL JOINT

LINE 3 SEISMIC REFRACTION LINE (N&M, 2020)

B-2 TD=60.0 LARGE DIAMETER BORING

A A' CROSS SECTION

N

FEET

0 60 120

REFERENCE: OCPW GIS.

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

2_211532001_SG.dwg 01/12/2021 GK

FIGURE 2

Ninyo & Moore

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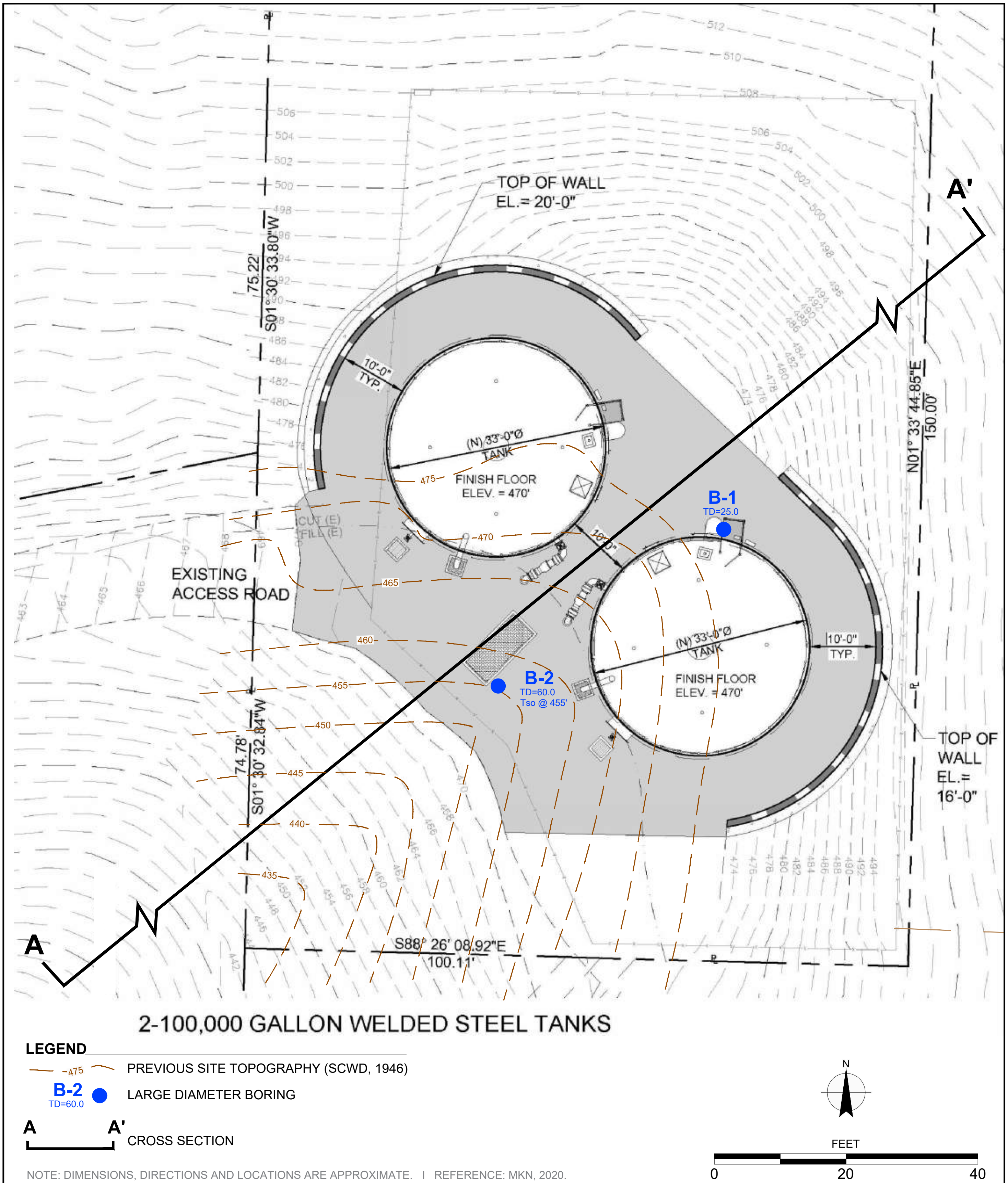
## SITE GEOLOGY

RESERVOIR 2B REPLACEMENT PROJECT
LAGUNA BEACH, CALIFORNIA

211532002 | 1/22




NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: MKN, 2020.

3_211532002_SP2.dwg 01/13/2022 GK

**FIGURE 3**

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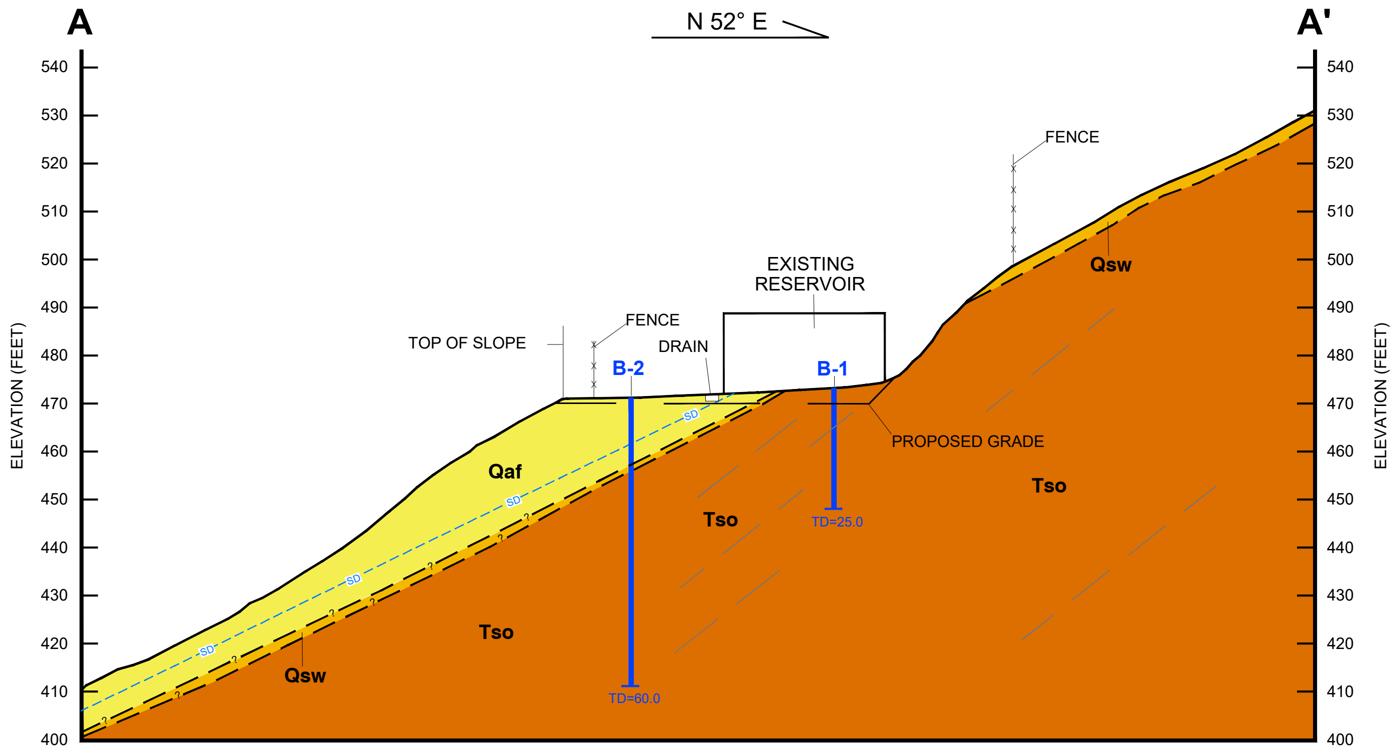
**PROPOSED SITE IMPROVEMENTS**

RESERVOIR 2B REPLACEMENT PROJECT
LAGUNA BEACH, CALIFORNIA

211532002 I 1/22




**LEGEND**

Qaf ARTIFICIAL FILL

Tso SAN ONOFRE BRECCIA

Qsw VERY YOUNG SLOPEWASH DEPOSITS

GEOLOGIC CONTACT; QUERIED WHERE INFERRED

APPARENT BEDDING

B-2 TD=60.0 LARGE DIAMETER BORING; TD=TOTAL DEPTH IN FEET

FEET 0 40 80

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

4_211532002_CS.dwg 01/12/2022 GK

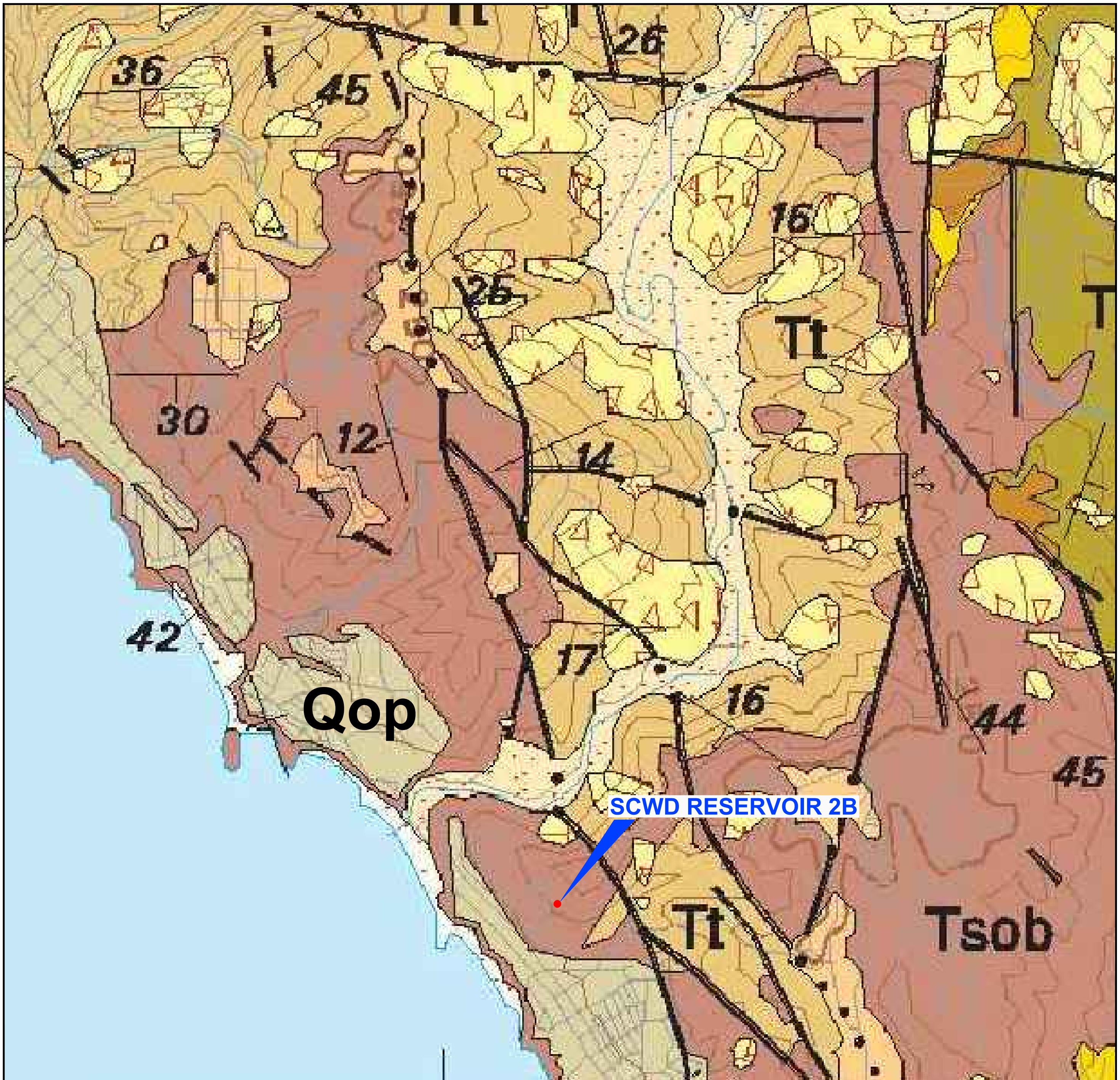
**FIGURE 4**

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

# CROSS SECTION A-A'

RESERVOIR 2B REPLACEMENT PROJECT
LAGUNA BEACH, CALIFORNIA

211532002 I 1/22




LEGEND

| | | | |
|---|---|---|---|
| Qop | OLD PARALIC DEPOSITS | | GEOLOGIC CONTACT |
| Tsob | SAN ONOFRE BRECCIA | | FAULT |
| Tt | TORREY SANDSTONE | | |

N

FEET

0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: MORTON AND MILLER, 2006.

