La Jolla View Reservoir Project
Environmental Impact Report

SCH No. 2018041020 - Project No. 331101

Appendix I

Geotechnical Evaluation

February 2020

GEOTECHNICAL EVALUATION
LA JOLLA VIEW RESERVOIR
REPLACEMENT PROJECT
LA JOLLA, CALIFORNIA

PREPARED FOR:

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PREPARED BY:

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July 11, 2014
Project No. 107314001

Ninyo & Moore
Geotechnical and Environmental Sciences Consultants

July 11, 2014
Project No. 107314001

Mr. Anders Egense
Infrastructure Engineering Corporation
14271 Danielson Street
Poway, California 92064

Subject: Geotechnical Evaluation
La Jolla View Reservoir Replacement Project
La Jolla, California

Dear Mr. Egense:

In accordance with your authorization, we have performed a geotechnical evaluation for the City of San Diego's La Jolla View Reservoir Replacement Project in La Jolla, California. This report presents our geotechnical findings, conclusions, and recommendations regarding the proposed project. Our report was prepared in accordance with our revised proposal dated August 27, 2012.

We appreciate the opportunity to be of service on this project.

Sincerely,
NINYO & MOORE

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1. INTRODUCTION

In accordance with your request, we have performed a geotechnical evaluation for the proposed La Jolla View Reservoir Replacement Project located in La Jolla, California (Figure 1). The project consists of construction of a new 3.11 million gallon (MG) water storage tank and associated pipelines as well as demolition of the existing La Jolla View and Exchange Place Reservoirs. This report presents our findings and conclusions regarding the geotechnical conditions at the subject site, and our recommendations for the design, earthwork, and construction of this project.

2. SCOPE OF SERVICES

Our scope of services for this evaluation included the following:

- Project coordination and review of readily available background materials pertaining to the site, including geologic and topographic maps, geotechnical reports, faulting and seismic hazard reports, and stereoscopic aerial photographs.
- Review of engineering plans, previous reports, and other data provided by the client.
- Performance of a site reconnaissance to observe and document existing conditions and to mark exploratory boring locations for utility clearance by Underground Service Alert.
- Acquisition of a boring permit from the County of San Diego Department of Environmental Health (DEH).
- Coordinating with environmental professionals to limit environmental impacts at the proposed boring locations.
- Performance of a subsurface exploration consisting of the drilling, logging, and sampling of eight exploratory borings. Specifically, we performed three large-diameter borings which were downhole logged by a Certified Engineering Geologist and five small-diameter borings. Bulk and in-place samples were collected and transported to our in-house geotechnical laboratory for testing.
- Performance of geotechnical laboratory testing on selected samples.
- Compilation and geotechnical analysis of data obtained from our research, subsurface exploration, and geotechnical laboratory testing.
- Preparation of this geotechnical report presenting our findings, conclusions, and recommendations for the design and construction of the proposed project.

3. SITE DESCRIPTION

The La Jolla View Reservoir Replacement Project site is generally located within the City of San Diego La Jolla Natural Park and along Country Club Drive in the La Jolla community of San Diego, California (Figure 1). Topography of the project area is characterized by native ridges and slopes incised by steep-sided erosional ravines. A large, westerly trending ravine is located along the north side of Country Club Drive. Drainage at the site is by sheet flow down the flanks of existing ridges to the ravine north of Country Club Drive. Elevations across the site range from approximately 240 feet above mean sea level (MSL) near the intersection of Soledad Avenue and Exchange Place to approximately 650 above MSL near the intersection of Brodiaea Way and Encelia Drive. The proposed reservoir site is located near the top of a ridge with north-, east-, and west-facing slopes covered with low-lying chaparral and succulents.

The project site is currently occupied by two existing reservoirs. The existing La Jolla View Reservoir is a 0.72 MG steel tank approximately 70 feet in diameter and 30 feet in height. This reservoir, constructed in approximately 1948, is located near the southern edge of the site near the upper end of the large ravine that cuts across the site from east to west. An asphalt-paved access road to the existing La Jolla View Reservoir is located northeast of the reservoir and extends to a gated entrance at Brodiaea Way and Encelia Drive. A buried utility water pipeline extends southwest from the reservoir to Country Club Drive. This reservoir is partially supported on fill soils that were placed at the time the reservoir was constructed. A roughly 30-foot-high cut slope constructed at a slope angle of approximately ½:1 (horizontal to vertical) is located on the east side of the existing tank. A geotechnical reconnaissance report was performed for the existing La Jolla View Reservoir site in 2001 (LawCrandall, 2001).

A second existing reservoir, the 0.98 MG Exchange Place Reservoir, is a buried rectangular tank located near the western end of the project, between Country Club Drive, Exchange Place, La Jolla Knolls Drive, and Al Bahr Drive. Slopes and retaining walls up to 25 feet in height are located on the north, east, and west sides of this reservoir.

4. PROJECT DESCRIPTION

We understand that, as part of the La Jolla View Reservoir Replacement project, the existing Exchange Place Reservoir and La Jolla View Reservoir will be demolished. Following demolition of the Exchange Place Reservoir, the below-grade portions of the concrete liner will be left in-place and the resulting excavation will be filled with cuttings generated from the new reservoir site. After demolition and removal of the tank, the site of the former La Jolla View Reservoir will be graded to approximately restore the pre-development hillside configuration.

A new 3.11 MG La Jolla View Reservoir is proposed to be constructed on a ridge approximately 500 feet northwest of the intersection of Brodiaea Way and Encelia Drive and approximately 600 feet northeast of the existing La Jolla View Reservoir (Figure 2). Based on our current understanding of the project, the new pre-stressed concrete reservoir will be approximately 120 feet in diameter and 40 feet in height and will be buried below the existing terrain. Excavation for the new tank pad is anticipated to result in temporary cut slopes of up to approximately 70 feet in height. These temporary cut slopes will be constructed at near-vertical and approximately 1:1 (horizontal to vertical) slope gradients. It is anticipated that soil nails may be used to provide stability to the vertical portions of the temporary cut slopes. A temporary access road will provide site access from Country Club Drive. Construction of the access road as well as a temporary stockpile along the road will include cuts and fills of up to 60 feet. A new utility water pipeline will be constructed between the new reservoir and the intersection of Brodiaea Way and Encelia Drive. We also understand that the existing access road northwest of Encelia Drive may either be removed or replaced with pervious pavement.

As part of the project, a new 2,800-foot long, 30-inch diameter water pipeline will be constructed. The new pipeline will extend southwest from the new reservoir to Country Club Drive. The pipeline will then extend to the northwest along Country Club Drive to the intersection of Exchange Place and Soledad Avenue (Figure 2). While most of the pipeline is anticipated to be constructed by open-trench methods, the section of the pipeline that crosses the deep ravine north of Country Club Drive may utilize trenchless methods (e.g., microtunneling, jack-and-bore, etc.).

5. SUBSURFACE EXPLORATION AND LABORATORY TESTING

Our subsurface exploration was conducted on February 19 and 20, 2014 and on March 26, 27, and 28, 2014 and consisted of the drilling, logging, and sampling of eight exploratory borings (Borings B-1 through B-8). Borings B-1 and B-2 were drilled in the area of the proposed reservoir, Borings B-3 and B-4 were drilled at the existing La Jolla View Reservoir site, Boring B-5 was drilled adjacent to the Muirlands Pump Station (along Country Club Drive) in the vicinity of the proposed pipeline, and Borings B-6 through B-8 were drilled at the existing Exchange Place Reservoir site. Borings B-1 through B-3 were drilled using a truck-mounted drill rig equipped with 30-inch diameter bucket and solid flight augers. These borings were downhole logged by a Certified Engineering Geologist (CEG). Borings B-4 through B-8 were drilled with a truck-mounted drill rig using 6-inch diameter hollow-stem augers. The exploratory borings were drilled to depths of up to approximately 84 feet. The approximate locations of our exploratory borings are shown on the maps presented in Figures 3A through 3D. Logs of the borings are presented in Appendix A.

Relatively undisturbed and bulk samples were collected at selected depths from the borings and transported to our in-house geotechnical laboratory for testing. Laboratory testing included in-situ moisture content and dry density, gradation, Atterberg limits, direct shear testing, expansion index, soil corrosivity, and R-value. The results of the in-situ moisture content and dry density tests are shown at the corresponding sample depths on the boring logs in Appendix A. The results of the other laboratory tests performed are presented in Appendix B.

6. GEOLOGY AND SUBSURFACE CONDITIONS

Our findings regarding regional and site geology, including groundwater conditions, faulting and seismicity, and landslides at the subject site are provided in the following sections. A regional geologic map, geologic cross-sections, a regional faulting map, and a geologic hazards map are presented on Figures 4 through 7.

6.1. Regional Geologic Setting

The project area is situated in the coastal section of the Peninsular Ranges Geomorphic Province. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California (Norris and Webb, 1990; Harden, 2004). The province varies in width from approximately 30 to 100 miles. In general, the province consists of rugged mountains underlain by Jurassic metavolcanic and metasedimentary rocks, and Cretaceous igneous rocks of the southern California batholith. The portion of the province in San Diego County that includes the project area is generally underlain by Cretaceous-, Tertiary-, and Quaternary-age sedimentary rock.

The Peninsular Ranges Province is traversed by a group of sub-parallel faults and fault zones trending roughly northwest. Several of these faults, which are shown on Figure 6, are considered active faults. The Whittier-Elsinore, San Jacinto, San Andreas, and Rose Canyon faults are active fault systems located east and northeast of the project area and the Agua Blanca-Coronado Bank, San Diego Trough, and San Clemente faults are active faults located west of the project area. Major tectonic activity associated with these and other faults within this regional tectonic framework consists primarily of right-lateral, strike-slip movement. Further discussion of faulting relative to the site is provided in the Faulting and Seismicity section of this report.

6.2. Site Geology

Based on our subsurface evaluation and our review of published maps and reports, earth units at the project site consist of fill and topsoil/colluvium, which mantle formational materials including very old paralic deposits (formerly designated the Lindavista Formation), the Mount Soledad Formation, Ardath Shale, and the Cabrillo Formation. Generalized descriptions of the earth units encountered during our field reconnaissance and subsurface exploration are provided in the subsequent sections. Additional descriptions of the subsurface units are provided on the boring logs in Appendix A.

6.2.1. Fill

Fill material was encountered in Borings B-2 through B-8 and is expected to underlie various portions of the proposed project. As encountered in Boring B-2 at the proposed reservoir site beneath the existing pavement, the fill is approximately 1-foot thick and consists of brown, damp, soft to firm sandy clay with gravel and cobbles. At the existing La Jolla View Reservoir site, fill was encountered beneath the existing pavement and extended to depths of greater than 20 feet. As encountered in Borings B-3 and B-4, the fill material consists of brown, damp to moist, medium dense to dense, clayey to sandy gravel with cobbles. In Boring B-5, the encountered fill materials consist of light brown to brown, damp, medium dense to dense, clayey sand with scattered cobbles. Similar fill materials are expected to underlie the pipeline alignment along Country Club Drive. At the existing Exchange Place Reservoir site, fill was encountered beneath the existing pavements and extended to the total depths explored (24 feet). As encountered in Borings B-6 through B-8, the fill materials consist of various shades of brown, damp to moist, medium dense to dense, silty and clayey sand and stiff to hard, gravelly clay. Abundant gravel and cobbles were encountered in the fill materials. Existing fill soils are assumed to be undocumented.

6.2.2. Topsoil/Colluvium

A mantle of topsoil/colluvium is anticipated to exist along ridges and slopes across the site. As encountered in Borings B-1 and B-2 at the proposed reservoir site, these materials are up to about 2 feet in thickness and consist of various shades of brown and gray, moist, stiff, sandy clay. These materials were also encountered in Boring B-5 beneath the fill materials and extended down to a depth of approximately 10 feet. As encountered, these materials consist of brown to reddish brown, moist, dense, clayey gravel.

6.2.3. Very Old Paralic Deposits

While not encountered in our exploratory borings, Pleistocene-age very old paralic deposits (Kennedy and Tan, 2008) are mapped in the vicinity of the new proposed La Jolla View Reservoir site and across portions of the proposed pipeline alignment. This unit was formerly designated as the Lindavista Formation on older geologic maps. The very old paralic deposits consist of reddish brown, moderately to well-cemented silty sandstone with numerous gravels and cobbles and sandy conglomerate. The very old paralic deposits unconformably overlie the Ardath Shale, Mount Soledad Formation, and Cabrillo Formation.

6.2.4. Ardath Shale

Although not encountered in our exploratory borings, materials of the Eocene-aged Ardath Shale are mapped southwest of the Country Club fault, underlying the western portion of the proposed pipeline alignment. The Ardath Shale generally consists of light gray to light reddish brown, finely bedded, moderately indurated, clayey siltstone with lesser amounts of moderately to well cemented, silty fine-grained sandstone. Resistant, well-cemented concretions and concretionary layers may be encountered within the Ardath Shale. The Ardath Shale is conformably underlain by the Mount Soledad Formation.

6.2.5. Mount Soledad Formation

The Eocene-aged Mount Soledad Formation is mapped within the central portion of the proposed water pipeline and was encountered in our Boring B-5. As encountered underlying fill and colluvium, this unit consists of light brown, brown, and yellowish brown, weakly cemented, sandy and clayey siltstone and silty sandstone and conglomerate. Resistant, well-cemented concretions and concretionary layers may be encountered within the Mount Soledad Formation. The Mount Soledad Formation is conformably overlain by the Ardath Shale and is unconformably underlain by the Cabrillo Formation.

6.2.6. Cabrillo Formation

Materials of the Cretaceous-aged Cabrillo Formation were encountered in our Borings B-1 through B-3 at the proposed and existing La Jolla View Reservoir sites. The Cabrillo Formation is also expected at several locations along the proposed water pipeline. As encountered, materials comprising the Cabrillo Formation generally consist of various shades of brown and gray, weakly to strongly cemented sandstone and light brown, poorly to moderately cemented cobble conglomerate. Lesser beds of siltstone and claystone were also encountered. The Cabrillo Formation is unconformably overlain by the Mount Soledad Formation.

6.3. Geologic Structure

The Mount Soledad area is an intensely folded and faulted area, due to uplift and deformation caused by the movement along the Rose Canyon fault. The project area is located on the western side of the Mount Soledad anticline, a shallow folded structure with limbs that dip to the northeast and southwest. The anticline is cut and deformed by several faults including the Rose Canyon fault, the Mount Soledad fault, the Country Club fault, and several unnamed faults.

Bedding within the Cabrillo Formation at the proposed reservoir site was observed to be undulatory and dipping up to 35°. Older fractures and faults were observed within the Cabrillo Formation and were found to generally dip 55° or more to the south and west. Excavations in areas crossed by these older fractures and faults may result in surficial block failures. Further recommendations regarding excavation conditions can be found in the Section 8.1.11 of this report. In addition, several clay seams were observed within intact portions of the Cabrillo Formation with orientations generally consistent with that of bedding. It is our opinion that these clay seams represent bedding-parallel shear zones (Hart, 2000) and are not indicative of recent landsliding. Graphic representations of structural features encountered during our subsurface evaluation are included on Figure 5.

6.4. Groundwater

Groundwater was not encountered in our exploratory borings. Based on the elevation and topography of the area and our experience in the vicinity of the site, static groundwater is likely to be encountered at depths greater than 85 feet below the proposed reservoir site. Static groundwater may be encountered at shallower depths within the ravines, especially the ravine northwest of Country Club Drive. Groundwater seepage at other elevations may be encountered and fluctuations in the groundwater level may occur due to variations in ground surface topography, subsurface geologic conditions and structure, rainfall, irrigation, and other factors.

6.5. Faulting and Seismicity

Like other areas of southern California, the project area is considered to be seismically active and the potential for strong ground motion is considered significant during the design life of the proposed project improvements. Based on our review of the referenced geologic maps and stereoscopic aerial photographs, as well as on our subsurface evaluation, the subject site is not located within a State of California Earthquake Fault Zone (EFZ), formerly known as an Alquist-Priolo Special Studies Zone (Hart and Bryant, 1997). However, two potentially active faults (i.e., faults that exhibit evidence of ground displacement in the last 2,000,000 years) have been mapped as crossing the proposed pipeline alignment (Figures 4 and 7). These faults, the Country Club fault and a shorter, unnamed fault, traverse portions of the proposed water pipeline alignment.

The nearest known active fault is the Rose Canyon fault, located approximately 0.4 miles northeast of the site. Table 1 lists selected principal known active faults that may affect the subject site, the maximum moment magnitude (M_{max}) and the fault types as published for the California Geological Survey (CGS) by Cao et al. (2003). The approximate fault-to-site distance was calculated by the United States Geological Survey (2008) National Seismic Hazard Maps database (web-based) or was assessed from the referenced geologic maps.

Table 1 – Principal Active Faults

Fault	Distance miles [1]	Moment Magnitude/ Fault Type [1,2]
Rose Canyon	0.4	7.2/B
Coronado Bank	13	7.6/B
Newport-Inglewood (Offshore)	24	7.1/B
Elsinore (Temecula Segment)	37	6.8/A
Elsinore (Julian Segment)	39	7.1/A
Earthquake Valley	46	6.5/B
Palos Verdes	49	7.3/B
Elsinore (Glen Ivy Segment)	54	6.8/A
Elsinore (Coyote Mountain)	54	6.8/A
San Joaquin Hills	57	6.6/B
San Jacinto (Coyote Creek Segment)	60	6.8/A

Notes:
[1] USGS (2008)
[2] Cao, et al. (2003)

In general, hazards associated with seismic activity include strong ground motion, ground surface rupture, liquefaction, and seismically induced settlement. These hazards are discussed in the following sections.

6.5.1. Strong Ground Motion

The 2013 California Building Code (CBC) recommends that the design of structures be based on the spectral response accelerations in the direction of maximum horizontal response (5 percent damped) having a 1 percent probability of collapse in 50 years. Such spectral response accelerations represent the Risk-Targeted Maximum Considered Earthquake (MCE_R) ground motion. The horizontal peak ground acceleration (PGA) that corresponds to the MCE_R for the site was calculated as 0.51g using the United States Geological Survey (USGS, 2013) seismic design tool (web-based). The mapped and design PGA were estimated to be 0.58g and 0.34g, respectively, using the USGS (2013) calculator and the American Society of Civil Engineers (ASCE) 7-10 Standard.

6.5.2. Ground Surface Rupture

Based on our review of the referenced reports, geologic maps, and stereoscopic aerial photographs, as well as on our geologic field reconnaissance and subsurface evaluation, active faults have not been mapped and were not observed at the proposed new La Jolla View Reservoir site. Based on this information, the potential for surface rupture on the site is considered low. However, as previously discussed, two potentially active faults (the Country Club fault and a shorter, unnamed fault), cross portions of the proposed water pipeline alignment. The Country Club fault has been mapped as generally following the ravine northwest of Country Club Drive. The short, unnamed fault parallels the Country Club fault, approximately 800 feet northeast of Country Club Drive. The mapped southeastern end of the unnamed fault is located roughly 200 feet southwest of the new reservoir site. While ground surface rupture is not considered likely at these locations, it is possible. In addition, lurching or cracking of the ground surface as a result of nearby seismic events is possible.

6.5.3. Liquefaction and Seismically Induced Settlement

Liquefaction of cohesionless soils can be caused by strong vibratory motion due to earthquakes. Research and historical data indicate that loose granular soils and non-plastic silts that are saturated by a relatively shallow groundwater table are susceptible to liquefaction. Based on the dense nature of the underlying formational materials and absence of a shallow groundwater table, it is our opinion that liquefaction and seismically induced settlement at the project site are not design considerations.

6.6. Landsliding

Numerous landslides have been mapped in the vicinity of the project area (Figures 4 and 7). Most of these mapped landslides appear to be the result of shallow earth flow type failures with some deeper translational landslides located north of the project area. Based on our review of published geologic literature and aerial photographs and our subsurface exploration, no deep-seated landslides or related features underlie the proposed reservoir site. Due to the proposed temporary steep slopes and the fractured nature of the Cabrillo Formation, the potential for shallow block failures to impact the site during construction should be anticipated. Slope stability is discussed further in our recommendations.

While landslides have not been mapped adjacent or beneath the new reservoir site, the Law-Crandall (2001) report indicates "...a landslide appears to underlie the existing (La Jolla View) reservoir site…" based on LawCrandall's review of aerial photographs and as-built reservoir plans (City of San Diego, 1948). Evidence of deep-seated landsliding was not observed in our exploratory borings drilled at the existing La Jolla View Reservoir site. However, the total extent of a possible landslide at the existing reservoir could not be evaluated at the time of our field activities due to drilling access restraints. Landsliding should be further evaluated during demolition of the existing reservoir and during earthwork for the former and new reservoir sites.

7. CONCLUSIONS

Based on our review of the referenced background data, subsurface exploration, and laboratory testing, it is our opinion that the proposed reservoir replacement project is feasible from a geotechnical standpoint provided the recommendations presented in this report are incorporated into the design and construction of the project. Due to the preliminary nature of the anticipated depth of the proposed improvements and the variable subsurface conditions, we recommend that our office re-evaluate our conclusions and recommendations once final plans are available. In general, the following conclusions were made:

- The proposed reservoir site is underlain by fill, topsoil, and formational materials of the Cabrillo Formation. Fill and topsoils are not considered suitable for structural support of the proposed improvements in their current condition. The Cabrillo Formation is considered suitable for structural support.

- Based on our subsurface exploration, excavation at the proposed reservoir site should be feasible with heavy-duty earthmoving equipment in good working condition. However, the contractor should anticipate heavy ripping or rock breaking in areas of concretionary or well cemented sandstone and conglomerate.

- The project site is located in a seismically active zone. Accordingly, the potential for strong seismic ground motions should be considered in the project design. A design PGA of 0.34g was calculated for the project site.

