Updated Geotechnical Evaluation

North Reservoirs 19BR and 19C Replacement Project

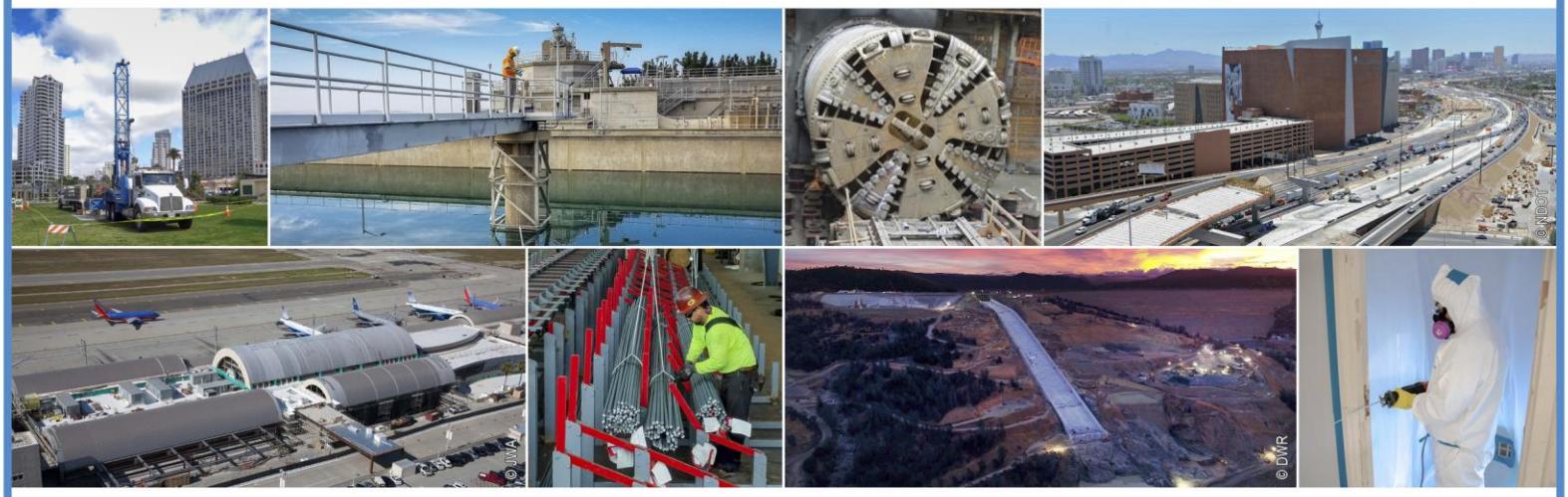
City of Glendora

Glendora, California

AKM Consulting Engineers (AKM)

553 Wald Street | Irvine, California 92618

May 5, 2023 | Project No. 212016001



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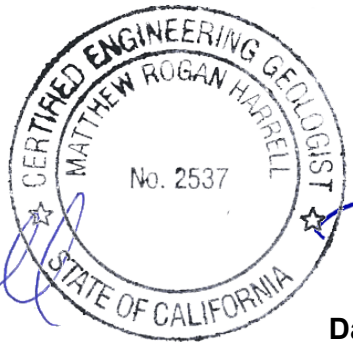



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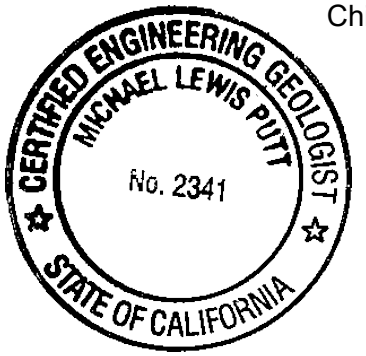

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CONTENTS

1 INTRODUCTION 1
2 SCOPE OF SERVICES 1
3 SITE DESCRIPTION AND BACKGROUND 2
4 PROPOSED CONSTRUCTION 2
5 SUBSURFACE EVALUATION AND LABORATORY TESTING 3
6 GEOLOGY AND SUBSURFACE CONDITIONS 4
6.1 Regional Geology 4
6.2 Site Geology 4
6.2.1 Artificial Fill (Qaf) 4
6.2.2 Slopewash (Qsw) 4
6.2.3 Bedrock – Granitic and Leucocratic Rock (Kgl) 5
6.2.4 Geologic Structure 5
6.3 Groundwater 5
7 FAULTING AND SEISMICITY 5
7.1 Surface Fault Rupture 6
7.2 Seismic Ground Motion 6
7.3 Earthquake-Induced Landslides and Slope Stability 7
8 SLOPE STABILITY ANALYSIS 7
8.1 Kinematic Analysis 8
8.2 Slope Stability Analysis 8
9 RIPPABILITY 9
10 CONCLUSIONS 9
11 RECOMMENDATIONS 11
11.1 Earthwork 11
11.1.1 Pre-Construction Conference 12
11.1.2 Demolition, Clearing, and Grubbing 12
11.1.3 Excavation Characteristics 12
11.1.4 Temporary Excavations 13
11.1.5 Foundation Subgrade Preparation for New Structures 13
11.1.5.1 Foundations for New Reservoirs 13

11.1.5.2 Foundations for New Retaining Wall, Minor Structures and Pavements 14
11.1.6 Fill Material 14
11.1.7 Fill Placement and Compaction 15
11.2 Underground Utilities 16
11.2.1 Pipe Bedding 16
11.2.2 Modulus of Soil Reaction 16
11.2.3 Lateral Pressures for Thrust Blocks 17
11.3 Site-Specific Seismic Design Considerations 17
11.4 Foundations 17
11.4.1 Ring Wall Foundation 17
11.4.2 Spread Footings 18
11.4.3 Retaining Wall Foundation 19
11.4.3.1 Lateral Earth Pressure 19
11.5 Lateral Earth Underground Structures 20
11.6 Corrosivity 20
11.7 Concrete Placement 20
11.8 Drainage 21
12 CONSTRUCTION OBSERVATION 21
13 ADDITIONAL WORK 22
14 LIMITATIONS 22
15 REFERENCES 24

TABLES

1 – Material Properties Used in Slope Stability Evaluation 8
2 – 2022 California Building Code Seismic Design Criteria 17

FIGURES

1 – Site Location
2 – Site Geology
3 – Proposed Improvements
4 – Cross Section A-A’
5 – Regional Geology
6 – Fault Locations

7 – Seismic Hazard Zones
8 – Thrust Block Lateral Earth Pressure Diagram
9 – Lateral Earth Pressures for Yielding Retaining Walls
10 – Retaining Wall Drainage Detail
11 – Lateral Earth Pressures for Underground Structures

APPENDICES

A – Test Pit Logs
B – Laboratory Testing
C – Seismic Refraction Profiles
D – Kinematic Analyses
E – Slope Stability Analyses

1 INTRODUCTION

In accordance with your request and authorization, we have performed a geotechnical evaluation for the proposed North Reservoirs 19BR and 19C Replacement Project located at the north terminus of Glendora Avenue in Glendora, California (Figure 1). The purpose of our study was to evaluate the soil and geologic conditions at the site and provide geotechnical recommendations for the proposed site grading, and the design and construction of the proposed replacement reservoir and other associated site improvements. The report was updated to address review comments and to provide geotechnical recommendations for design and construction of a retaining wall that will be constructed at the toe of the proposed cut slope. 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 preliminary plans provided by the client.
- A site reconnaissance to observe the general site conditions, mark-out the proposed test pit locations, and coordination with Underground Service Alert for utility clearance.
- Performed a geotechnical reconnaissance and geologic mapping by a California Certified Engineering Geologist of the project area to evaluate the surface conditions from a geotechnical perspective.
- Performed a seismic refraction survey of the project area to evaluate the bedrock conditions for excavatability and rippability.
- Subsurface exploration consisting of the excavation, sampling, and logging of four exploratory test pits to depths of up to approximately 7 feet. The test pits were logged in the field by our representative and bulk samples were collected and returned to our laboratory for evaluation and testing. The test pits were backfilled with the excavated soil.
- Laboratory testing on selected soil samples including evaluation of Atterberg, gradation, 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 site is located near the northern terminus of Glendora Avenue in the foothills of the San Gabriel Mountains. The site is accessed by an asphalt driveway from Glendora Avenue. An approximately 1½:1 (horizontal to vertical) natural slope ascends from the northwest side of Reservoir 19B to the Azusa-Glendora Ridge Mountainway to an elevation of approximately 2550 feet above mean sea level (MSL) (USGS, 1981). On the east side of the reservoir site, a natural slope descends at 2:1 or flatter to the east from the access road to an elevation of approximately 1360 feet above MSL to canyon bottoms and residential properties. In addition, a natural slope located about 250 feet southwest of the site descends at an approximately 2:1 or flatter to an elevation of approximately 1,250 feet above MSL to a canyon. Residential properties and Glendora Avenue are to the south at elevation of approximately 1360 feet above MSL. The slopes have a relatively thick growth of native brush.

We understand that the existing bolted steel tank reservoir, which is approximately 47½ feet in diameter and approximately 24 feet in height, estimated to hold 250,000 gallons, was constructed in the 1990's and is experiencing leaks and will be replaced with a new reservoir. Based on our review of the as-built drawings (Giron Engineers, Inc., 1998), grading for the existing reservoir generally involved excavating into the hillside to create a level pad for the reservoir and exporting over 7,000 cubic yards of soil/bedrock. The grading created an approximately 75-foot-high, 1½ :1 (horizontal to vertical) cut slope on the west side of the reservoir. The as-built drawings indicate that the reservoir pad area is in a cut condition, presumably so that the reservoir foundations would be founded in bedrock.

4 PROPOSED CONSTRUCTION

We understand that it is the City's desire to keep the existing reservoir in-service during construction and that in order to accommodate this, the new reservoir is to be constructed to the southwest of the existing reservoir. After the new reservoir 19C is constructed and the existing reservoir 19B is demolished, a second reservoir, reservoir 19BR will be added at the location of the current reservoir.

Based on our review of the of construction drawings prepared by AKM (2023), we understand that the new reservoir will have similar dimensions and capacity as the existing reservoir. In order to construct the new reservoir, rough grading will be performed to enlarge the existing reservoir pad.

The construction drawing indicates that the existing 1½:1 (horizontal to vertical) cut slope will be expanded to the southwest and that the new cut slope will be up to approximately 100 feet high at the same 1½:1 (horizontal to vertical) slope ratio. Up to approximately 35 feet of cut will be needed to reach the proposed pad elevation of 1390 feet MSL. In addition, the existing approximately 3-foot-high slough wall will be replaced with a new retaining wall ranging in height from approximately 3 feet at the existing reservoir to approximately 12 feet high around the new reservoir.

We understand that the new reservoirs will be supported on a ring foundation, which is anticipated to be supported on bedrock based on the proposed grading. New piping, an inlet/outlet pipe vault, and drainage improvements are also planned. The existing site conditions, previous site topography, proposed improvements, and proposed grading are shown on Figures 2 and 3, and in Cross Section A-A' on Figure 4.

5 SUBSURFACE EVALUATION AND LABORATORY TESTING

Our site exploration activities were performed on September 16 and 20, 2022 and consisted of one seismic refraction profile, the excavation, logging, and sampling of four test pits (TP-1 through TP-4), and geologic mapping. The test pits were excavated to depths ranging from approximately 5½ feet to 7 feet, using a track-mounted Kubota 080 excavator with a 24-inch-wide bucket. The test pits were logged in the field by a representative of Ninyo & Moore during excavation to the depths shown on the logs. Representative bulk soil and bedrock samples were collected from the test pits for laboratory testing and transported to our laboratory. Several attempts were made to collect relatively undisturbed samples of the bedrock with a hand-driven modified split-barrel drive sampler, however, the samples either became disturbed during driving or the sampler could not penetrate a sufficient depth into the relatively hard bedrock. Logs of the test pits are presented in Appendix A.

Laboratory testing was performed on representative samples collected from the test pits. The laboratory testing included evaluation of gradation, Atterberg limits, and soil corrosivity characteristics (including pH, resistivity, and water soluble sulfates and chlorides). The laboratory testing results are presented in Appendix B.

Our subsurface evaluation included a seismic refraction profile to evaluate excavatabilty and rippability of the granitic bedrock material, which is discussed in Section 9 of this report. The approximate locations of the seismic refraction profile and test pits are presented on Figures 2 and 3. The seismic refraction profile is presented in Appendix C.

6 GEOLOGY AND SUBSURFACE CONDITIONS

6.1 Regional Geology

The project site is located in the Transverse Ranges Geomorphic Province of Southern California (Norris and Webb, 1990). The province is characterized by generally east-west trending mountain ranges, fault zones, and structural basins 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 Cretaceous-age granitic and leucocratic rocks including pegmatite, aplite, alaskite, and biotite monzogranite bedrock that is weakly to moderately foliated (Morton and Miller, 2006), as shown on Figure 5. However, a relatively large dormant old/relict landslide has been mapped in the area of the site (Tremaine and Willis, 2007). The landslide is described as questionable with 50 percent confidence that it is a landslide, but the geomorphic feature that is suggestive of a landslide could be the result of other processes. The landslide covers a wide area, well beyond the limits of the reservoir site, and includes the nearby roadways and residential properties to the south. During our site exploration and geologic mapping, we did not observe evidence of significant slope instability such as existing landslides, scarps or tension cracks.

6.2 Site Geology

Earth materials observed during our geologic mapping and subsurface exploration consisted primarily of slope wash and granitic bedrock materials. While not directly observed, artificial fill associated with prior construction activities is anticipated to be present. 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 test pit 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)

Although we did not observe artificial fill during our geologic mapping and subsurface exploration, it is anticipated to be present for retaining wall backfill, trench backfill and in driveway areas to allow for site drainage. The fill material was probably derived from the slopewash and granitic material and is anticipated to consist of silty sand with gravel.

6.2.2 Slopewash (Qsw)

Slopewash was observed at the ground surface, along the top and edges of the existing cut 1½:1 cut slope and within the test pits. The slopewash generally consists of silty sand with

gravel and was observed to be relatively thin, approximately 2 feet thick, based on the test pits and where exposed in the existing cut slope.

6.2.3 Bedrock – Granitic and Leucocratic Rock (Kgl)

Granitic bedrock materials were observed in the test pits and are exposed in the existing cut slope beneath a relatively thin mantle of slopewash. The bedrock materials that were observed generally consisted of fine to medium grained granodiorite with aplite and quartz dikes. The bedrock materials were moderately soft and weathered in the upper 5 to 6 feet and became less weathered and moderately hard below this depth. In addition, the aplite and quartz dikes were generally harder than the surrounding granitic rock. The relatively small track-mounted Kuboto 080 excavator had difficulty excavating below depths of 5 to 6 feet.

6.2.4 Geologic Structure

The granitic bedrock materials were generally observed to be weakly to moderately foliated. The foliation was observed to be thinly spaced and dipping steeply to the northeast, northwest, and southwest at angles ranging from approximately 50 to 74 degrees. The bedrock was also observed to be moderately fractured. The fractures were observed to be undulatory and relatively short/discontinuous, with highly variable dips ranging from approximately 17 degrees to vertical in varying directions. The fractures were generally observed to be tight and rough; however, some fractures near the surface were slightly open and infilled with soil from the overlying slopewash. The geologic structure is shown on Figure 2 and Cross Section A-A', Figure 4.

6.3 Groundwater

Groundwater was not encountered during the excavation of the exploratory test pits. Groundwater is anticipated to be more than 100 feet deep in the hillside terrain. However, seepage along the contact 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 ¾ mile northeast of the Sierra Madre 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 2022 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 710 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.XXg using the American Society of Civil Engineers (ASCE) ASCE 7 Hazard Tool (ASCE, 2022 [web-based]).

The 2022 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 7-22 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 0.79g

using the American Society of Civil Engineers (ASCE) ASCE 7 Hazard Tool (ASCE, 2022 [web-based]).

7.3 Earthquake-Induced Landslides and Slope Stability

The State of California Seismic Hazards Zones Maps indicates that the site is located adjacent to areas that are mapped as susceptible to seismically induced landslides (Figure 7). However, the area where the existing cut slope will be expanded, and the slope that descends below the reservoir site to the east, are not located within the seismic-induced landslide hazard zone. 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 or other planes of weakness 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 granitic bedrock. Where observed in the existing cut slope and in our test pits, the bedrock was moderately soft to moderately hard, moderately foliated and moderately fractured. The foliation is steeply dipping. The majority of the fractures were also observed to be steeply dipping in varying directions to vertical. However, some fractures were observed to be dipping at shallow angles of 17 to 27 degrees to the southeast. In general, the fracture surfaces were rough and tight, but some slightly open fractures with soil infilling were noted near the surface. However, the fractures were generally short and discontinuous. 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

Our slope stability evaluation included review of geologic mapping data, field measurements of geologic structure, preparation of representative geologic cross sections and computer modeling. The stability of slopes comprised of granitic bedrock vary depending on the slope inclinations, slope directions, rock weathering and fracturing. Steeper slopes generally involve higher risk of instability depending on the integrity of the rock. In general, fractures or intersecting fractures that dip toward the face of a slope may cause instability. Our analyses included preparation of geologic Cross Sections A-A' in Figure 4. The geologic data, including fracture trends, were used to perform stereonet analyses using the computer program Dips version 8.013 (Rocscience, 2021).