5_211532002_RG.dwg 01/11/2022 JDP, GK

FIGURE 5

Ninyo & Moore

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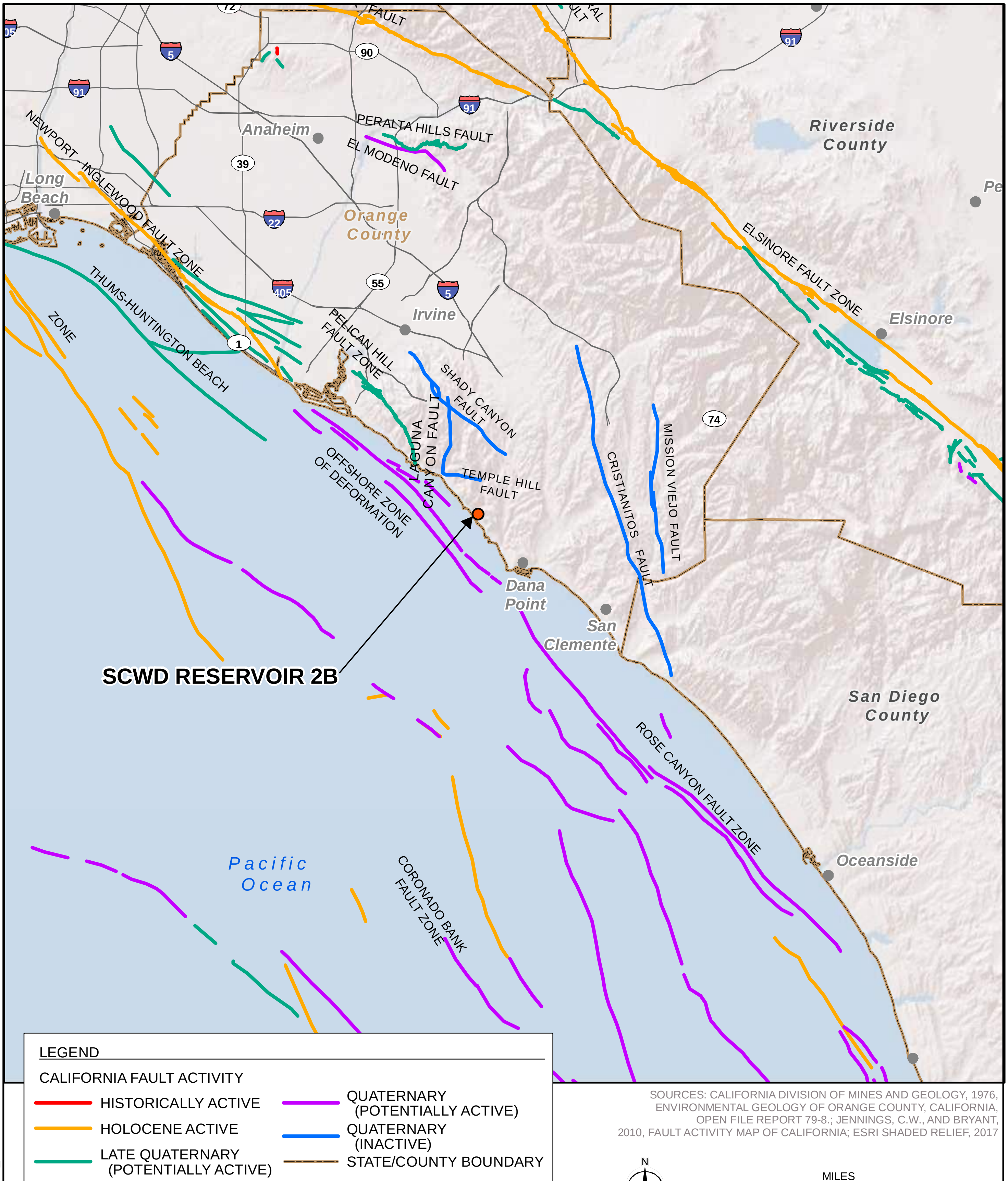
## REGIONAL GEOLOGY

RESERVOIR 2B REPLACEMENT PROJECT
LAGUNA BEACH, CALIFORNIA

211532002 I 1/22

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N

MILES

0 8 16

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

211532002_FL.mxd 1/10/2022 GK

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FIGURE 6

**FAULT LOCATIONS**

RESERVOIR 2B REPLACEMENT PROJECT
LAGUNA BEACH, CALIFORNIA

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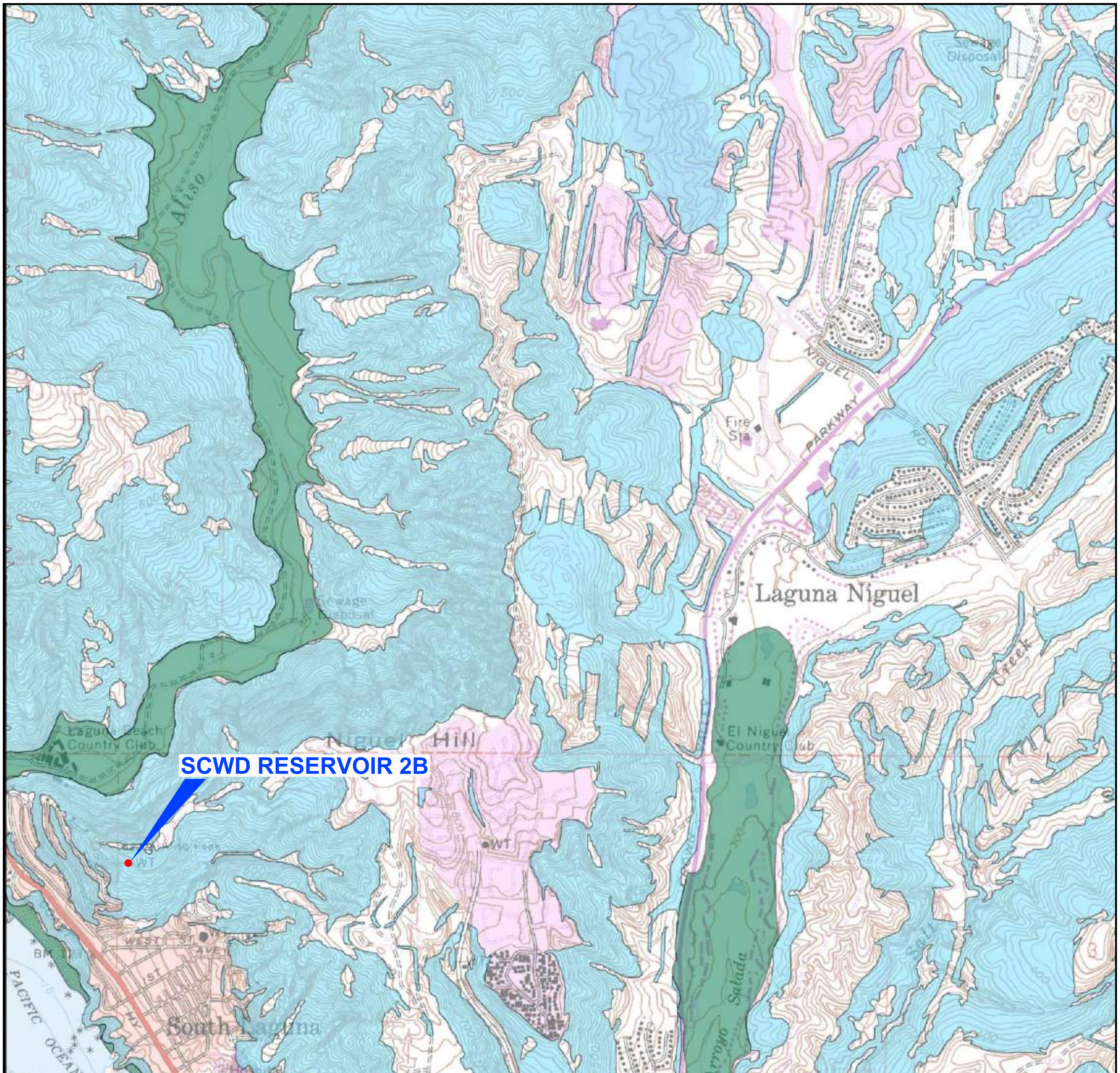


LEGEND

**EARTHQUAKE-INDUCED LANDSLIDES**
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

**LIQUEFACTION**
Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: CGS, 2001.

N

FEET
0 1,000 2,000

7_211532002_SHZ.dwg 01/11/2022 JDP, GK

**FIGURE 7**

## SEISMIC HAZARD ZONES

RESERVOIR 2B REPLACEMENT PROJECT
LAGUNA BEACH, CALIFORNIA

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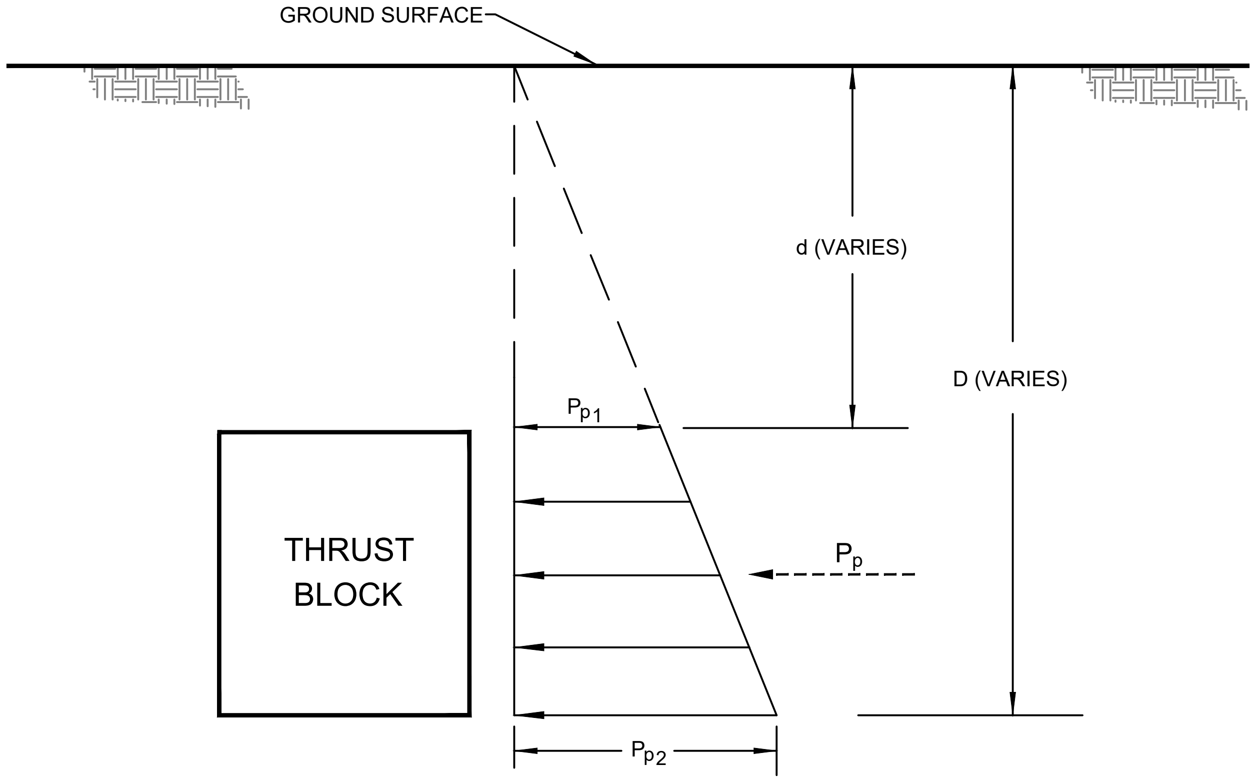


NOTES:

1. ASSUMES GROUNDWATER BELOW BLOCK
   $P_p = 200\,(D^2 - d^2)$ lb/ft
2. ASSUMES BACKFILL IS GRANULAR MATERIAL
3. ASSUMES THRUST BLOCK IS ADJACENT TO COMPETENT MATERIAL
4. D AND d ARE IN FEET