- Groundwater was not encountered in our borings. Accordingly, we do not anticipate that groundwater will be a constraint for the proposed construction.

- On-site excavations are anticipated to generate oversized materials. Additional processing (i.e., screening or crushing) should be anticipated prior to usage as engineered fill.

- Based on our limited laboratory testing, the encountered soils and formational materials are considered corrosive.

8. RECOMMENDATIONS

Based on our understanding of the project, the following recommendations are provided for the design and construction of the proposed improvements.

8.1. Earthwork

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

8.1.1. Pre-Construction Conference

We recommend that a pre-construction conference be held in order to discuss the recommendations presented in this report. The owner and/or their representative, the governing agencies' representatives, the civil engineer, Ninyo & Moore, and the contractor should be in attendance to discuss the work plan, project schedule, and earthwork requirements.

8.1.2. Site Preparation

Prior to excavation and placement of fill, the project site should be cleared of abandoned utilities (if present), and stripped of rubble, debris, vegetation, any loose, wet, or otherwise unstable soils, as well as surface soils containing organic material. Obstructions that extend below the finished grade, if any, should be removed and the resulting holes filled with com-

compacted soil. Materials generated from the clearing operations should be removed from the site and disposed of at a legal dumpsite away from the project area.

8.1.3. Remedial Grading

In general, we recommend that the on-site existing fill, topsoil, and colluvium be removed down to competent materials in those areas where improvements or additional fill soils are planned. The extent and depths of removals should be evaluated by Ninyo & Moore's representative in the field based on the materials exposed. Where required, the resulting excavation may be filled with engineered fill soils (either on-site derived or imported) that meet the recommendations presented in the following sections. Precautions should be taken by the contractor when grading adjacent to existing structures.

8.1.4. Excavation Characteristics

Our evaluation of the excavation characteristics of the on-site materials is based on the results of our exploratory borings, our site observations, and our experience with similar materials in the general vicinity of the site. Due to the presence of gravel, cobbles, and possible boulders along with strongly cemented zones within the materials of very old paralic deposits, the Mount Soledad Formation, and the Cabrillo Formation, difficulty in the performance of excavations should be anticipated. The presence of rock masses, and/or strongly cemented or concretionary zones can be problematic in a narrow trench and should be anticipated. Consequently, special excavation methods, including heavy ripping or rock breaking may be needed.

8.1.5. Cut/Fill Transition

The proposed reservoir should not straddle a cut/fill transition. In the instance that a cut/fill transition condition is created beneath the reservoir through grading, one of the two following recommendations may be implemented. The first option consists of the overexcavation of the pad area for the new reservoirs. The overexcavation should extend to a depth of 2 feet below the bottom of the footing elevation or one-third of the largest thickness of fill, whichever is deeper. Engineered fill may then be moisture-conditioned, placed in the overexcavation, and compacted in accordance with the recommendations herein. The reservoir pad area is defined as the structure footprint and extending 5 feet horizontally outside of the structure footprint plus the depth of the overexcavation.

As an alternative to overexcavation, the fill portion of the cut/fill transition may be removed and replaced with a controlled low strength material (CLSM). The reservoir base may be supported on the CLSM.

8.1.6. Materials for Fill

The on-site soils with an organic content of less than approximately 3 percent by volume (or 1 percent by weight) are suitable for reuse as engineered fill. In general, fill material should not contain rocks or lumps over approximately 3 inches, and not more than approximately 30 percent larger than ¾-inch. Oversize material generated during excavations should be disposed of off-site or broken into acceptable size material. Imported fill material, if needed for the project, should generally be granular soils with a very low to low expansion potential (ASTM International [ASTM] 4829). Import material should also be non-corrosive in accordance with the Caltrans (2012) and American Concrete Institute (ACI) 318 corrosion guidelines. Materials for use as fill should be evaluated by Ninyo & Moore’s representative prior to filling or importing.

8.1.7. Compacted Fill

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

Fill materials should be moisture-conditioned to generally above the laboratory optimum moisture content prior to placement. The optimum moisture content will vary with material type and other factors. Moisture-conditioning of fill soils should be generally consistent within the soil mass. As noted, wet soils may be encountered during construction and aeration/processing should be anticipated.

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

Compacted fill should be placed in horizontal lifts of approximately 8 inches in loose thickness. Prior to compaction, each lift should be watered or dried as needed to achieve a moisture content generally above the laboratory optimum, mixed, and then compacted by mechanical methods, using sheepsfoot rollers, multiple-wheel pneumatic-tired rollers or other appropriate compacting rollers, to a relative compaction of 90 percent as evaluated by ASTM D 1557. Successive lifts should be treated in a like manner until the desired finished grades are achieved.

We understand that structural foam blocks (i.e., lightweight fill) are being considered for use as backfill for the reservoir excavation. If structural foam blocks are used, we recommend that they be placed in accordance with the manufacturer's guidelines/specifications.

8.1.8. Fill Slopes

Fill slopes for the project should be constructed at an inclination of 2:1 (horizontal to vertical) or flatter. We performed global slope stability of the proposed temporary 2:1 (horizontal to vertical) stockpile slope using the two-dimensional stability analysis program GSTABL7 with STEDwin (version 2). The slope stability calculations of the stockpile slope (section B-B') are presented in Appendix C. The results of our analysis indicate that the proposed 2:1 (horizontal to vertical) fill slopes will be globally stable if the fills are derived from suitable materials and properly constructed as recommended in this report.

Compaction of the face of temporary and permanent fill slopes should be performed by backrolling at intervals of 4 feet or less in vertical height, or as dictated by the capability of the available equipment, whichever is less. Fill slopes should be backrolled utilizing a conventional sheepsfoot-type roller. Care should be taken to maintain the desired moisture conditions and/or reestablish them, as needed, prior to backrolling. The placement, moisture-conditioning, and compaction of fill slope materials should be done in accordance with the recommendations presented in the Compacted Fill section of this report. Slopes and other exposed ground surfaces should be appropriately planted with a protective ground cover. To enhance surficial stability, fill slopes should be planted as soon as feasible subsequent to grading. Erosion control and drainage devices should be installed in compliance with the requirements of the local governing agencies as soon as feasible subsequent to grading.

8.1.9. Fill Placement on Sloping Ground

Fills constructed on sloping ground having an inclination steeper than 5:1 should be keyed and benched into competent materials underlying loose soils. Keys should generally be 15 feet in width or greater and extend 3 feet or more into the competent material. The actual width of the keys and extent of removal of any existing loose surficial soil or other native materials should be evaluated by Ninyo & Moore or the client's designated representative in the field during construction. In addition, key excavations should be observed by Ninyo & Moore or the client's designated representative prior to placing fill.

8.1.10. Cut Slopes

The stability of cut slopes is generally affected by local geologic conditions, the gradient of the overall slope, groundwater seepage conditions and also by the excavation technique used in creating the slope. We performed global slope stability of the proposed temporary 1:1 (horizontal to vertical) cut slope using the two-dimensional stability analysis program GSTABL7 with STEDwin (version 2). The slope stability calculations of the cut slope (section C-C') are presented in Appendix C. The results of our analysis indicate that the proposed 1:1 (horizontal to vertical) cut slopes will be globally stable if the cuts are made into suitable materials of the Cabrillo Formation or similar formational materials and properly constructed as recommended in this report. Permanent cut slopes within the Cabrillo Formation should be constructed at an inclination of 2:1 (horizontal to vertical) or flatter. Excavation of cut slopes should include removal of near-surface residual soils and/or weathered materials. It is recommended that cut slopes be observed by Ninyo & Moore during grading to further evaluate their stability, the presence of geologic planes of weakness, and to provide appropriate mitigation recommendations as needed.

Typically, in slopes excavated in ripped formational materials, loose materials may be present in slope faces. Finish slopes should be groomed to reduce spalling of loose materials from the slope faces.

Surface runoff should not be permitted to flow over the tops of slopes. Positive drainage should be established away from the top of slopes. This may be accomplished by utilizing brow ditches placed at the top of cut slopes to divert surface runoff away from the slope face where drainage devices are not otherwise available.

8.1.11. Temporary Excavations and Shoring

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

Fill and Colluvium	*Type C*
Very Old Paralic Deposits, Mount Soledad Formation, Ardath Shale, Cabrillo Formation	*Type B*

Upon making the excavations, the soil classifications and excavation performance should be evaluated in the field by the geotechnical consultant in accordance with the OSHA regulations. Temporary excavations should be constructed in accordance with OSHA recommendations. For trenches, jacking and receiving pits, or other excavations, OSHA requirements regarding personnel safety should be met using appropriate shoring (including trench boxes) or by laying back the slopes to no steeper than 1.5:1 in fill colluvial materials and 1:1 for formational materials including very old paralic deposits, Mount Soledad Formation, Ardath Shale, and Cabrillo Formation.

Figure 8 presents recommended lateral earth pressures for the design of braced shoring. The recommended design pressures are based on the assumptions that the shoring system is constructed without raising the ground surface elevation behind the shoring, that there are no surcharge loads (such as soil stockpiles and construction materials), and that no loads act above a 1:1 plane extending up and back from the base of the shoring system. The contractor should include the effect of any surcharge loads on the lateral pressures against the shoring.

Temporary excavations that encounter seepage may be shored or stabilized by placing sandbags or gravel along the base of the seepage zone. Excavations encountering seepage should be evaluated on a case-by-case basis. On-site safety of personnel is the responsibility of the contractor.

Older fractures and faults were encountered within the Cabrillo Formation during our subsurface evaluation. Due to the proposed steep temporary slopes and the fractured and weathered nature of the Cabrillo Formation, the potential for surficial block failures to impact the slopes during construction should be anticipated.

8.1.12. Backfilling of Exchange Place Reservoir

Based on our communications with Infrastructure Engineering Corporation, we understand that demolition of the Exchange Place Reservoir will include the demolition of the above-grade portions of the existing reservoir and leaving portions of the existing concrete liner in-place. We further understand that the portions of the liner left in-place will be cored in several places to help facilitate drainage of water, and that the resulting excavation will be backfilled with materials excavated from the La Jolla View Reservoir site.

In addition to the coring of the liner (as described above) and to further reduce the potential for the accumulation of subsurface water upon the liner (and to reduce the potential for seepage of this water through the slope face and onto the adjacent properties), we recommend that a subdrain be installed beneath the cored liner prior to the placement of the backfill. This subdrain should consist of a 6-inch-diameter perforated PVC pipe (Schedule 40 or approved equivalent), which is surrounded by 3 cubic feet per lineal foot of clean crushed gravel, and wrapped in Mirafi 140N filter fabric (or approved equivalent). The subdrain should allow gravity flow of water to a nearby storm drain or other suitable outlet.

8.1.13. Soil Nail Retaining Wall

It is our understanding that the City of San Diego is considering the use of a soil nail wall as a method to shore portions of the proposed near-vertical temporary excavation at the reservoir site. The wall will be approximately 350 feet in length and extend along the eastern and southern sides of the reservoir excavation. Preliminary soil nail recommendations for the subject wall are presented below. These recommendations were developed based on the subsurface information obtained from our exploratory borings at the proposed reservoir location (B-1 and B-2) and surface data gathered from our geologic reconnaissance of the site. These soil nail recommendations are also based on our understanding of the general wall profile and sections at the proposed reservoir location. It should be noted that based on our review of the grading plans, the steep inclination of the existing descending slope adjacent to the eastern portion of the wall may result in day-lighting of soil nails installed near the top of the wall. Recommendations regarding soil nails in this area of the site are presented in Section 8.1.13.4. We assume that no additional fill or surcharge will be placed above the soil nail wall. In the event that additional fill materials are to be placed and the retained heights are to be increased, the following recommendations will need to be revisited and refined, as needed.

Based on our understanding of the proposed wall configuration, we anticipate that the soil nail wall will utilize a typical nail spacing of 4 feet on center in both the horizontal and vertical directions. An ultimate unit pullout resistance of 2,100 psf and an allowable unit pullout resistance of 1,050 psf is estimated for the bonded portion of the nail in the materials comprising the Cabrillo Formation. This pullout resistance is based on Table 3.10 of the Federal Highway Administration soil nail design guideline (FHWA, 2003) and assumes rotary drilling methods and gravity grouting of the soil nails. We recommend that soil nails be inclined at an angle of 15 degrees below the horizontal plane with a minimum grout diameter of 6 inches. A nail bar diameter of 1 inch is recommended for design.

For preliminary design purposes, we recommend that soil nails be designed using an unbonded length of 5 feet and a bonded length of 5 feet or more into the materials comprising the Cabrillo Formation. The actual bonded length should be determined by the wall designer based on anticipated loading considerations. The design bond length may be revised based on the pullout testing of the nails during construction. Design of the soil nail walls should be performed in accordance with FHWA soil nail wall design guideline (FHWA, 2003) and should utilize minimum factors of safety presented in Table 5.3 of the FHWA guideline.

In the event the actual nail spacing chosen for the project is different than that recommended herein, the recommendations presented above should be reevaluated and modified as needed. The nail lengths may also need modification if underground utilities are present near the retained areas. While preparing the soil nail layout profile, special care should be taken in limiting the non-anchored portion of the wall (i.e., the top, bottom and the edge segments of the wall) to less than the nail spacing (i.e., less than 4 feet in this case).

8.1.13.1. Materials

The nail bars should conform to ASTM A615 - Grade 75 or ASTM A722 - Grade 150. The soil nails should be epoxy coated in accordance with ASTM A775. The epoxy coat should have a minimum thickness of approximately 0.3-inch, and should be electro-statically applied. If potentially aggressive ground conditions (i.e., low electrical resistivity) are encountered, the use of encapsulation should be considered. Encapsulation should be achieved by grouting the steel bar inside a corrugated HDPE or PVC sleeve. A neat cement grout containing admixtures to control water bleed from the grout should be used to fill the annular space between the bar and the sleeve. The cement grout for the nail should consist of either neat cement or a sand-cement mixture with a minimum three-day compressive strength of 1,500 pounds per square inch (psi) and a minimum 28-day compressive strength of 3,000 psi per ASTM C109. The cement should conform to ASTM C150, Type II/V Portland cement. Fine aggregate for the grout mix should comply with ASTM C33. Water used for grout should potable, clean and free from substances deleterious to concrete and steel. Testing of nail grout during construction should be performed at a frequency no less than one test for every 50 cubic yards of grout placed or once a week, whichever occurs first.

8.1.13.2. Nail Testing

The soil nails should be tested during construction to evaluate the design assumptions and the nail capacities. The contractor should provide equipment and instrumentation needed to check the adequacy of the nails. A dial gauge capable of measuring displacements to 0.01-inch precision should be used to measure the nail movement. A hydraulic jack and pump should be used to apply the test load, and the jack and a calibrated pressure gauge should be used to measure the load. The standard testing procedures typically consist of the following methods:

- <u>Verification Test</u> – These tests are typically performed on a limited number of sacrificial nails to check that 1) the design test load (DTL) may be safely carried, 2) effective bond length corresponds to the design requirements, and 3) the residual movement is within tolerable range. The verification test consists of a single cycle of incremental loading to a maximum test load of 200 percent of the DTL in accordance with the guideline presented in Appendix B1 of FHWA Manual for Design and Construction Monitoring of Soil Nail Walls (Publication No. FHWA-SA-96-069R, dated October 1998). The verification test should be conducted on at least three soil nails within the wall. The location of the verification test nails will be provided by Ninyo & Moore once the soil nail wall profile is developed. The sacrificial nails used for verification tests should not be used as production nails.

- Proof Test – These tests are typically performed on about 5 percent of the production nails in each row to check that the load-deflection behavior of the production nail is consistent with the specified acceptance criteria. The proof test consists of a single cycle of incremental loading to a maximum test load of 150 percent of the DTL in accordance with the guideline presented in Appendix B1 of the aforementioned FHWA Manual (1998). The location of the proof test nails will be evaluated by Ninyo & Moore once the soil nail wall profile is developed.

The verification and proof test schedules for the nails, including the acceptance criteria and repair mechanism of failed test nails, should be developed by the contractor utilizing his experience on similar projects, the nail design/testing recommendations presented here, and the FHWA (1998) guidelines. In general, the acceptance criteria for the tested nails should be based on the following aspects.

- For verification tests, a total creep movement of less than 2 millimeters (mm) per log cycle of time between the 6- and 60-minute readings is measured during creep testing and the creep rate is linear or decreasing throughout the creep test load hold period.

- For proof tests, a total creep movement of less than 1 mm is measured between the 1- and 10-minute readings or a total creep movement of less than 2 mm is measured between the 6- and 60-minute readings and the creep rate is linear or decreasing throughout the creep test load hold period.

- The total measured movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the test nail unbonded length.

- A pullout failure does not occur at the maximum test load. Pullout failure is defined as the load at which attempts to further increase the test load result in continued pullout movement of the test nail. The pullout failure load shall be recorded as part of the test data.

The test schedules of the soil nails and the acceptance criteria should be included in the project plans. Ninyo & Moore should be given the opportunity to review the project plans to check compliance with design and construction recommendations presented herein.

8.1.13.3. Shotcrete Cover

The shotcrete cover for the soil nail wall should be constructed as a reinforced concrete wall at vertical or near-vertical inclination. The wall should be structurally connected to the soil nails through the bearing plate and the nail head assembly. The portion of the wall below the nail will act as a non-structural façade to the cut slope and may not need a footing for vertical or lateral support.

The shotcrete wall should be provided with appropriate drainage in order to reduce build-up of hydrostatic pressure behind the wall. A limited area may exist behind the wall for installation of a conventional pipe and gravel subdrain. Therefore, a drain mat such as Miradrain 6000 or equivalent should be considered. The drain mat should be installed between the soil nails along the back of the shotcrete wall. In the vertical direction, the drain mats should be connected to facilitate the downslope flow of water under gravity. A perforated subdrain should be placed behind the bottom of the shotcrete wall, and it should be partially or wholly wrapped inside the lower edges of the drain mat. Water collected by the perforated subdrain system should be routed to a suitable discharge point.

8.1.13.4. Construction Considerations

The nail installation and shotcrete wall construction should be performed by a specialized contractor having significant experience in nail installations. The construction should be performed in a phased, “top down” manner starting at the top of the wall and proceeding gradually to the bottom. In each phase, nail installation in a row should be preceded by removal of unsuitable materials (e.g., loose fill and/or weathered bedrock) from the slope face and followed by placement of rebar cage and drainage mat for the shotcrete wall, spraying of shotcrete onto the rebar, and finishing and/or sculpturing the wall. Slope excavations should be performed in accordance with the published guidelines of the OSHA.

Based on our review of the grading plans, the steep inclination of the existing descending slope adjacent to the eastern portion of the proposed wall may result in daylighting of soil nails installed near the top of the wall. Therefore, we recommend that soil nail installation should occur no less than 15 feet below the top of the wall along the eastern side. This upper area may be stabilized during construction by sloping back the cut, thereby reducing the slope angle and height of the slope. An alternative method to ensure installation of soil nails into formational materials may include reducing the length of soil nails in this area. In order to adequately reduce the length of the soil nails, the spacing of soil nails should also be reduced.

The nails should be installed at the design angle below the horizontal plane with a tolerance of ±2 degrees at the bearing plate. The bonded portion of the nail should be filled with neat cement grout or a sand-cement mixture following placement of the steel bar inside the nail hole. The unbonded portion of the nail may be filled with lean concrete or slurry. However, the unbonded portion of the nail should be free to move.

The performance of the soil nail should be monitored during construction. The contractor should develop a monitoring plan and submit it to the City for review and approval. As a minimum, the monitoring should include observations for and measurements of 1) lateral movements of the wall face using survey markers, 2) vertical and lateral movements of the top of the wall facing and the ground surface behind the shotcrete facing using optical survey methods, and 3) ground cracks and other signs of disturbance in the wall backfill zone by daily visual inspection.

Zones containing more resistant cobbly, concretionary or well cemented sandstone and conglomerate should be anticipated during construction of the soil nail wall. Consequently, nail construction in such zones would be expected to necessitate coring or percussion drilling. We recommend that an experienced specialty contractor be used for construction of the soil nail wall.

Nail installation and wall construction should be observed by Ninyo & Moore or the City's designated representative. The contractor should provide equipment and instrumentation needed to check placement of steel bars and concrete within the nail holes. The quantity of grout and the grout pressure, if applicable, should be recorded by the contractor for each nail. The nails should be tested to check for the design considerations presented here.

8.1.14. Pipe Bedding and Modulus of Soil Reaction

It is our recommendation that the new pipelines, where constructed in open excavations, be supported on 6 or more inches of granular bedding material. Granular pipe bedding should be provided to distribute vertical loads around the pipe. Pipe bedding should have a Sand Equivalent (SE) of 30 or greater, and be placed around the sides and the crown of the pipe. In addition, the pipe bedding material should extend 1 foot or more above the crown of 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 governing agency.

The modulus of soil reaction is used to characterize the stiffness of soil backfill placed at the sides of buried flexible pipes for the purpose of evaluating deflection caused by the weight of the backfill over the pipe (Hartley and Duncan, 1987). A soil reaction modulus of 1,000 pounds per square inch (psi) may be used for an excavation depth of up to about 5 feet when backfilled with granular soil compacted to a relative compaction of 90 percent as evaluated by the ASTM D 1557. A soil reaction modulus of 1,400 psi may be used for trenches deeper than 5 feet.

8.1.15. Trench Zone Backfill

Based on our subsurface exploration, the on-site earth materials may be generally suitable for re-use as trench zone backfill provided they are free of organic material, clay lumps, debris, and rocks greater than approximately 3 inches in diameter. We recommend that trench backfill materials be in conformance with the “Greenbook” (Standard Specifications for Public Works) specifications for structure backfill. Soils classified as silts or clays should not be used for backfill in the pipe zone. Fill should be moisture-conditioned to generally above the laboratory optimum. Trench backfill should be compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557 except for the upper 12 inches of the backfill in pavement areas that should be compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557. Lift thickness for backfill will depend on the type of compaction equipment utilized, but fill should generally be placed in lifts not exceeding 8 inches in loose thickness. Special care should be exercised to avoid damaging the pipe during compaction of the backfill.