8.1 Kinematic Analysis

Kinematic analysis of the existing slope was performed along geologic Cross Sections A-A'. The slope was modeled as a 100-foot-high slope with a 33-degree inclination measured from the horizontal for Cross Section A-A'. A total of 32 discontinuities, including joints and foliation measured by us were used in the analysis. The kinematic analyses were performed using the computer program Dips version 8.013 (Rocscience, 2021). The program uses stereonet plots to evaluate whether the slope is kinematically stable or unstable with respect to planar, wedge-type and toppling failure. The equations used to calculate planar and wedge-type failure were based on limit equilibrium methods developed by Hoek and Bray (1981). Our analyses indicated the proposed cut rock slope is stable. The results of our kinematic analyses are presented in Appendix D.

8.2 Slope Stability Analysis

Slope stability analyses were performed on geologic Cross Sections A-A' based on the modified Bishop Method of Slices using the computer program GSTABL7 (Gregory, G.H., 2005). 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 shear strength parameters used in our stability analyses for the bedrock materials were selected based on 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)
Granitic Rock	140	200	40

Notes:
psf – pounds per square feet

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 E.

9 RIPPABILITY

As a part of our study, Ninyo & Moore evaluated the rippability of the bedrock materials expected to be encountered during grading for the proposed Reservoir 19BR and 19C. We performed a seismic refraction profile (Appendix C) at the location shown on Figures 2 and 3. The profile was 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 5,500 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. In addition, the quartz and aplite dikes may be more difficult to excavate and may generate more oversize rock as compared to the host granitic rock. 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 construction of the new reservoirs and retaining wall at the site 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 North Reservoirs 19BR and 19C site include the excavatability/rippability of the bedrock materials, handling of oversize rock generated during earthwork, and the long term slope stability of the proposed cut slope.

As described in Section 9, our seismic refraction survey indicates that the bedrock materials should be excavatable by a Caterpillar D-8 dozer, or equivalent, with a single-shank ripper to depths of approximately 40 feet, which is more than the planned excavation depth. Excavation of the bedrock, particularly as the depth of cut increases, is anticipated to involve moderate to heavy ripping. The aplite and quartz dikes are generally harder and may be more difficult to excavate. Due to the bedrock hardness, excavations into the granitic bedrock is anticipated to generate oversize materials. The harder and less fractured aplite and quartz dikes may generate more oversize materials than the surrounding granitic rock. Oversize rock should be screened from general fill as well as trench and retaining wall backfill. Backhoes may not be practical for

foundation and utility trench excavations, particularly on the west side of the new reservoir where deeper cuts will be needed to create the larger tank pad, and a larger excavator may be better suited. Overexcavation around the perimeter of the tank footprint and replacing the materials as compacted fill to the depths of the deeper utilities can be considered to reduce the potential difficult trenching conditions for new pipelines and vaults.

As described in Section 8, our slope stability analysis indicates that the expanded 1½:1 (horizontal to vertical) cut slope is anticipated to be grossly stable. Furthermore, during our geologic mapping of the existing cut slope, we did not observe evidence of slope instability, such as tension cracks or wedge failures of the bedrock. Additionally, we did not observe significant amounts of eroded debris at the base of the existing cut slope, indicating that this slope has performed well since it was graded in the 1990's.

Based on our review of the as-built construction drawings for the existing reservoir (Giron Engineers, 1998), the existing reservoir is probably supported by competent granitic bedrock. Therefore, significant remedial grading to support a new reservoir is not anticipated on the existing pad when the second new reservoir will be constructed in its place. However, demolition of the existing reservoir foundations, buried pipelines and vaults, may leave existing fill or voids from the removed improvements. A representative of Ninyo & Moore should observe the foundation demolition and removal of buried pipes and vaults to evaluate the suitability of the existing pad for support of the new reservoir. Additional test pits may also be appropriate to evaluate the pad condition following demolition. Updated remedial grading and foundation design recommendations may be needed to address the construction of the second new reservoir.

In general, the following additional conclusions were made:

- The site is underlain by slope wash deposits, and granitic bedrock materials. The slope wash deposits generally consist of moist, loose to medium dense, silty sand with gravel. The bedrock at the site can be described as moist, moderately soft to moderately hard, weathered, weak to moderately foliated granitics with leucocratic dikes.

- Excavations in the on-site soils and bedrock should be feasible with earthmoving equipment in good working condition. Moderate to heavy ripping should be anticipated in the bedrock and significant amounts oversized material may be generated during site grading, utility trench excavation, and during excavations for retaining walls.

- The on-site sandy soils and bedrock should be suitable for re-use as general fill and 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 retaining wall construction.

- Groundwater was not encountered during our subsurface exploration and groundwater is not anticipated to be a design consideration for the project. During the wet season, some seepage along the contact between the slopewash and the bedrock, and along fracture surface, may be encountered. 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 including structural details were not available at the time this report was prepared. We recommend that the final construction drawings with structural details 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 and review of the site plan (AKM, 2023), earthwork at the site is anticipated to consist of site clearing, excavations of up to approximately 35 feet deep for the construction of the enlarged reservoir pad and retaining wall, remedial grading associated with the preparation of equipment pads, excavations for the reservoir 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 vegetation. Other structures, reservoir improvements, AC pavements where present, rubble and debris, abandoned utilities, surface obstructions, and other deleterious materials within the limits of the new grading should also be removed, as needed. 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 slope wash materials generally consisted of medium dense, silty sand with gravel. The bedrock at the site can generally be described as moderately soft to moderately hard, weathered, granitic rock. 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. Difficult excavating conditions were encountered in our test pits, and the difficulty generally increased with depth; therefore, excavations will involve moderate to heavy ripping, particularly at depth and within the harder quartz and aplite dikes. However, zones of hard, 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.

Oversized granitic materials 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 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

Based on our review of the preliminary construction drawings, competent bedrock is anticipated to be exposed across the enlarged reservoir pad at the completion of site grading for the new reservoir. The following sections provide our recommendations for the new reservoirs and other minor structures. As described in the Conclusions section of this report, updated earthwork and foundation recommendations may be needed for the second new reservoir constructed after the existing reservoir and associated improvements are removed.

11.1.5.1 Foundations for New Reservoirs

As described in the previous sections, the foundation for the new reservoir is planned to be founded on the granitic bedrock materials. Based on the proposed cuts shown on the conceptual site plan, we anticipate that the reservoir foundation will be excavated into competent bedrock and additional remedial grading should not be needed. A Ninyo & Moore representative should observe the foundation excavations during construction to evaluate the depth to competent bedrock.

As described in Section 10, a representative of Ninyo & Moore should observe the demolition activities when the existing Reservoir 19B and associated improvements are removed to construct the new Reservoir 19BR. Based on our review of the as-built construction drawings for the existing reservoir (Giron Engineers, 1998), the existing reservoir is probably supported by competent granitic bedrock. Therefore, significant remedial grading to support Reservoir 19BR is not anticipated. However, if existing fill or voids from the demolition are present, the existing fill should be removed to competent bedrock and replaced as new compacted fill. Depending on the locations of the existing fill and/or voids, it may be appropriate to overexcavate the foundation area for the new Reservoir 19BR to provide a consistent compacted fill mat below the new foundations.

For planning purposes, the contractor should anticipate overexcavating the new Reservoir 19BR foundation area to the depth of the bottom of the foundation for the existing reservoir. Additional test pits may also be appropriate to evaluate the pad condition following demolition. Updated remedial grading and foundation design recommendations may be needed to address the construction of the second new reservoir and will be provided following our observations after demolition, as appropriate.

11.1.5.2 Foundations for New Retaining Wall, Minor Structures and Pavements

We anticipate competent bedrock will be exposed at the retaining wall foundation subgrade and additional remedial grading should not be needed. Furthermore, 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 slope wash material, 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.

In new pavement areas, we recommend that the subgrade soils are scarified, moisture-conditioned, and recompacted to a depth of approximately 8 to 12 inches prior to placing pavements. The scarification, moisture conditioning and recompaction should extend up to the existing pavements that will remain in-place, or 12 inches beyond the new pavements where they do not join existing pavements.

11.1.6 Fill Material

Grading of the site is anticipated to mainly consist of cuts to construct the enlarged reservoir pad and significant fills are not anticipated. 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 removing rocks or lumps of material in excess of 4 inches in diameter from general fill and backfill. 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).

Although not anticipated, imported materials, if used, 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

As described above, we anticipate that the future enlarged reservoir pad will involve cuts in the bedrock to reach finish pad grade. However, some fill placement may be needed depending on the final design of the site grading. Fill placed to construct the enlarged reservoir pad or beneath other site improvements such as pipe vaults 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, 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 or unsuitable slopewash materials, old fill and slopewash should be removed to expose competent bedrock and the overexcavated materials replaced as compacted fill to the pipeline or vault subgrades.

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 (2022) guidelines.

Table 2 – 2022 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	1.4
Mapped Spectral Response Acceleration at 0.2-second Period, S_s	1. 89g
Mapped Spectral Response Acceleration at 1.0-second Period, S_1	0.65g
Site-Modified Spectral Response Acceleration at 0.2-second Period, S_{MS}	1.99g
Site-Modified Spectral Response Acceleration at 1.0-second Period, S_{M1}	0.93g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.32g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.62g
Mapped Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration, PGA_M	0.79g

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 reservoir 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 10,000 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 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 500 psf per foot of depth for level ground condition up to a value of 5,000 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 vaults and other miscellaneous at-grade equipment pads may be supported on shallow spread footings bearing on competent bedrock or 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 underlain by competent bedrock may be designed using the net allowable bearing capacities described above. However, where footings are supported by compacted fill, they should 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.

11.4.3 Retaining Wall Foundation

As noted, a new retaining wall will be constructed in order to reduce the height of the planned cut-slope for the new reservoir. The proposed retaining wall may be supported by spread footings bearing on bedrock. If highly weathered bedrock or slopewash materials are exposed near the ends of the proposed retaining wall where less excavation is needed to reach the foundation subgrade, the retaining wall foundation should be deepened through the unsuitable material down to competent bedrock. Foundations should be designed in accordance with the recommendations presented in Section 11.4 and the structural considerations. In addition, requirements of the appropriate governing jurisdictions and applicable building codes should be considered in the design of the structures.

11.4.3.1 Lateral Earth Pressure

Lateral earth pressures recommended for the design of yielding retaining walls are provided on Figure 9. In addition, the designer should include any surcharge. Retaining walls should be backfilled with free-draining, granular, non-expansive material (Expansion Index 20 or less). Measures should be taken to reduce the potential for build-up of moisture behind the retaining walls. Drainage design should include free-draining backfill materials and perforated drains as described on Figure 10.

11.5 Lateral Earth Underground Structures

Below-grade vault walls 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 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 5.5. The electrical resistivity was measured 3,232 ohm-cm. The chloride content was measured at 85 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.

11.7 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.

11.8 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 our geologic mapping and the subsurface conditions observed in our exploratory 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.
- A Ninyo & Moore geologist should perform geologic mapping during grading to evaluate the conditions exposed on the proposed cut slope and to evaluate that competent bedrock has been exposed at the finish pad elevation.
- Observe foundation excavations for the reservoir and cleaning prior to placement of reinforcing steel or concrete.
- Observe 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 ADDITIONAL WORK

At the time of this study, the existing reservoir is planned for demolition following construction of the new reservoir. The original reservoir may be replaced with a second reservoir of similar dimensions. As described in the Conclusions section of this report, the pad area of the existing reservoir should be evaluated during the demolition activities.

During demolition of the existing reservoir, we recommend Ninyo & Moore observes the removal of the existing reservoir foundation and utility pipes, and that the pad area is geologically mapped to document the limits of the voids created by their removal. Performing test pits in the pad area may also be appropriate. Updated recommendations for earthwork and foundation design parameters will be provided for construction of the second reservoir will be provided, as appropriate.

14 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.

15 REFERENCES

AKM, 2022, Site Plan, Layout Option 1, Approximate Limits, City of Glendora, North Glendora Reservoir Replacement, dated July.

AKM, 2023, Plans for Construction of North Glendora Reservoirs 19BR and 19C, City of Glendora, Plan No. 1368, dated March.

American Concrete Institute (ACI), 2016, ACI Manual of Concrete Practice.

American Society of Civil Engineers (ASCE), 2022, Minimum Design Loads and Associated Criteria for Building and Other Structures, ASCE Standard 7-22.

American Society of Civil Engineers (ASCE), 2022, ASCE Hazard Tool, ASCE Standard 7-22, https://asce7hazardtool.online/

Applied Technology Council, 2021, Hazards by Location, https://hazards.atcouncil.org/

Building Seismic Safety Council, 2009, National Earthquake Hazards Reduction Program (NEHRP) Recommended Seismic Provisions for New Buildings and Other Structures (FEMA P-750).

California Building Standards Commission, 2022, California Building Code: California Code of Regulations, Title 24, Part 2, Volumes 1 and 2, based on the 2021 International Building Code.

California Department of Conservation, Division of Mines and Geology (CDMG), 1998, Seismic Hazard Zone Report for the Glendora 7.5-Minute Quadrangle, Los Angeles County, California: Seismic Hazard Zone Report 025.

California Department of Conservation, Division of Mines and Geology (CDMG), 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, CGS Special Publication 117A.

California Geological Survey (CGS), 1999, Earthquake Zones of Required Investigation, Glendora Quadrangle, 7.5-Minute Series, Scale 1:24,000, dated March 25.

California Geological Survey (CGS), 2022, CGS Map Sheet 48: Shear-wave Velocity in Upper 30m of Surficial Geology (Vs30), https://gis.data.ca.gov/datasets/cadoc::cgs-map-sheet-48-shear-wave-velocity-in-upper-30m-of-surficial-geology-vs30/about.

California Department of Transportation, 2021, Corrosion Guidelines, Version 3.2, dated May.

Caterpillar, 2000, Handbook of Ripping, Twelfth Edition, dated February

Caterpillar, 2018, Caterpillar Performance Handbook, Edition 48, dated June.

Giron Engineers, Inc., 1998, As-Built, Grading and Drainage Plan, Reservoir 19B Site – Northerly Terminus of Glendora Avenue, City of Glendora, dated April 22.

Google, 2020, Website for Viewing Aerial Photographs; http://maps.google.com.

Gregory, G.H., 2005, GSTABL7 with STEDwin v. 2005.3 for Slope Stability Analysis.

Hart, E.W., and Bryant, W.A., 2007, Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zones Maps: California Department of Conservation, California Geological Survey, Special Publication 42, with Supplement 1 added in 2012, Supplement 2 added in 2014, Supplement 3 added in 2015, and Supplement 4 added in 2016.

Hendron, A.J., and Oriard, L.L., 1972, Specifications for Controlled Blasting in Civil Engineering Projects: Proceedings of Rapid Excavation and Tunneling Conference, Chicago

Jennings, C.W., and Bryant, W.A., 2010, Fault Activity Map: California Geological Survey, California Geologic Data Map Series, Map No. 6, Scale 1:750,000.

Morton, D.M. and Miller, F.K., 2006, Geologic Map of the San Bernardino and Santa Ana 30' by 60' Quadrangles, California, Version 1.0: United States Geological Survey Open-File Report 2006-1217, Scale 1:100,000.

Ninyo & Moore, 2022, Revised Proposal for Geotechnical Evaluation, Reservoirs 19B Replacement Project, North Terminus of Glendora Avenue, City of Glendora, California, Proposal No. 04-03586, dated March 23.

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

Post-Tensioning Institute, 2004, PTI DC35.1-04: Recommendations for Prestressed Rock and Soil Anchors.

Public Works Standard, Inc., 2018, The "Greenbook": Standard Specifications for Public Works Construction, 2018 Edition, with Errata No. 1 dated 2019.

RocScience, Inc., 2021a, Dips – Graphical and Statistical Analysis of Orientation Data. Version 8.013.

Southern California Earthquake Data Center, 2005, SCEC Community Velocity Model, Version 4.

Structural Engineers Association of California/Office of Statewide Health Planning and Development, 2019, Seismic Design Maps, https://seismicmaps.org/.

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

United States Geological Survey, 1966, Glendora, California Quadrangle Map, 7.5 Minute Series: Scale 1:24,000.