NOT TO SCALE

8_211532002_LEP-TB.dwg 01/13/2021 GK

**FIGURE 8**

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# THRUST BLOCK LATERAL EARTH PRESSURE DIAGRAM

RESERVOIR 2B REPLACEMENT PROJECT
LAGUNA BEACH, CALIFORNIA

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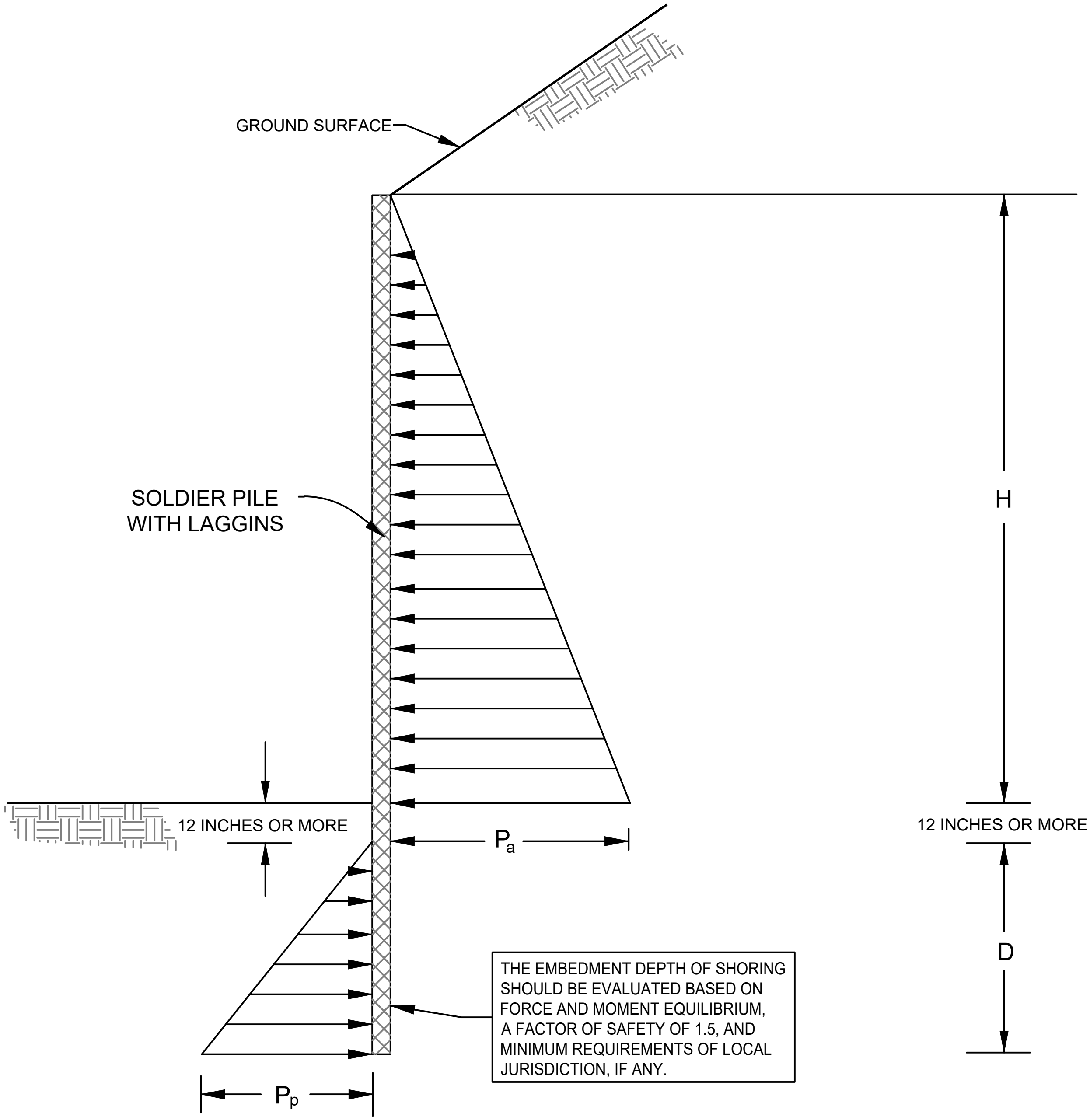


NOTES:

1. ACTIVE LATERAL EARTH PRESSURE, $P_a$
   $P_a$ = 36H psf
2. PASSIVE LATERAL EARTH PRESSURE, $P_p$
   $P_p$ = 550D psf
3. ASSUMES GROUNDWATER IS NOT PRESENT
4. H AND D ARE IN FEET

NOT TO SCALE

9_211532002_LEP-CS1.dwg 01/04/2021 GK

FIGURE 9

# LATERAL EARTH PRESSURES FOR SOLDIER PILE WALL

RESERVOIR 2B REPLACEMENT PROJECT
LAGUNA BEACH, CALIFORNIA

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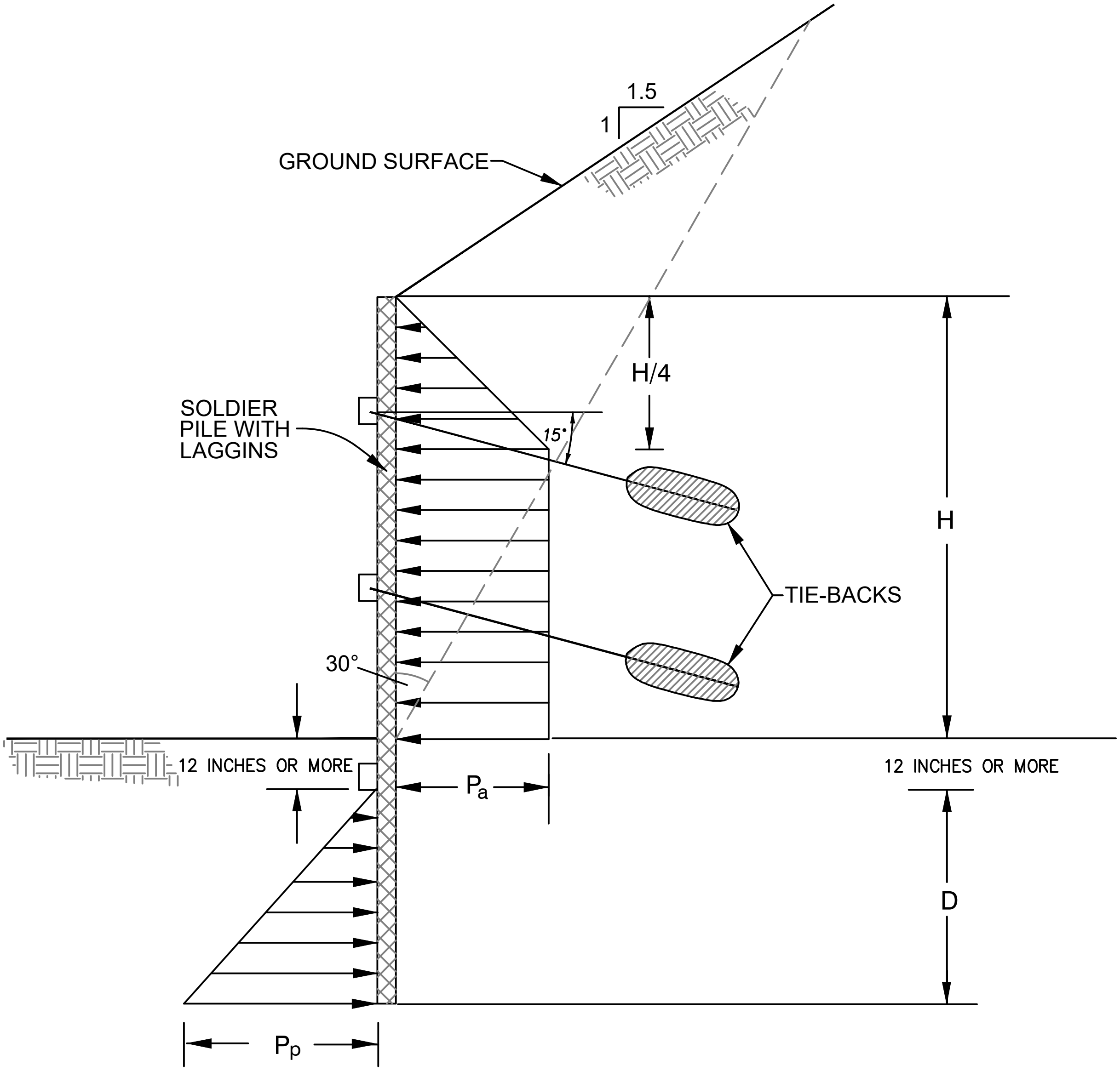


NOTES:

1. APPARENT LATERAL EARTH PRESSURE, $P_a$
$P_a$ = 24 H Psf

2. PASSIVE LATERAL EARTH PRESSURE, $P_p$
$P_p$ = 550 D Psf

2. ASSUMES GROUNDWATER IS NOT PRESENT

4. H AND D ARE IN FEET

NOT TO SCALE

10_211532002_LEP-TBSW.dwg 01/26/2022 GK

FIGURE 10

**LATERAL EARTH PRESSURES FOR SOLDIER PILE WALL WITH TIE-BACK**

RESERVOIR 2B REPLACEMENT PROJECT
LAGUNA BEACH, CALIFORNIA

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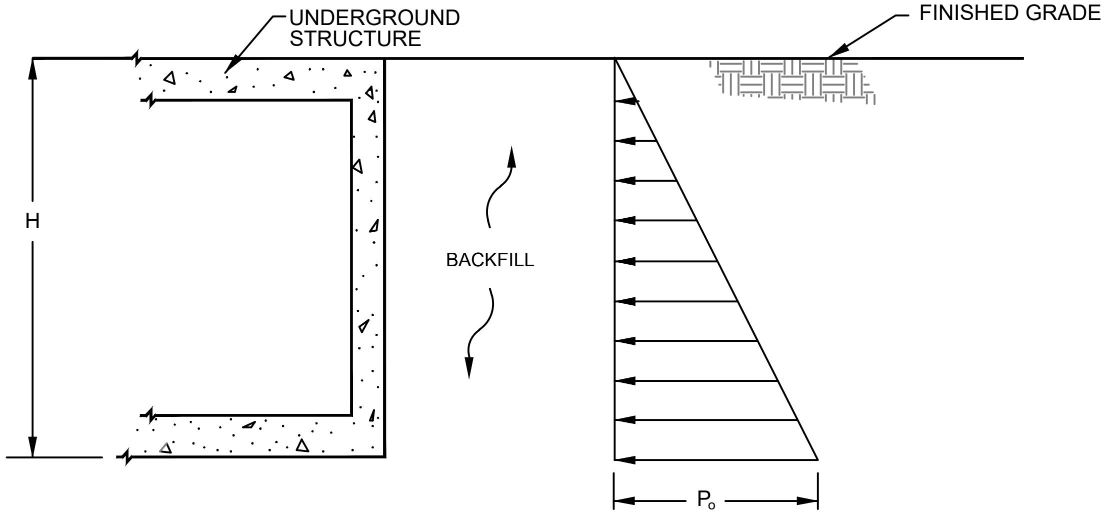
Geotechnical & Environmental Sciences Consultants

211532002 I 1/22




AT-REST LATERAL EARTH

NOTES:

1. APPARENT LATERAL EARTH PRESSURE
   $P_o$ = 56H psf
2. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
3. H IS IN FEET

NOT TO SCALE

11_211532002_LEP-US.dwg 01/26/2021 GK

FIGURE 11

# LATERAL EARTH PRESSURES FOR UNDERGROUND STRUCTURES

RESERVOIR 2B REPLACEMENT PROJECT
LAGUNA BEACH, CALIFORNIA

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211532002 I 1/22

# APPENDIX A

Boring Logs

# APPENDIX A

## BORING LOGS

**<u>Field Procedure for the Collection of Disturbed Samples</u>**
Disturbed soil samples were obtained in the field using the following methods.

**<u>Bulk Samples</u>**
Bulk samples of representative earth materials were obtained from the exploratory drilling. The samples were bagged and transported to the laboratory for testing.

**<u>Field Procedure for the Collection of Relatively Undisturbed Samples</u>**
Relatively undisturbed soil samples were obtained in the field using the following method.