8.1.16. Pipe Jacking and Thrust Blocks

As noted above, the section of the pipeline that crosses the deep ravine north of Country Club Drive may be installed utilizing trenchless methods (e.g., directional drilling, jack-and-bore, tunneling, etc.). If trenchless methods are employed, jacking and receiving pits will be installed at each end of the trenchless segment. Due to seasonal variations in groundwater, the pits may require dewatering during excavation. It should be anticipated that more resistant cobbly, concretionary or well cemented sandstone and conglomerate will also be encountered during excavation and that excavation in such zones would necessitate heavy ripping, rock breaking, or coring. In addition, resistant cobbles, concretions, and/or well cemented zones could affect the installation of the jacked portions of the pipelines by deflecting the bore-and-jack equipment away from pipeline's design alignment. We recommend that an experienced specialty contractor be used for the jack-and-bore operations.

Minor ground surface settlements may occur from the pipe jacking operations. However, due to the anticipated depth of the proposed pipeline, these settlements are not anticipated to impact existing improvements provided that an experienced contractor performs the work.

In order to evaluate the load factors on the proposed pipeline, the loading presented in the following table should be used.

Table 2 – Loading on Trenchless Segment of Pipeline

Approximate Depth from Existing Ground Surface to Top of Pipeline (feet)	Load on 36-inch Pipeline/ Casing (pounds/lineal foot of pipe)	Load on 24-inch Pipeline/ Casing (pounds/lineal foot of pipe)
5	1,500	900
10	2,500	1,400
15	3,100	1,600
20	3,500	1,700

Notes:
Linear interpolation may be used to obtain loading between the depths shown. Loading assumes 36-inch and 24-inch sleeve diameter of the trenchless section. Loading may need to be modified for different sleeve sizes.

Following installation of the pipeline, the jacking and receiving pits should be backfilled in accordance with recommendations contained in Section 8.1.7.

8.1.17. Lateral Earth Pressure for Thrust Blocks and Jacking

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 magnitude and distribution of passive lateral earth pressures presented on Figure 9.

Thrust blocks should be backfilled with granular backfill material and compacted following the recommendations presented in this report.

8.1.18. Seismic Design Considerations

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. The following table presents the seismic design parameters for the site in accordance with CBC (2013) and adjusted MCE_R spectral response acceleration parameters (USGS, 2013).

Table 3 – 2013 California Building Code Seismic Design Criteria

Site Coefficients and Spectral Response Acceleration Parameters	Values
Site Class	C
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.3
Mapped Spectral Response Acceleration at 0.2-second Period, S_S	1.283g
Mapped Spectral Response Acceleration at 1.0-second Period, S_1	0.496g
Spectral Response Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	1.283g
Spectral Response Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.647g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.855g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.431g

8.2. Foundations

Foundation recommendations presented in the following sections are for shallow, spread footings bearing on compacted fill or competent materials comprising the Cabrillo Formation. Based on our review of the project plans (IEC, 2013; 2014), foundations supporting the proposed reservoir are anticipated to bear within materials comprising the Cabrillo Formation. Foundations should be designed in accordance with structural considerations and the following recommendations. Requirements of the appropriate governing jurisdictions and applicable building codes should be considered in the design of the structures.

8.2.1. Shallow Footings

Shallow, spread or continuous footings, founded in compacted fill materials may be designed using a net allowable bearing capacity of 3,000 pounds per square foot (psf).

Footings founded in materials comprising the Cabrillo Formation may be designed using a net allowable bearing capacity of 4,500 psf. Spread footings should be founded 24 inches below the lowest adjacent grade. Continuous footings should have a width of 18 inches and isolated footings should be 24 inches in width. The allowable bearing capacity may be increased by 250 psf for every foot of increase in width or 600 psf for each additional foot of embedment up to a value of 8,000 psf. These allowable bearing capacities may be increased by one-third when considering loads of short duration such as wind or seismic forces. The spread footings should be reinforced in accordance with the recommendations of the project structural engineer. If cemented or concretionary zones are encountered and removed during footing excavation, the resulting voids may be backfilled with CLSM. Concrete footings may be placed on CLSM.

8.2.2. Lateral Resistance

For resistance of footings to lateral loads, we recommend a passive pressure of 350 psf per foot of depth be used with a value of up to 3,500 psf for footings founded in compacted fill. For portions of the footings embedded in the Cabrillo Formation, a passive pressure of 500 psf per foot of depth can be used with a value of up to 5,000 psf. These values assume that the ground is horizontal for a distance of 10 feet, or three times the height generating the passive pressure, whichever is greater. In those areas where the ground surface slopes downward from the footing/retaining wall at an inclination of 2:1 (horizontal to vertical), a passive pressure of 150 psf per foot is recommended. At those locations where the ground surface slopes downward at an inclination of 1.5:1 (horizontal to vertical), we recommend a passive pressure of 95 psf per foot. We recommend that the upper one-foot of soil not protected by pavement or a concrete slab be neglected when calculating passive resistance.

For frictional resistance to lateral loads, we recommend that a coefficient of friction of 0.35 be used between engineered fill and concrete and 0.45 be used between formational materials and concrete. The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed one-half of the total allowable resistance. The passive resistance values may be increased by one-third when considering loads of short duration such as wind or seismic forces.

8.2.3. Static Settlement

We estimate that the proposed facilities, designed and constructed as recommended herein, will undergo total settlement on the order of 1 inch. Differential settlement on the order of 1/2 inch over a horizontal span of 40 feet should be expected.

8.3. Retaining Wall Earth Pressures

We understand that the perimeter walls of the reservoirs will act as retaining walls. Retaining walls may also be constructed at the valve locations. For the design of a yielding retaining wall that is not restrained against movement by rigid corners or structural connections, an active pressure represented by an equivalent fluid weight of 65 pounds per cubic foot (pcf) may be assumed for 2:1 backfill and 40 pounds per cubic foot (pcf) may be assumed for level backfill. Restrained walls (non-yielding) may be designed for at-rest pressure represented by an equivalent fluid weight of 87 pcf for 2:1 backfill and 60 pcf for level backfill. Seismic loading can be modeled assuming an inverted triangular loading. Should dynamic earth pressures be considered in the design, an inverse triangular pressure distribution with an equivalent fluid weight of 14 pcf may be used. For retaining walls with heights of 6 feet or less, dynamic earth pressures do not need to be considered in the design. These pressures assume low-expansive, granular backfill as defined in the Materials for Fill section of this report. Wall backfill should be moisture-conditioned and compacted to a relative compaction of 90 percent at a moisture content near the optimum as evaluated by ASTM D 1557. A drain should be provided behind the wall as shown on Figure 10. The drain should be connected to an appropriate outlet.

8.4. Slabs–on-Grade

We recommend that conventional slabs-on-grade, underlain by compacted fill materials of generally very low to low expansion potential, be 5 inches in thickness and be reinforced with No. 4 reinforcing bars spaced 18 inches on center each way. The reinforcing bars should be placed near the middle of the slab. As a means to help reduce shrinkage cracks, we recommend that the slabs be provided with expansion joints at intervals of approximately 12 feet each way. The final slab thickness, reinforcement, and expansion joint spacing should be designed by the project structural engineer.

8.5. Concrete Flatwork

Exterior concrete flatwork should be 4 inches in thickness and should be reinforced with No. 3 reinforcing bars placed at 24 inches on center both ways. To reduce the potential manifestation of distress to exterior concrete flatwork due to movement of the underlying soil, we recommend that such flatwork be installed with crack-control joints at appropriate spacing as designed by the structural engineer. Exterior slabs should be underlain by 4 inches of clean sand. The subgrade soils should be scarified to a depth of 12 inches, moisture-conditioned to generally above the laboratory optimum moisture content, and compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557. Positive drainage should be established and maintained adjacent to flatwork.

8.6. Corrosion

The corrosion potential of the near-surface site soils was evaluated using the results of representative samples obtained from our borings. Laboratory testing was performed to evaluate pH, minimum electrical resistivity, soluble sulfate and chloride contents. The pH and electrical resistivity tests were performed in accordance with California Test (CT) 643 and the sulfate and chloride content tests were performed in accordance with CT 417 and 422, respectively. The laboratory test results are presented in Appendix B.

The results of the corrosivity testing indicated electrical resistivity ranging from 310 to 420 ohm-cm, soil pH of 5.7 to 8.9, chloride content of 1,040 to 1,920 parts per million (ppm), and sulfate content of 0.003 to 0.068 percent (i.e., 30 ppm to 680 ppm). Based on the laboratory test results and Caltrans (2012) and American Concrete Institute (ACI) 318 corrosion criteria, the project site can be classified as a corrosive site. Corrosive soils are defined as soils with more than 500 ppm chlorides, more than 0.20 percent sulfates (i.e., 2,000 ppm), a pH of 5.5 or less, or an electrical resistivity of 1,000 ohm-centimeters or less.

8.7. Concrete Placement

Concrete in contact with soil or water that contains high concentrations of soluble sulfates can be subject to chemical and/or physical deterioration. Based on the CBC criteria (CBC, 2013), the potential for sulfate attack is negligible for water-soluble sulfate contents in soil ranging from 0.00 to 0.10 percent by weight (i.e., 0 to 1,000 ppm). As noted, the soil samples tested for this evaluation had water-soluble sulfate contents of approximately 0.003 to 0.068 percent by weight (i.e., 30 ppm to 680 ppm). Accordingly, the on-site soils are considered to have a negligible potential for sulfate attack. Due to the variable nature of the on-site soils and possible changes in the soil conditions during grading, we recommend that Type II/V cement with a water/cement ratio of 0.50 or less, be considered for the project.

8.8. Drainage

Roof, pad, and slope drainage should be directed such that runoff water is diverted away from slopes and structures to suitable discharge areas by nonerodible devices (e.g., gutters, downspouts, concrete swales, etc.). Positive drainage adjacent to structures should be established and maintained. Positive drainage may be accomplished by providing drainage away from the foundations of the structure at a gradient of 2 percent or steeper for a distance of 5 feet or more outside the structure perimeter, and further maintained by a graded swale leading to an appropriate outlet, in accordance with the recommendations of the project civil engineer and/or landscape architect.

Surface drainage on the site should be provided so that water is not permitted to pond. A gradient of 2 percent or steeper should be maintained over the pad area and drainage patterns should be established to divert and remove water from the site to appropriate outlets. Care should be taken by the contractor during final grading to preserve any berms, drainage terraces, interceptor swales or other drainage devices of a permanent nature on or adjacent to the property.

Drainage patterns established at the time of final grading should be maintained for the life of the project. The property owner and the maintenance personnel should be made aware that altering drainage patterns might be detrimental to slope stability and foundation performance.

8.9. Pavement Design

For design of flexible pavements, we have used Traffic Indices (TI) of 5, 6, and 7 to represent the volume and loading of the traffic for site pavements. If traffic loads are different from those assumed, the pavement design should be re-evaluated. Actual pavement recommendations should be based on R-value tests performed on bulk samples of the soils exposed at the finished subgrade elevations once grading operations have been performed.

The resistance (R-value) characteristics of the subgrade soils were evaluated by conducting laboratory testing on a representative soil sample obtained from our soil boring. The test result indicated an R-value of 10, which was utilized in our analysis. The preliminary recommended flexible pavement sections are as follows:

Table 4 – Recommended Pavement Sections

Traffic Index	R-value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
5.0	10	3.0	9.0
6.0	10	3.0	12.5
7.0	10	4.0	14.0

We recommend that the upper 12 inches of the subgrade and aggregate base materials be compacted to 95 percent relative compaction as evaluated by ASTM D 1557. The pavement sections should provide an approximate pavement life of 20 years.

In areas where concrete pavement is anticipated, as well as drainage swales and gutters, we recommend a rigid pavement section consisting of 6 inches of Portland cement concrete underlain by 12 inches of subgrade soils compacted to a relative compaction of 95 percent, as evaluated by ASTM D 1557.

9. CONSTRUCTION OBSERVATION

The recommendations provided in this report are based on our understanding of the proposed project and on our evaluation of the data collected based on subsurface conditions disclosed by widely spaced exploratory borings. It is imperative that the geotechnical consultant checks the interpolated subsurface conditions during construction. We recommend that Ninyo & Moore review the project plans and specifications prior to construction. It should be noted that, upon review of these documents, some recommendations presented in this report may be revised or modified.

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

- Observing clearing, grubbing, and removals.
- Observing excavation, placement, and compaction of fill.
- Evaluating imported materials prior to their use as fill (if used).
- Performing field tests to evaluate fill compaction.
- Observing cut slopes for fractures, joints, and other geologic planes of weakness.
- Observing placement of soil nail wall or other shoring methods.
- Observing installation of portions of the pipe using trenchless methods.
- Observing foundation excavations for bearing materials and cleaning prior to placement of reinforcing steel or concrete.
- Performing material testing services, including concrete compressive strength and steel tensile strength tests and inspections.

The recommendations provided in this report assume that Ninyo & Moore will be retained as the geotechnical consultant during the construction phase of this project. If another geotechnical consultant is selected, we request that the selected consultant indicate to the owner and to our firm in writing that our recommendations are understood and that they are in full agreement with our recommendations.

10. LIMITATIONS

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

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. 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.

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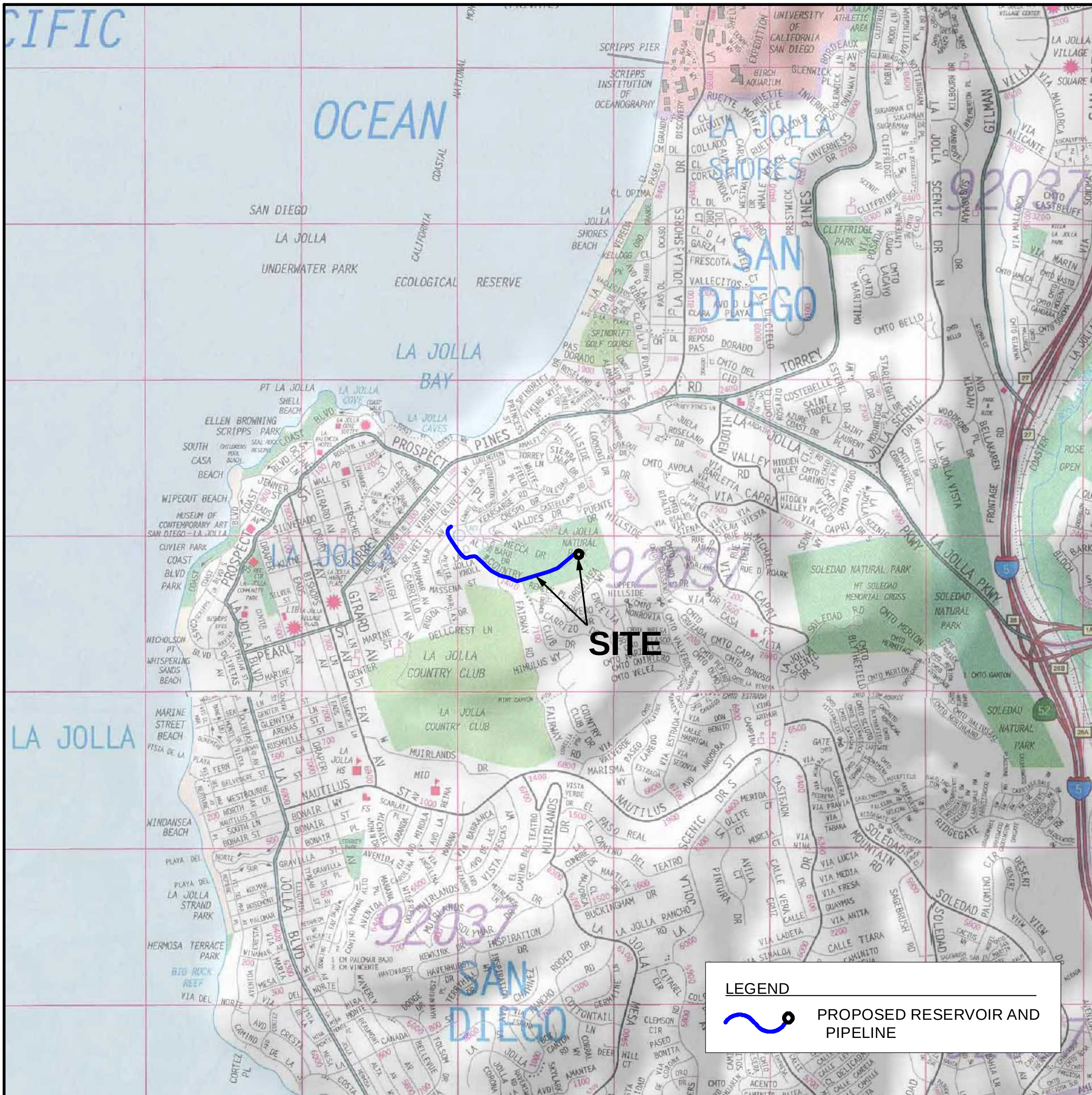
AERIAL PHOTOGRAPHS

Source	Date	Flight	Numbers	Scale
County of San Diego	1928	52	C-1 and CA-1	1:15,000
USDA	4-11-1953	AXN-8M	89 and 90	1:20,000
County of San Diego	10-09-1970	4	3 and 4	1:10,000
County of San Diego	11-25-1973	31	17 and 18	1:10,000
County of San Diego	10-23-1978	17B	53 and 54	1:20,000



SOURCE: 2008 THOMAS GUIDE FOR SAN DIEGO COUNTY, STREET GUIDE AND DIRECTORY; MAP © RAND MCNALLY, R.L.07-S-129.

MAP INDEX

N

SCALE IN FEET

0 2,400 4,800

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE

Ninyo & Moore

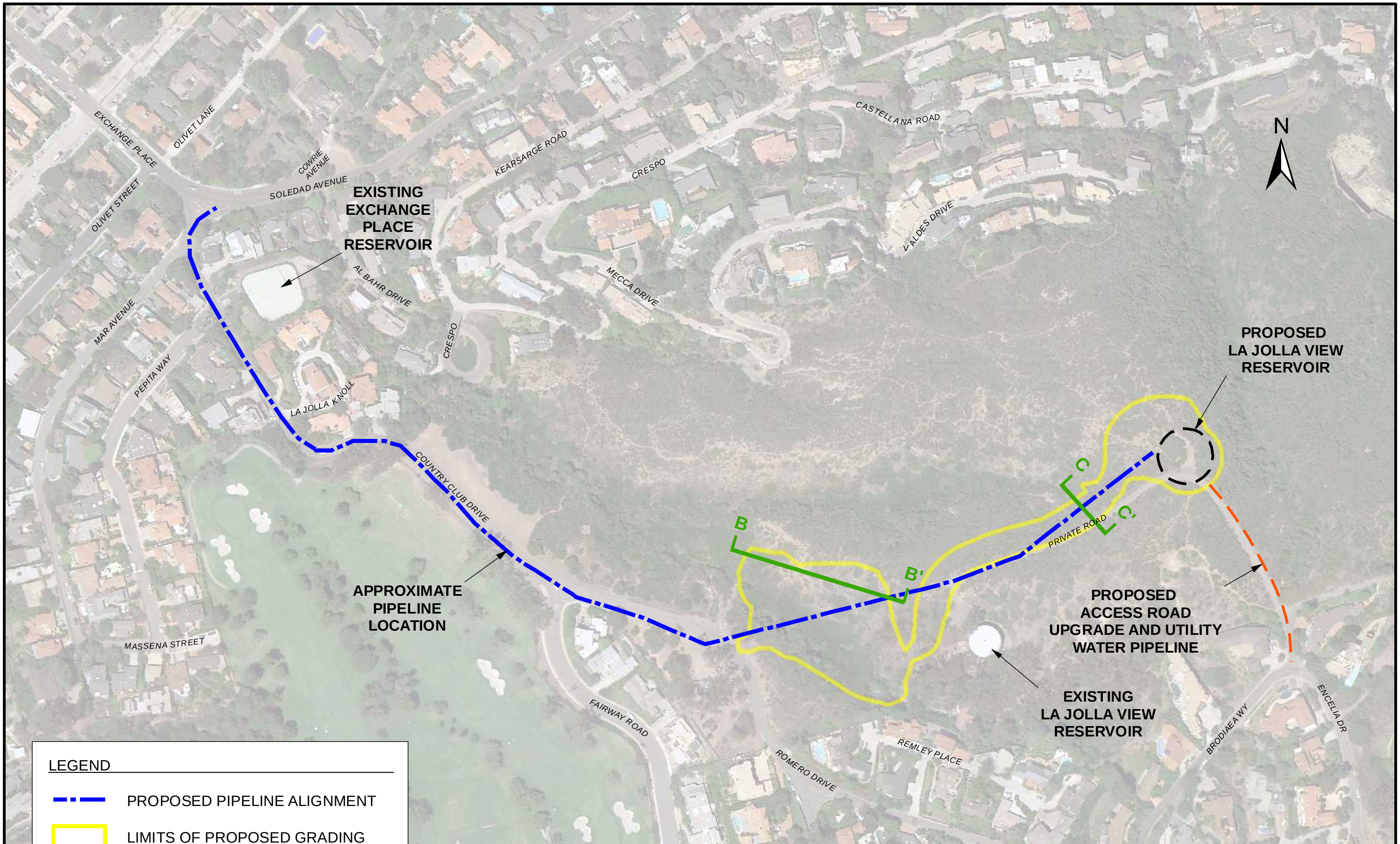
SITE LOCATION

FIGURE

PROJECT NO.	DATE
107314001	7/14

LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT
LA JOLLA, CALIFORNIA

1



SOURCE: AERIAL IMAGERY - (C) GOOGLE EARTH, 2013.