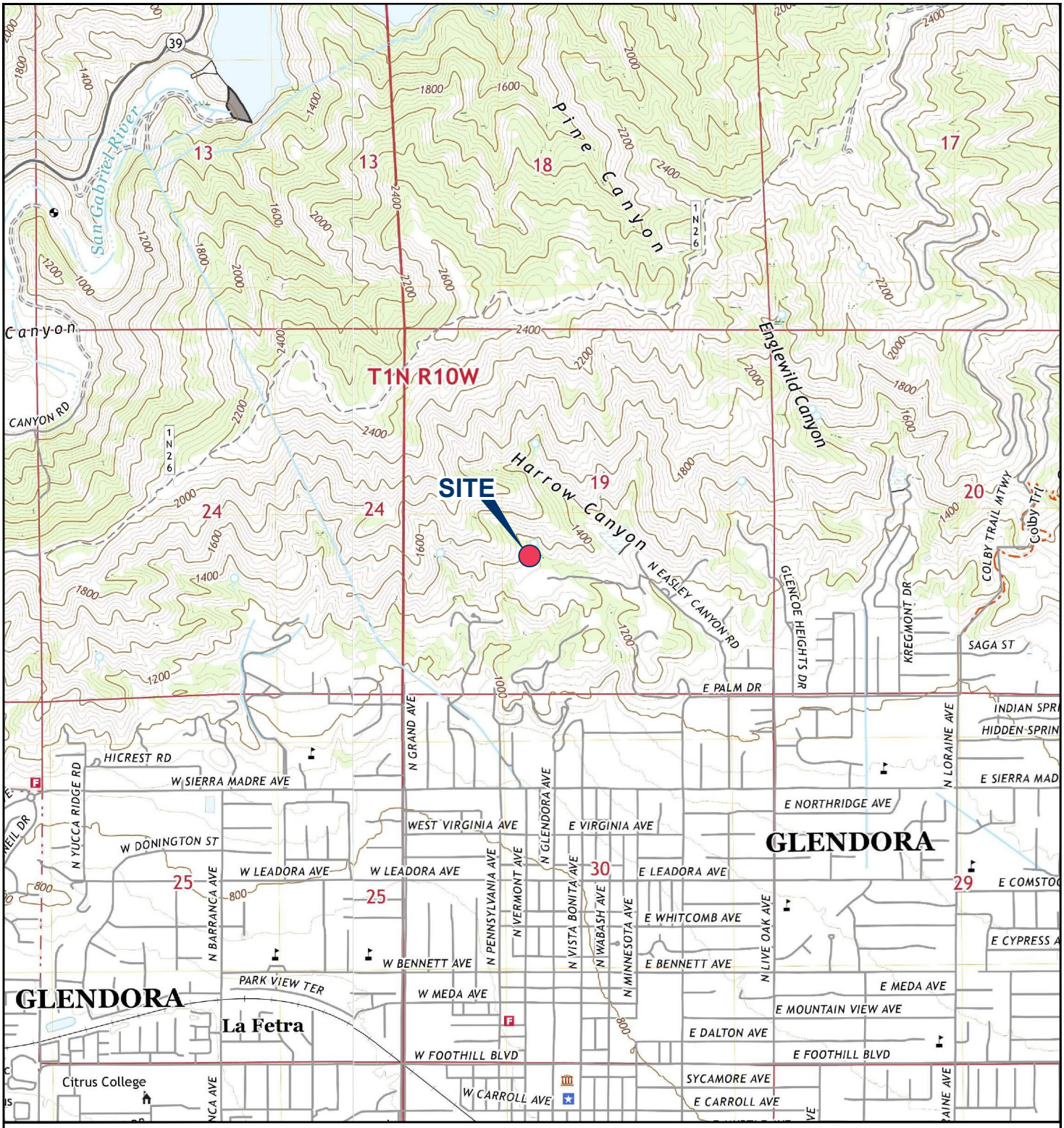
United States Geological Survey, 1995, Glendora, California Quadrangle Map, 7.5 Minute Series: Scale 1:24,000.

United States Geological Survey, 2018, Glendora, California Quadrangle Map, 7.5 Minute Series: Scale 1:24,000.

United States Geological Survey, 2019c, Slope Based Vs30 Map Viewer; https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=8ac19bc334f747e486550f32837578e1

Wills, C.J., and Clahan, L.B., 2006, Developing a Map of Geologically Defined Site-Condition Categories for California, Bulletin of the Seismological Society of America, v. 96, no. 4A, p. 1483–1501.

FIGURES



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

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FEET
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FIGURE 1

SITE LOCATION

NORTH RESERVOIRS 19BR AND 19C REPLACEMENT
GLENDORA, CALIFORNIA

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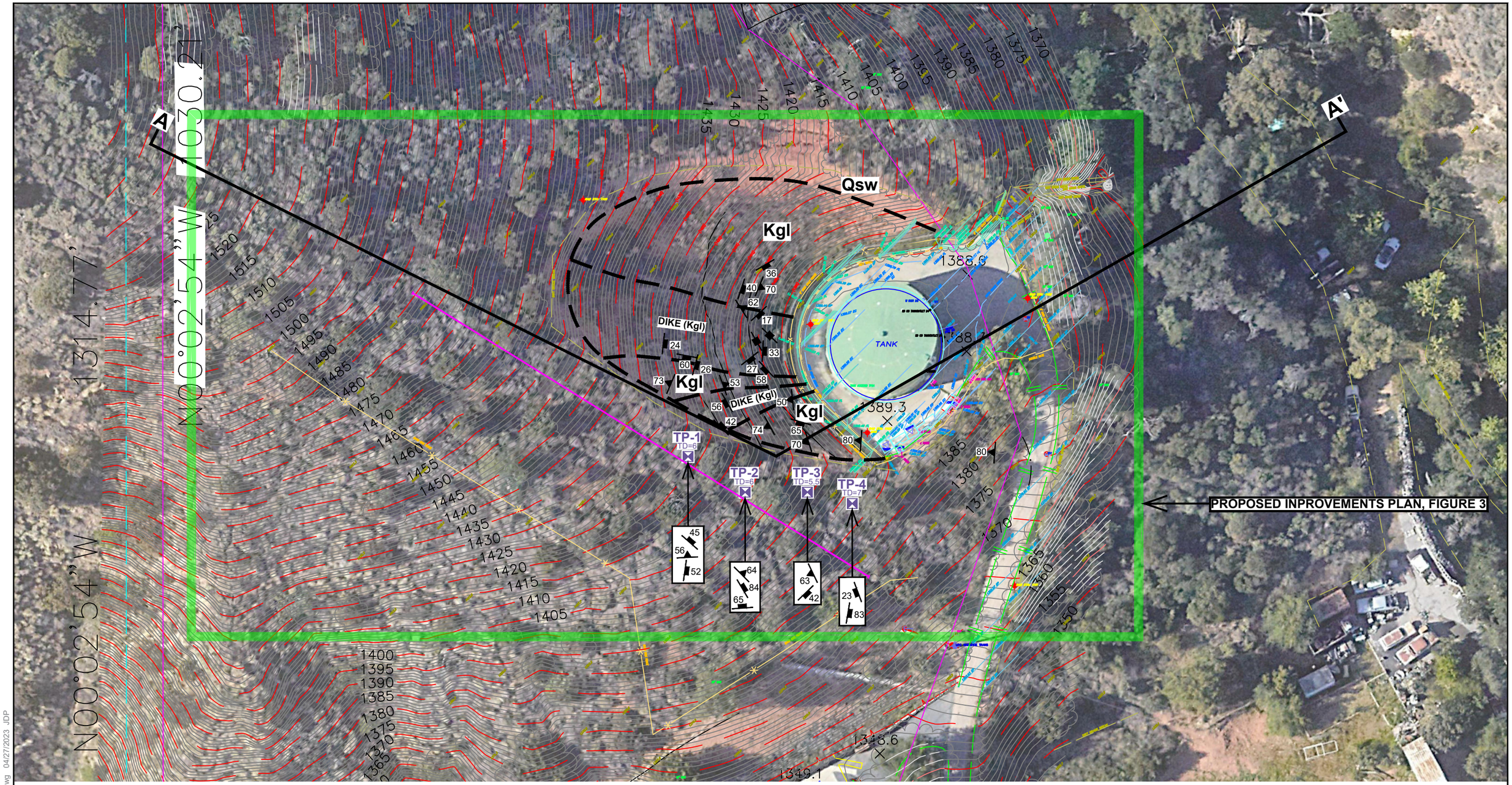
Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



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NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: GOOGLE EARTH, 2022.

LEGEND

- TP-4 (TD=7): TEST PIT; TD=TOTAL DEPTH IN FEET
- GEOLOGIC CONTACT
- Kgl: MIXED LEUCOCRATIC AND GRANITIC ROCKS (CRETACEOUS)
- Qsw: SLOPE WASH
- SEISMIC LINE
- 65: JOINTING ATTITUDE AND DIP
- 70: FOLIATION ATTITUDE AND DIP

N

FEET 0 40 80

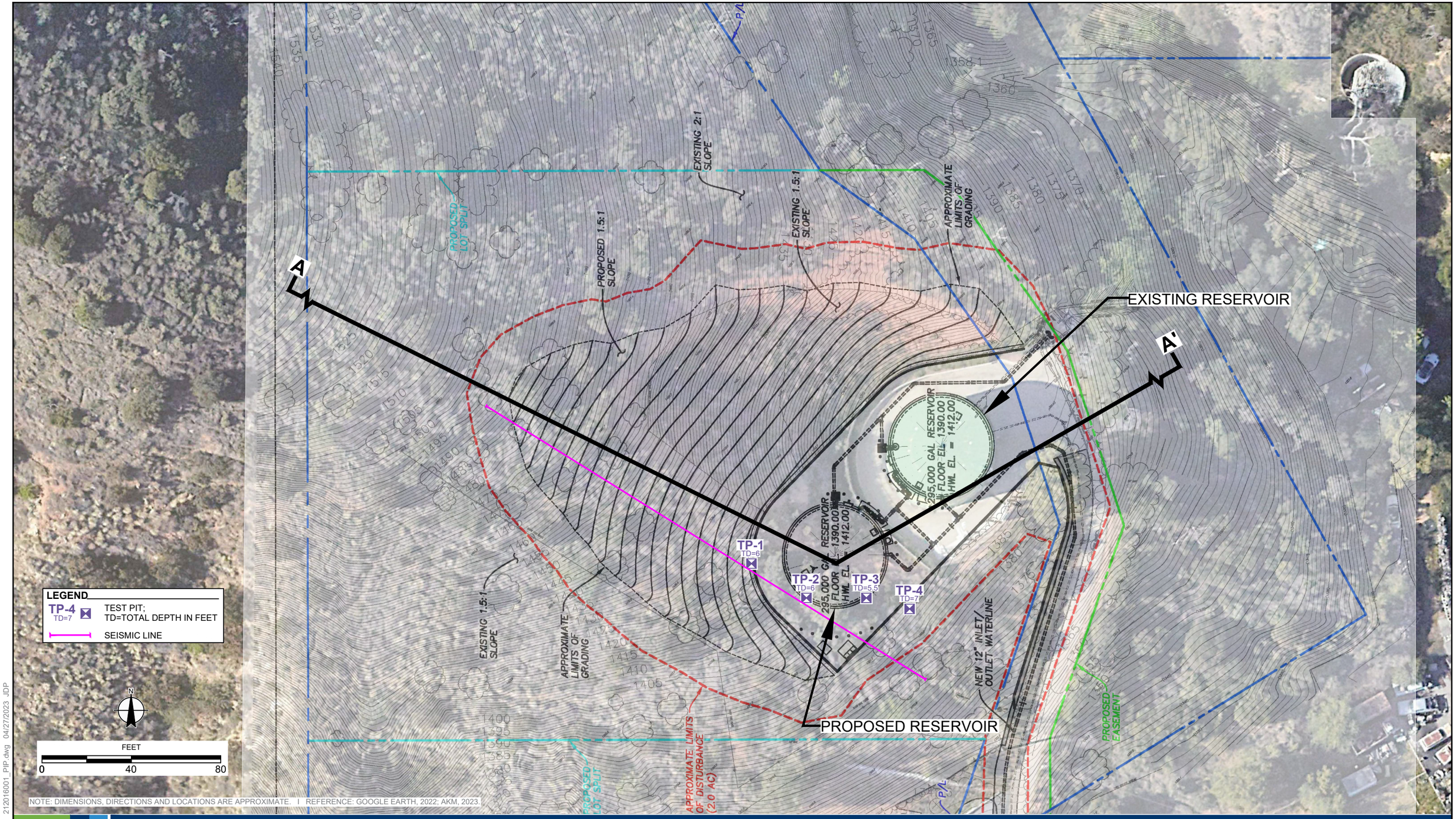
FIGURE 2

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

SITE GEOLOGY AND EXPLORATION LOCATIONS

NORTH RESERVOIRS 19BR AND 19C REPLACEMENT
GLENDORA, CALIFORNIA

212016001 I 5/23



NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: GOOGLE EARTH, 2022; AKM, 2023.

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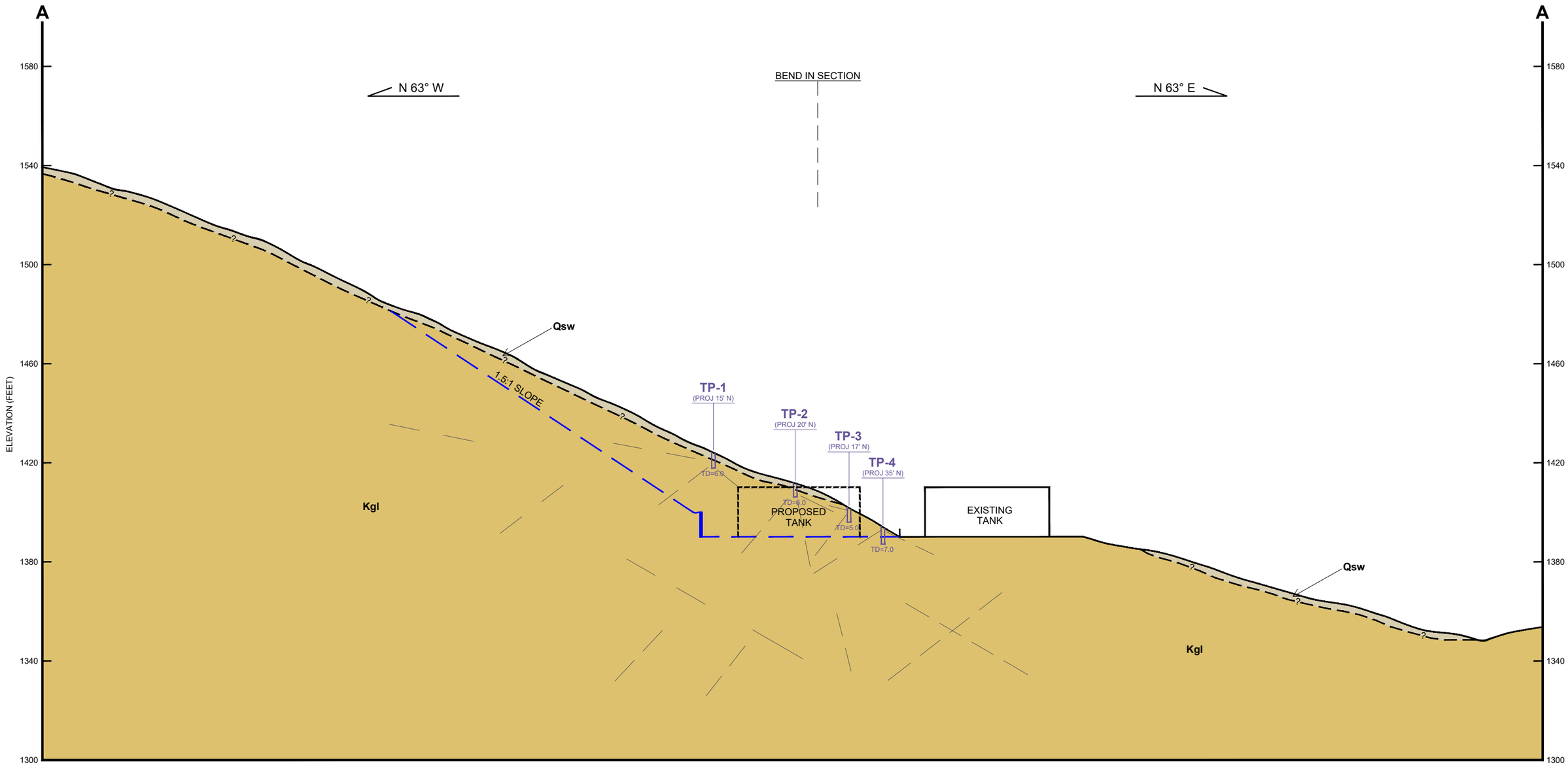
FIGURE 3

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

PROPOSED IMPROVEMENTS PLAN

NORTH RESERVOIRS 19BR AND 19C REPLACEMENT
GLENDORA, CALIFORNIA

212016001 I 5/23



LEGEND

TP-4 / TD=7.0 — TEST PIT; TD=TOTAL DEPTH IN FEET

Qsw — SLOPE WASH

Kgl — LEUCOCRATIC AND GRANITIC ROCKS

GEOLOGIC CONTACT; QUERIED WHERE INFERRED

GROUND SURFACE

DESIGN GRADE

APPARENT DIP OF JOINTS AND FOLIATION

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

FEET
0 40 80

212016001_CSA.dwg 04/27/2023 JDP

FIGURE 4

CROSS SECTION A-A'