**<u>The Modified Split-Barrel Drive Sampler</u>**
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.

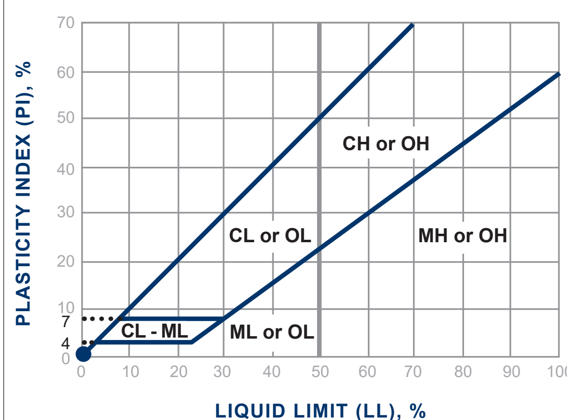
## Soil Classification Chart Per ASTM D 2488

| Primary Divisions | | | Secondary Divisions: Group Symbol | Secondary Divisions: Group Name |
|---|---|---|---|---|
| COARSE-GRAINED SOILS more than 50% retained on No. 200 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: SPT (blows/foot) | Spooling Cable or Cathead: Modified Split Barrel (blows/foot) | Automatic Trip Hammer: SPT (blows/foot) | Automatic Trip Hammer: 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

| Consistency | Spooling Cable or Cathead: SPT (blows/foot) | Spooling Cable or Cathead: Modified Split Barrel (blows/foot) | Automatic Trip Hammer: SPT (blows/foot) | Automatic Trip Hammer: 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 |

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**USCS METHOD OF SOIL CLASSIFICATION**

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | BORING LOG EXPLANATION SHEET |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | Bulk sample. |
| | | | | | | | | Modified split-barrel drive sampler. |
| | | | | | | | | No recovery with modified split-barrel drive sampler. |
| | | | | | | | | Sample retained by others. |
| | | | | | | | | Standard Penetration Test (SPT). |
| 5 | | | | | | | | No recovery with a SPT. |
| | | | XX/XX | | | | | Shelby tube sample. Distance pushed in inches/length of sample recovered in inches. |
| | | | | | | | | No recovery with Shelby tube sampler. |
| | | | | | | | | Continuous Push Sample. |
| | | | | | | | | Seepage. |
| 10 | | | | | | | | Groundwater encountered during drilling. |
| | | | | | | | | Groundwater measured after drilling. |
| | | | | | | | SM | MAJOR MATERIAL TYPE (SOIL):<br>Solid line denotes unit change. |
| | | | | | | | CL | Dashed line denotes material change.<br><br>Attitudes: Strike/Dip<br>b: Bedding<br>c: Contact<br>j: Joint<br>f: Fracture<br>F: Fault<br>cs: Clay Seam<br>s: Shear<br>bss: Basal Slide Surface<br>sf: Shear Fracture<br>sz: Shear Zone<br>sbs: Shear Bedding Surface |
| 15 | | | | | | | | |
| | | | | | | | | The total depth line is a solid line that is drawn at the bottom of the boring. |
| 20 | | | | | | | | |

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**BORING LOG**

| DATE DRILLED | 11/9/21 | BORING NO. | B-1 |
|---|---|---|---|
| GROUND ELEVATION | 473' ± (MSL) | SHEET | 1 OF 1 |
| METHOD OF DRILLING | 24" Bucket Auger (Roy Bros. Drilling) | | |
| DRIVE WEIGHT | See Notes | DROP | 30" |
| SAMPLED BY | GM | LOGGED BY | GM/MRH |
| REVIEWED BY | MRH/MLP | | |

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | SAN ONOFRE BRECCIA: Bluish gray, moist, moderately hard, BRECCIA with cobble and boulder-sized clasts of igneous and metamorphic rocks; massive, no discernible bedding; significant fractures were not observed. |
| | | | 6/6" | | | | | @ 8': Imbricated boulder. Approximate bedding orientation of N53°W, 40°S, based on flat elongated schist boulder with foliations; massive below. |
| 10 | | | | | | | | |
| 20 | | | | | | | | Continues massive, no discernible bedding; difficult drilling in hard breccia. |
| | | | | | | | | Increase in drilling difficulty and rock hardness. Drilling refusal at 25 feet. |
| | | | | | | | | Total Depth = 25.0 feet (Refusal). Groundwater was not encountered during drilling. Downhole logged on 11/9/21 to 25 feet. Backfilled with on-site soil on 11/9/21. |
| 30 | | | | | | | | Notes: 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. |
| 40 | | | | | | | | |

| | | | | |
|---|---|---|---|---|
| DATE DRILLED | 11/8/21 | BORING NO. | B-2 | |
| GROUND ELEVATION | 471' ± (MSL) | SHEET | 1 | OF 2 |
| METHOD OF DRILLING | 24" Bucket Auger (Roy Bros. Drilling) | | | |
| DRIVE WEIGHT | See Notes | DROP | 30" | |
| SAMPLED BY | GM | LOGGED BY | GM/MRH | REVIEWED BY MRH/MLP |

| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|
| 0 | | | | | | SM | FILL: Grayish to yellowish brown, moist, medium dense, silty SAND with gravel; few clay pockets; few cobbles; trace to few boulders. |
| | ■ | | | | | | |
| | | ■ | 3 | 5.9 | 122.3 | | |
| | ■ | | | | | | |
| | | | | | | | Medium dense to dense. |
| 10 | | ■ | 3 | 8.6 | 114.6 | | |
| | ■ | | | | | | From 14 to 16 feet, fill transitions into San Onofre Breccia. Rootlets and gravel; slight increase in clay. |
| | | | | | | SC | SLOPE WASH: Yellowish brown to red brown, moist, medium dense, clayey SAND with gravel and cobbles; trace rootlets. |
| | | ■ | 5 | 7.5 | 120.0 | | |
| | | | | | | | SAN ONOFRE BRECCIA: Yellowish brown to grayish brown, moist, moderately hard, BRECCIA; with cobble and boulder-sized clasts of metamorphic and igneous rocks; massive, no discernible bedding; no significant fracturing observed. |
| 20 | | | | | | | Yellowish brown. |
| | | ■ | 9/4" | | | | |
| 30 | | | | | | | Continues massive, no discernible bedding; difficult drilling in hard breccia. |
| | | ■ | 11 | | | | |
| 40 | | | | | | | |



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| DEPTH (feet) | SAMPLES Bulk | SAMPLES Driven | BLOWS/FOOT | MOISTURE (%) | DRY DENSITY (PCF) | SYMBOL | CLASSIFICATION U.S.C.S. | DESCRIPTION/INTERPRETATION |
|---|---|---|---|---|---|---|---|---|
| 40 | | | | | | | | SAN ONOFRE BRECCIA: (Continued) Bluish gray, moist, hard, BRECCIA with cobble and boulder-sized clasts of metamorphic and igneous rocks; oxidized reddish brown staining; massive; no significant fractures observed. |
| | | | | | | | | Sampler refusal; difficult drilling. |
| | | | | | | | | Increase in gravel and cobble content. |
| 50 | | | | | | | | |
| 60 | | | | | | | | Total Depth = 60.0 feet. Groundwater was not encountered during drilling. Downhole logged to 60 feet. Backfilled with on-site soil on 11/8/21. |
| | | | | | | | | Notes: 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. |
| 70 | | | | | | | | |
| 80 | | | | | | | | |

DATE DRILLED 11/8/21 BORING NO. B-2

GROUND ELEVATION 471' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 24" Bucket Auger (Roy Bros. Drilling)

DRIVE WEIGHT See Notes DROP 30"

SAMPLED BY GM LOGGED BY GM/MRH REVIEWED BY MRH/MLP

# APPENDIX B

Laboratory Testing

# APPENDIX B

## 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 borings 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 borings in Appendix A.

**Gradation Analysis**
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 6913. The grain-size distribution curves are shown on Figure B-1. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

**Percent Finer than No. 200 Sieve**
An evaluation of the percentage of particles finer than the No. 200 sieve in selected soil samples was performed in general accordance with ASTM D 1140. The results of the tests are presented on Figure B-2.

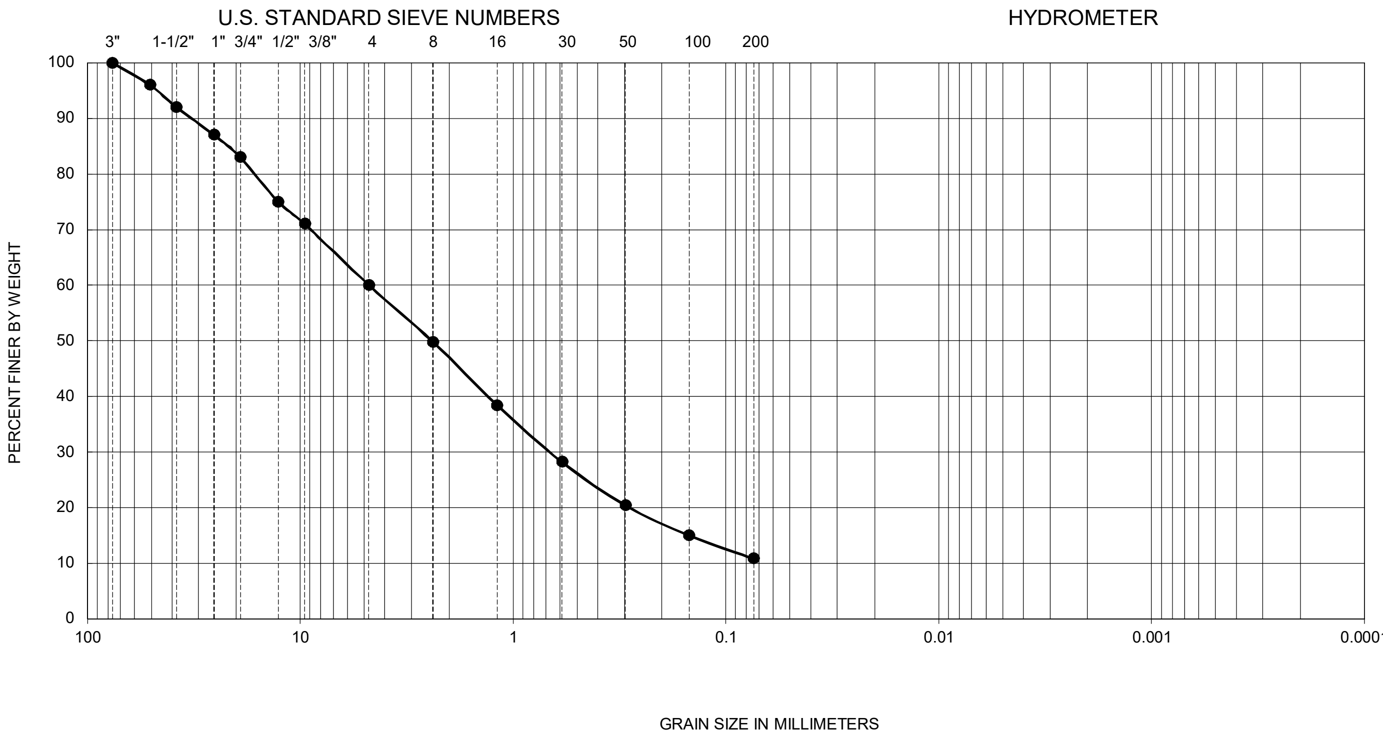
**Proctor Density Tests**
The maximum dry density and optimum moisture content of selected representative soil samples were evaluated using the Modified Proctor method in general accordance with ASTM D 1557. The results of these tests are summarized on Figure B-3.

**Direct Shear Test**
Direct shear tests were performed on selected relatively undisturbed and remolded samples in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected materials. The samples were inundated during shearing to represent adverse field conditions. The results are shown on Figures B-4 and B-5.

**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 B-6.

**Soil Corrosivity Tests**
Soil pH and resistivity tests were performed on a representative sample in general accordance with California Test (CT) 643. The soluble sulfate and chloride content of the selected sample were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-7.