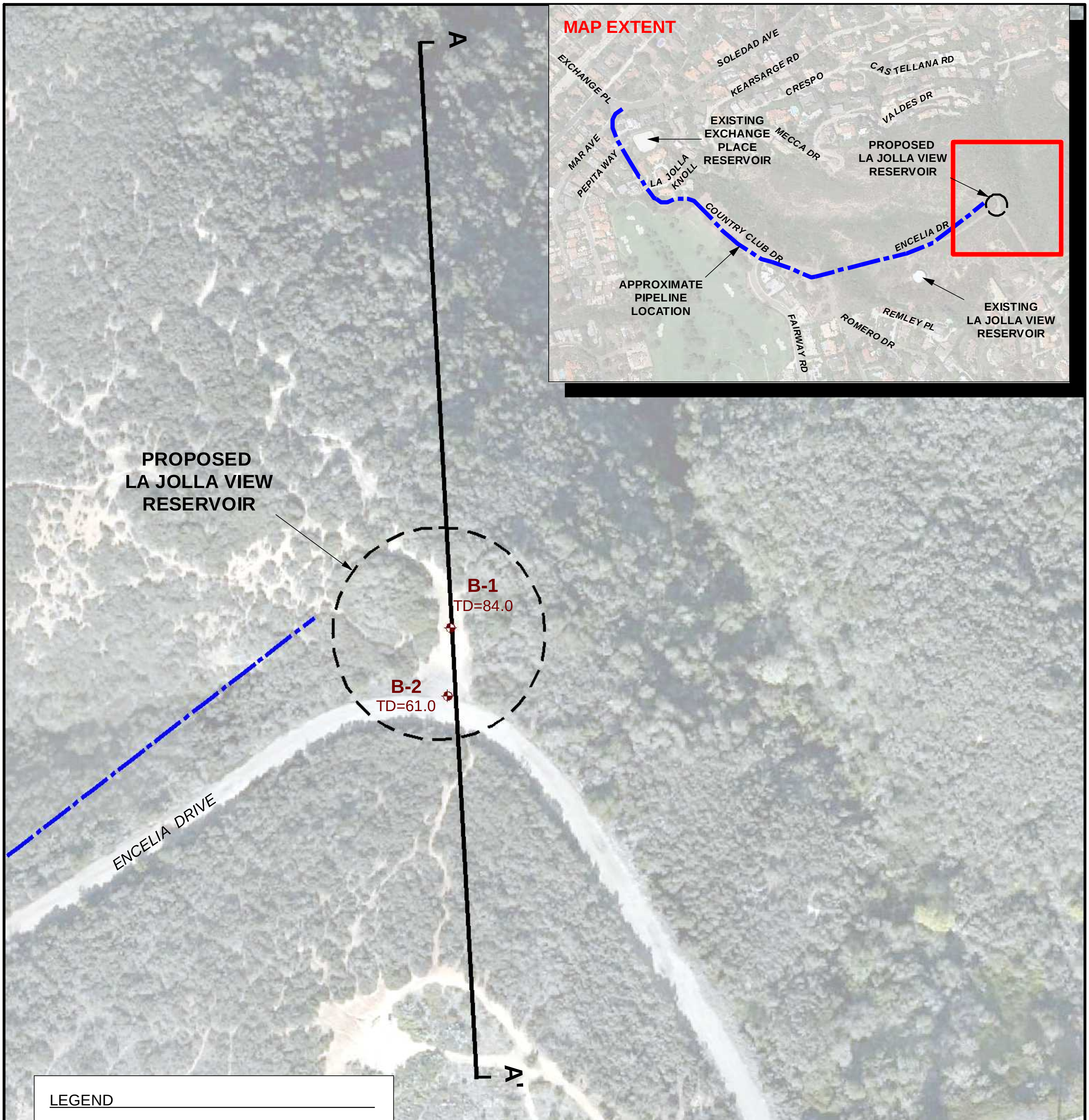
SCALE IN FEET

0 300 600

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

Ninyo & Moore		SITE PLAN	FIGURE
PROJECT NO.	DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	2
107314001	7/14		

2_107314001_SP.mxd 5/27/2014 3:16:25 PM JDL



LEGEND

B-2 TD=61.0	BORING LOCATION TD=TOTAL DEPTH IN FEET
	PIPELINE ALIGNMENT
A A'	CROSS-SECTION

SOURCE: AERIAL IMAGERY - GOOGLE EARTH, 2014.

N

SCALE IN FEET

0 80 160

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

3A_107314001_PB.mxd 5/27/2014 3:20:17 PM JDL

Ninyo & Moore	BORING LOCATIONS	FIGURE
PROJECT NO. 107314001 / DATE 7/14	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	3A

SOURCE: AERIAL IMAGERY - GOOGLE EARTH, 2014.

LEGEND

B-4 TD=20.0 BORING LOCATION TD=TOTAL DEPTH IN FEET

PIPELINE ALIGNMENT

N

SCALE IN FEET

0 40 80

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

Ninyo & Moore

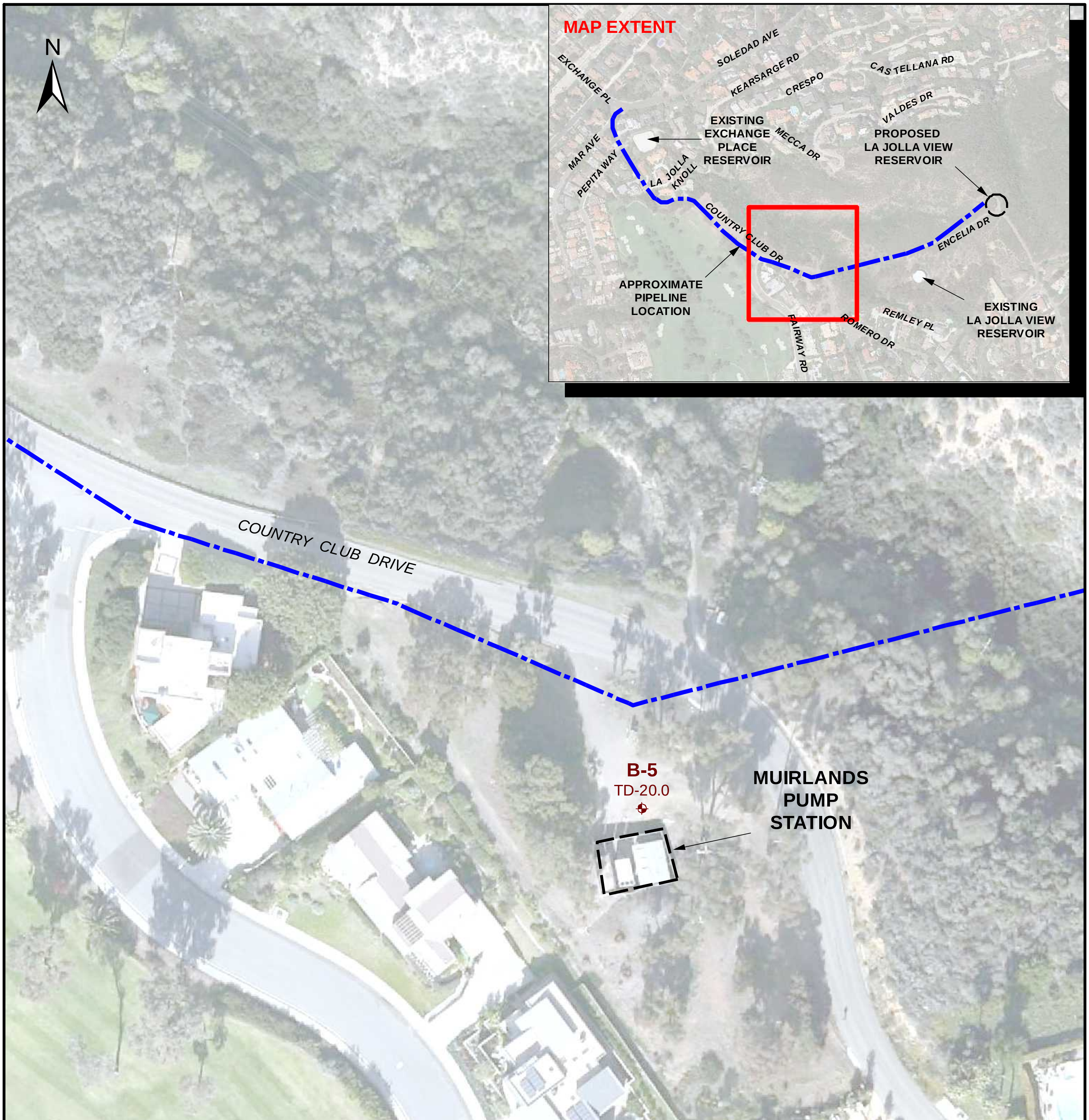
BORING LOCATIONS

PROJECT NO.	DATE
107314001	7/14

LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT
LA JOLLA, CALIFORNIA

FIGURE 3B

3B_107314001_PB.mxd 7/8/2014 JDL



SOURCE: AERIAL IMAGERY - GOOGLE EARTH, 2014.

LEGEND

B-5 TD-20.0 — BORING LOCATION, TD=TOTAL DEPTH IN FEET

PIPELINE ALIGNMENT

N

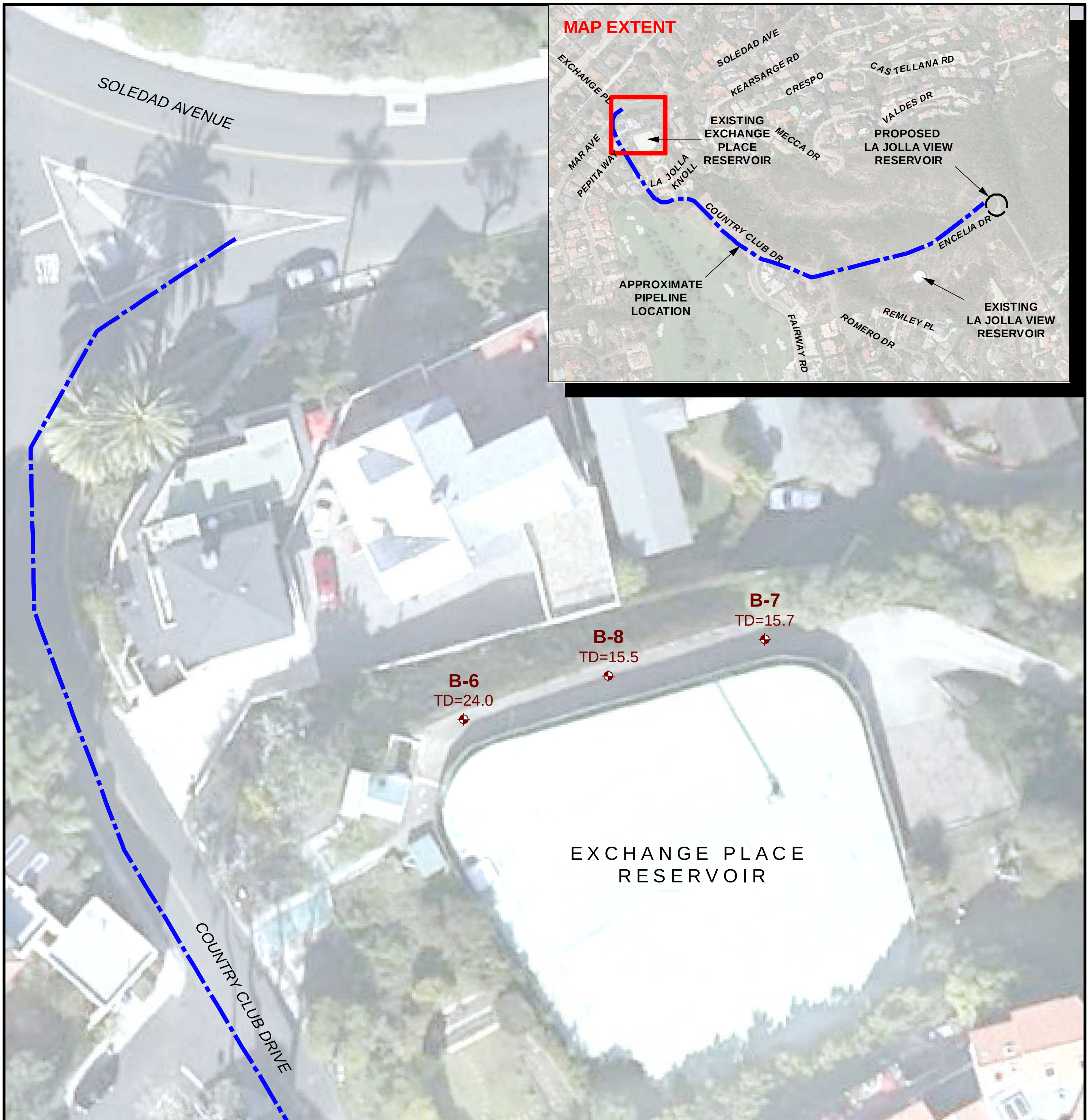
SCALE IN FEET

0 80 160

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

3C_107314001_PB.mxd 5/27/2014 3:24:32 PM JDL

Ninyo & Moore		BORING LOCATIONS	FIGURE
PROJECT NO.	DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	3C
107314001	7/14		



SOURCE: AERIAL IMAGERY - GOOGLE EARTH, 2014.

LEGEND

B-8 TD=15.5 BORING LOCATION
TD=TOTAL DEPTH IN FEET

PIPELINE ALIGNMENT

N

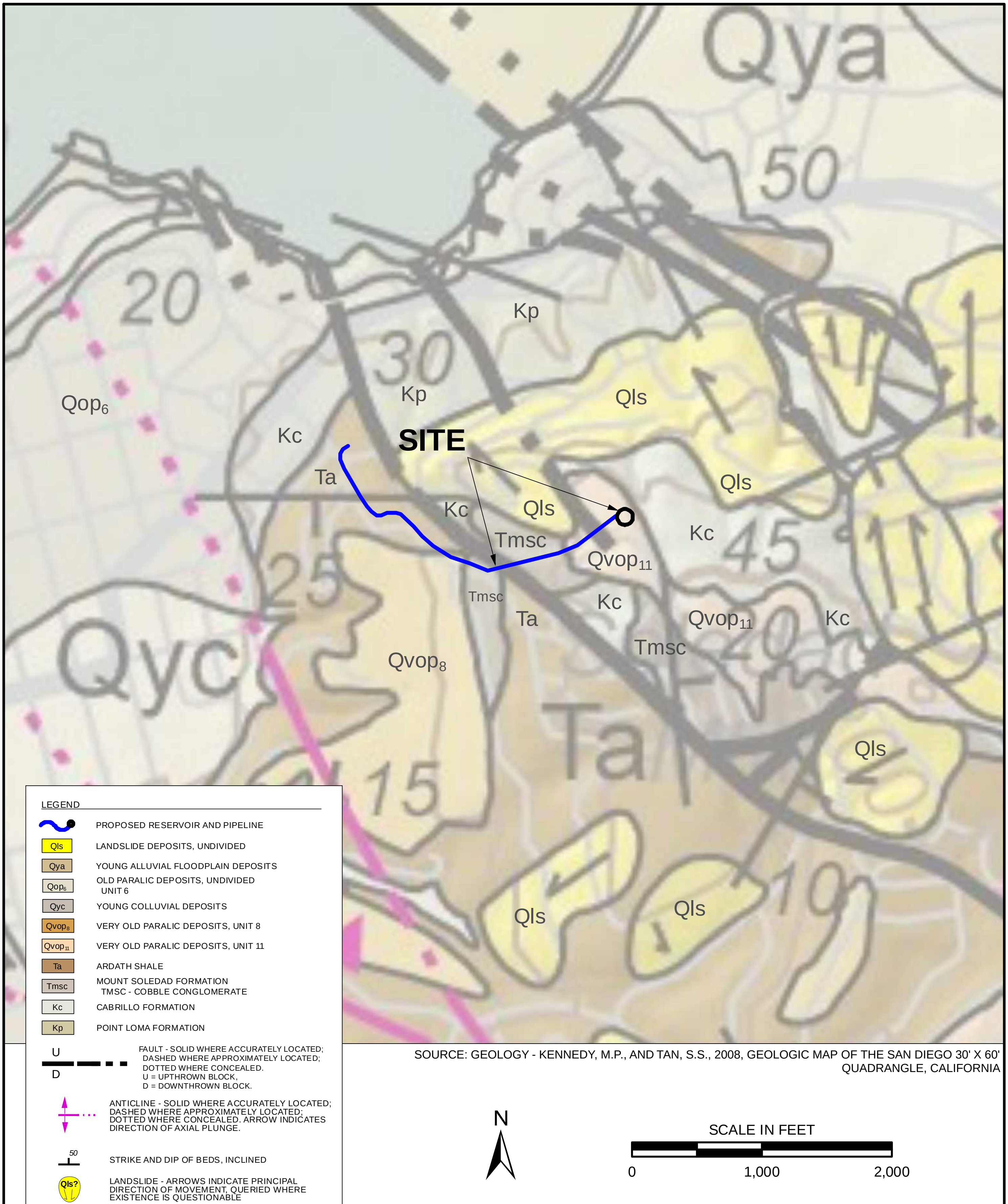
SCALE IN FEET

0 40 80

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

3D_107314001_PB.mxd 5/27/2014 3:25:00 PM JDL

Ninyo & Moore		BORING LOCATIONS	FIGURE
PROJECT NO.	DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	3D
107314001	7/14		



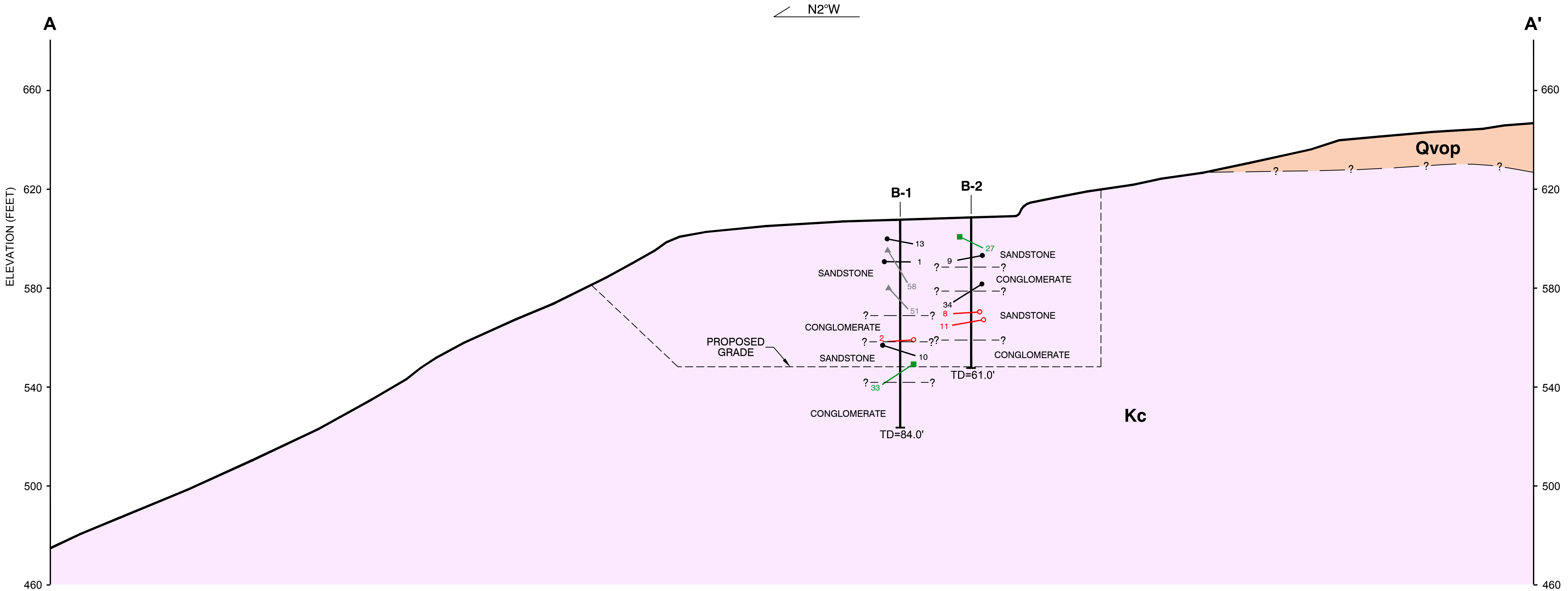
SOURCE: GEOLOGY - KENNEDY, M.P., AND TAN, S.S., 2008, GEOLOGIC MAP OF THE SAN DIEGO 30' X 60' QUADRANGLE, CALIFORNIA

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

Ninyo & Moore

REGIONAL GEOLOGY

PROJECT NO.	DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	FIGURE
107314001	7/14		4



LEGEND

B-2 TD=61.0'	BORING TD=TOTAL DEPTH IN FEET	20	APPARENT DIP OF BED
		20	APPARENT DIP OF JOINT/FRACTURE
		20	APPARENT DIP OF FAULT
Qvop	VERY OLD PARALIC DEPOSITS	20	APPARENT DIP OF CLAY SEAM
Kc	CABRILLO FORMATION		