NORTH RESERVOIRS 19BR AND 19C REPLACEMENT
GLENDORA, CALIFORNIA

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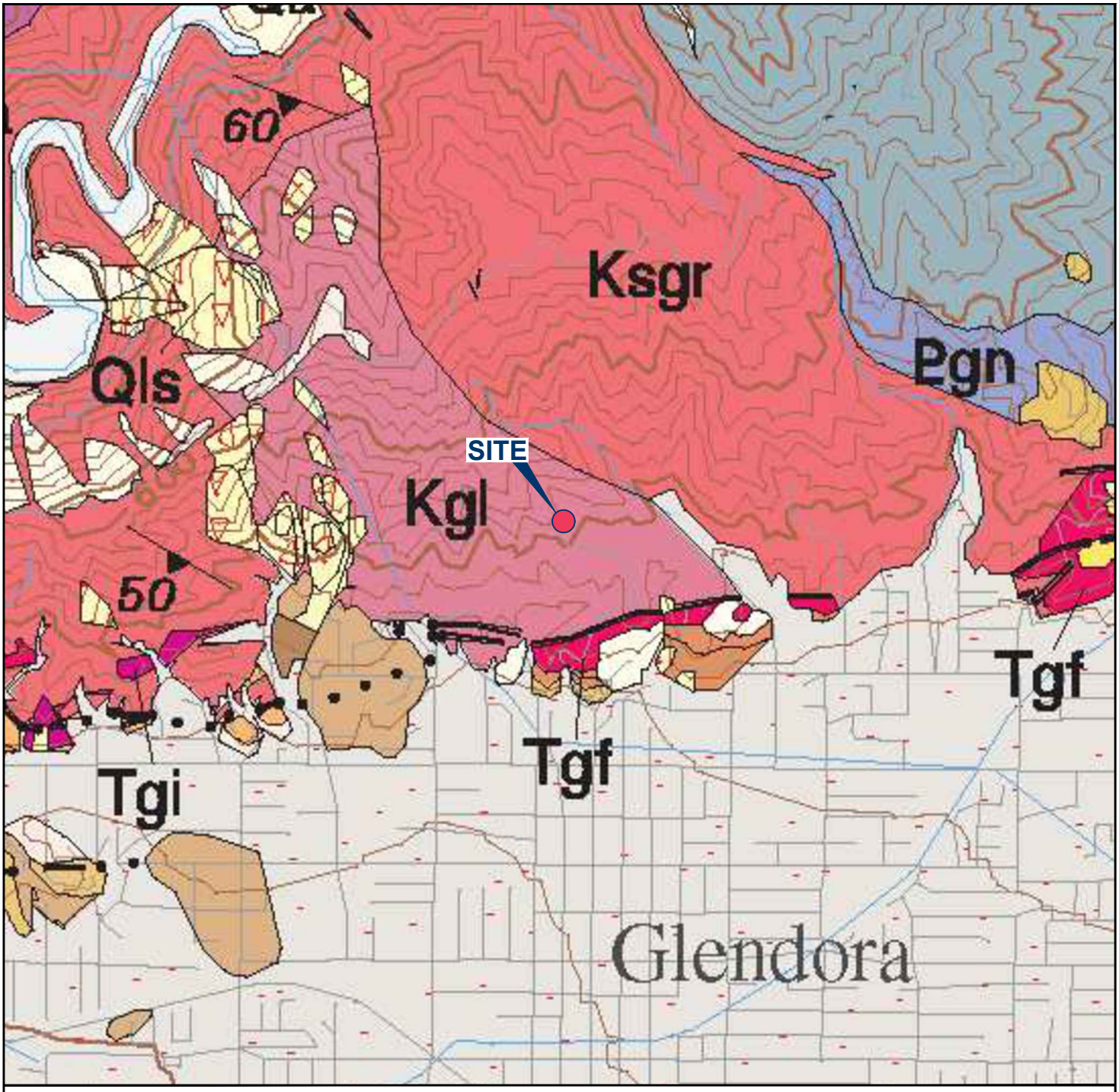

LEGEND

Kgl — **MIXED LEUCOCRATIC AND GRANITIC ROCKS (CRETACEOUS)** -- LEUCOCRATIC DIKE ROCKS. INCLUDES PEGMATITE, APLITE, ALASKITE, AND BIOTITE MONZOGRANITE. WEAKLY TO MODERATELY FOLIATED; MANY EXHIBIT BUCKLING OR BOUDINAGE STRUCTURES. MOST NOT LARGE ENOUGH TO SHOW AT 1:100,000 SCALE. INTRUDE CRETACEOUS QUARTZ DIORITE AND OLDER ROCKS BETWEEN SAN GABRIEL AND SAN ANTONIO CANYONS

GEOLOGIC CONTACT

FAULT

N

FEET

0 2,000 4,000

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

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FIGURE 5

REGIONAL GEOLOGY

NORTH RESERVOIRS 19BR AND 19C REPLACEMENT
GLENDORA, CALIFORNIA

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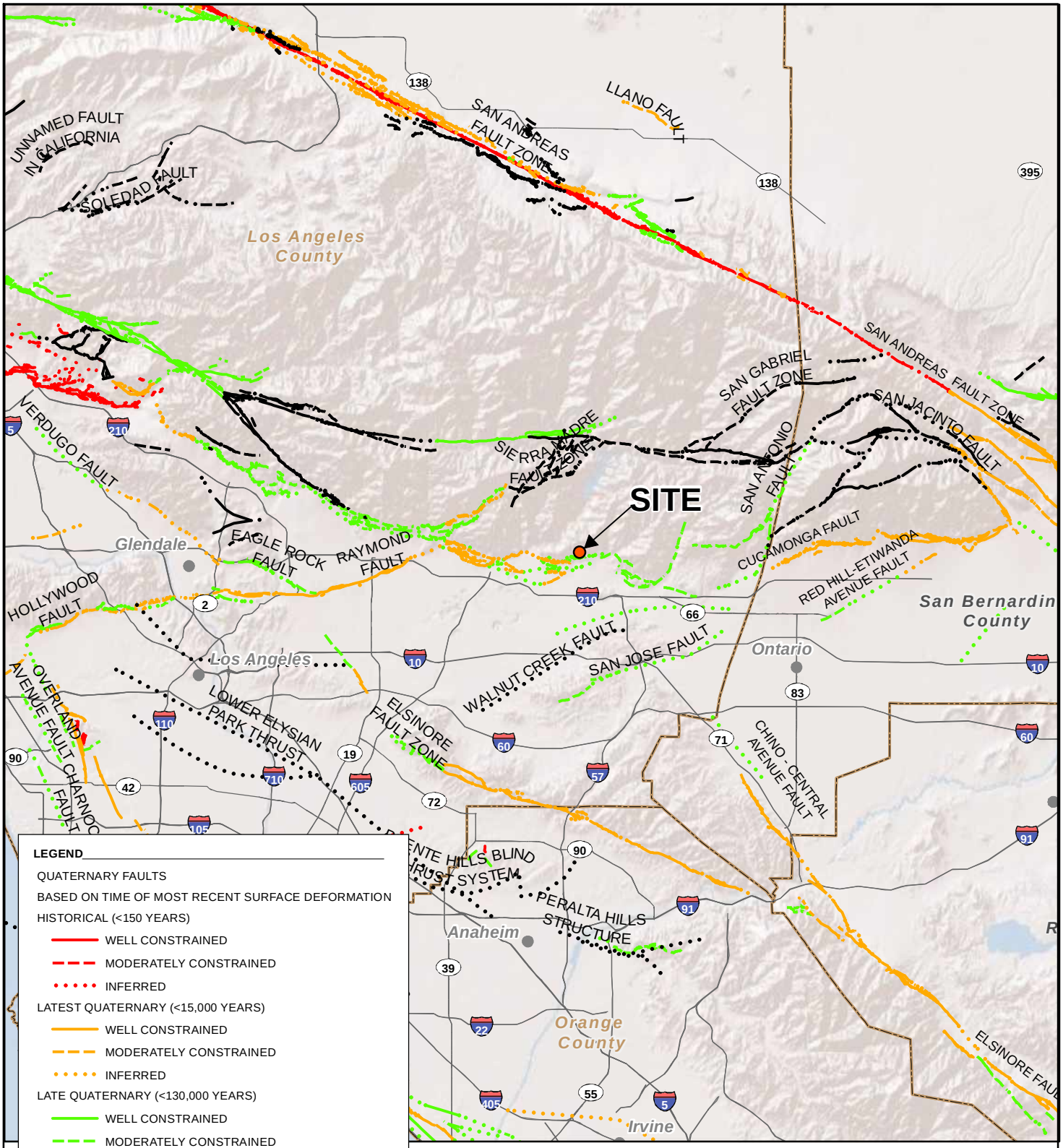

212016001_FL.mxd 4/27/2023 JDP

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

SOURCES: CALIFORNIA GEOLOGICAL SURVEY, ACCESSED OCTOBER 28, 2022, AT: https://www.usgs.gov/natural-hazards/earthquake-hazards/faults; ESRI, 2021.

FIGURE 6

FAULT LOCATIONS

NORTH RESERVOIRS 19BR AND 19C REPLACEMENT
GLENDORA, CALIFORNIA

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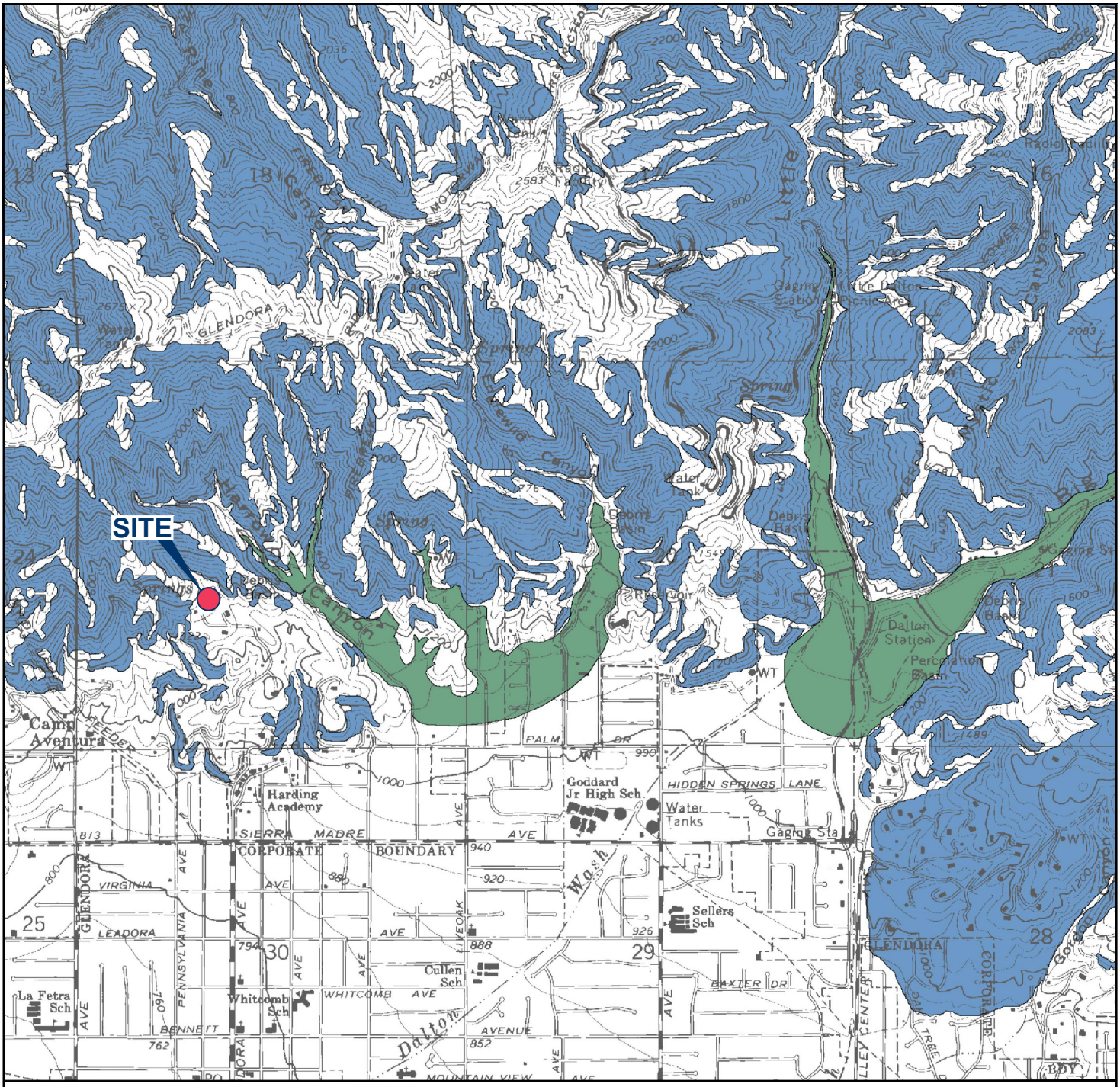

LEGEND

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: CDMG, 1999.

N

FEET
0 2,000 4,000

212016001_SHZ.dwg 04/27/2023 JDP

FIGURE 7

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SEISMIC HAZARD ZONES

NORTH RESERVOIRS 19BR AND 19C REPLACEMENT
GLENDORA, CALIFORNIA

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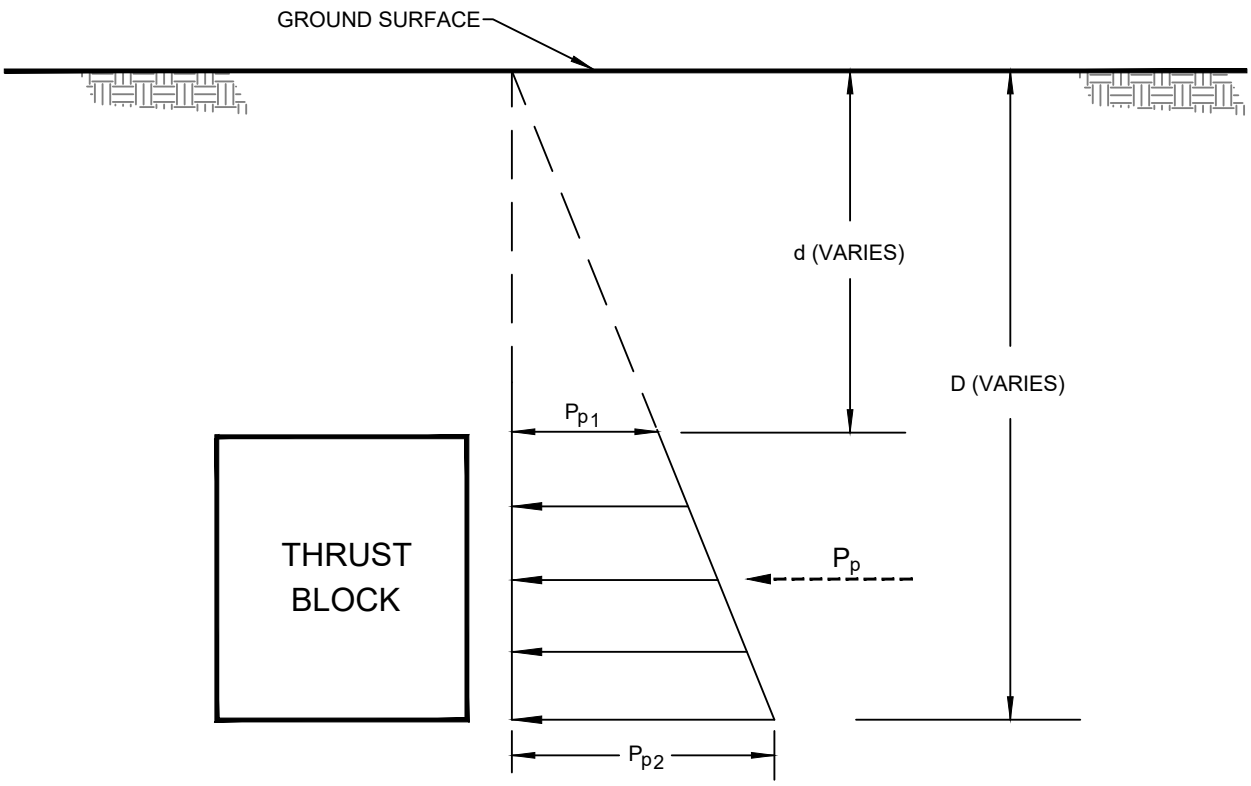

NOTES:

1. 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

FIGURE 8

THRUST BLOCK LATERAL EARTH PRESSURE DIAGRAM

NORTH RESERVOIRS 19BR AND 19C REPLACEMENT
GLENDORA, CALIFORNIA

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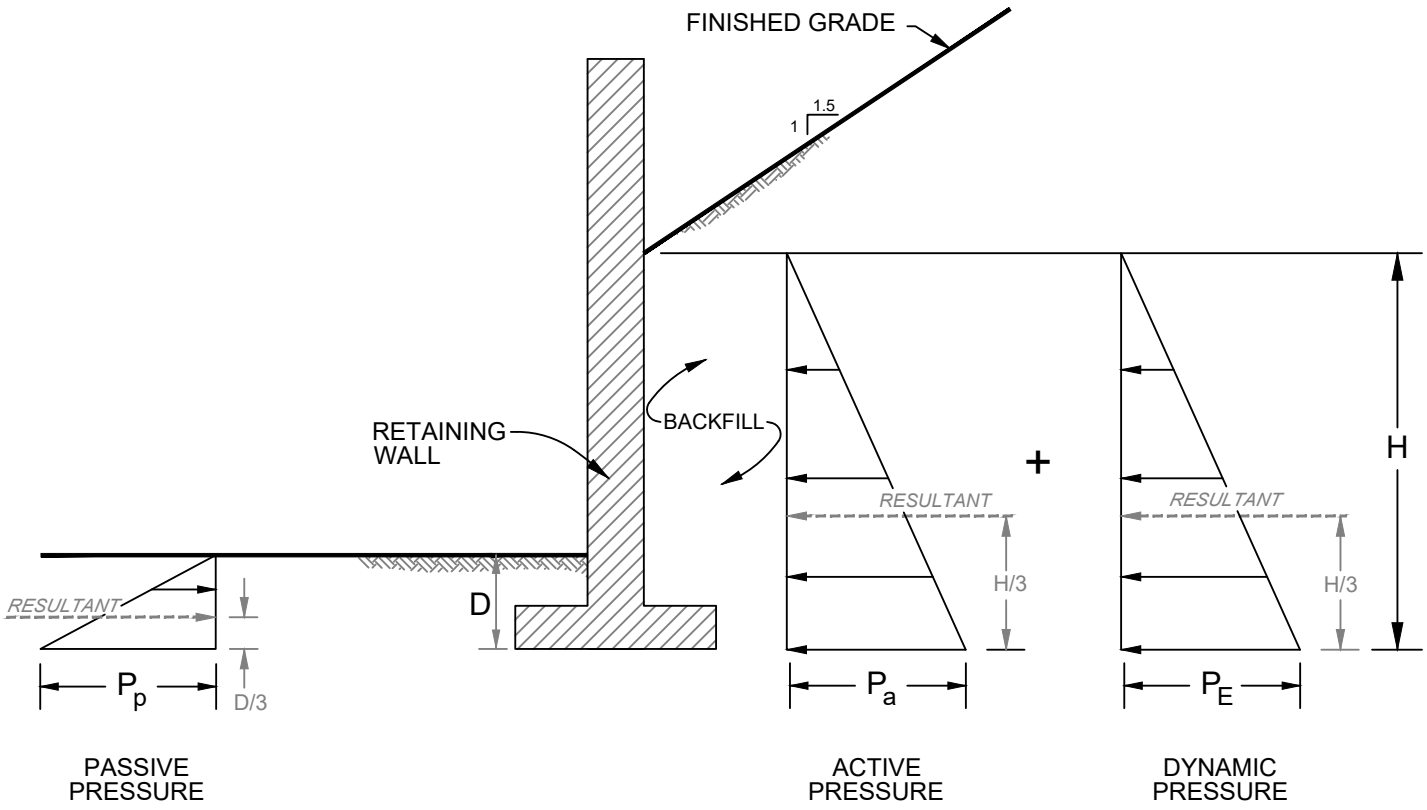
212016001_LEP-TB.dwg 04/27/2023



NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. STRUCTURAL, GRANULAR BACKFILL MATERIALS AS SPECIFIED IN GREENBOOK SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. DYNAMIC LATERAL EARTH PRESSURE IS BASED ON A MAPPED DESIGN PEAK GROUND ACCELERATION OF 0.58 g
5. P_E IS CALCULATED IN ACCORDANCE WITH THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), AND ATIK AND SITAR (2010)
6. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
7. H AND D ARE IN FEET

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure ($lb/ft^2/ft$)[1]	
P_a	Level Backfill with Granular Soils [2]	1.5H:1V Sloping Backfill with Granular Soils [2]
	37H	57H
P_E	20H	20H
P_p	Level Ground	
	350D	

NOT TO SCALE

212016001_LEP-YRW1.dwg 05/05/2023 JDP

FIGURE 9

LATERAL EARTH PRESSURES FOR YIELDING RETAINING WALLS

NORTH RESERVOIRS 19BR AND 19C REPLACEMENT
GLENDORA, CALIFORNIA

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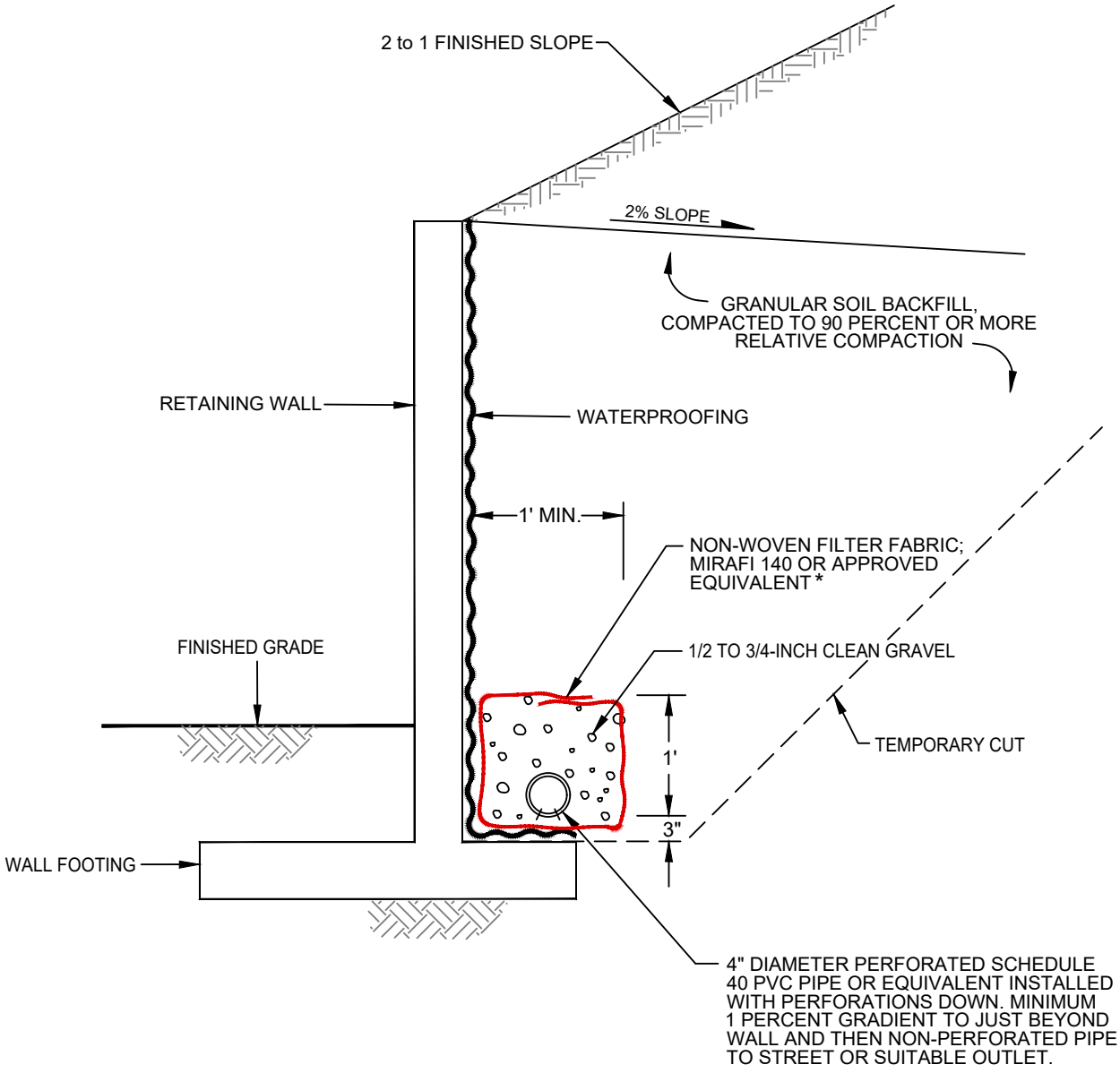

* AS AN ALTERNATIVE, CLASS 2 PERMEABLE FILTER MATERIAL MAY BE USED WITHOUT FILTER FABRIC IN LIEU OF GRAVEL.

NOT TO SCALE

212016001_RWDD1.dwg 04/27/2023

FIGURE 10

RETAINING WALL DRAIN DETAIL

NORTH RESERVOIRS 19BR AND 19C REPLACEMENT
GLENDORA, CALIFORNIA

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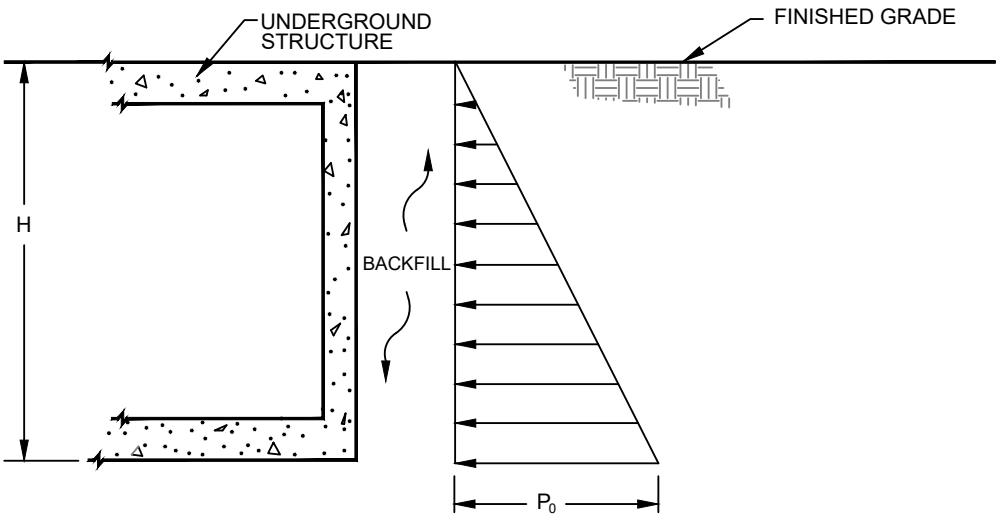

AT-REST PRESSURE

NOTES:

1. AT-REST LATERAL EARTH PRESSURE, $P_0 = 56H$
2. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
3. H IS IN FEET

NOT TO SCALE

FIGURE 11

LATERAL EARTH PRESSURES FOR UNDERGROUND STRUCTURES

NORTH RESERVOIRS 19BR AND 19C REPLACEMENT
GLENDORA, CALIFORNIA

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APPENDIX A

Test Pit Logs

APPENDIX A

TEST PIT LOGS

Field Procedure for the Collection of Disturbed Samples

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

Bulk Samples

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

LOCATION: North Reservoir 19B Replacement

GROUND ELEVATION: 1424± (msl)

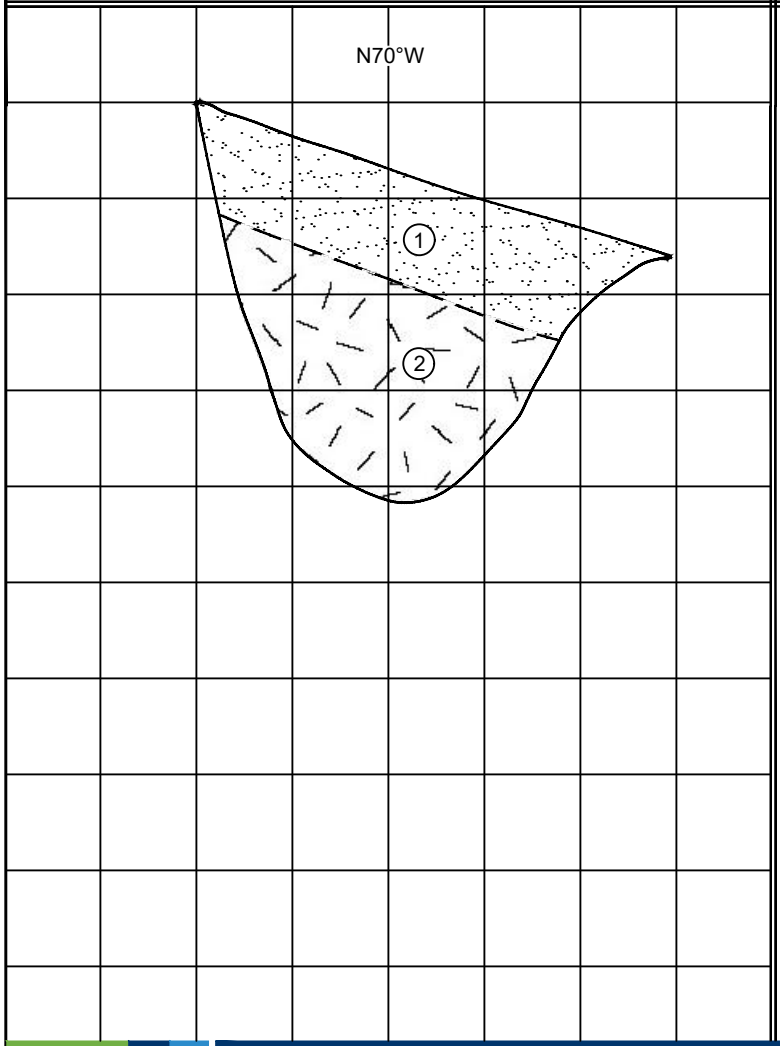
METHOD OF EXCAVATION: Kubota 080 Excavator

TEST PIT NO.: TP-1

LOGGED BY: MRH

DATE EXCAVATED: 9/20/22

TEST PIT DIAGRAM



DEPTH (FEET)	SAMPLES: Bulk	SAMPLES: Driven	SAMPLES: Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.
0						
2		X				
4		X				
6						
8						
10						
12						
14						
16						
18						

DESCRIPTION

SLOPEWASH (Qsw)

(1) Light yellowish brown, dry, loose to medium dense, silty SAND with rootlets, trace gravel.

@1': Becomes yellowish brown, moist, medium dense, silty SAND with gravel.

BEDROCK (Kgl)

(2) Yellowish brown to pale gray to white, moist, moderately soft, Granodiorite, weathered, moderately fractured and foliated, some fractures have soil in-fill near contact with overlying slopewash.

@2.5': Joint, N50W, 45NE.
@2.5': Foliation, N85E, 56NW.
@2.5': Joint, N19E, 52NW.
@5': Difficult excavating.

Bottom of Test Pit at 6 feet.
Test Pit backfilled 9/20/2022.
Groundwater not encountered during excavation. 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.

SCALE: 1"=4'

FIGURE A-1

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TEST PIT LOG

NORTH RESERVOIRS 19BR AND 19C REPLACEMENT
GLENDORA, CALIFORNIA

212016001 I 5/23

212016001_TP1.dwg 04/27/2023 JDP

LOCATION: North Reservoir 19B Replacement

GROUND ELEVATION: 1412± (msl)

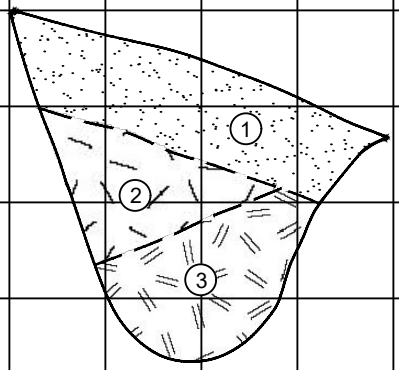
METHOD OF EXCAVATION: Kubota 080 Excavator

TEST PIT NO.: TP-2

LOGGED BY: MRH

DATE EXCAVATED: 9/20/22

TEST PIT DIAGRAM



DEPTH (FEET)	SAMPLES: Bulk	SAMPLES: Driven	SAMPLES: Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.
0						
2		X				
4						
6						
8						
10						
12						
14						
16						
18						

DESCRIPTION

SLOPEWASH (Qsw)

(1) Light yellowish brown, dry, loose to medium dense, silty SAND with rootlets, trace gravel.

@1': Becomes yellowish brown, moist, medium dense, silty SAND with gravel.

BEDROCK (Kgl)

(2) Yellowish brown to pale gray to white, moist, moderately soft, Granodiorite, weathered, moderately fractured and foliated, some fractures have soil in-fill near contact with overlying slopewash.

@3': Foliation, N40W, 64NE.
@3': Joint, N32W, 84NE.
@3': Joint, N75E, 65NW.
@5': Difficult excavating.

(3) White to light brown, moist, moderately strong, Aplite, weathered, fractures and joints not readily discernable.

Bottom of Test Pit at 6 feet.
Test Pit backfilled 9/20/2022.
Groundwater not encountered during excavation. 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.