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




| Symbol | Hole No. | Depth (ft) | Liquid Limit | Plastic Limit | Plasticity Index | $D_{10}$ | $D_{30}$ | $D_{60}$ | $C_u$ | $C_c$ | Passing No. 200 (percent) | USCS (Equivalent) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | B-1 | 2.0-3.0 | -- | -- | -- | -- | 0.67 | 4.76 | -- | -- | 11 | SP-SM |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

**FIGURE B-1**

## GRADATION TEST RESULTS

RESERVOIR 2B REPLACEMENT PROJECT
LAGUNA BEACH, CALIFORNIA

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| SAMPLE LOCATION | SAMPLE DEPTH (ft) | DESCRIPTION | PERCENT PASSING NO. 4 | PERCENT PASSING NO. 200 | USCS (TOTAL SAMPLE) |
|---|---|---|---|---|---|
| B-2 | 1.0-3.0 | SILTY SAND WITH GRAVEL | 82 | 21 | SM |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

FIGURE B-2

## NO. 200 SIEVE ANALYSIS TEST RESULTS

RESERVOIR 2B REPLACEMENT PROJECT
LAGUNA BEACH, CALIFORNIA

211532002 | 1/22

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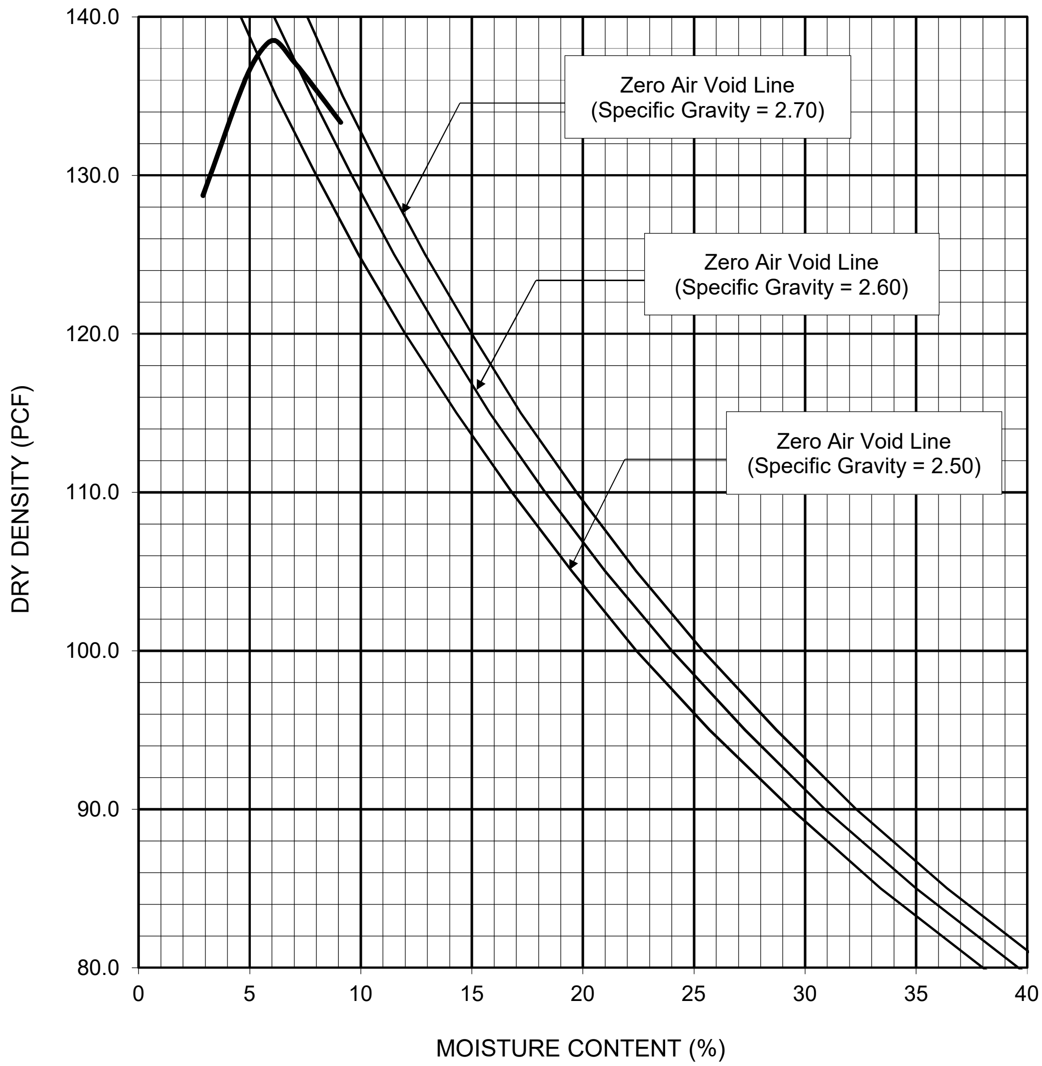


| Sample Location | Depth (ft) | Soil Description | Maximum Dry Density (pcf) | Optimum Moisture Content (percent) |
|---|---|---|---|---|
| B-2 | 1.0-3.0 | Yellowish Brown Silty SAND w/ Gravel | 138.5 | 6.0 |
| Dry Density and Moisture Content Values Corrected for Oversize (ASTM D 4718) | | | 140.5 | 5.5 |

PERFORMED IN GENERAL ACCORDANCE WITH ☑ ASTM D 1557 ☐ ASTM D 698 METHOD ☐ A ☐ B ☑ C

**FIGURE B-3**

**PROCTOR DENSITY TEST RESULTS**

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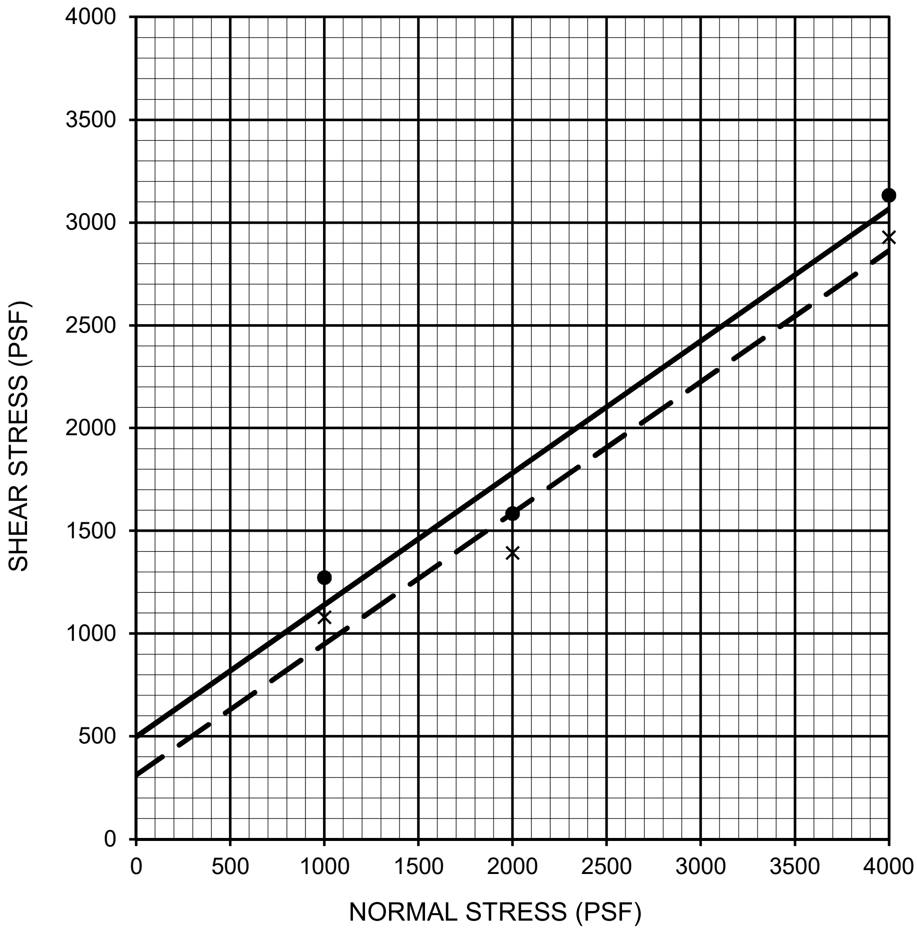
RESERVOIR 2B REPLACEMENT PROJECT
LAGUNA BEACH, CALIFORNIA

211532002 | 1/22




| Description | Symbol | Sample Location | Depth (ft) | Shear Strength | Cohesion (psf) | Friction Angle (degrees) | Soil Type |
|---|---|---|---|---|---|---|---|
| SILTY SAND W/ GRAVEL | ——●—— | B-2 | 1.0-3.0 | Peak | 498 | 33 | SM |
| SILTY SAND W/ GRAVEL | – – X – – | B-2 | 1.0-3.0 | Ultimate | 312 | 33 | SM |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080 ON REMOLDED (90% RELATIVE COMPACTION) SOIL SAMPLES

**FIGURE B-4**

**DIRECT SHEAR TEST RESULTS**

RESERVOIR 2B REPLACEMENT PROJECT
LAGUNA BEACH, CALIFORNIA

211532002 | 1/22

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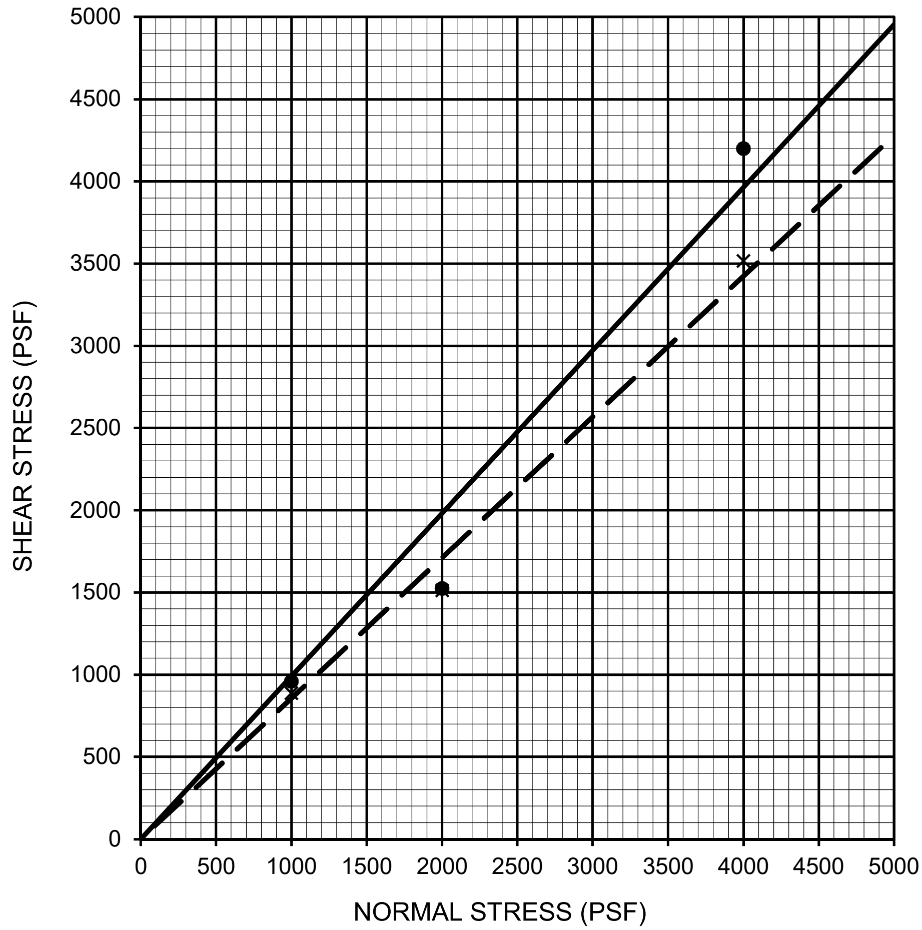
Geotechnical & Environmental Sciences Consultants




| Description | Symbol | Sample Location | Depth (ft) | Shear Strength | Cohesion (psf) | Friction Angle (degrees) | Soil Type |
|---|---|---|---|---|---|---|---|
| SILTY SAND | —●— | B-2 | 10.0-11.5 | Peak | 0 | 48 | SM |
| SILTY SAND | – – X – – | B-2 | 10.0-11.5 | Ultimate | 0 | 42 | SM |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

**FIGURE B-5**

**DIRECT SHEAR TEST RESULTS**

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RESERVOIR 2B REPLACEMENT PROJECT
LAGUNA BEACH, CALIFORNIA

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| SAMPLE LOCATION | SAMPLE DEPTH (ft) | EQUIVALENT SOIL TYPE | R-VALUE |
|---|---|---|---|
| B-1 | 2.0 - 3.0 | SP/SM | 72 |

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

FIGURE B-6

## R-VALUE TEST RESULTS

RESERVOIR 2B REPLACEMENT PROJECT
LAGUNA BEACH, CALIFORNIA

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| SAMPLE LOCATION | SAMPLE DEPTH (ft) | pH [1] | RESISTIVITY [1] (ohm-cm) | SULFATE CONTENT [2] | | CHLORIDE CONTENT [3] (ppm) |
|---|---|---|---|---|---|---|
| | | | | (ppm) | (%) | |
| B-1 | 2.0-3.0 | 7.9 | 12,956 | 10 | 0.001 | 35 |
| B-2 | 1.0-3.0 | 8.4 | 12,956 | 10 | 0.001 | 15 |