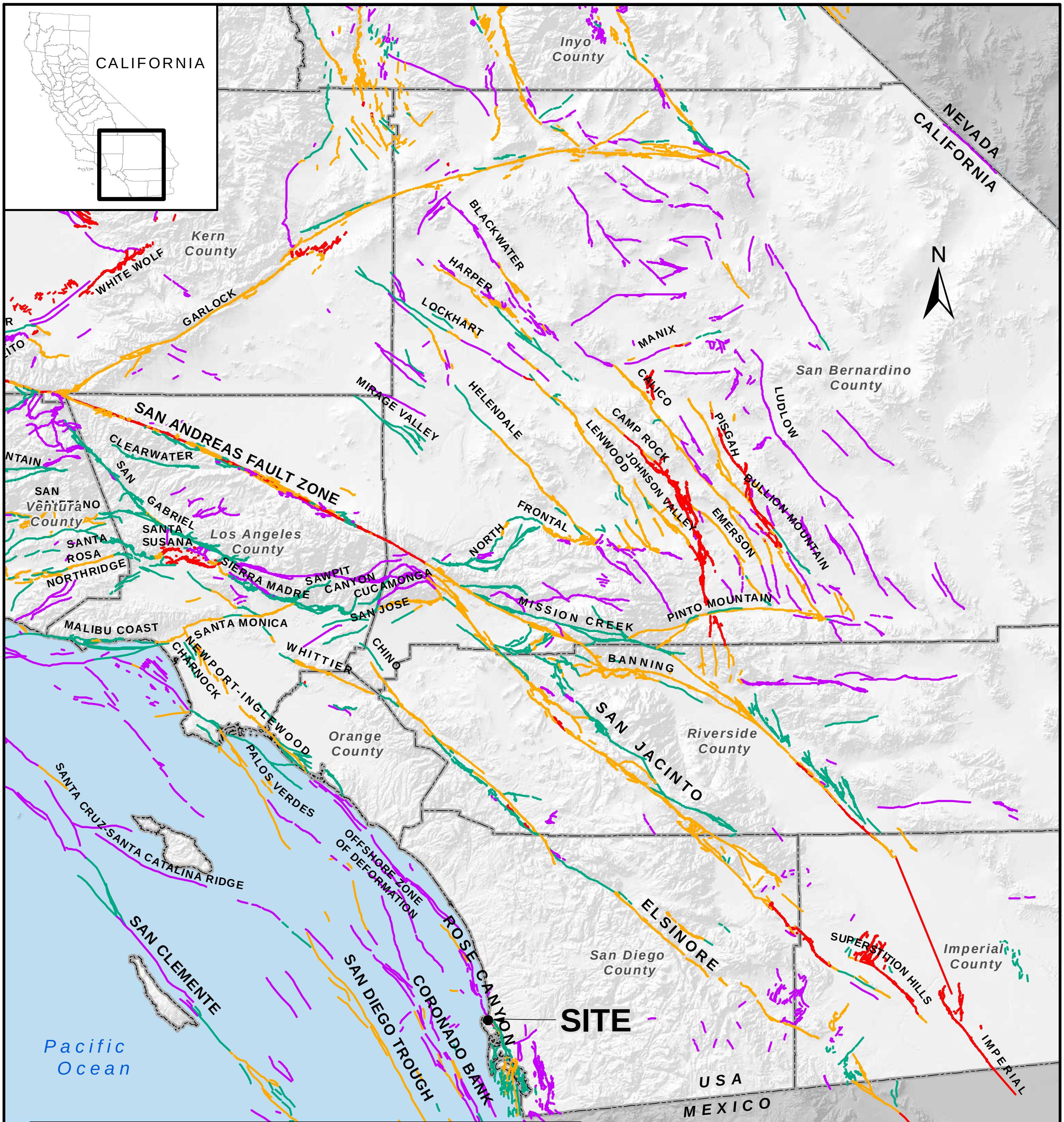
SCALE IN FEET

0 40 80

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE

5B 107314001 cs b-b'.dwg AOB

Ninyo & Moore		GEOLOGIC CROSS-SECTION A-A'	FIGURE
PROJECT NO.	DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	5
107314001	7/14		



LEGEND

CALIFORNIA FAULT ACTIVITY

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

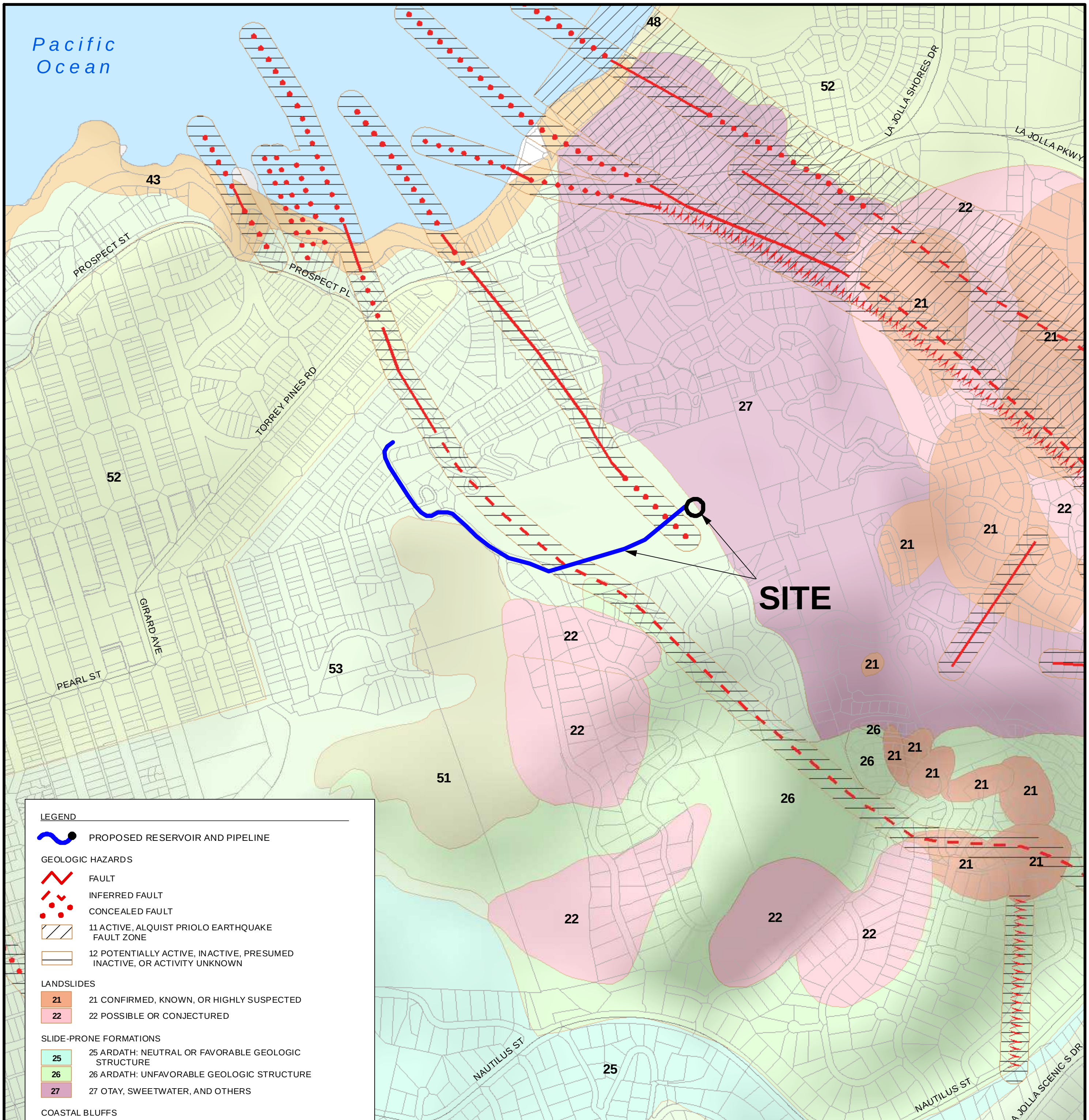
SOURCE: U.S. GEOLOGICAL SURVEY AND CALIFORNIA GEOLOGICAL SURVEY, 2006, QUATERNARY FAULT AND FOLD DATABASE FOR THE UNITED STATES.

SCALE IN MILES

0 30 60

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

Ninyo & Moore		FAULT LOCATIONS	FIGURE
PROJECT NO.	DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	6
107314001	7/14		



SOURCE: SANGIS, 2008, CITY OF SAN DIEGO SEISMIC SAFETY STUDY GEOLOGIC HAZARDS AND FAULTS.

N

SCALE IN FEET

0 1,000 2,000

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

Ninyo & Moore		GEOLOGIC HAZARDS	FIGURE
PROJECT NO.	DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	7
107314001	7/14		

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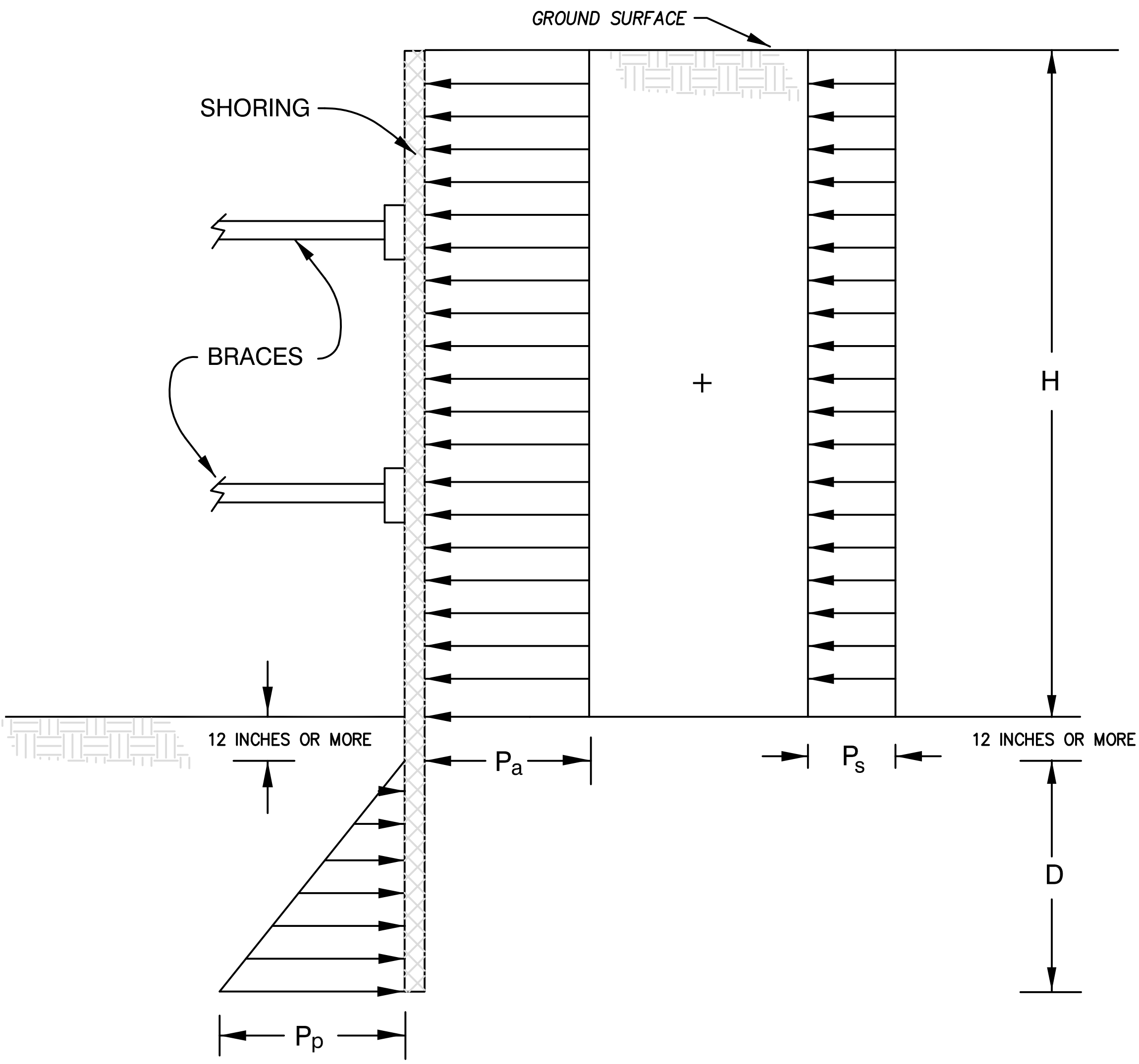


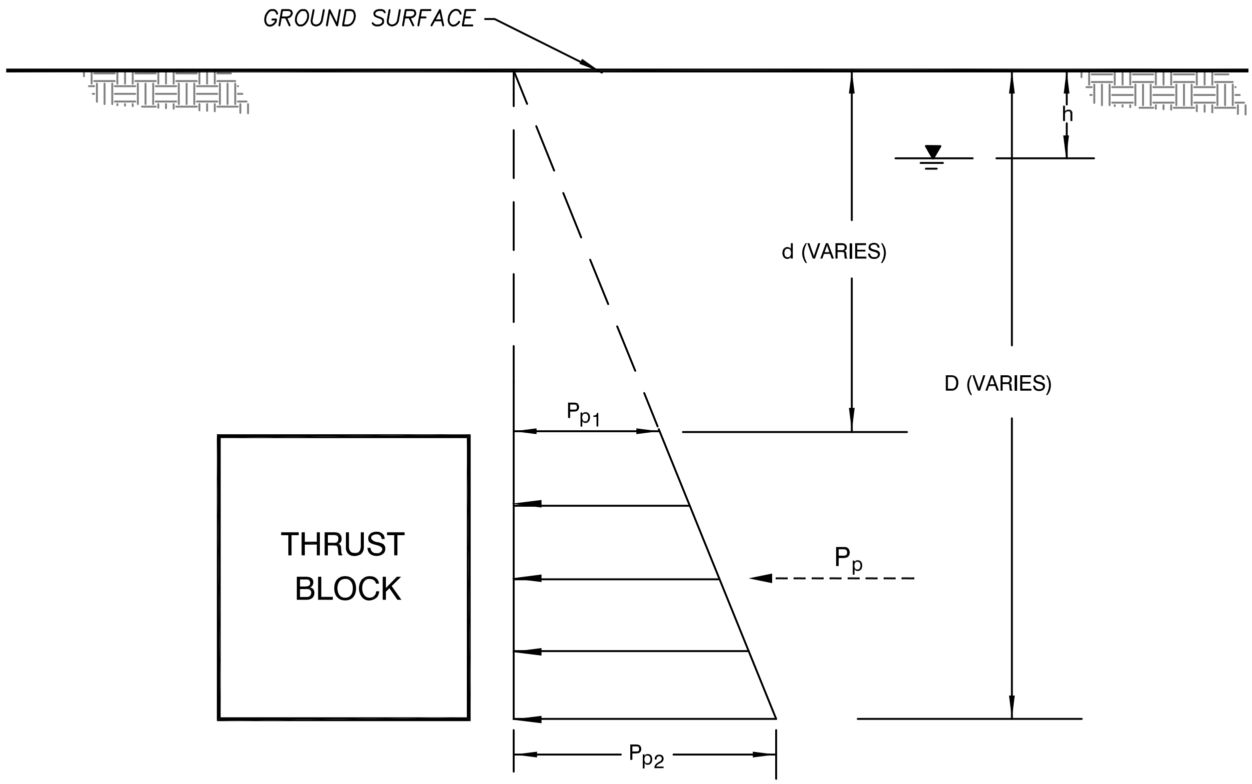
NOTES:

1. APPARENT LATERAL EARTH PRESSURE, P_a
   $P_a = 25\ H$ psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   $P_s = 120$ psf
3. PASSIVE LATERAL EARTH PRESSURE, P_p
   $P_p = 350\ D$ psf
4. ASSUMES GROUNDWATER IS NOT PRESENT
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H AND D ARE IN FEET

NOT TO SCALE

8 107314001 d-beg.dwg

Ninyo & Moore		LATERAL EARTH PRESSURES FOR BRACED EXCAVATION (GRANULAR SOIL)	FIGURE
PROJECT NO.	DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	8
107314001	7/14		



NOTES:

1. GROUNDWATER BELOW BLOCK

   $P_p = 175\ (D^2\text{-}d^2\ )$ lb/ft

2. GROUNDWATER ABOVE BLOCK

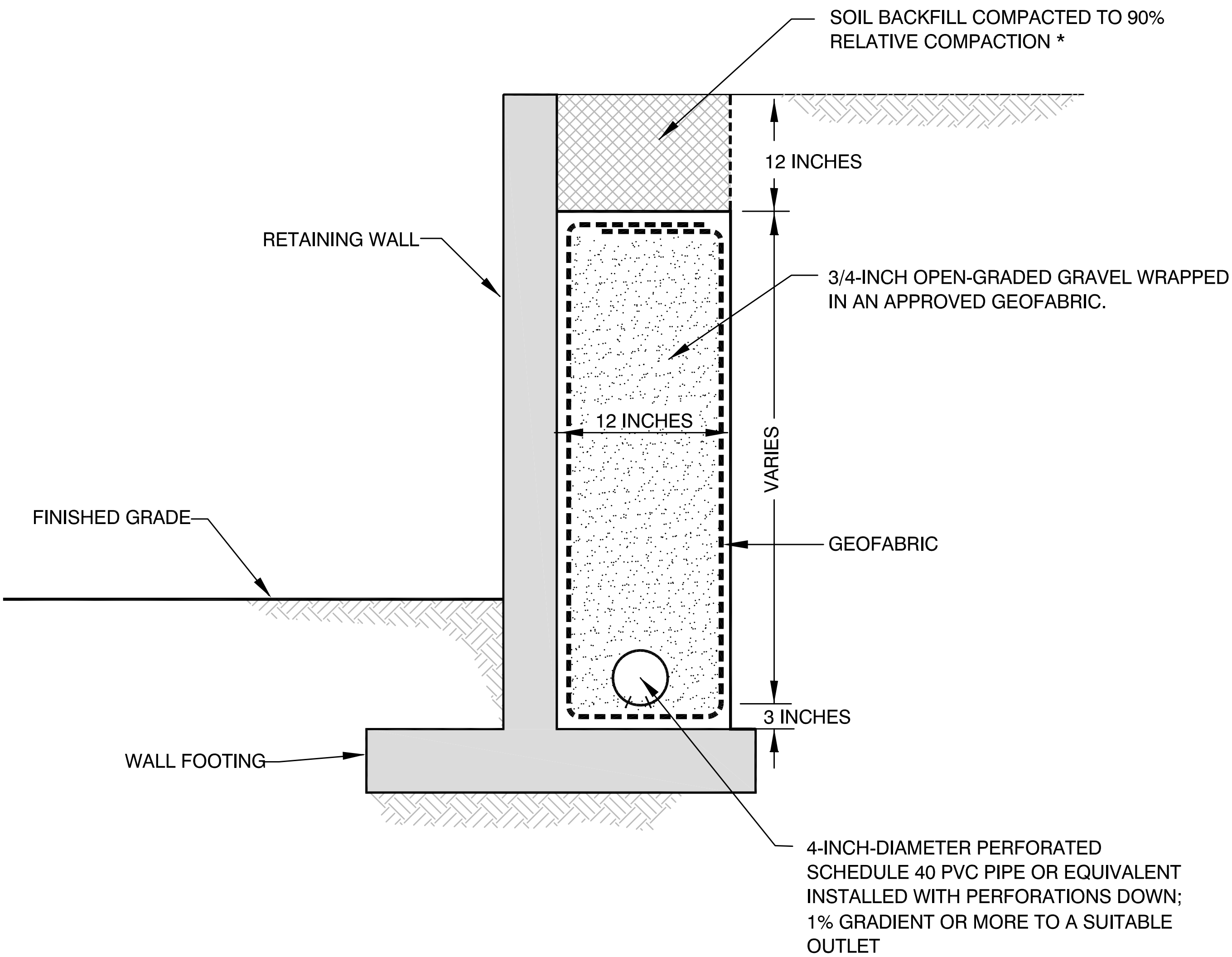
   $P_p = 1.5\ (\ D - d\ )[\ 124.8h + 57.6\ (\ D{+}d\ )]$ lb/ft

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

9 107314001 d-tb.dwg

NOT TO SCALE

Ninyo & Moore	THRUST BLOCK LATERAL EARTH PRESSURE DIAGRAM	FIGURE
PROJECT NO. 107314001 / DATE 7/14	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	9



*BASED ON ASTM D1557

NOT TO SCALE

10 107314001_d-rwd.dwg

NOTE: AS AN ALTERNATIVE, AN APPROVED GEOCOMPOSITE DRAIN SYSTEM MAY BE USED.