SCALE: 1"=4'

212016001_TP2.dwg 04/27/2023 JDP

FIGURE A-2

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TEST PIT LOG

NORTH RESERVOIRS 19BR AND 19C REPLACEMENT
GLENDORA, CALIFORNIA

212016001 I 5/23

LOCATION: North Reservoir 19B Replacement

GROUND ELEVATION: 1403± (msl)

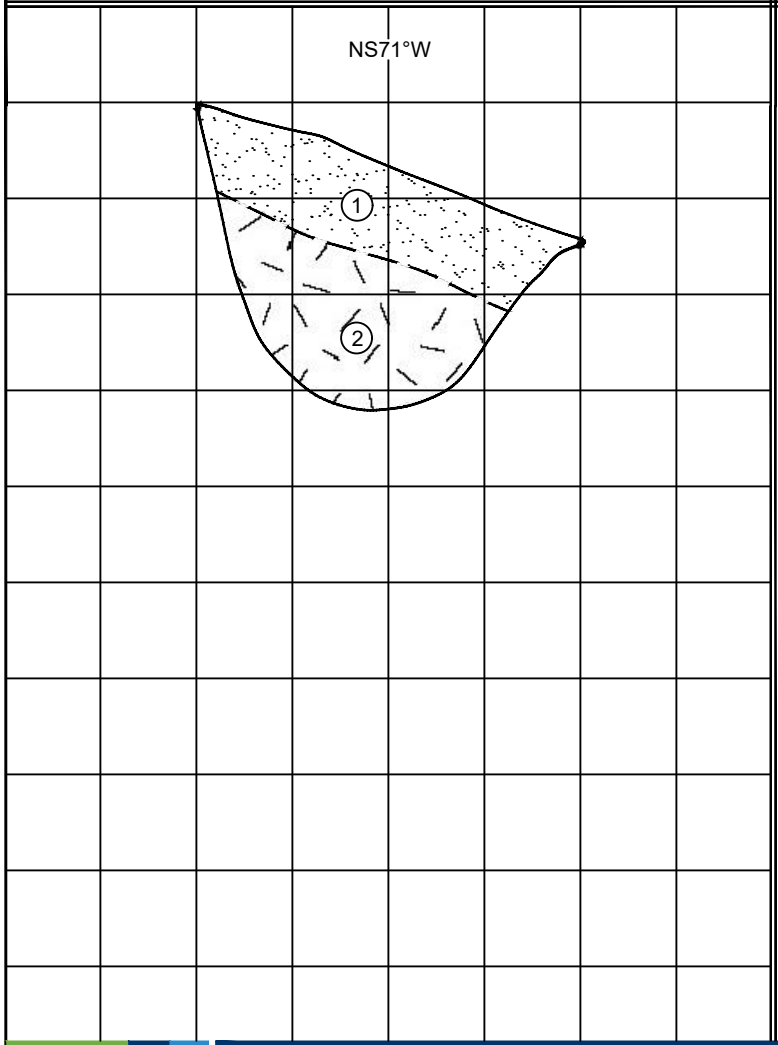
METHOD OF EXCAVATION: Kubota 080 Excavator

TEST PIT NO.: TP-3

LOGGED BY: MRH

DATE EXCAVATED: 9/20/22

TEST PIT DIAGRAM



DEPTH (FEET)	SAMPLES: Bulk	SAMPLES: Driven	SAMPLES: Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.
0						
2						
4						
6						
8						
10						
12						
14						
16						
18						

DESCRIPTION

SLOPEWASH (Qsw)

(1) Light yellowish brown, dry, loose to medium dense, silty SAND with rootlets, trace gravel.

@1': Becomes yellowish brown, moist, medium dense, silty SAND with gravel.

BEDROCK (Kgl)

(2) Yellowish brown to pale gray to white, moist, moderately soft, Granodiorite, weathered, moderately fractured and foliated, some fractures have soil in-fill near contact with overlying slopewash.

@2.5': Foliation, N25W, 53SW.
@2.5': Joint, N50E, 42SE.
@5': Difficult excavating.

Bottom of Test Pit at 5.5 feet.
Test Pit backfilled 9/20/2022.
Groundwater not encountered during excavation. 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.

SCALE: 1"=4'

212016001_TP3.dwg 04/27/2023 JDP

FIGURE A-3

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TEST PIT LOG

NORTH RESERVOIRS 19BR AND 19C REPLACEMENT
GLENDORA, CALIFORNIA

212016001 I 5/23

LOCATION: North Reservoir 19B Replacement

GROUND ELEVATION: 1400± (msl)

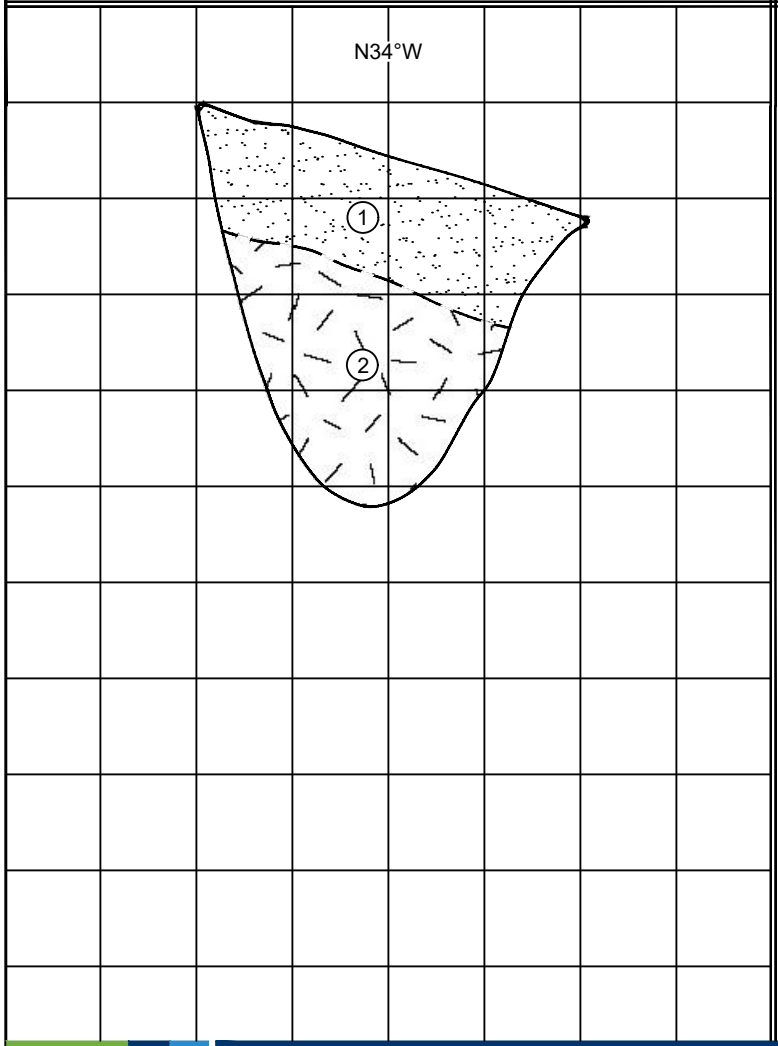
METHOD OF EXCAVATION: Kubota 080 Excavator

TEST PIT NO.: TP-4

LOGGED BY: MRH

DATE EXCAVATED: 9/20/22

TEST PIT DIAGRAM



DEPTH (FEET)	SAMPLES: Bulk	SAMPLES: Driven	SAMPLES: Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.
0						
2						
4						
6						
8						
10						
12						
14						
16						
18						

DESCRIPTION

SLOPEWASH (Qsw)

(1) Light yellowish brown, dry, loose to medium dense, silty SAND with rootlets, trace gravel.

@1': Becomes yellowish brown, moist, medium dense, silty SAND with gravel.

BEDROCK (Kgl)

(2) Yellowish brown to pale gray to white, moist, moderately soft, Granodiorite, weathered, moderately fractured, weakly foliated, some fractures have soil in-fill near contact with overlying slopewash.

@2.5': Joint, N24W, 33SW.
@2.5': Joint, N10E, 33SE.
@6': Difficult excavating.

Bottom of Test Pit at 7 feet.
Test Pit backfilled 9/20/2022.
Groundwater not encountered during excavation. 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.

SCALE: 1"=4'

212016001_TP4.dwg 04/27/2023 JDP

FIGURE A-4

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TEST PIT LOG

NORTH RESERVOIRS 19BR AND 19C REPLACEMENT
GLENDORA, CALIFORNIA

212016001 I 5/23

APPENDIX B

Laboratory Testing

APPENDIX B

LABORATORY TESTING

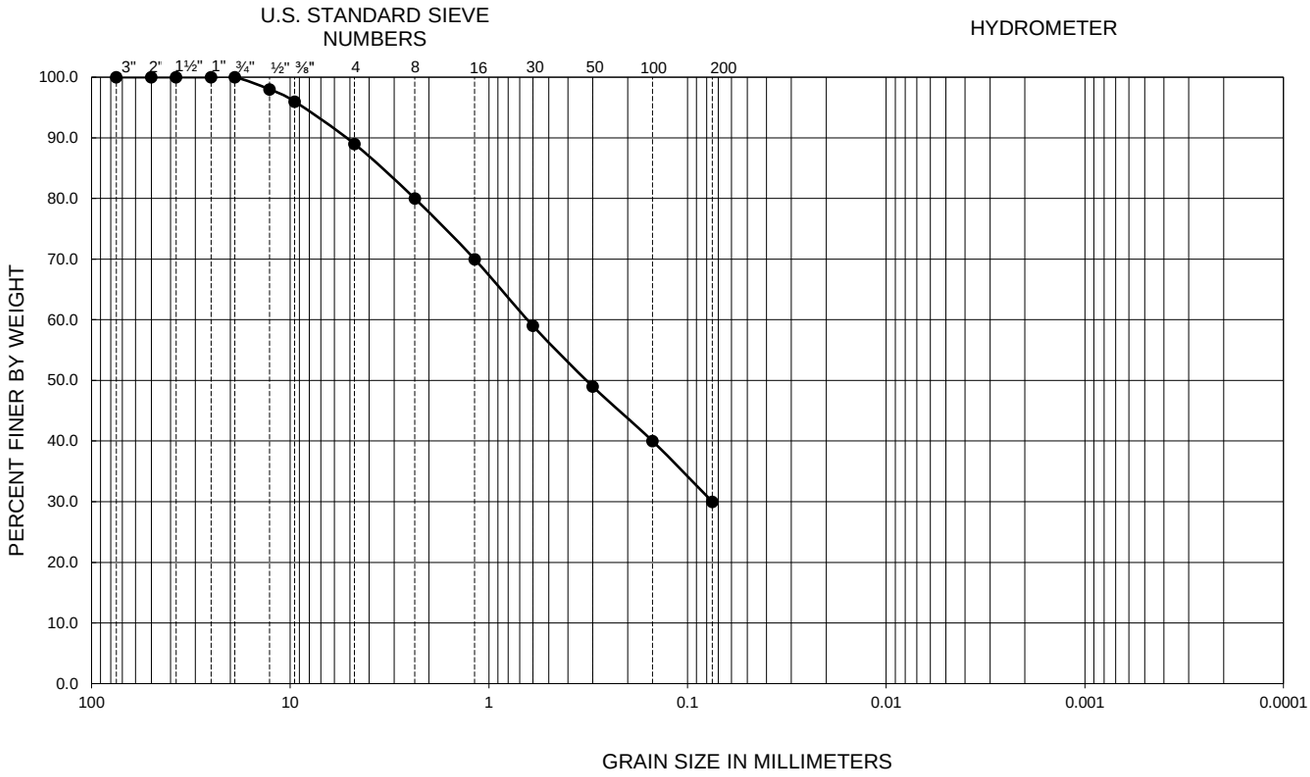
Classification
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

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 Figures B-1 and B-2. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

Atterberg Limits
Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the USCS. The test results and classifications are shown on Figure B-3.

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-4.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	TP-3	0.0-1.0	25	24	1	--	--	--	--	--	30	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

FIGURE B-1

GRADATION TEST RESULTS

NORTH RESERVOIRS 19BR AND 19C REPLACEMENT
GLENDORA, CALIFORNIA

212016001 | 5/23

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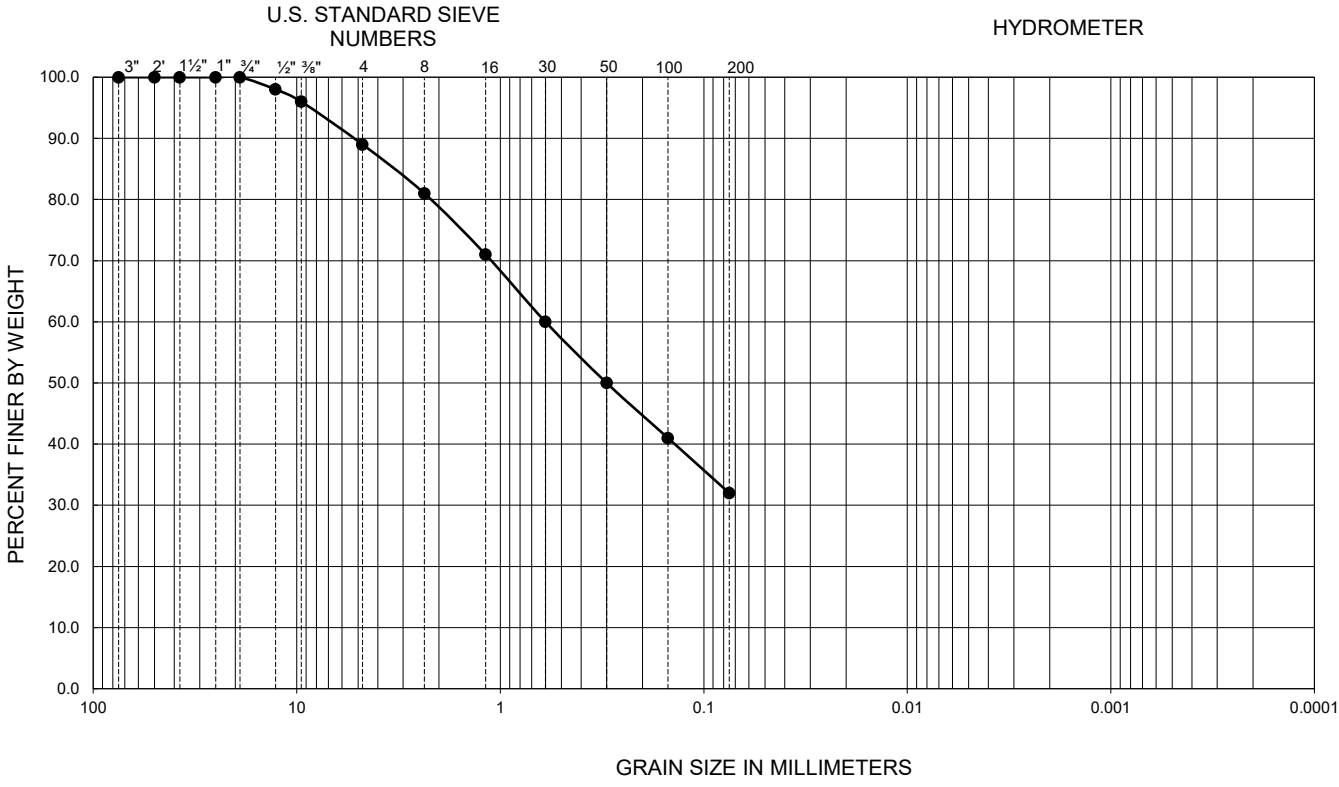
Geotechnical & Environmental Sciences Consultants

212016001 Fig B-1 SIEVE w No 8 @ TP-3 0.0-1.0

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	TP-3	1.0-2.0	--	--	--	--	--	--	--	--	32	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

FIGURE B-2

GRADATION TEST RESULTS

NORTH RESERVOIRS 19BR AND 19C REPLACEMENT
GLENDORA, CALIFORNIA

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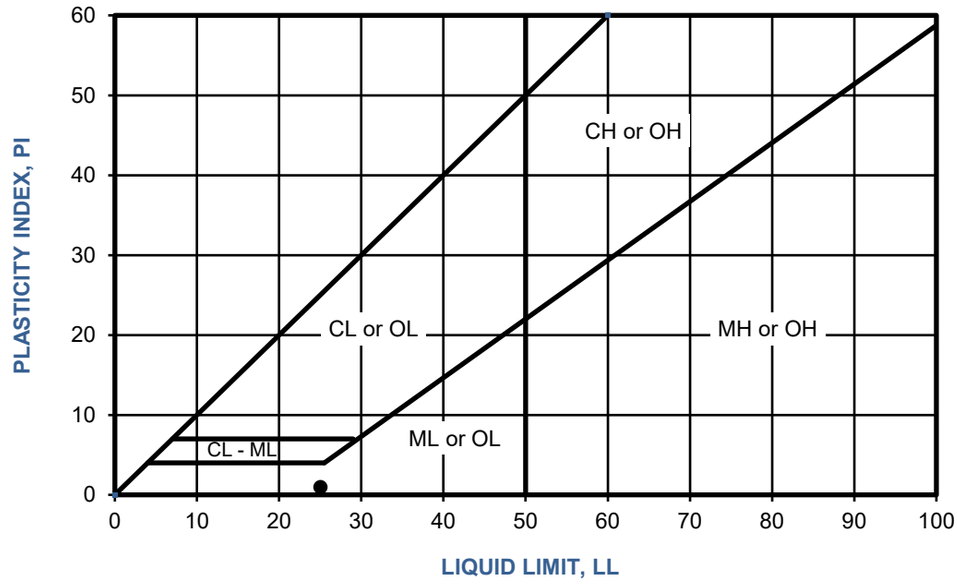
SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	TP-3	0.0-1.0	25	24	1	ML	SM

NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

FIGURE B-3

ATTERBERG LIMITS TEST RESULTS

NORTH RESERVOIRS 19BR AND 19C REPLACEMENT
GLENDORA, CALIFORNIA

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
CS-1	0.0-1.0	5.5	3,232	10	0.001	85