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

**FIGURE B-7**

## CORROSIVITY TEST RESULTS

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RESERVOIR 2B REPLACEMENT PROJECT
LAGUNA BEACH, CALIFORNIA

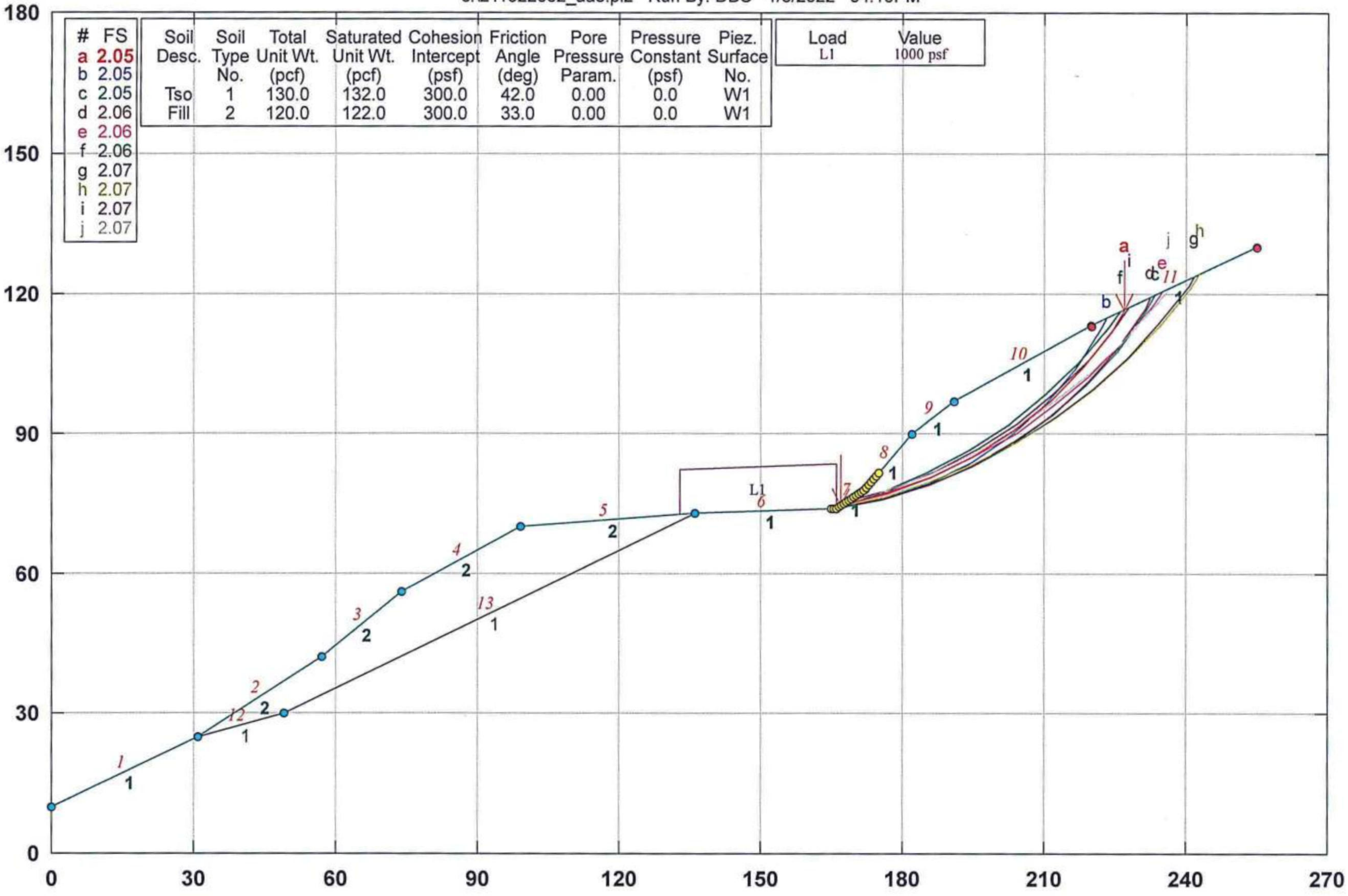
211532002 | 1/22

# APPENDIX C

## Seismic Refraction Profiles

**MKN/SCWD Reservoir 2B, Laguna Beach, CA Cross-section A-A'**

c:\211522002_aa3.pl2 Run By: DBC 1/6/2022 04:10PM

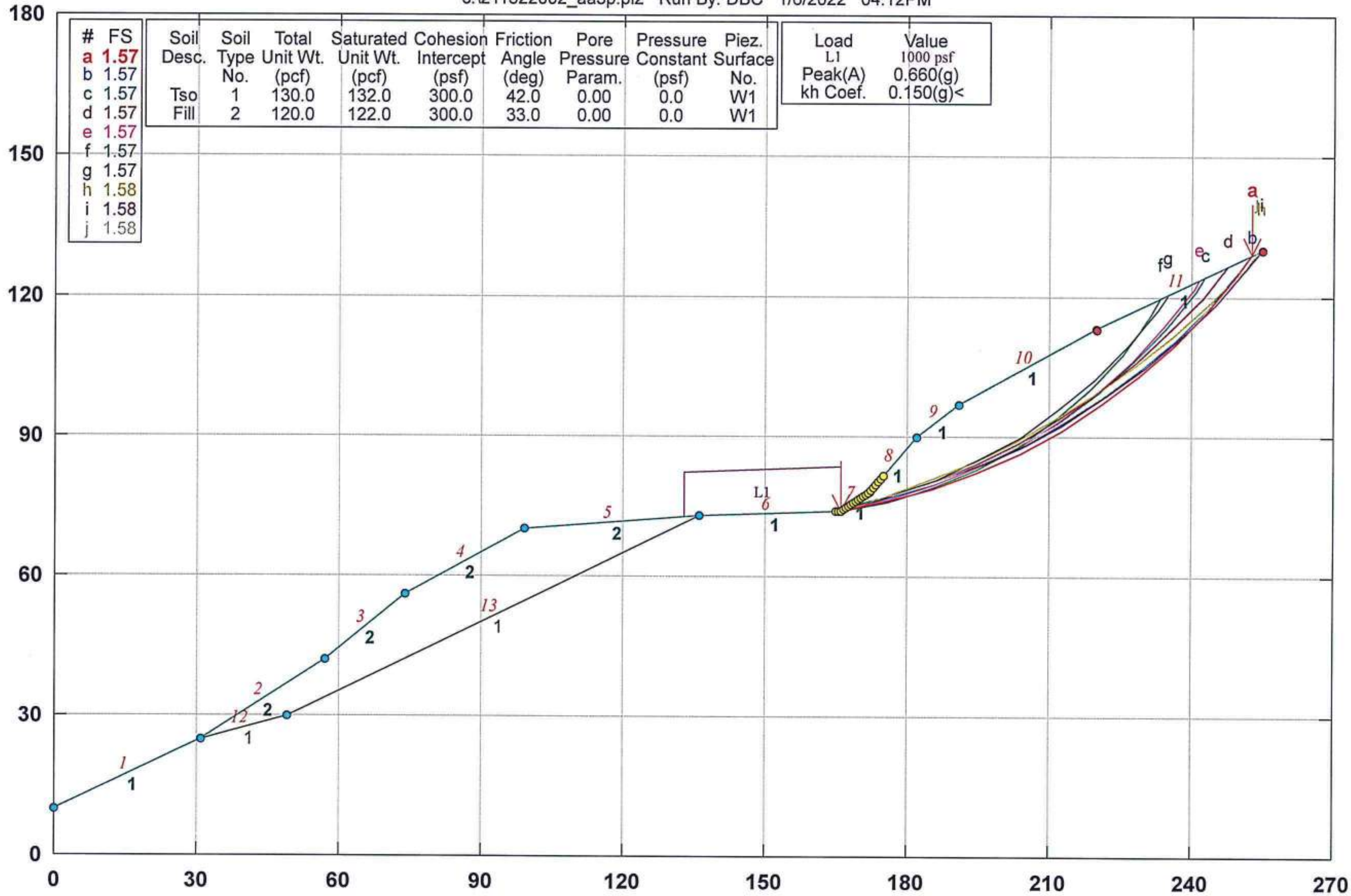



GSTABL7 v.2 FSmin=2.05

**Safety Factors Are Calculated By The Modified Bishop Method**

## MKN/SCWD Reservoir 2B, Laguna Beach, CA Cross-section A-A', pseudo-static Analys

c:\211522002_aa3p.pl2 Run By: DBC 1/6/2022 04:12PM

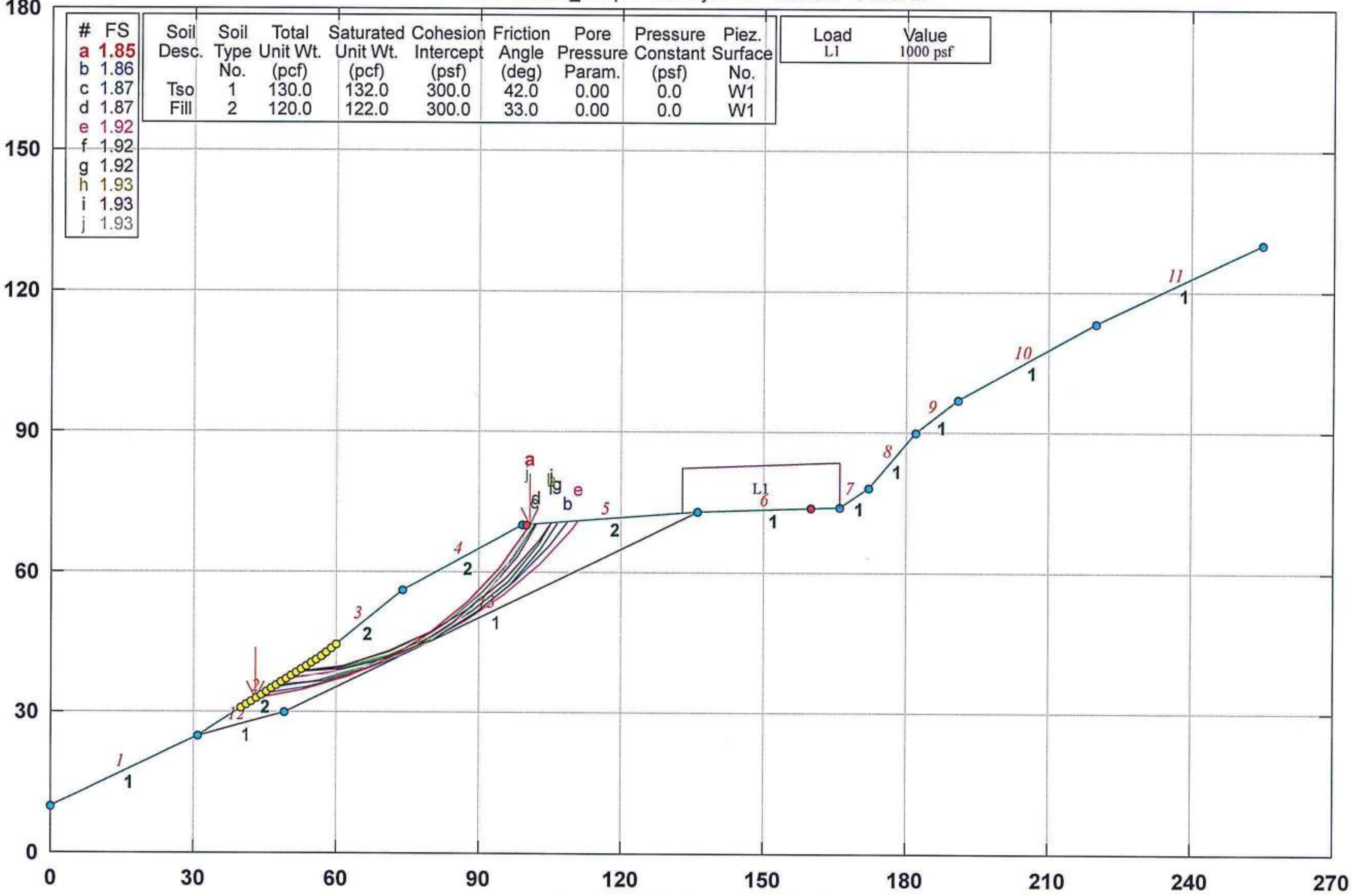



GSTABL7 v.2 FSmin=1.57

Safety Factors Are Calculated By The Modified Bishop Method