Ninyo & Moore		RETAINING WALL DRAINAGE DETAIL	FIGURE
PROJECT NO.	DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	10
107314001	7/14		

APPENDIX A

BORING 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 borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler

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

Field Procedure for the Collection of Relatively Undisturbed Samples

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

The Modified Split-Barrel Drive Sampler

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

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES: Bulk	SAMPLES: Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0								Bulk sample.
								Modified split-barrel drive sampler.
								2-inch inner diameter split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler, or 2-inch inner diameter split-barrel drive sampler.
								Sample retained by others.
5								Standard Penetration Test (SPT).
								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
10								Seepage.
								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	MAJOR MATERIAL TYPE (SOIL): Solid line denotes unit change.
							CL	Dashed line denotes material change.
15								Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
								The total depth line is a solid line that is drawn at the bottom of the boring.
20								

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BORING LOG

Explanation of Boring Log Symbols

PROJECT NO.	DATE	FIGURE

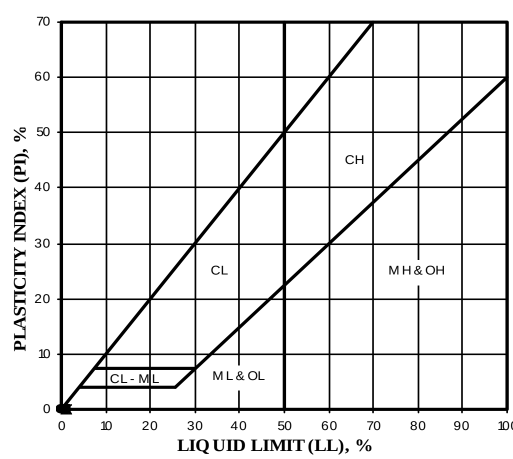
U.S.C.S. METHOD OF SOIL CLASSIFICATION

MAJOR DIVISIONS		SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (More than 1/2 of soil > No. 200 Sieve Size)	GRAVELS (More than 1/2 of coarse fraction > No. 4 sieve size	GW	Well graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (More than 1/2 of coarse fraction < No. 4 sieve size	SW	Well graded sands or gravelly sands, little or no fines
		SP	Poorly graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (More than 1/2 of soil < No. 200 sieve size)	SILTS & CLAYS Liquid Limit <50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS Liquid Limit >50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity, organic silty clays, organic silts
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils

GRAIN SIZE CHART

CLASSIFICATION	RANGE OF GRAIN	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	306 to 76.2
GRAVEL	3" to No. 4	76.2 to 4.76
Coarse	3" to 3/4"	76.2 to 19.1
Fine	3/4" to No. 4	19.1 to 4.76
SAND	No. 4 to No. 200	4.76 to 0.075
Coarse	No. 4 to No. 10	4.76 to 2.00
Medium	No. 10 to No. 40	2.00 to 0.420
Fine	No. 40 to No. 200	0.420 to 0.075
SILT & CLAY	Below No. 200	Below 0.075

PLASTICITY CHART



Ninyo & Moore

U.S.C.S. METHOD OF SOIL CLASSIFICATION

Updated Nov. 2011

DATE DRILLED	3/26/14	BORING NO.	B-1
GROUND ELEVATION	608' ± (MSL)	SHEET	1 OF 3
METHOD OF DRILLING	30" Bucket Auger (Pacific Drilling - EZ Bore)		
DRIVE WEIGHT	See notes	DROP	12"
SAMPLED BY	BTM	LOGGED BY	BTM/RDH
REVIEWED BY	RDH		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							CL	TOPSOIL: Mottled dark brown and gray, moist, stiff, silty CLAY; abundant roots extending to 2 feet in depth.
								CABRILLO FORMATION: Mottled light brown and white, damp, moderately cemented, sandy SILTSTONE; scattered gravel. Layer of rounded gravel 1 to 4 inches in diameter.
		X	8	10.0	117.2			Light brown, damp, moderately cemented, silty fine-grained SANDSTONE; scattered gravel. Generally massive with near vertical fracturing infilled with white calcium carbonate. Fractures are approximately 1/2-inch wide and extend from 3 feet to 11 feet.
								@9'-10':Alternating yellowish brown to gray beds; undulatory bedding.
10	X	X	5	11.0	111.6			
								@13': Distinct concretions on north and south sides approximately 1 foot in diameter at 13.1 feet and 13.8 feet, at 14 feet to 15 feet (south side), and at 13.5 feet to 14 feet (north side); well cemented. @13'-17': Tight clay lined fractures 1/8-inch wide on southwest side of hole; undulatory; near vertical. @15': Clay rip-up clasts 2 to 6 inches in diameter. @15'-16': 2 to 6 inch gravel and cobble clasts. @15'-17': Concretions on north side. @17': b: N 10°W, 6°SW.
20		X	5	9.8	113.0			@18'-20': Discontinuous concretion on south side; clay rip-up clasts 2 inches in diameter. Massive. @21'-30.7': Fault zone of undulatory polished clay-lined surfaces within sandstone. Clay is moist and soft; sandstone on NE side of fault zone and sandstone with claystone on SW side. @20.2': F: N 55°W, 63°S. @22': Concretionary zones on south side; rip-up clasts. @22'-24': 1/16- to 1/8-inch thick clay lined fractures.
30		X	6	10.3	120.1			Gravel. @30.7': F: N 60°W, 55°S; undulatory.
	X							@33': Discontinuous concretionary layer approximately 8 inches thick; claystone rip-up clasts approximately 6 inches in diameter within massive sandstone. Rip-up clasts and gravel more numerous with depth.
								@36': Gradational contact to light brown, damp, moderately cemented, sandy CONGLOMERATE; clasts up to 6 inches in diameter in matrix of fine silty sand; clasts are well rounded and composed of granitic and metavolcanic rock.
40								

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BORING LOG

LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT
LA JOLLA, CALIFORNIA

PROJECT NO.	DATE	FIGURE
107314001	7/14	A-1

DATE DRILLED 3/26/14 BORING NO. B-1

GROUND ELEVATION 608' ± (MSL) SHEET 2 OF 3

METHOD OF DRILLING 30" Bucket Auger (Pacific Drilling - EZ Bore)

DRIVE WEIGHT See notes DROP 12"

SAMPLED BY BTM LOGGED BY BTM/RDH REVIEWED BY RDH

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40	■							CABRILLO FORMATION: (Continued) Light brown, damp, moderately cemented, sandy CONGLOMERATE. @44': Clasts composed of granitic rock. @45'-46.5': Sandstone layer on north side. @48': Well cemented concretion approximately 2.5 feet in diameter along southwest side.
	■							
		■	10	13.9	118.7			@48'-48.8': Base of conglomerate. North side is a mixture of sandstone and claystone rip-up clasts.
50								@49': Continuous zone of soft to stiff clay approximately 4 inches thick at contact with conglomerate; undulatory. @49.3': cs N15°W, 5-15°W; 1/4- to 1/2-inch thick; well developed polished parting surfaces. Below clay sharp contact to light gray to brown, moist, moderately cemented, silty fine-grained SANDSTONE. @50': Sandstone becomes well cemented; trace of clay. @52': Fine clay-filled fractures; very tight. @52.2': b: N35°E, 10-22°SE; crossbedding.
60		■	15	10.3	111.5			@61': f: N 20°E, 60°W; tight fracture lined with clay.
								Undulatory contact (sharp) to brown, damp, weakly cemented, sandy CONGLOMERATE; clasts 1-6 inches in diameter with a matrix of fine silty sand.
70								
								Undulatory contact to dark gray, damp, well indurated, silty CLAYSTONE; 6-8 inches of light gray, damp, moderately cemented, silty fine-grained sandstone above and below claystone.
	■							
								Brown, damp, weakly cemented, sandy CONGLOMERATE.
80								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
80								CABRILLO FORMATION: (Continued) Brown, damp, weakly cemented, sandy CONGLOMERATE.
								Total Depth = 84 feet. (Logged to 82 feet) Groundwater not encountered during drilling. Backfilled with bentonite and soil cuttings on 3/27/14. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. Drive Weight: 0 to 27 feet: 4,500 lbs.; 27 to 50 feet: 3,700 lbs.; 50 to 80 feet: 1,500 lbs.
90								
100								
110								
120								

DATE DRILLED 3/26/14 BORING NO. B-1

GROUND ELEVATION 608' ± (MSL) SHEET 3 OF 3

METHOD OF DRILLING 30" Bucket Auger (Pacific Drilling - EZ Bore)

DRIVE WEIGHT See notes DROP 12"

SAMPLED BY BTM LOGGED BY BTM/RDH REVIEWED BY RDH

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BORING LOG

LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT
LA JOLLA, CALIFORNIA

PROJECT NO.	DATE	FIGURE
107314001	7/14	A-3

DATE DRILLED	3/27/14	BORING NO.	B-2
GROUND ELEVATION	608' ± (MSL)	SHEET	1 OF 2
METHOD OF DRILLING	30" Bucket Auger (Pacific Drilling - EZ Bore)		
DRIVE WEIGHT	See notes	DROP	12"
SAMPLED BY	BTM	LOGGED BY	BTM/RDH
REVIEWED BY	RDH		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							CL	ASPHALT CONCRETE: Approximately 2.5 inches thick.
	X						CL	FILL: Brown, damp, soft to firm, sandy CLAY; scattered roots, gravel, and cobbles.
								TOPSOIL: Brown, moist, stiff, sandy CLAY.
								CABRILLO FORMATION: Light brown, damp, moderately cemented, silty fine-grained SANDSTONE; massive. @4.6'-7.3': f: N18°E, 55°W; calcium carbonate infilled fractures; 1-2 inches wide; N 18° E and 55°W.
10								@9.4': f: N15°W, 66°W; fracture filled with calcium carbonate.
								@11': Carbonate layer 1/4-inches thick; semi-continuous; yellowish brown to gray; 6-inch thick concretionary layer extends approximately 1/2 hole.
								@13': Calcium carbonate and 8-inch thick concretion.
								@14.5': 6-inch thick concretionary layer.
		X	4	11.9	115.5			
								@16': b: N 70°W, 10° N.
								@18'-18.5': Concretion.
20								Sharp, undulatory contact to brown, damp, moderately cemented, sandy CONGLOMERATE; matrix of fine silty sand; rounded clasts 2-6 inches in diameter.
30								Undulatory contact to light brown, damp, moderately cemented, silty fine-grained SANDSTONE; c: N78°E, 34°N.
								@31': Scattered gravel and cobbles; trace of clay.
								@32.6': Gravel layer. @33': Light gray; no gravel.
		X	8	12.2	108.2			
								@38'-39.5': Disturbed zone; with seams of dark gray, moist, soft clay; 1/16-1/8 inches thick; undulatory. @38.5': sz: N25°W, 80°NE; approximately 1/4-inch discontinuous layer of soft clay. @39': cs: N 25° to 30°W and 15° to 22° NE.
40								

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BORING LOG

LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT
LA JOLLA, CALIFORNIA

PROJECT NO.	DATE	FIGURE
107314001	7/14	A-4

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 3/27/14 BORING NO. B-2
								GROUND ELEVATION 608' ± (MSL) SHEET 2 OF 2
								METHOD OF DRILLING 30" Bucket Auger (Pacific Drilling - EZ Bore)
								DRIVE WEIGHT See notes DROP 12"
								SAMPLED BY BTM LOGGED BY BTM/RDH REVIEWED BY RDH
								DESCRIPTION/INTERPRETATION
40	■							CABRILLO FORMATION: (Continued) Light gray, damp, moderately cemented, silty fine-grained SANDSTONE. @41': cs: N45°W, 35°NE; soft clay; base of disturbed zone. @41.9': cs: N45°E, 15°NW, 1/8-1/2-inch thick, dark gray, moist, soft clay; sandstone above and below; undulatory. @43': Scattered gravel. @43'-44': Rip-up claystone clasts; 1-2 inches in diameter.
		■	8	8.8	115.6			@46.2': 1/8-inch thick red stained laminate; undulatory; approximately horizontal. @49': Gravelly; grades to conglomerate.
50								Brown, damp, moderately cemented, sandy CONGLOMERATE. @51'-52': Sandstone layer with few gravel clasts.
60								Total Depth = 61 feet. (Logged to 59 feet) Groundwater not encountered during drilling. Backfilled with bentonite and soil cuttings on 3/28/14. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. Drive Weight: 0 to 27 feet: 4,500 lbs.; 27 to 50 feet: 3,700 lbs.; 50 to 61 feet: 1,500 lbs.
70								
80								

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BORING LOG

LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT
LA JOLLA, CALIFORNIA

PROJECT NO.	DATE	FIGURE
107314001	7/14	A-5

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							GC	ASPHALT CONRETE: Approximately 4 inches thick.
								FILL: Approximately 3 inches thick. Brown, moist, medium dense, clayey GRAVEL.
								CABRILLO FORMATION: Light brown, damp, moderately cemented, sandy CONGLOMERATE.
10								Sharp contact to gray to reddish brown, moist, weakly to moderately indurated clayey SILTSTONE.
								Light brown to light gray, damp to moist, weakly cemented, silty fine-grained SANDSTONE; massive. @11.7-12.2': Scattered concretions. @12.2': Moderately cemented. @15'-17': f: N28°E, near vertical fracture infilled with clay; 1/16-inch wide.
		X	4	13.0	111.1			@17': b: N55°E, 35°W; light brown with reddish staining. @17'-27': Mostly massive sandstone; scattered siltstone/claystone rip-up clasts; near vertical fractures with iron oxide staining.
20								Light gray.
								@23.2'-23.8': Scattered concretions; light brown.
								Dark gray to reddish brown, damp, moderately to strongly indurated, silty CLAYSTONE. @25'-26.3': Concretion.
								Grayish brown, damp, weakly to moderately indurated, clayey SILTSTONE; near horizontal, sharp contact with iron oxide staining; massive.
30								
								Total Depth = 35 feet. (Logged to 33 feet) Groundwater not encountered during drilling. Backfilled with bentonite and soil cuttings on 3/31/14. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. Drive Weight: 0 to 27 feet: 4,500 lbs.; 27 to 35 feet: 3,700 lbs.
40								

DATE DRILLED 3/31/14 BORING NO. B-3

GROUND ELEVATION 512' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 30" Bucket Auger (Pacific Drilling - EZ Bore)

DRIVE WEIGHT See notes DROP 12"

SAMPLED BY BTM LOGGED BY BTM/FOM REVIEWED BY RDH

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BORING LOG

LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT
LA JOLLA, CALIFORNIA

PROJECT NO.	DATE	FIGURE
107314001	7/14	A-6

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/3/4"	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 10 inches thick.
							GP	FILL: Brown, damp, dense, sandy to silty GRAVEL; with cobbles. Difficult drilling.
			50/6"					
10								
								Auger refusal on cobbles.
20								Total Depth = 20 feet. (Refusal) Groundwater not encountered during drilling. Backfilled shortly after drilling on 2/19/14. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
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40								

DATE DRILLED 2/19/14 BORING NO. B-4

GROUND ELEVATION 507' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 6" Diameter Hollow Stem Auger (Pacific) (Wolverine)

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

SAMPLED BY GS LOGGED BY GS REVIEWED BY RDH

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BORING LOG

LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT
LA JOLLA, CALIFORNIA

PROJECT NO.	DATE	FIGURE
107314001	7/14	A-8

DATE DRILLED 2/19/14 BORING NO. B-5

GROUND ELEVATION 397' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 6" Diameter Hollow Stem Auger (Pacific) (Wolverine)

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

SAMPLED BY GS LOGGED BY GS REVIEWED BY RDH

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/3/4"	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SC	FILL: Light brown to brown, damp, medium dense, clayey fine to medium SAND; scattered chunks of asphalt; little cobbles.
							Medium dense to dense.
							Grinding; difficult drilling.
	Bulk; Driven	68				GC	COLLUVIUM: Brown to reddish brown, moist, very dense, clayey GRAVEL; some sand.
			5.6				Reddish brown.
10	Driven	50/5"					MOUNT SOLEDAD FORMATION: Yellowish brown, moist, weakly cemented, fine sandy SILTSTONE; some gravel.
	Driven	31	16.0	111.0			
	Driven	43					Light brown, moist, weakly cemented, silty SANDSTONE; few gravel.
	Driven	40					Brown, moist, weakly cemented, clayey SILTSTONE. Abundant gravel and cobbles.
20							Total Depth = 20 feet. (Refusal) Groundwater not encountered during drilling. Backfilled shortly after drilling on 2/19/14.
							Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
30							
40							

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BORING LOG

LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT
LA JOLLA, CALIFORNIA

PROJECT NO.	DATE	FIGURE
107314001	7/14	A-9

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/3/4"	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.

DATE DRILLED 2/20/14 BORING NO. B-6

GROUND ELEVATION 268' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 6" Diameter Hollow Stem Auger (Pacific) (Wolverine)

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

SAMPLED BY GS LOGGED BY GS REVIEWED BY RDH

DESCRIPTION/INTERPRETATION

DEPTH (feet)	BLOWS/3/4"	MOISTURE (%)	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0				ASPHALT CONCRETE: Approximately 4 inches thick.
			SM	FILL: Yellowish brown, damp, medium dense, silty fine SAND; slightly clayey; scattered gravel.
			CL	Gray brown, moist, stiff, gravelly CLAY; scattered to abundant gravel and cobbles.
	10	9.9		
10				
				More cobbles; difficult drilling.
20				
	50/6"			Hard; scattered pieces of siltstone; some cobbles.
				Refusal on cobbles.
				Total Depth = 24 feet. (Refusal) Groundwater not encountered during drilling. Backfilled with approximately 7 cubbic feet of grout shortly after drilling on 2/20/14. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
30				
40				

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BORING LOG

LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT
LA JOLLA, CALIFORNIA

PROJECT NO.	DATE	FIGURE
107314001	7/14	A-10

DATE DRILLED	2/20/14	BORING NO.	B-7
GROUND ELEVATION	267' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	6" Diameter Hollow Stem Auger (Pacific) (Wolverine)		
DRIVE WEIGHT	140 lbs. (Auto-Trip)	DROP	30"
SAMPLED BY GS	LOGGED BY GS	REVIEWED BY	RDH

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/3/4"	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SC	ASPHALT CONCRETE: Approximately 2 inches thick.
	X			6.2				FILL: Brown, moist, medium dense to dense, clayey SAND; scattered to abundant gravel and cobbles.
		X	24					Grayish brown; dense; damp to moist.
10	X	X	50/6"					
		X	50/3"				SM	Grayish brown and yellowish brown, damp, dense, sandy GRAVEL; numerous cobbles.
								Total Depth = 15.7 feet. (Refusal) Groundwater not encountered during drilling. Backfilled with on 2/20/14. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
20								
30								
40								

Ninyo & Moore

BORING LOG

LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT
LA JOLLA, CALIFORNIA

PROJECT NO.	DATE	FIGURE
107314001	7/14	A-11

DATE DRILLED	2/20/14	BORING NO.	B-8
GROUND ELEVATION	271' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	6" Diameter Hollow Stem Auger (Pacific) (Wolverine)		
DRIVE WEIGHT	140 lbs. (Auto-Trip)	DROP	30"
SAMPLED BY	GS	LOGGED BY GS	REVIEWED BY RDH

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/3/4"	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							CL	ASPHALT CONCRETE: Approximately 2 inches thick.
								FILL: Brown, moist, stiff, gravelly CLAY.
								Grayish brown; damp to moist.
10			30	6.7				Hard; scattered pieces of siltstone, cobbles, and sandstone.
			50/6"					Grinding on numerous cobbles; refusal.
								Total Depth = 15.5 feet. (Refusal) Groundwater not encountered during drilling. Backfilled with on 2/20/14. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
20								
30								
40								

Ninyo & Moore

BORING LOG

LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT
LA JOLLA, CALIFORNIA

PROJECT NO.	DATE	FIGURE
107314001	7/14	A-12

APPENDIX B

LABORATORY TESTING

Classification
Soils and formational 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 boring logs in Appendix A.

Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 422. The grain-size distribution curves are shown on Figures B-1 through B-5. These test result were utilized in evaluating the soil classifications in accordance with the USCS.

Atterberg Limits
Tests were performed on selected representative fine-grained 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-6.

Direct Shear Tests
Direct shear tests were performed on relatively undisturbed samples in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of selected materials. The samples were inundated during shearing to represent adverse field conditions. The results are shown on Figures B-7 through B-12.

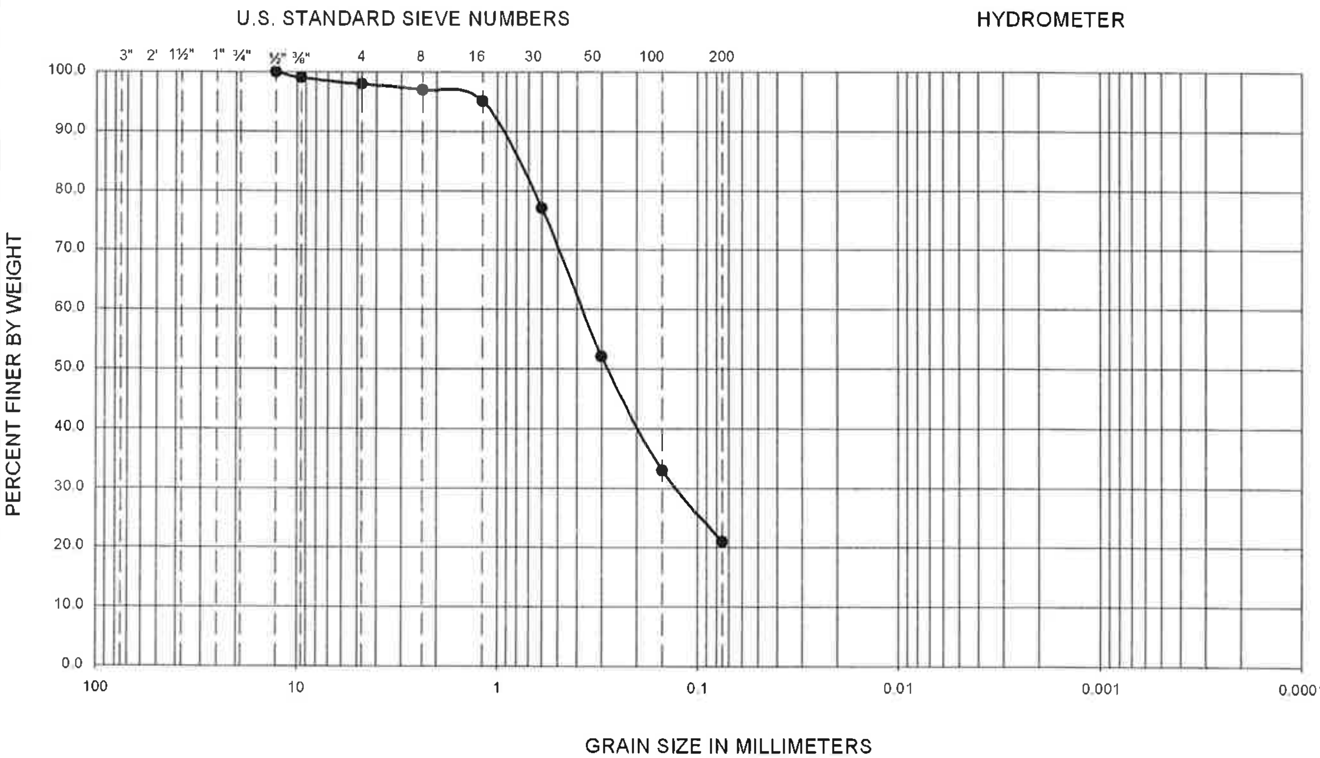
Expansion Index Tests
The expansion index of selected materials was evaluated in general accordance with ASTM D 4829. A specimen was molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimen was loaded with a surcharge of 144 pounds per square foot and was inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of this test are presented on Figure B-13.