[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-4

CORROSIVITY TEST RESULTS

NORTH RESERVOIRS 19BR AND 19C REPLACEMENT
GLENDORA, CALIFORNIA

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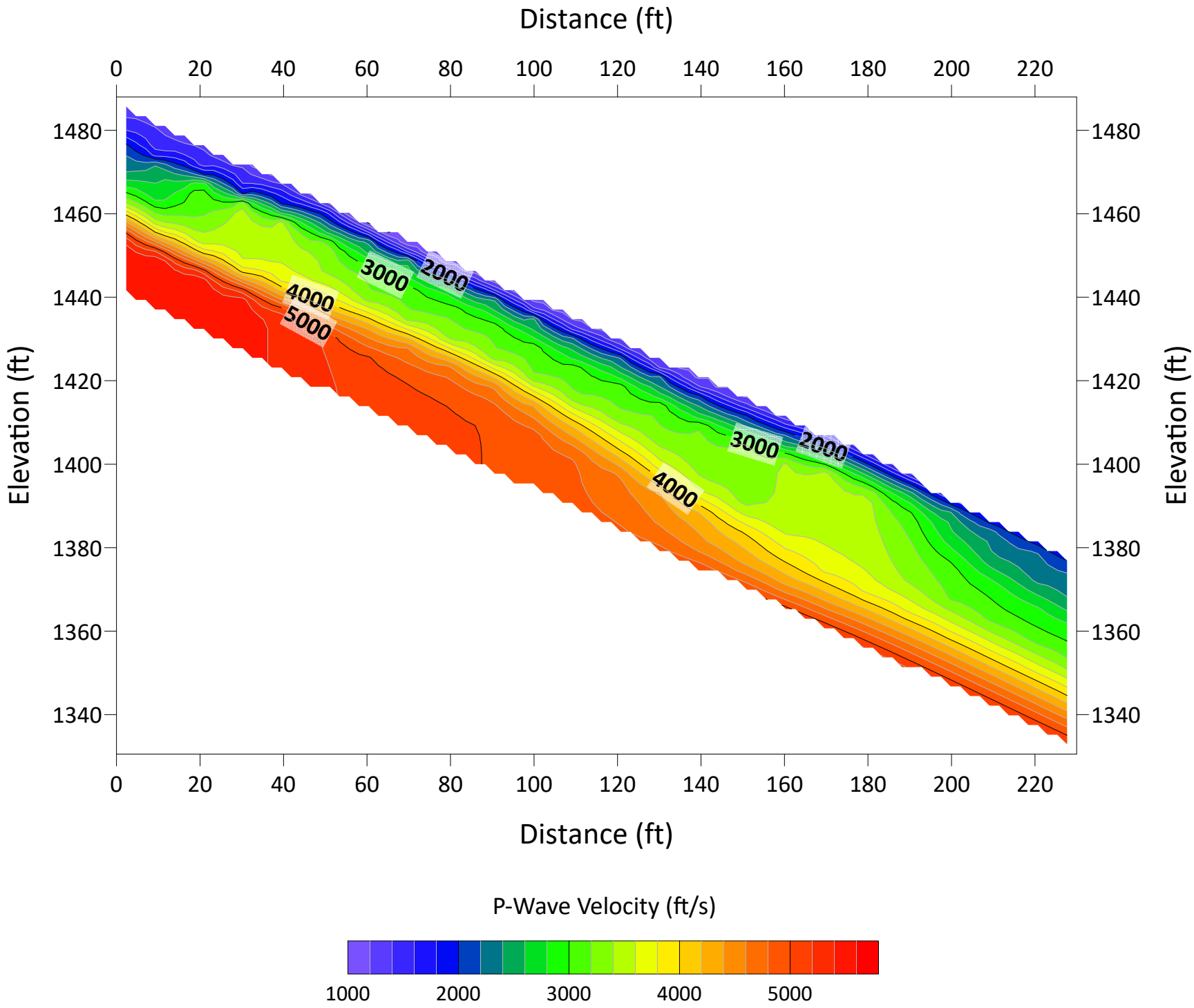
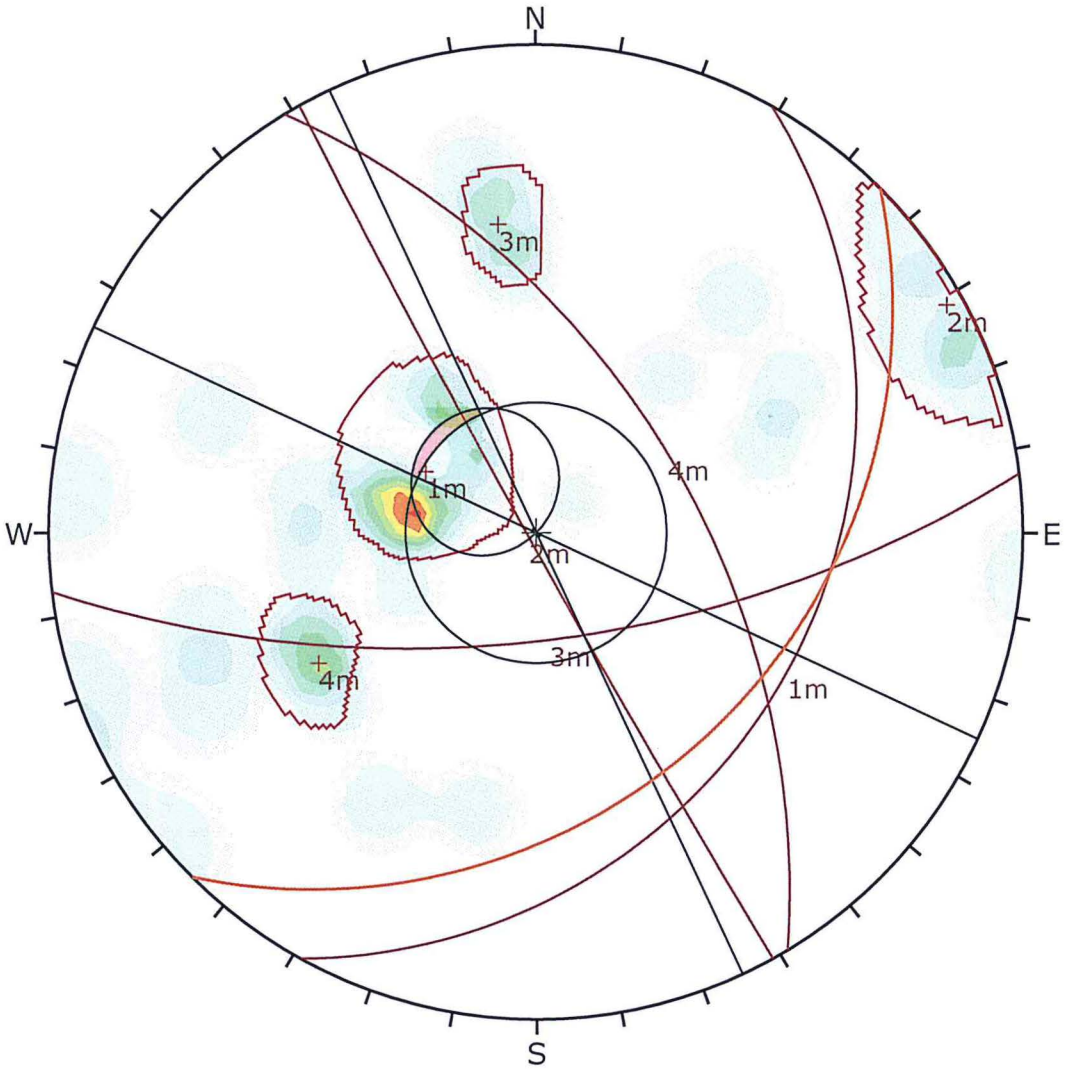
APPENDIX C

Seismic Refraction Profile


Distance (ft)
0
20
40
60
80
100
120
140
160
180
200
220
Elevation (ft)
1480
1460
1440
1420
1400
1380
1360
1340
2000
3000
4000
5000
P-Wave Velocity (ft/s)
1000
2000
3000
4000
5000

APPENDIX D

Kinematic Analyses



Color	Density Concentrations
	0.00 - 1.20
	1.20 - 2.40
	2.40 - 3.60
	3.60 - 4.80
	4.80 - 6.00
	6.00 - 7.20
	7.20 - 8.40
	8.40 - 9.60
	9.60 - 10.80
	10.80 - 12.00

Contour Data	Pole Vectors
Maximum Density	11.09%
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Planar Sliding		
Slope Dip	33		
Slope Dip Direction	135		
Friction Angle	30°		
Lateral Limits	20°		
	Critical	Total	%
Planar Sliding (All)	0	32	0.00%

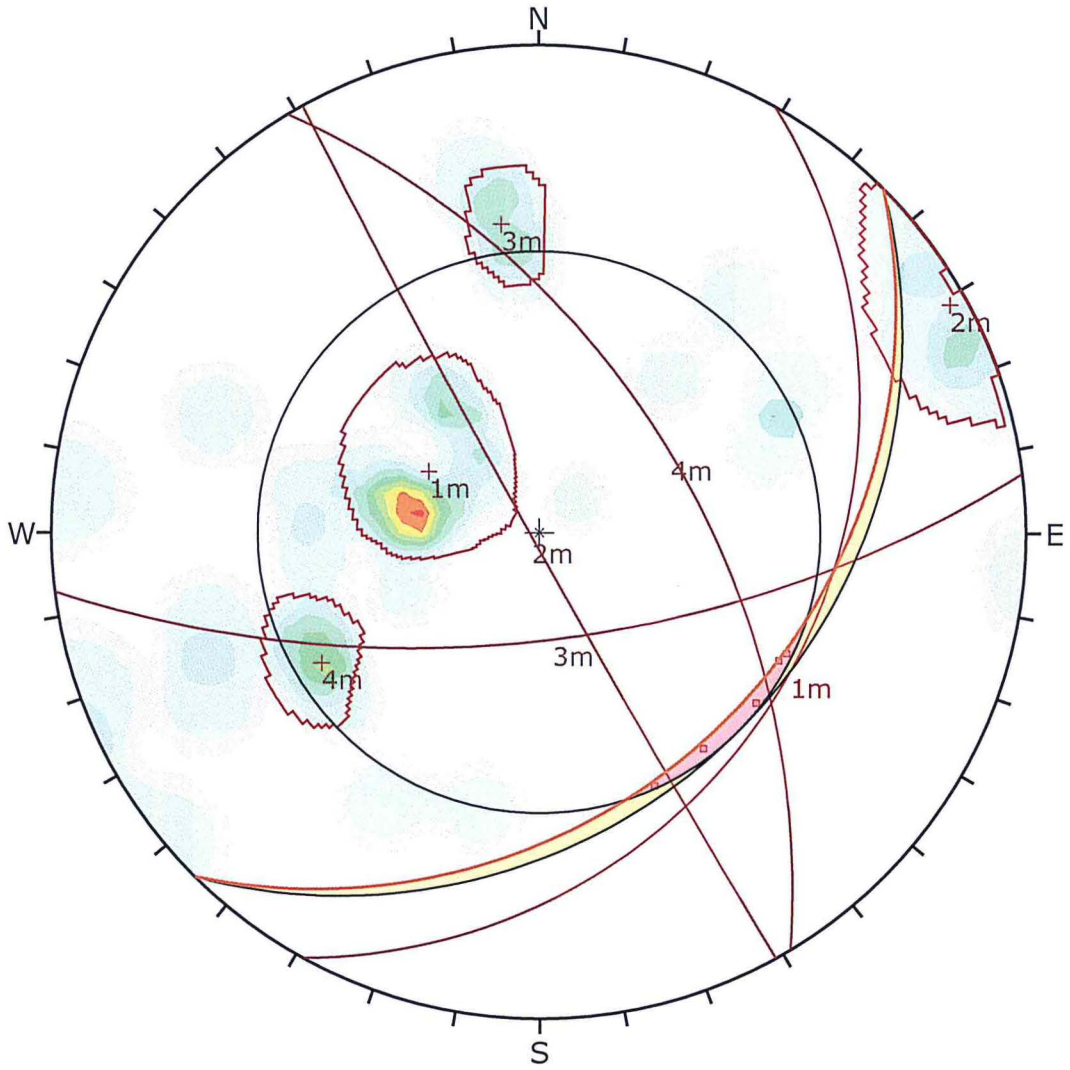
	Color	Dip	Dip Direction	Label
Mean Set Planes				
1m		29	119	
2m		88	241	
3m		65	173	
4m		55	59	

Plot Mode	Pole Vectors
Vector Count	32 (32 Entries)
Hemisphere	Lower
Projection	Equal Angle

rocscience

DIPS 8.021

Project	AKM/North Reservoir 19B Replacement, Glendora, California		
Analysis Description	1.5:1 Cut slope in Granitic Rock		
Drawn By	DBC	Company	Ninyo & Moore
Date	11/3/2022, 11:02:25 AM	File Name	AA-1.dips8



Symbol	Feature
▫	Critical Intersection

Color	Density Concentrations
	0.00 - 1.20
	1.20 - 2.40
	2.40 - 3.60
	3.60 - 4.80
	4.80 - 6.00
	6.00 - 7.20
	7.20 - 8.40
	8.40 - 9.60
	9.60 - 10.80
	10.80 - 12.00

Contour Data	Pole Vectors
Maximum Density	11.09%
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Wedge Sliding
Slope Dip	33
Slope Dip Direction	135
Friction Angle	30°

	Critical	Total	%
Wedge Sliding	7	496	1.41%

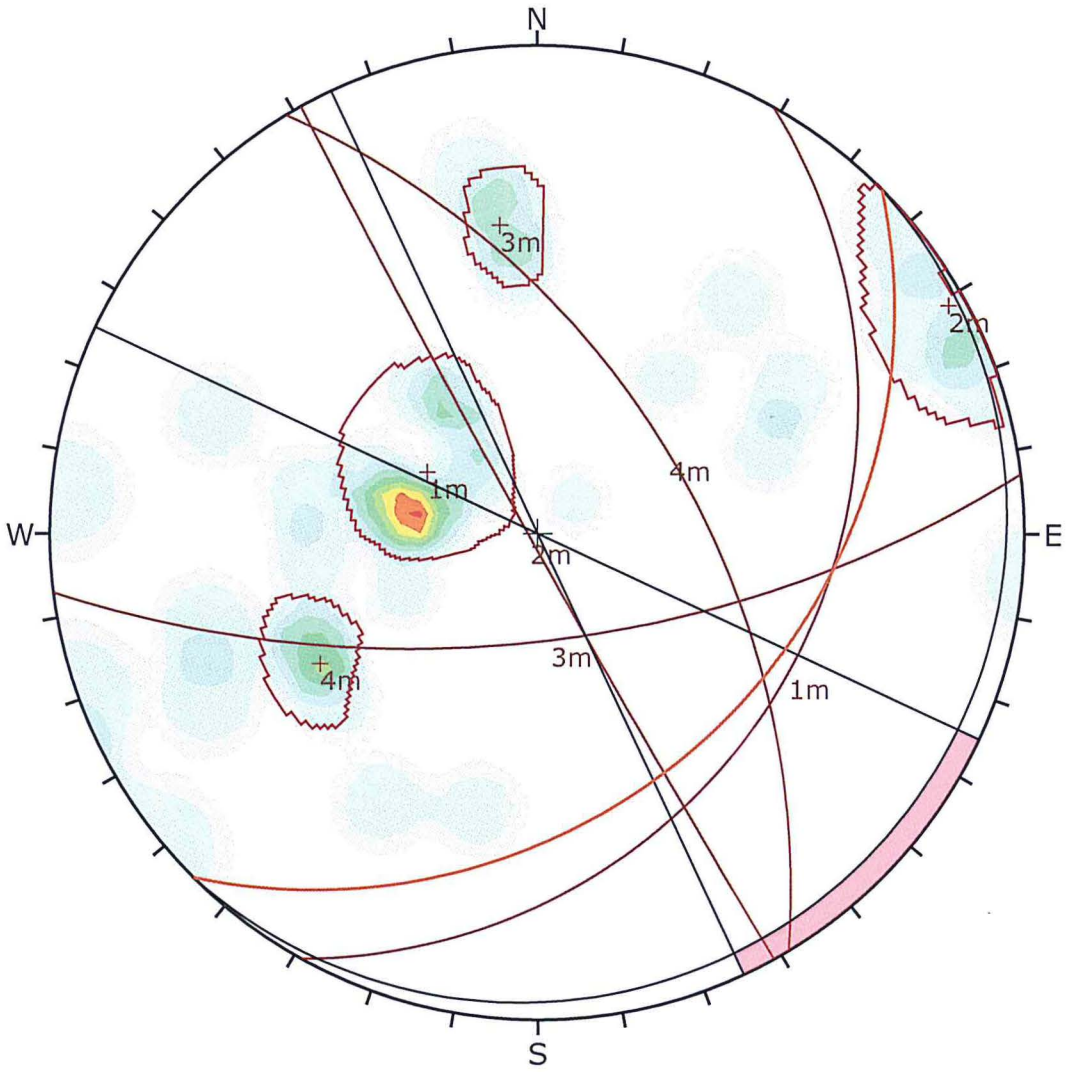
	Color	Dip	Dip Direction	Label
Mean Set Planes				
1m		29	119	
2m		88	241	
3m		65	173	
4m		55	59	

Plot Mode	Pole Vectors
Vector Count	32 (32 Entries)
Intersection Mode	Grid Data Planes
Intersections Count	496
Hemisphere	Lower
Projection	Equal Angle

rocscience

DIPS 8.021

Project	AKM/North Reservoir 19B Replacement, Glendora, California		
Analysis Description	1.5:1 Cut slope in Granitic Rock		
Drawn By	DBC	Company	Ninyo & Moore
Date	11/3/2022, 11:02:25 AM	File Name	AA-1.dips8



Color	Density Concentrations
	0.00 - 1.20
	1.20 - 2.40
	2.40 - 3.60
	3.60 - 4.80
	4.80 - 6.00
	6.00 - 7.20
	7.20 - 8.40
	8.40 - 9.60
	9.60 - 10.80
	10.80 - 12.00

Contour Data	Pole Vectors
Maximum Density	11.09%
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Flexural Toppling
Slope Dip	33
Slope Dip Direction	135
Friction Angle	30°
Lateral Limits	20°

	Critical	Total	%
Flexural Toppling (All)	0	32	0.00%

	Color	Dip	Dip Direction	Label
Mean Set Planes				
1m		29	119	
2m		88	241	
3m		65	173	
4m		55	59	

Plot Mode	Pole Vectors
Vector Count	32 (32 Entries)
Hemisphere	Lower
Projection	Equal Angle

rocscience

DIPS 8.021

Project	AKM/North Reservoir 19B Replacement, Glendora, California		
Analysis Description	1.5:1 Cut slope in Granitic Rock		
Drawn By	DBC	Company	Ninyo & Moore
Date	11/3/2022, 11:02:25 AM	File Name	AA-1.dips8

Dips Analysis Information

AKM/North Reservoir 19B Replacement, Glendora, California

Project Summary

File Name: AA-1
Last saved with Dips version: 8.021
Project Title: AKM/North Reservoir 19B Replacement, Glendora, California
Analysis: 1.5:1 Cut slope in Granitic Rock
Author: DBC
Company: Ninyo & Moore
Date Created: 11/3/2022, 11:02:25 AM

General Settings

Data Format: Dip / Dip Direction
Magnetic Declination (E pos): 0°
Multiple Data Flag (Quantity): OFF
Distance Column: OFF
Extra Data Columns: 0
Units: Metric
Poles: 32
Entries: 32

Traverses

No traverse information available.