## MKN/SCWD Reservoir 2B, Laguna Beach, CA Cross-section A-A'

c:\211522002_aa2.pl2 Run By: DBC 1/6/2022 04:13PM

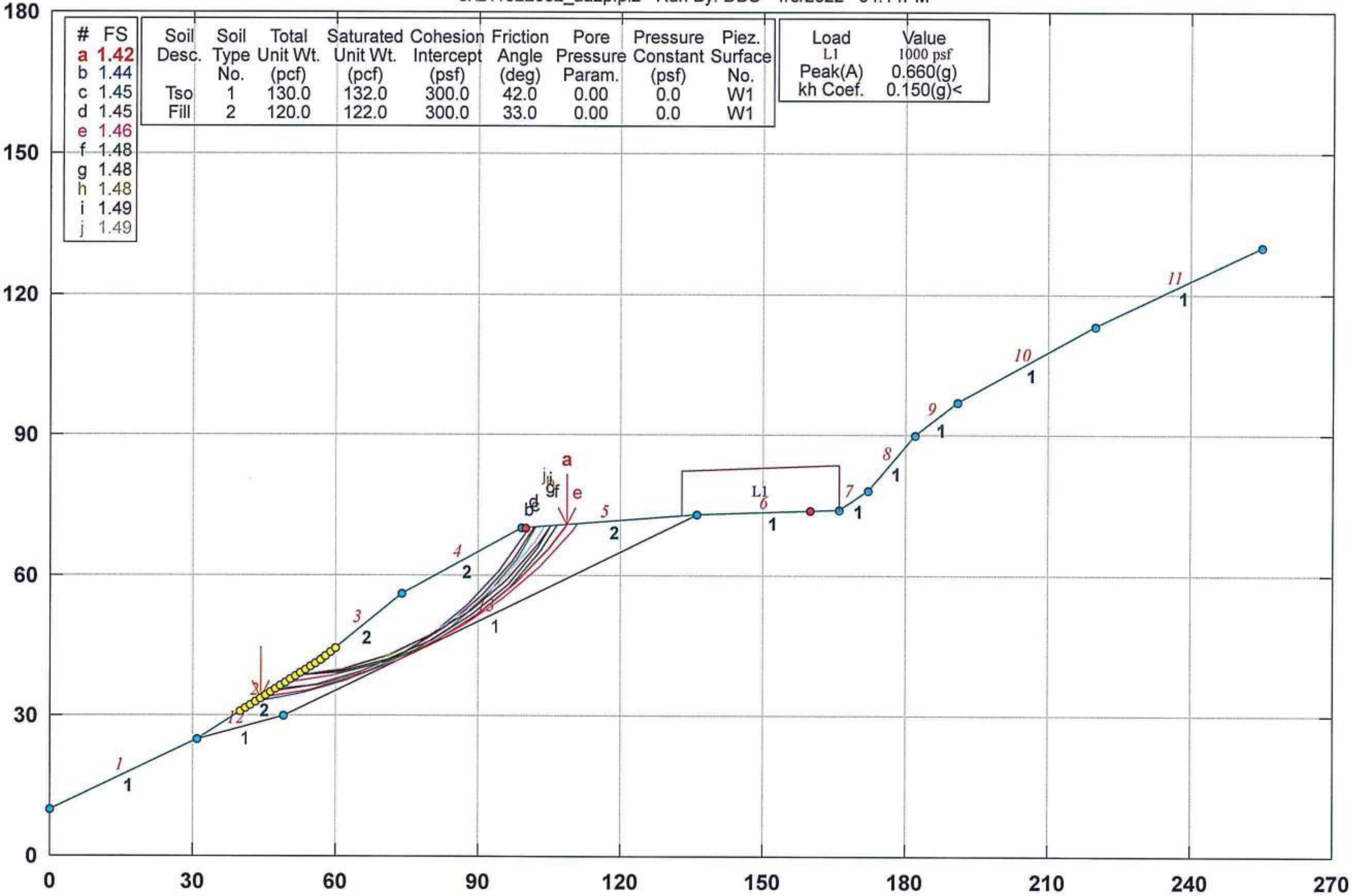



GSTABL7 v.2 FSmin=1.85

**Safety Factors Are Calculated By The Modified Bishop Method**

## MKN/SCWD Reservoir 2B, Laguna Beach, CA Cross-section A-A', pseudo-static analys

c:\211522002_aa2p.pl2 Run By: DBC 1/6/2022 04:14PM




| # | FS |
|---|---|
| a | 1.42 |
| b | 1.44 |
| c | 1.45 |
| d | 1.45 |
| e | 1.46 |
| f | 1.48 |
| g | 1.48 |
| h | 1.48 |
| i | 1.49 |
| j | 1.49 |

| Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Param. | Pressure Constant (psf) | Piez. Surface No. |
|---|---|---|---|---|---|---|---|---|
| Tso | 1 | 130.0 | 132.0 | 300.0 | 42.0 | 0.00 | 0.0 | W1 |
| Fill | 2 | 120.0 | 122.0 | 300.0 | 33.0 | 0.00 | 0.0 | W1 |

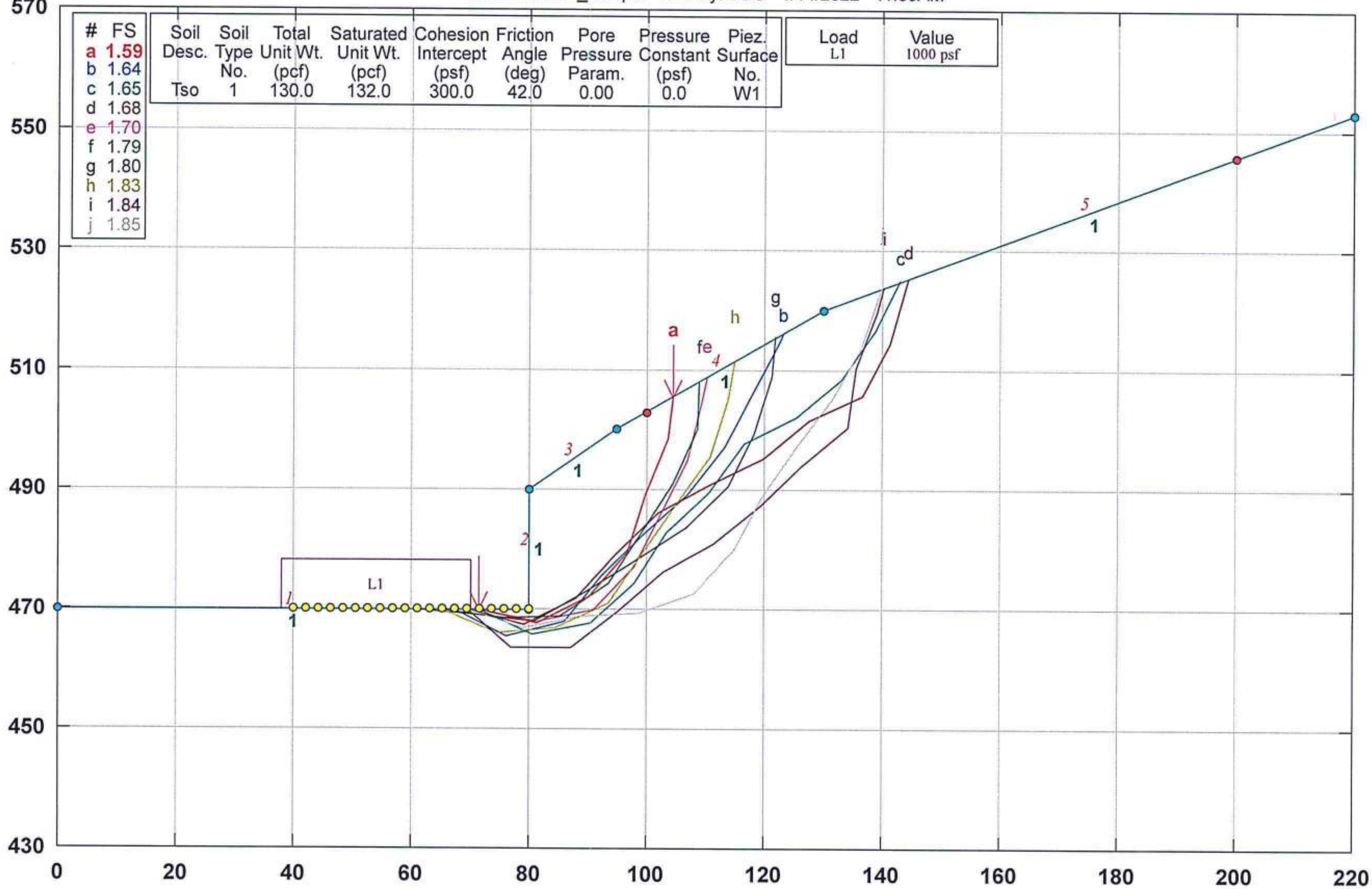
| Load | Value |
|---|---|
| L1 | 1000 psf |
| Peak(A) | 0.660(g) |
| kh Coef. | 0.150(g)< |