Soil Corrosivity Tests
Soil pH, and resistivity tests were performed on representative samples in general accordance with California Test (CT) 643. The chloride content of the selected samples was evaluated in general accordance with CT 422. The sulfate content of the selected samples was evaluated in general accordance with CT 417. The test results are presented on Figure B-14.

R-Value

The resistance value, or R-value, for site soils was evaluated in general accordance with CT 301. A representative sample was prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test results are shown on Figure B-15.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



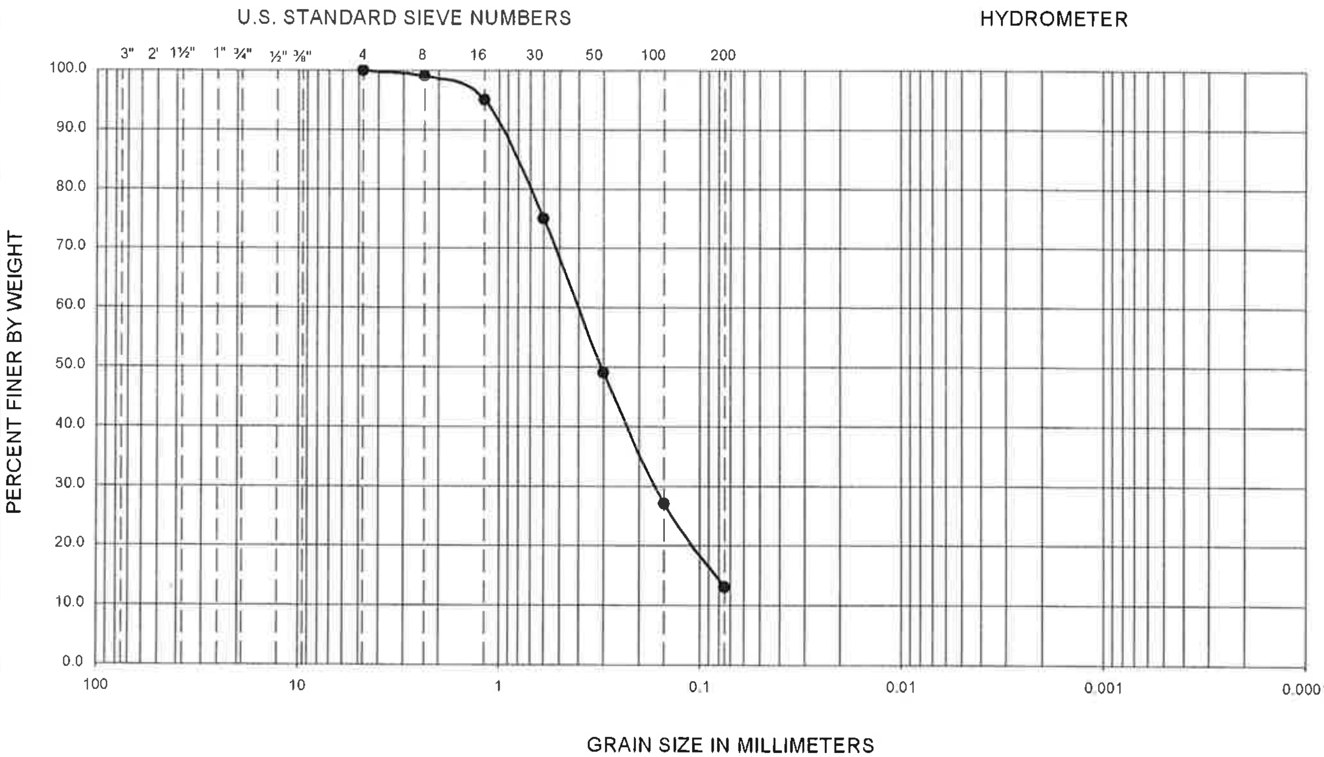
Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	Equivalent USCS
●	B-1	10.0-12.0	--	--	--	--	--	--	--	--	21	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	B-1
107314001	7/14		

107314001_SIEVE B-1 @ 10.0-12.0.xls

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



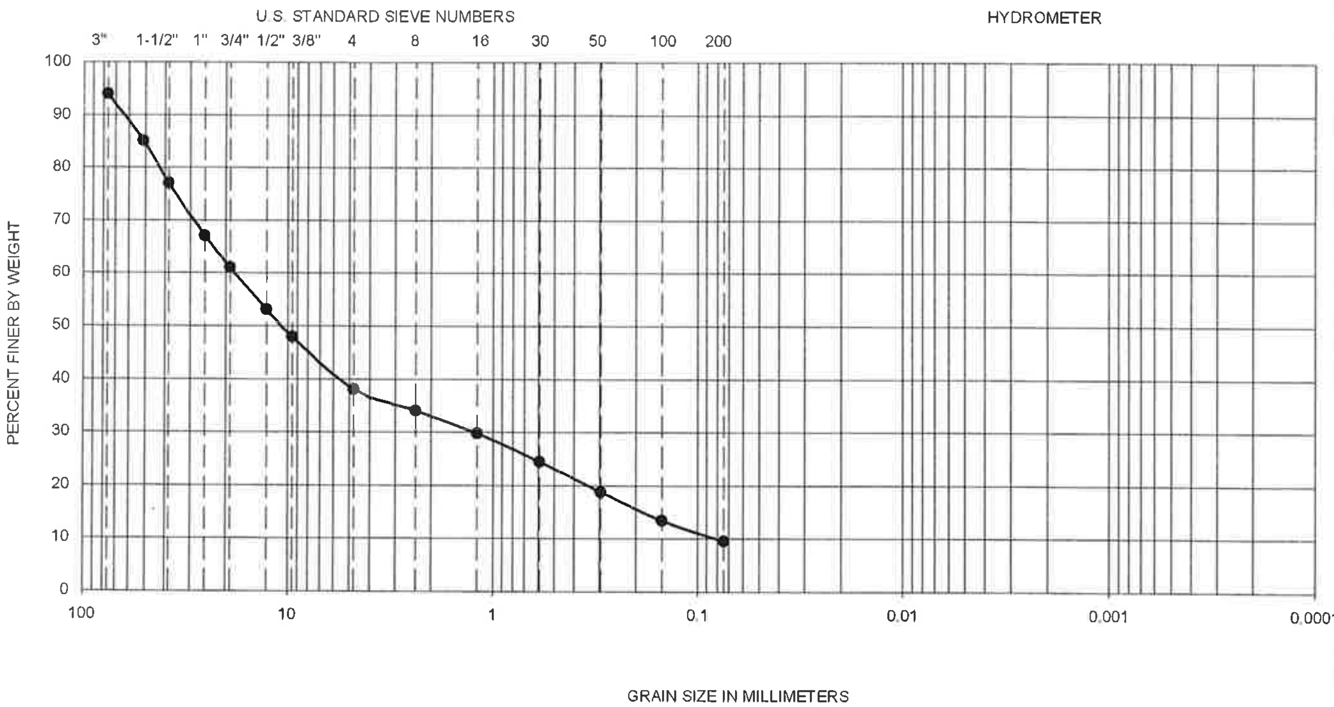
Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	Equivalent USCS
●	B-1	30.0-31.0	--	--	--	--	--	--	--	--	13	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	B-2
107314001	7/14		

107314001_SIEVE B-1 @ 30.0-31.0.xls

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

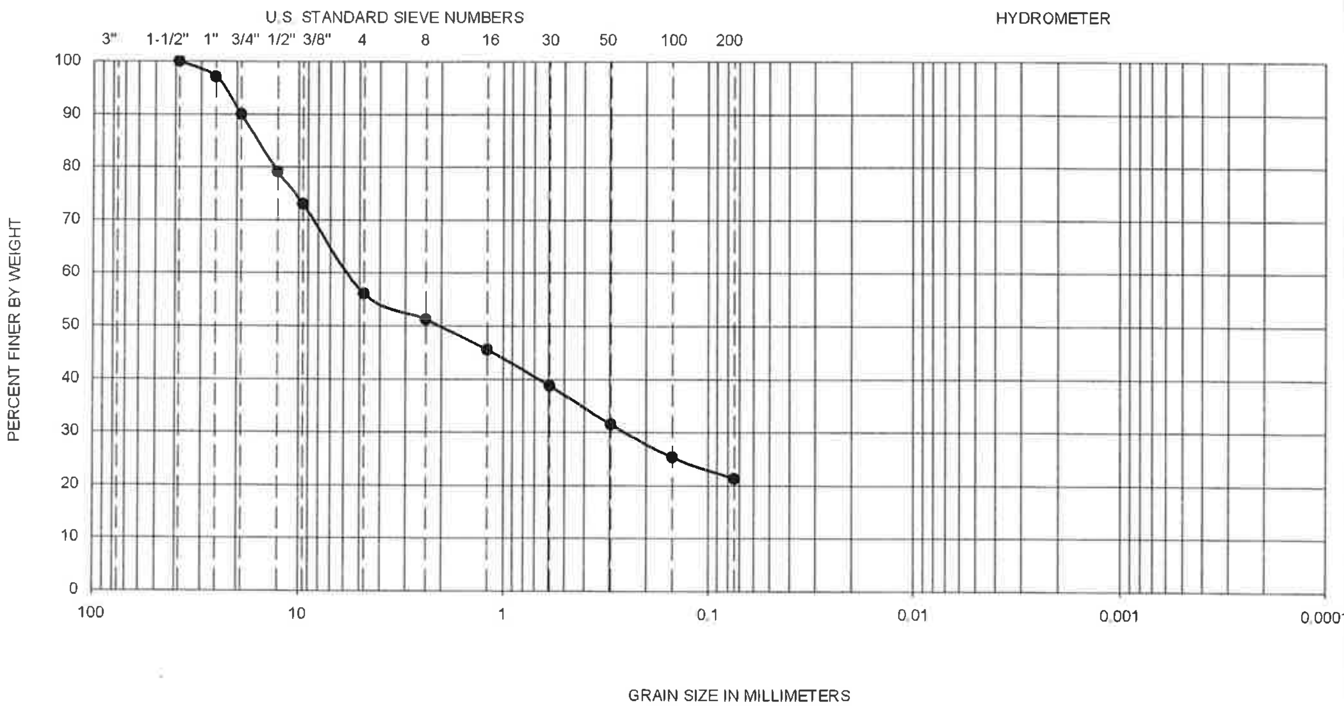


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	Equivalent USCS
●	B-1	40.0-41.0	--	--	--	0.08	1.19	18.10	241.3	1.0	10	GW-GM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	B-3
107314001	7/14		

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

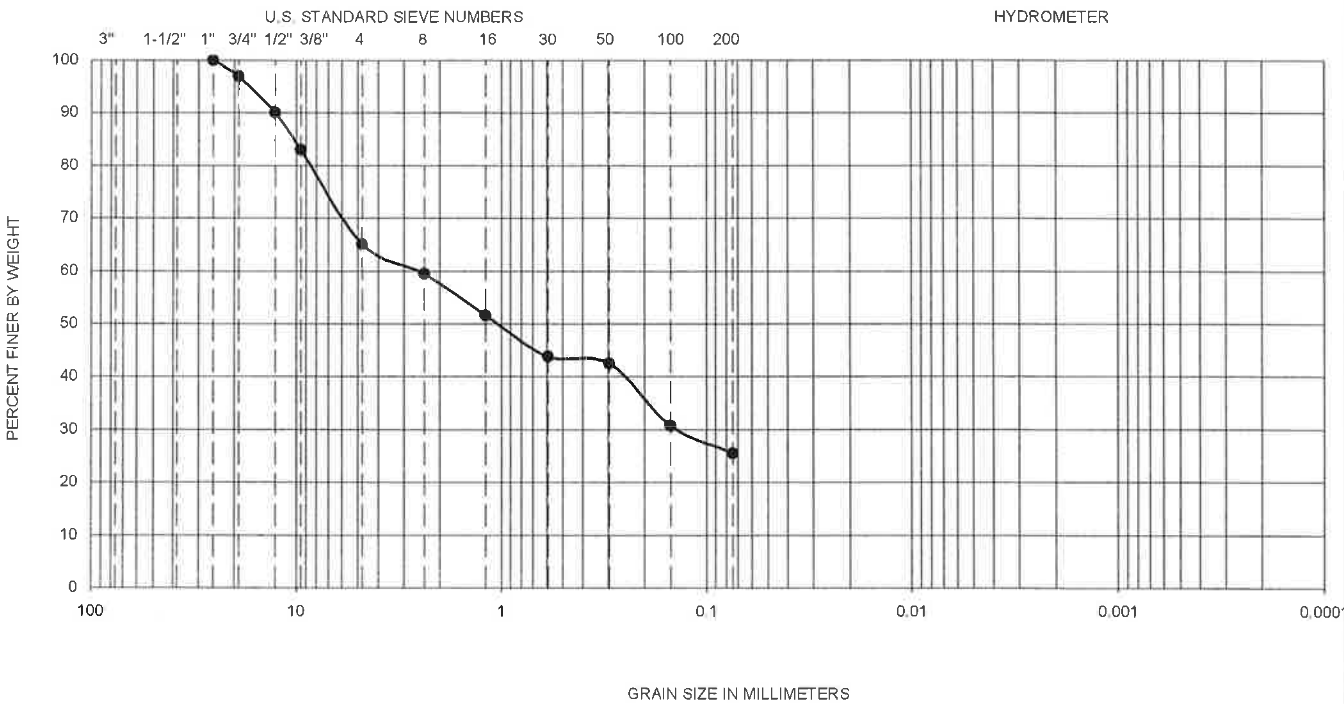


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-5	5.0-10.0	--	--	--	--	--	--	--	--	21	GC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	B-4
107314001	7/14		

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



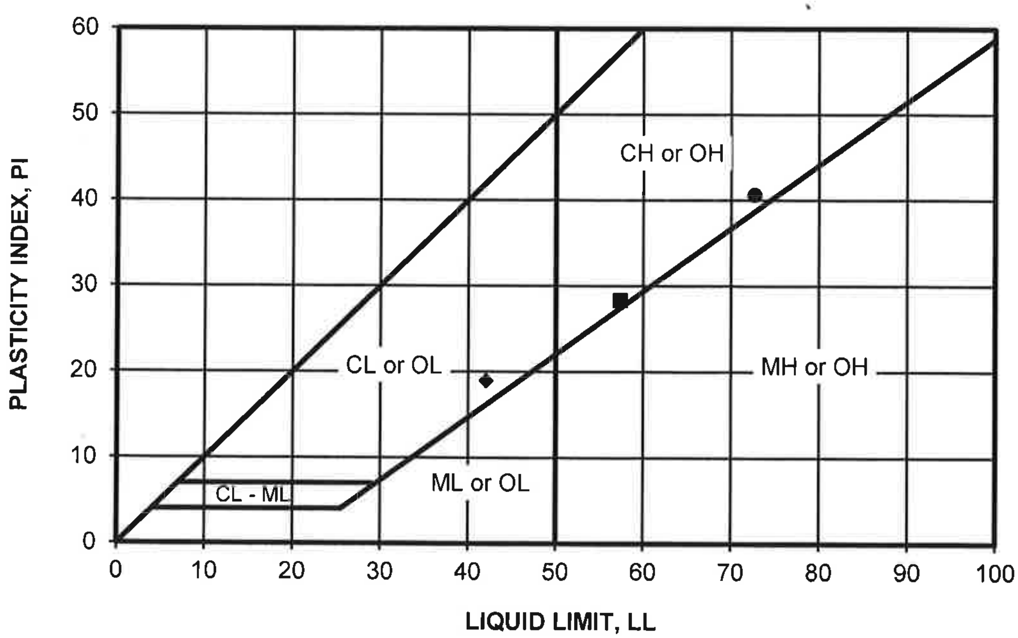
Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-7	1.0-5.0	--	--	--	--	--	--	--	--	25	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore	GRADATION TEST RESULTS	FIGURE
PROJECT NO. / DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	B-5
107314001 / 7/14		

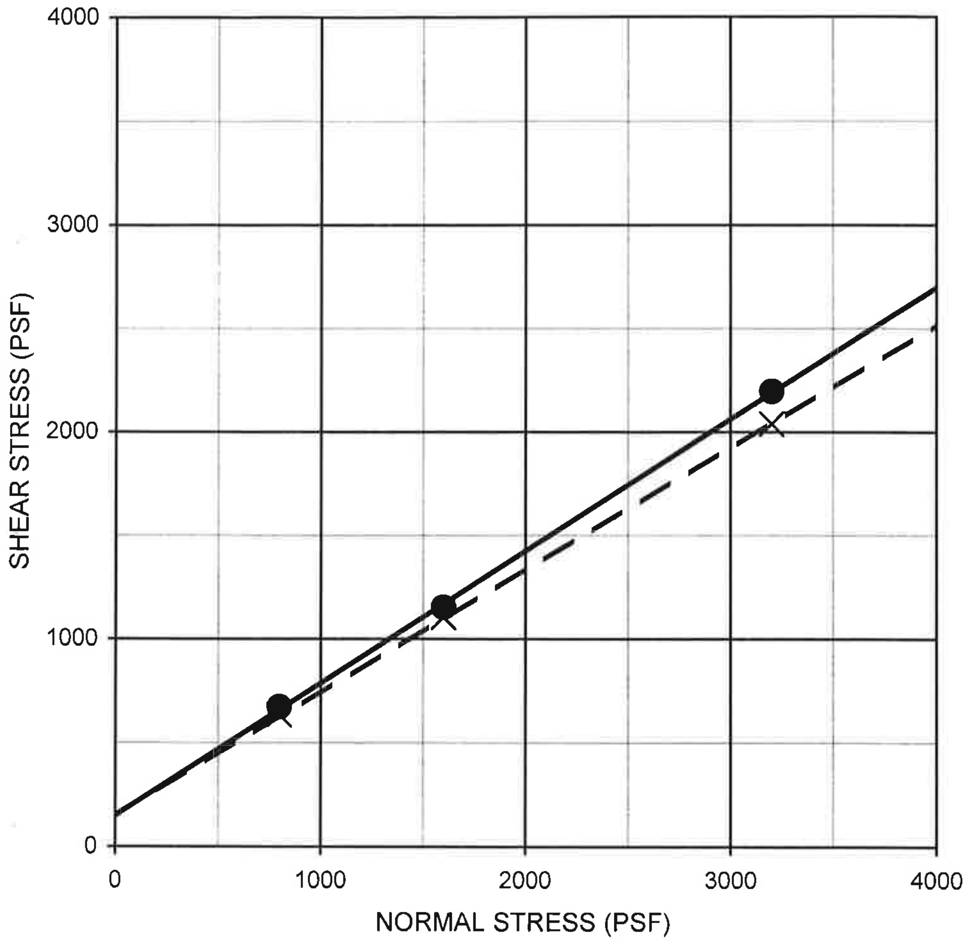
107314001_SIEVE B-7 @ 1.0-5.0.xls

SYMBOL	LOCATION	DEPTH (FT)	LIQUID LIMIT, LL	PLASTIC LIMIT, PL	PLASTICITY INDEX, PI	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS (Entire Sample)
●	B-1	49.0-50.0	73	32	41	CH	CH
■	B-1	77.0-78.0	57	29	28	CL	CL
◆	B-2	41.0-42.0	42	23	19	CH	CH



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

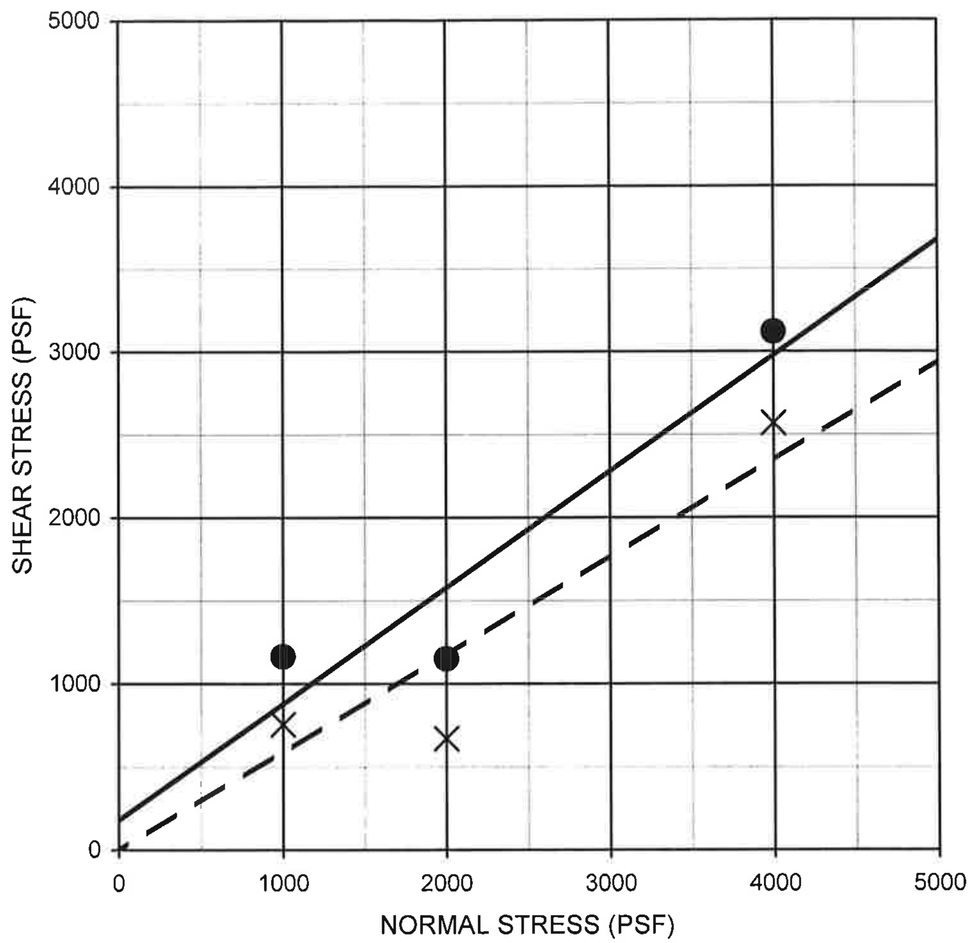
Ninyo & Moore		ATTERBERG LIMITS TEST RESULTS	FIGURE
PROJECT NO.	DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	B-6
107314001	7/14		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Silty SANDSTONE	—●—	B-1	5.0-6.0	Peak	150	33	Formation
Silty SANDSTONE	- - x - -	B-1	5.0-6.0	Ultimate	150	31	Formation