Global Mean

	Dip	Dip Direction
Unweighted	25.80	124.65
Weighted	25.80	124.65

Global Best Fit

Unweighted			
	Dip	Dip Direction	Eigenvalue
S1	44.73	86.52	0.554791
S2	59.25	213.43	0.341614
S3	61.08	322.62	0.103595

Woodcock S1 / S3 = 5.355
Woodcock K = 0.406
Woodcock C = 1.678

	Weighted		
	Dip	Dip Direction	Eigenvalue
S1	44.73	86.52	0.554791
S2	59.25	213.43	0.341614
S3	61.08	322.62	0.103595

Woodcock S1 / S3 = 5.355
Woodcock K = 0.406
Woodcock C = 1.678

Mean Set Planes

ID	Dip	Dip Direction	Label
1m	29.12	119.21	
2m	87.98	241.00	
3m	64.65	173.22	
4m	54.83	58.93	

Set Statistics

Set: 1m:

Poles: 9
Entries: 9
Fisher's K: 33.7139

	68.26%	95.44%	99.74%	50%
Variability Limit	14.9922°	24.7134°	34.5685°	11.6384°
Confidence Limit	5.05182°	8.29126°	11.5209°	3.92565°

Set: 2m:

Poles: 3
Entries: 3
Fisher's K: 53.5474

	68.26%	95.44%	99.74%	50%
Variability Limit	11.8834°	19.5527°	27.272°	9.22892°
Confidence Limit	6.89579°	11.323°	15.7448°	5.35796°

Set: 3m:

Poles: 3
Entries: 3
Fisher's K: 82.9364

	68.26%	95.44%	99.74%	50%
Variability Limit	9.54247°	15.6837°	21.8392°	7.41277°
Confidence Limit	5.52738°	9.07274°	12.6088°	4.29508°

Set: 4m:

Poles: 3
Entries: 3
Fisher's K: 145.198

	68.26%	95.44%	99.74%	50%
Variability Limit	7.20838°	11.8374°	16.4624°	5.60071°
Confidence Limit	4.16952°	6.84198°	9.50461°	3.24016°

Set Windows

Set ID	Window	Type	Wrapped
1	1a	Freehand	No
2	2a	Freehand	No
3	3a	Freehand	No
4	4a	Freehand	No

Intersections

Intersection Type	Number
Grid Data Planes	496
All Set Planes	108
Set 1 vs Set 2 Planes	27
Set 1 vs Set 3 Planes	27
Set 1 vs Set 4 Planes	27
Set 2 vs Set 3 Planes	9
Set 2 vs Set 4 Planes	9
Set 3 vs Set 4 Planes	9
User and Mean Set (Unweighted) Planes	6
User Planes	0
Mean Set (Unweighted) Planes	6

Kinematic Analysis

Slope Dip: 33
Slope Dip Direction: 135
Friction Angle: 30°
Lateral Limit Angle: 20°

Planar Sliding

Planar Sliding	Critical	%	Total
All Vectors	0	0.00%	32

Planar Sliding (No Limits)

Planar Sliding	Critical	%	Total
All Vectors	0	0.00%	32

Wedge Sliding

Critical 1 = Wedge Sliding (Both Planes)
Critical 2 = Wedge Sliding (One Plane)

Intersection Type	Critical 1	%	Critical 2	%	Total
Grid Data Plane Intersections	7	1.41%	0	0.00%	496
All Set Planes	3	2.78%	0	0.00%	108
Set 1 vs Set 2 Planes	1	3.70%	0	0.00%	27
Set 1 vs Set 3 Planes	0	0.00%	0	0.00%	27
Set 1 vs Set 4 Planes	2	7.41%	0	0.00%	27
Set 2 vs Set 3 Planes	0	0.00%	0	0.00%	9
Set 2 vs Set 4 Planes	0	0.00%	0	0.00%	9
Set 3 vs Set 4 Planes	0	0.00%	0	0.00%	9
User and Mean Set (Unweighted) Plane Intersections	0	0.00%	0	0.00%	6
User Plane Intersections	No results				
Mean Set Plane (Unweighted) Intersections	0	0.00%	0	0.00%	6

Flexural Toppling

Flexural Toppling	Critical	%	Total
All Vectors	0	0.00%	32

Direct Toppling

Base Plane	Critical	%	Total
All Vectors	4	12.50%	32
Set 1	4	44.44%	9

Critical 1 = Direct Toppling (Intersection)
Critical 2 = Oblique Toppling (Intersection)

Intersection Type	Critical 1	%	Critical 2	%	Total
Grid Data Plane Intersections	1	0.20%	6	1.21%	496
All Set Planes	0	0.00%	0	0.00%	108
Set 1 vs Set 2 Planes	0	0.00%	0	0.00%	27
Set 1 vs Set 3 Planes	0	0.00%	0	0.00%	27
Set 1 vs Set 4 Planes	0	0.00%	0	0.00%	27
Set 2 vs Set 3 Planes	0	0.00%	0	0.00%	9
Set 2 vs Set 4 Planes	0	0.00%	0	0.00%	9
Set 3 vs Set 4 Planes	0	0.00%	0	0.00%	9
User and Mean Set (Unweighted) Plane Intersections	0	0.00%	0	0.00%	6
User Plane Intersections	No results				
Mean Set Plane (Unweighted) Intersections	0	0.00%	0	0.00%	6

APPENDIX E

Slope Stability Analyses

AKM/North Reservoir 19B Replacement, Glendora, CA Cross section A-A'

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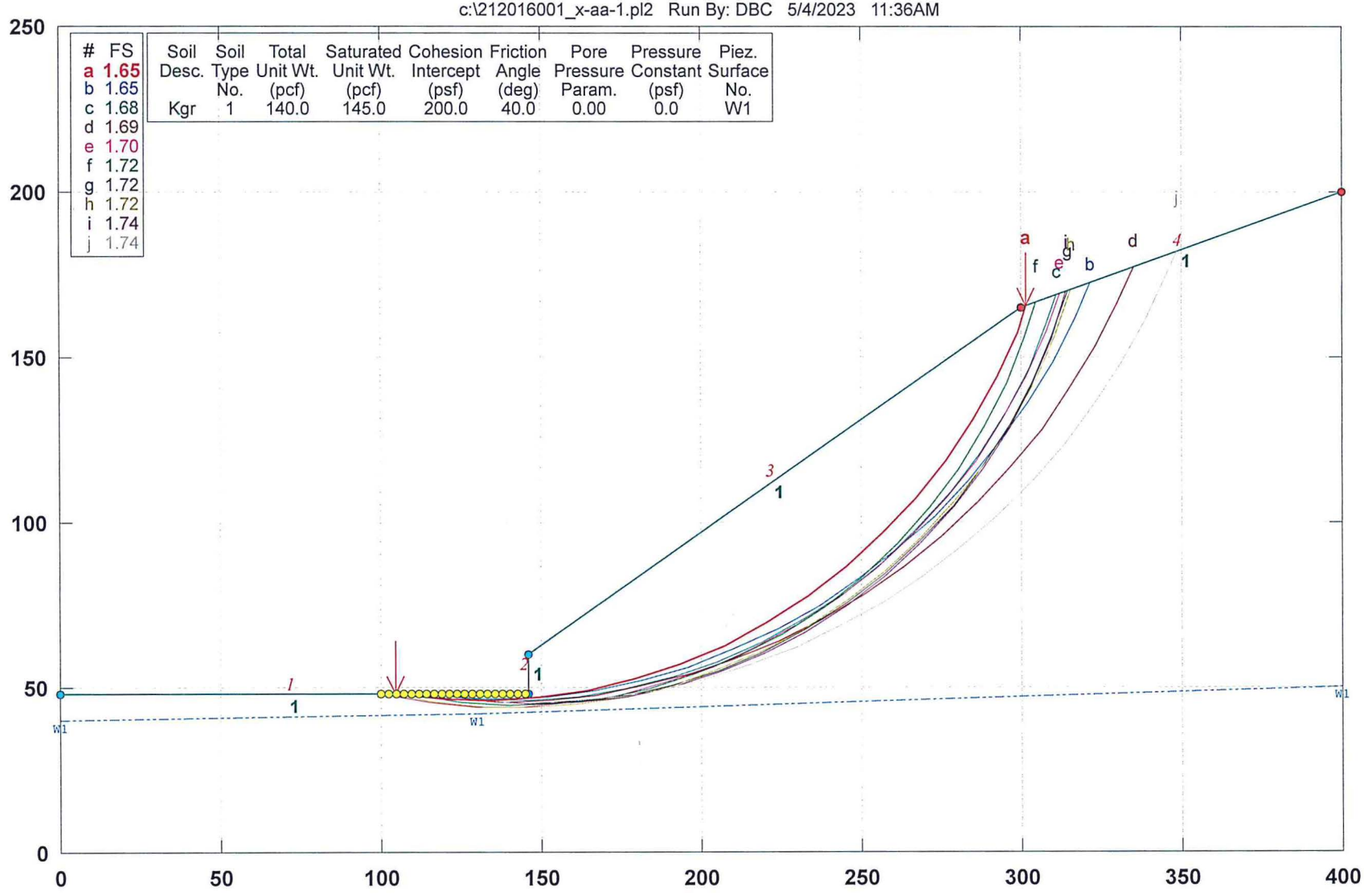
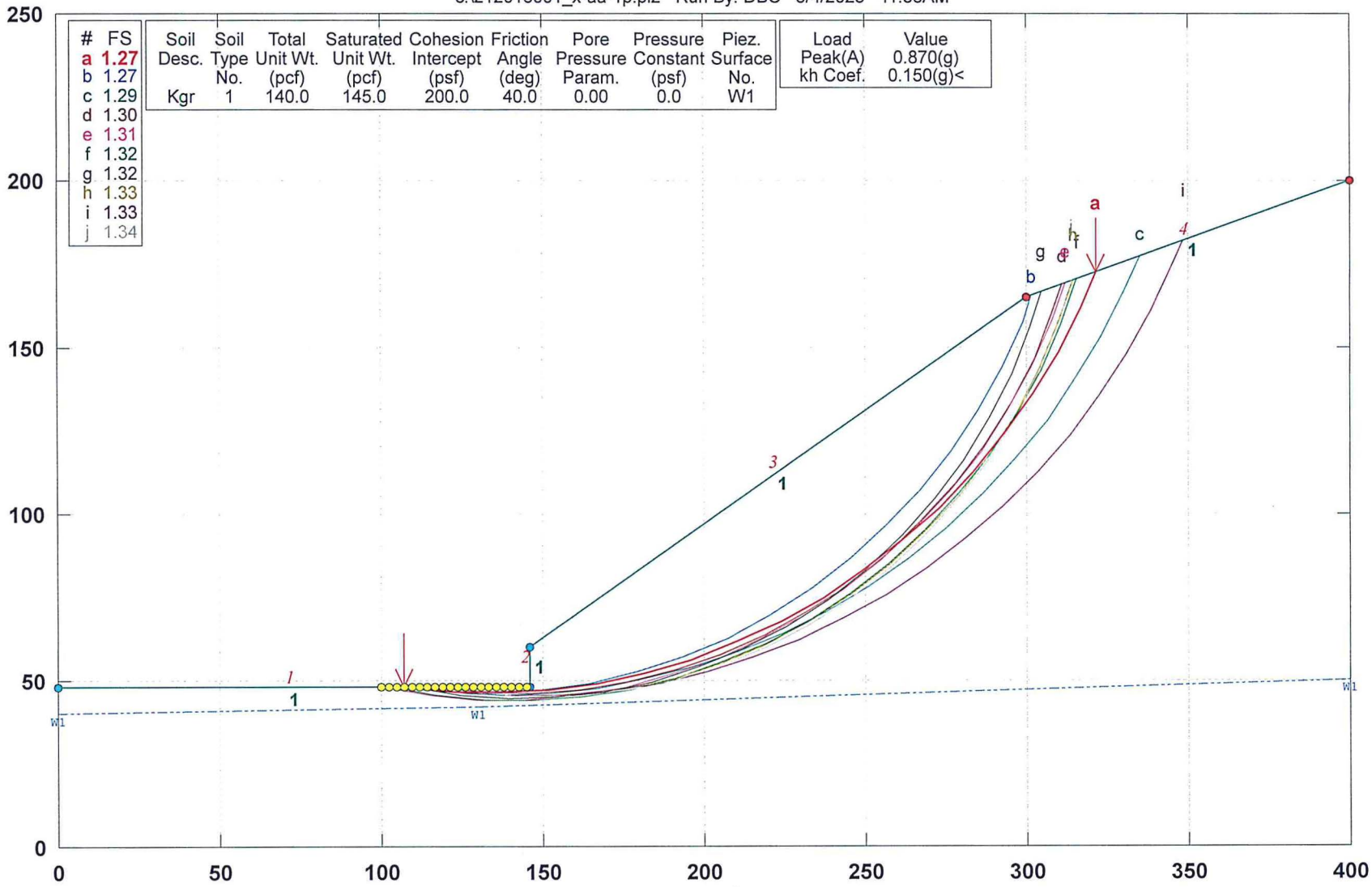


GSTABL7 v.2 FSmin=1.65

Safety Factors Are Calculated By The Modified Bishop Method

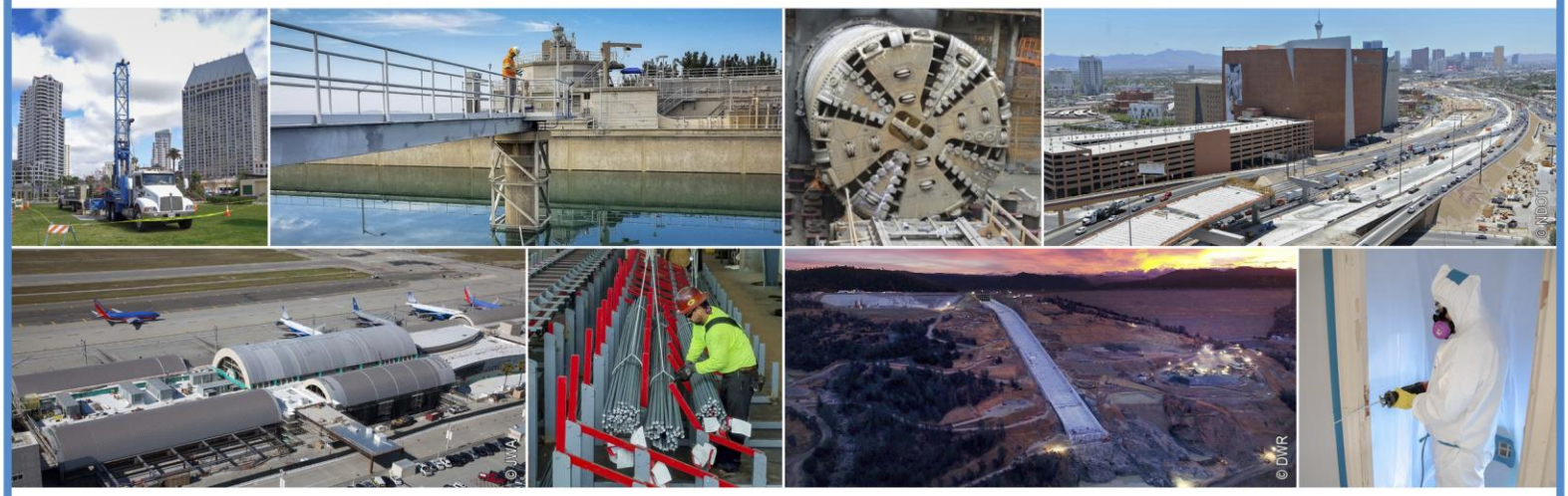
AKM/North Reservoir 19B Replacement, Glendora, CA Cross section A-A', pseudo

c:\212016001_x-aa-1p.pl2 Run By: DBC 5/4/2023 11:38AM



GSTABL7 v.2 FSmin=1.27

Safety Factors Are Calculated By The Modified Bishop Method



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Geotechnical & Environmental Sciences Consultants