GSTABL7 v.2 FSmin=1.42

**Safety Factors Are Calculated By The Modified Bishop Method**

## MKN/SCWD Reservoir 2B, Laguna Beach, CA Slope stability with Soldier Pile Wall

c:\211532002_bb1.pl2 Run By: DBC 1/14/2022 11:59AM

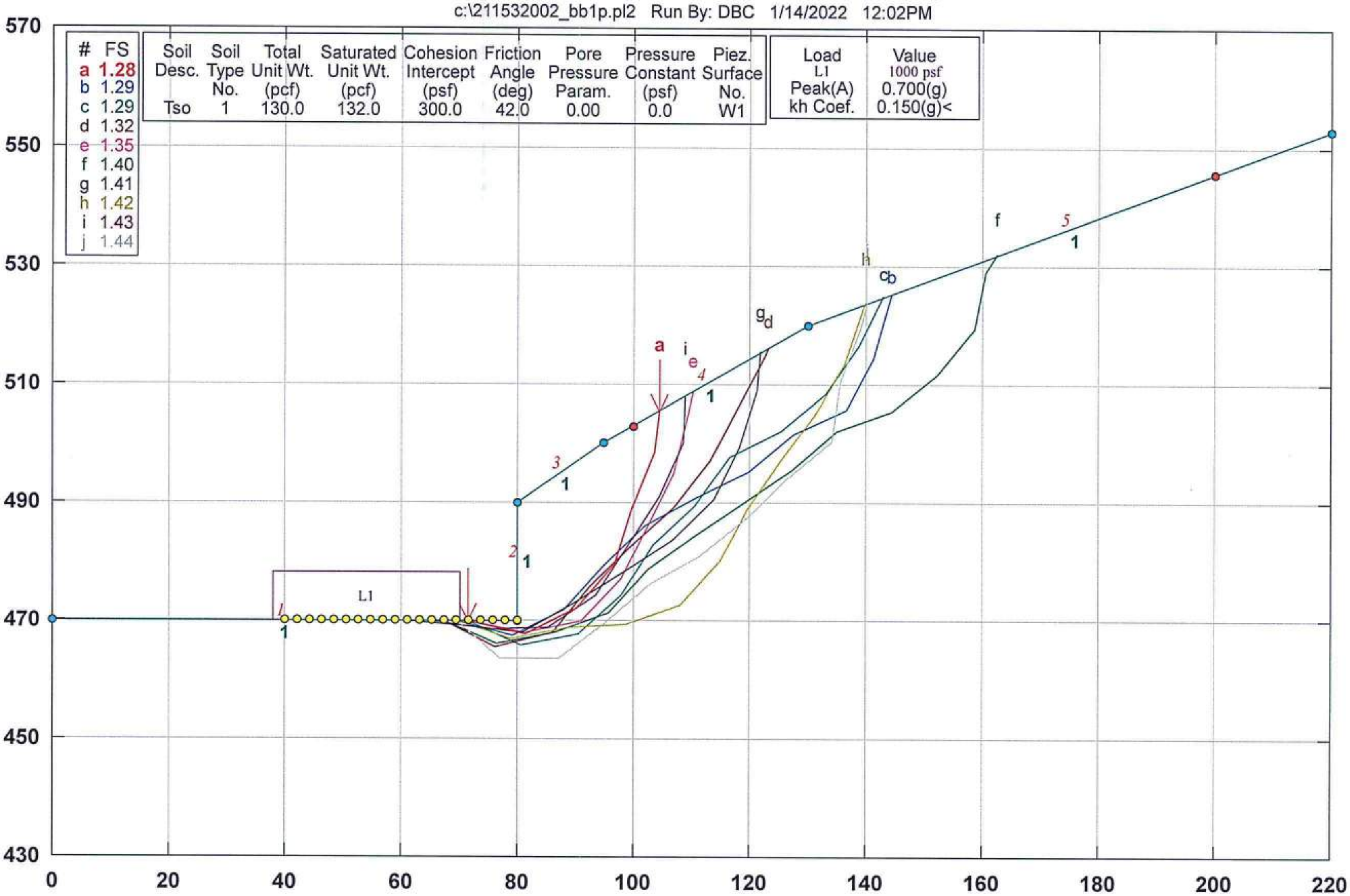



GSTABL7 v.2 FSmin=1.59

Safety Factors Are Calculated By The Simplified Janbu Method

# MKN/SCWD Reservoir 2B, Laguna Beach, CA Slope stability with Soldier Pile Wall

c:\211532002_bb1p.pl2 Run By: DBC 1/14/2022 12:02PM

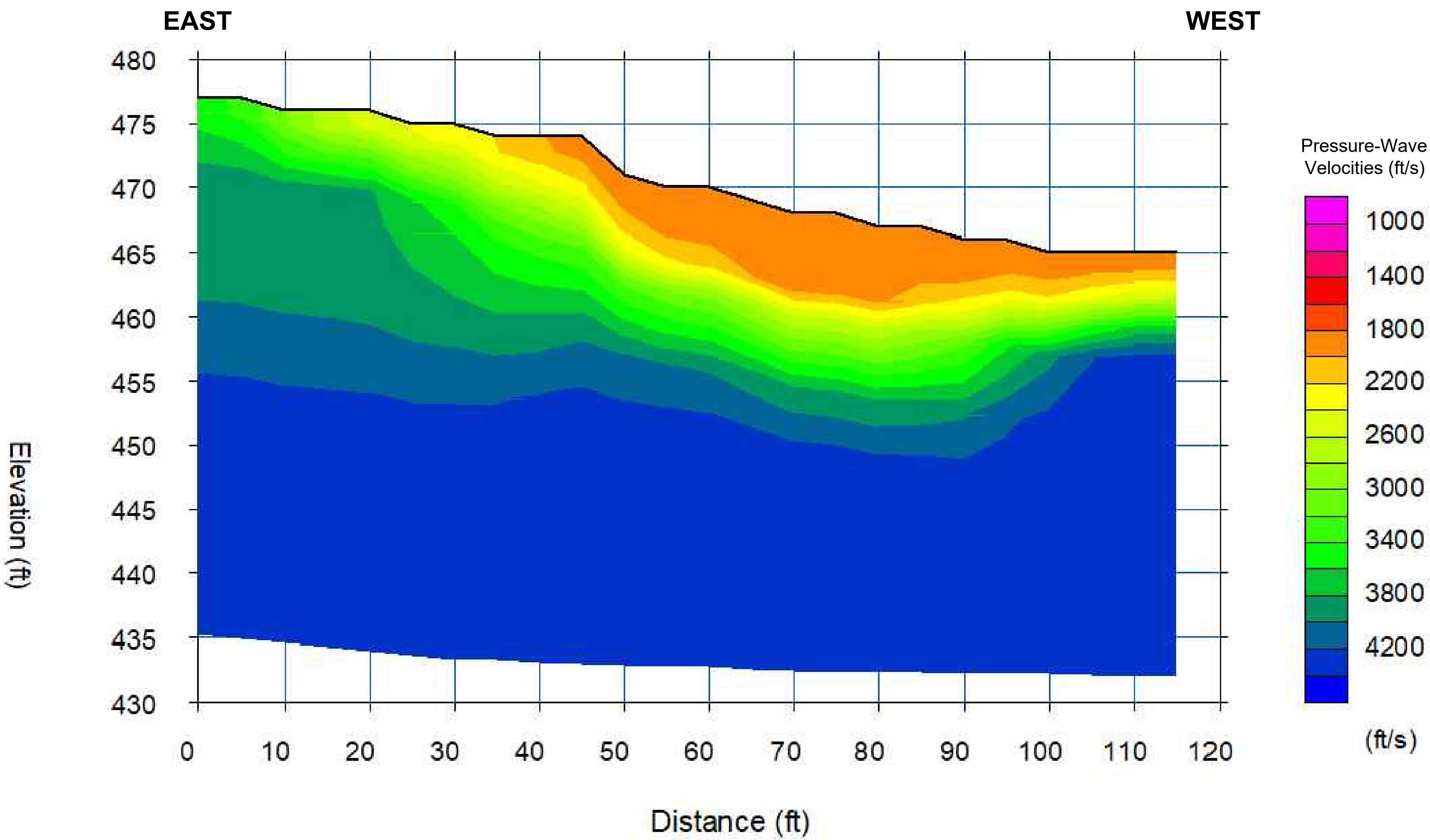



GSTABL7 v.2 FSmin=1.28

Safety Factors Are Calculated By The Simplified Janbu Method

# APPENDIX D

## Slope Stability Analyses




211532001_SRL1.dwg 11/13/2020 JDP

FIGURE 7

## SEISMIC REFRACTION LINE 1

RESERVOIRS 2B AND 3B REPLACEMENT PROJECT
LAGUNA BEACH, CALIFORNIA

211532001 I 11/20

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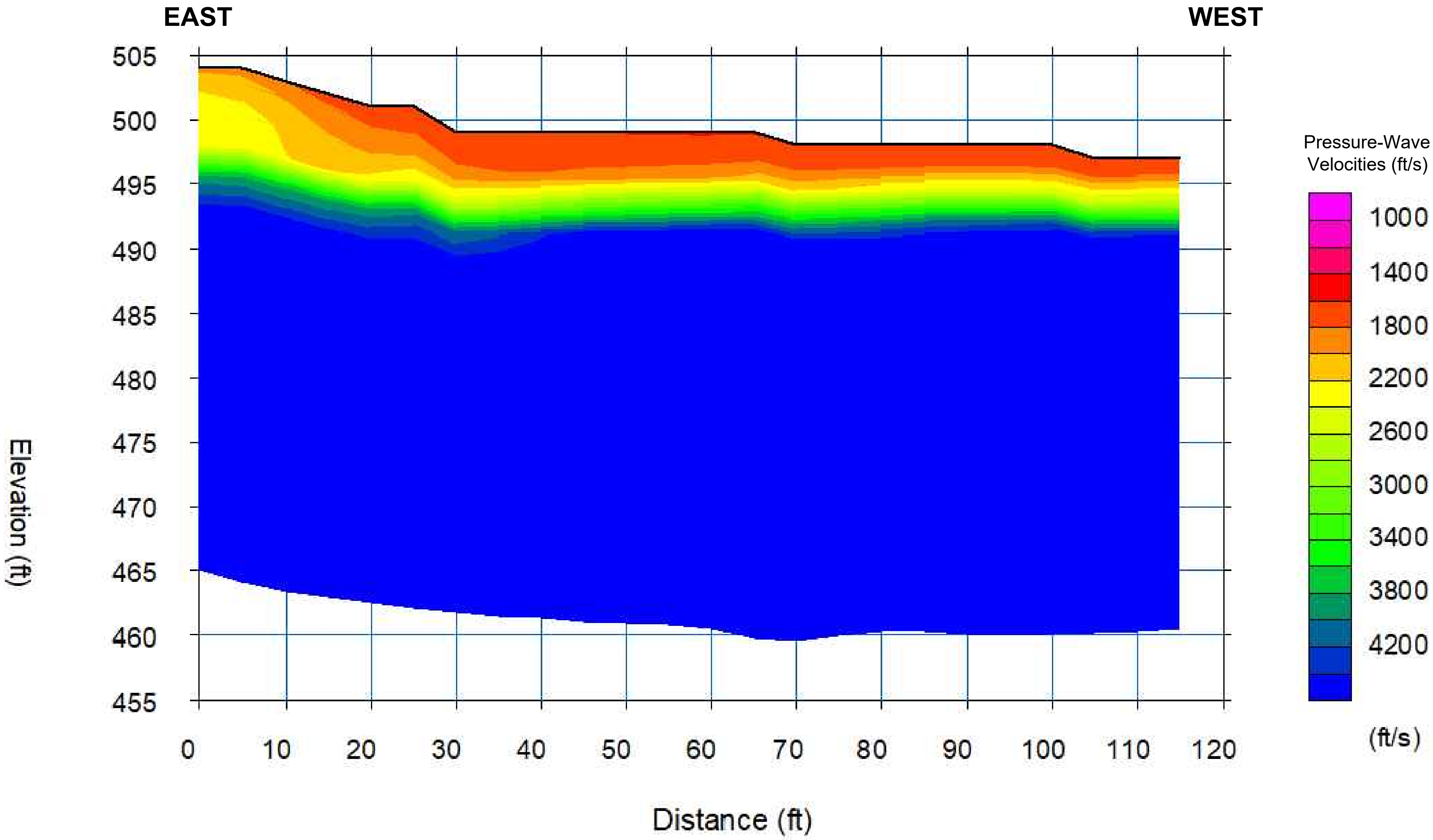




FIGURE 8

## SEISMIC REFRACTION LINE 2

RESERVOIRS 2B AND 3B REPLACEMENT PROJECT
LAGUNA BEACH, CALIFORNIA



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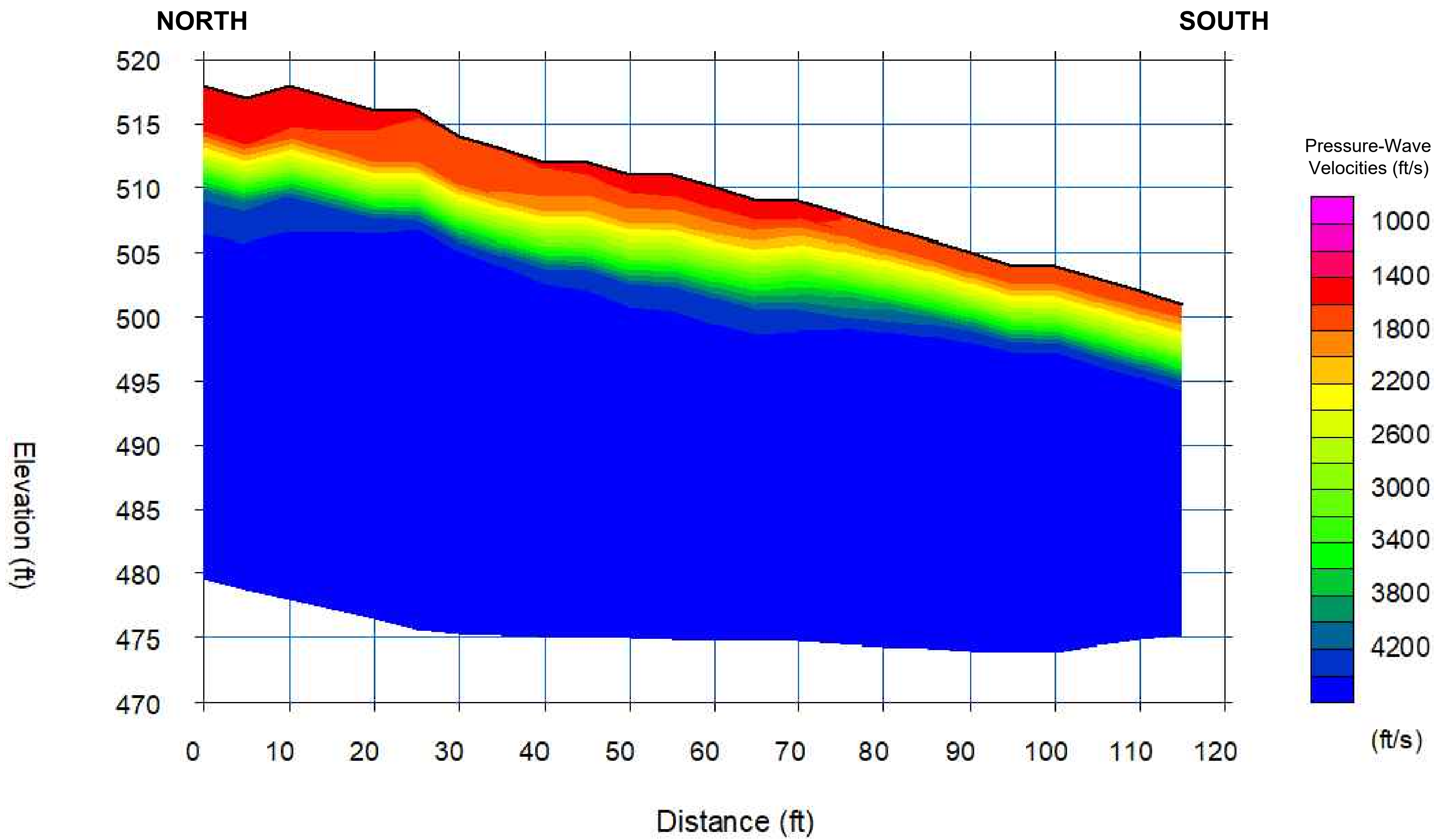


211532001_SRL3.dwg 11/13/2020 JDP

FIGURE 9

**SEISMIC REFRACTION LINE 3**

RESERVOIRS 2B AND 3B REPLACEMENT PROJECT
LAGUNA BEACH, CALIFORNIA

211532001 I 11/20

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