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

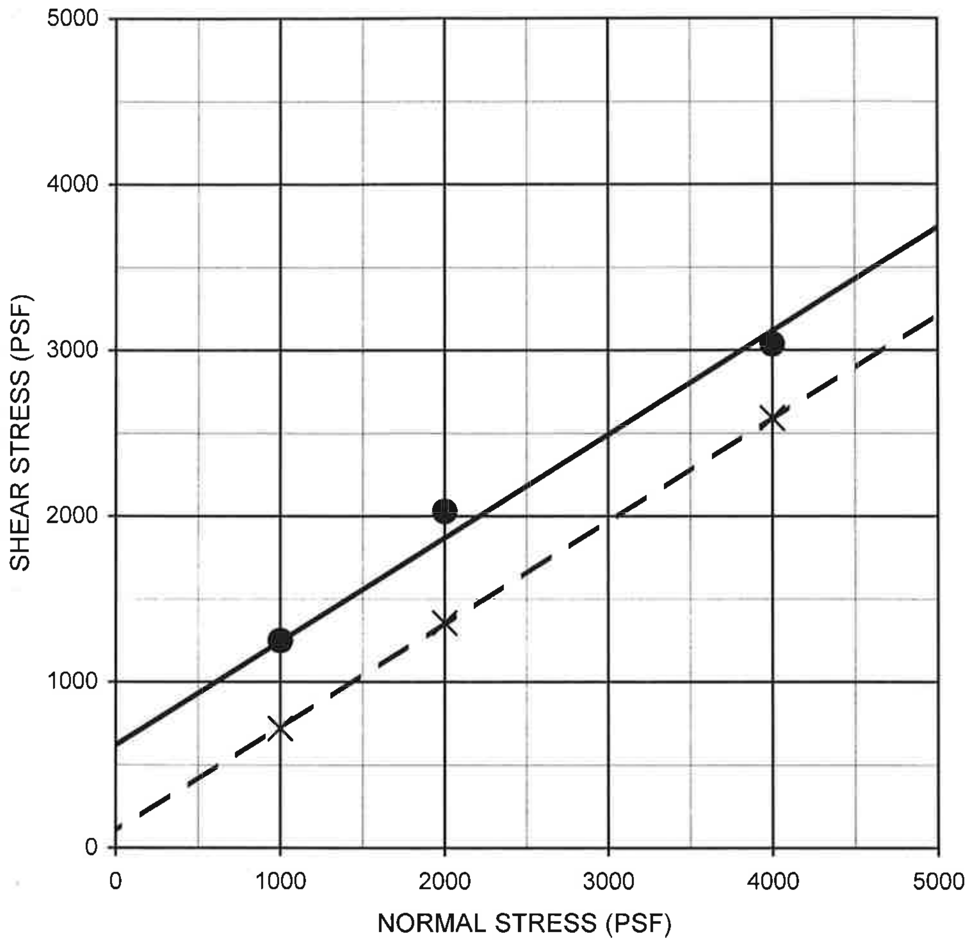
Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT NO.	DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	B-7
107314001	7/14		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Silty SANDSTONE	—●—	B-1	20.0-21.0	Peak	180	34	Formation
Silty SANDSTONE	- - X - -	B-1	20.0-21.0	Ultimate	0	31	Formation

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

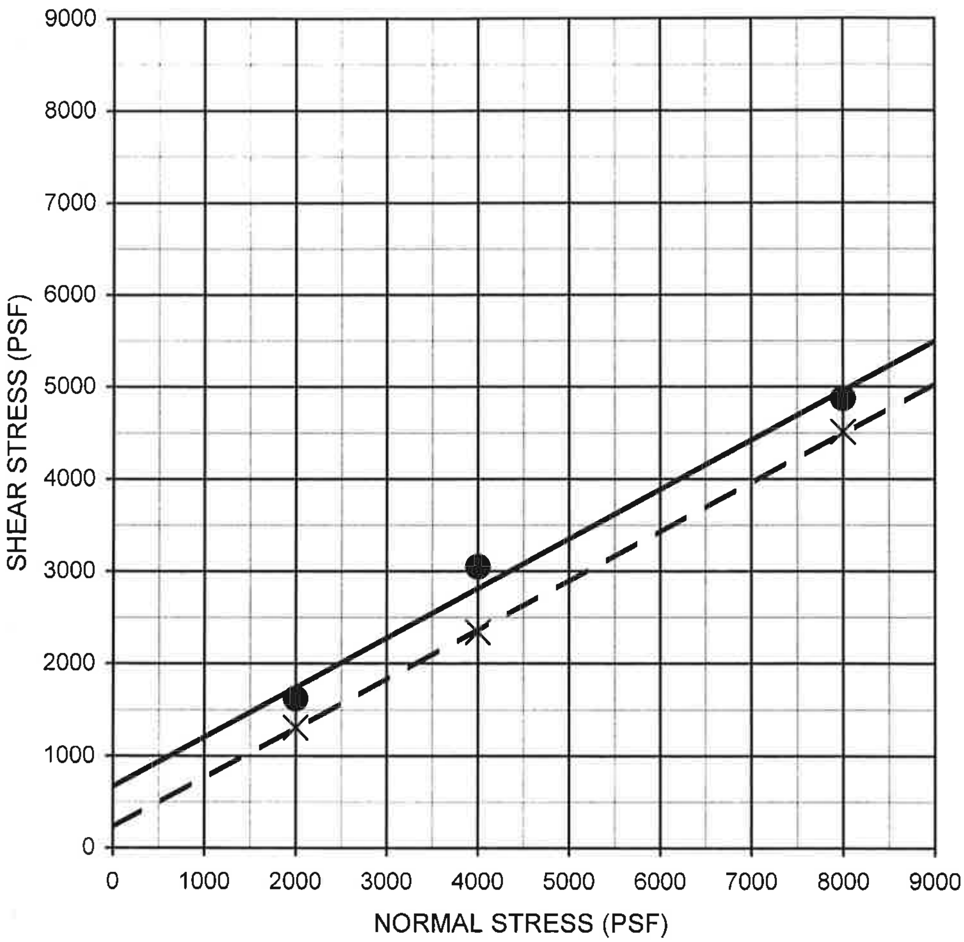
Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT NO.	DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	B-8
107314001	7/14		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Silty SANDSTONE	—●—	B-2	15.0-16.0	Peak	620	32	Formation
Silty SANDSTONE	- - X - -	B-2	15.0-16.0	Ultimate	100	32	Formation

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

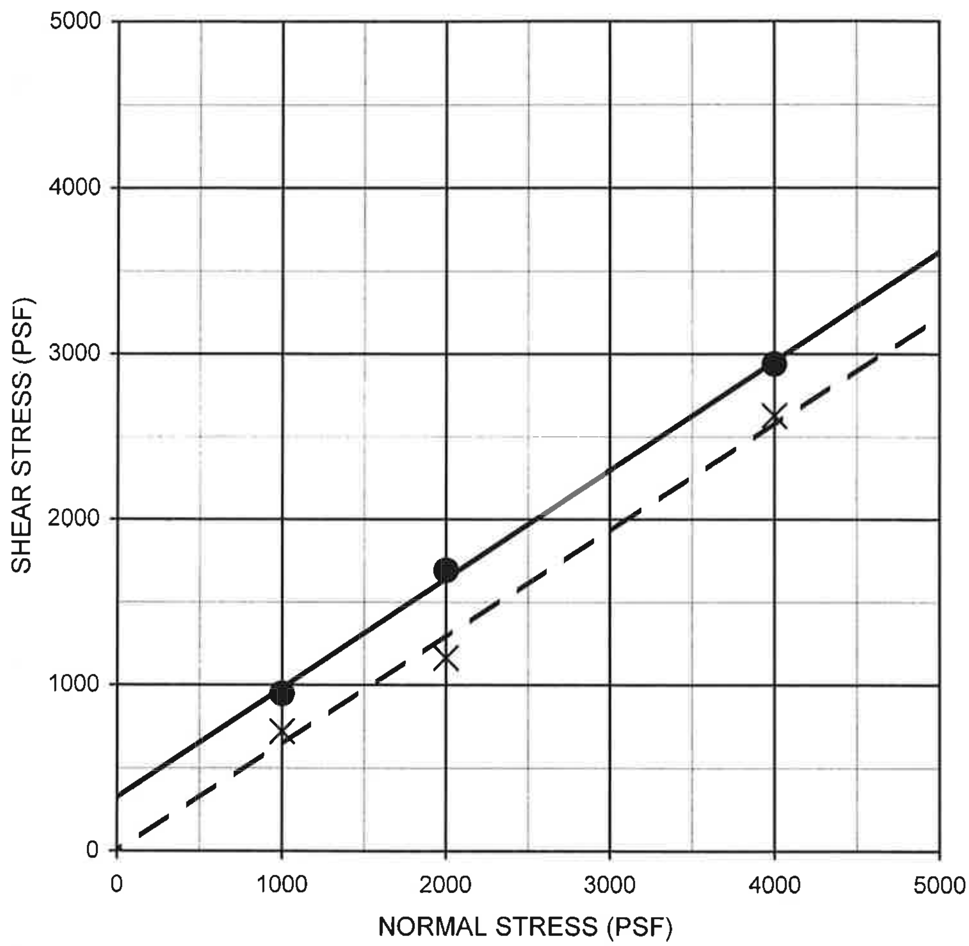
Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT NO.	DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	B-9
107314001	7/14		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Silty SANDSTONE	—●—	B-2	35.0-36.0	Peak	670	28	Formation
Silty SANDSTONE	- - X - -	B-2	35.0-36.0	Ultimate	220	28	Formation

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

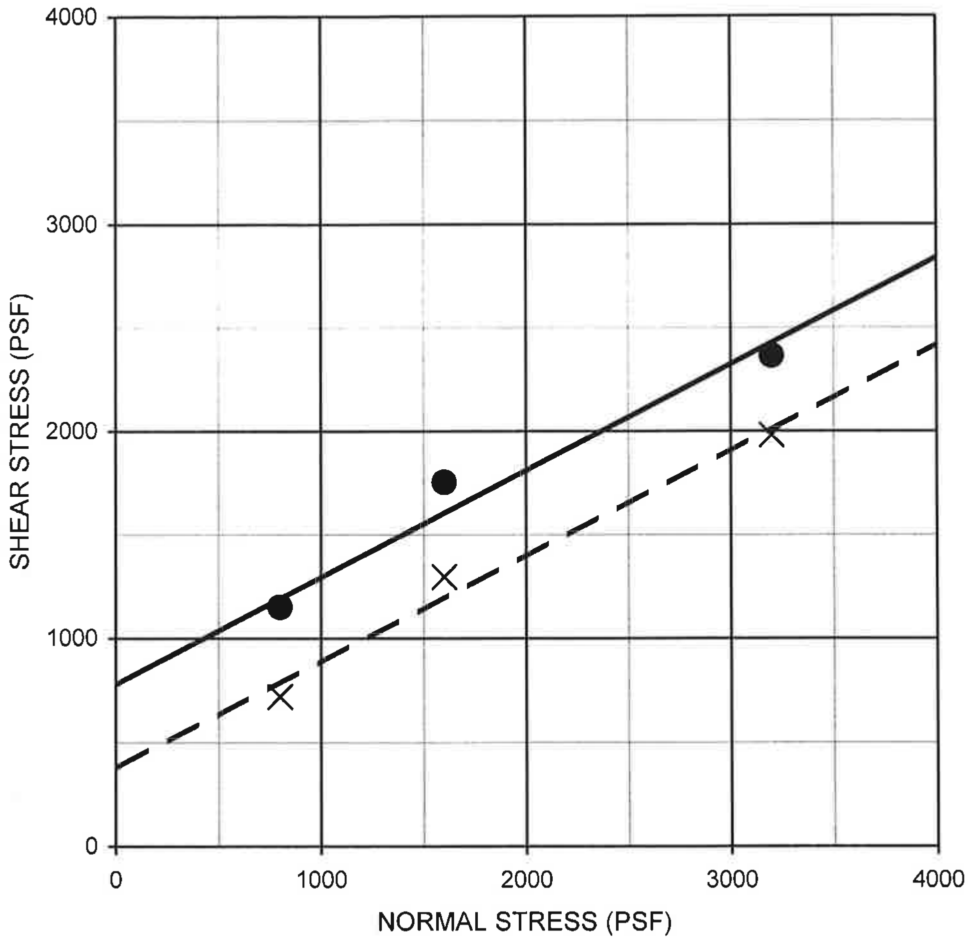
Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT NO.	DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	B-10
107314001	7/14		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Silty SANDSTONE	——●——	B-3	17.0-18.0	Peak	320	33	Formation
Silty SANDSTONE	- - X - -	B-3	17.0-18.0	Ultimate	0	33	Formation

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT NO.	DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	B-11
107314001	7/14		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, φ (degrees)	Soil Type
Sandy SILTSTONE	—●—	B-5	12.0-13.5	Peak	780	27	Formation
Sandy SILTSTONE	- - X - -	B-5	12.0-13.5	Ultimate	380	27	Formation

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT NO.	DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	B-12
107314001	7/14		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (PCF)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (IN)	EXPANSION INDEX	POTENTIAL EXPANSION
B-2	1.5-3.0	13.5	98.3	28.7	0.128	129	High

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

Ninyo & Moore	EXPANSION INDEX TEST RESULTS	FIGURE
PROJECT NO. / DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT	B-13
107314001 / 7/14	LA JOLLA, CALIFORNIA	

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH [1]	RESISTIVITY [1] (Ohm-cm)	SULFATE CONTENT [2] (ppm)	SULFATE CONTENT [2] (%)	CHLORIDE CONTENT [3] (ppm)
B-1	32.0-33.0	8.9	420	30	0.003	1,040
B-2	1.5-3.0	5.7	310	510	0.051	1,040
B-5	5.0-10.0	6.8	400	680	0.068	1920

[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

Ninyo & Moore

CORROSIVITY TEST RESULTS

PROJECT NO.	DATE
107314001	7/14

LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT
LA JOLLA, CALIFORNIA

FIGURE

B-14

SAMPLE LOCATION	SAMPLE DEPTH (FT)	SOIL TYPE	R-VALUE
B-2	1.5-3.0	CLAY (CL)	10

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

Ninyo & Moore		R-VALUE TEST RESULTS	FIGURE
PROJECT NO.	DATE	LA JOLLA VIEW RESERVOIR REPLACEMENT PROJECT LA JOLLA, CALIFORNIA	B-15
107314001	7/14		

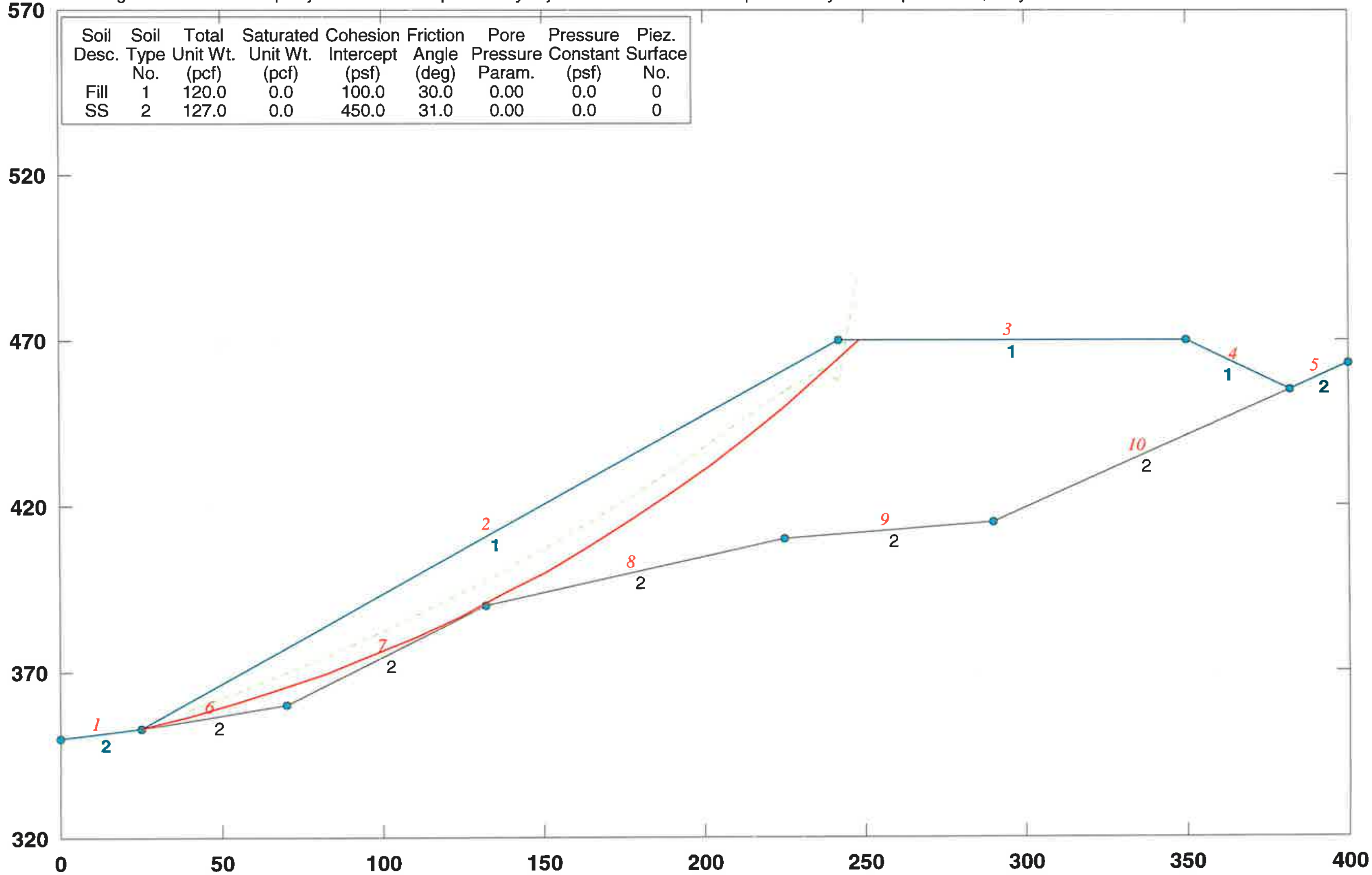
APPENDIX C

SLOPE STABILITY ANALYSIS

La Jolla View Reservoir Project B-B' La Jolla, California

g:\file share\ckv.temp\la jolla reservoir\slope stability\la jolla view b-b' surface #1.plt   Run By: Christopher Vonk, Ninyo & Moore   5/21/2014   03:29PM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	120.0	0.0	100.0	30.0	0.00	0.0	0
SS	2	127.0	0.0	450.0	31.0	0.00	0.0	0



GSTABL7 v.2 FSmin=1.263

Factor Of Safety Is Calculated By GLE (Spencer`s) Method (0-2)

GSTABL7

***** GSTABL7 *****

** GSTABL7 by Dr. Garry H. Gregory, Ph.D.,P.E.,D.GE **
** Original Version 1.0, January 1996; Current Ver. 2.005.2, Jan. 2011 **

****

SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

****

Analysis Run Date: 5/21/2014
Time of Run: 03:29PM
Run By: Christopher Vonk, Ninyo & Moore
Input Data Filename: G:\File Share\CKV.temp\La Jolla Reservoir\Slope Stability\la jolla view b-b' Surface #1.in
Output Filename: G:\File Share\CKV.temp\La Jolla Reservoir\Slope Stability\la jolla view b-b' Surface #1.OUT
Unit System: English
Plotted Output Filename: G:\File Share\CKV.temp\La Jolla Reservoir\Slope Stability\la jolla view b-b' Surface #1.PLT
PROBLEM DESCRIPTION: La Jolla View Reservoir Project B-B'
La Jolla, California

BOUNDARY COORDINATES
5 Top Boundaries
10 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	350.00	25.00	353.00	2
2	25.00	353.00	242.00	470.00	1
3	242.00	470.00	350.00	470.00	1
4	350.00	470.00	382.00	455.00	1
5	382.00	455.00	400.00	463.00	2
6	25.00	353.00	70.00	360.00	2
7	70.00	360.00	132.00	390.00	2
8	132.00	390.00	225.00	410.00	2
9	225.00	410.00	290.00	415.00	2
10	290.00	415.00	382.00	455.00	2

User Specified Y-Origin = 320.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	0.0	100.0	30.0	0.00	0.0	0
2	127.0	0.0	450.0	31.0	0.00	0.0	0

Trial Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	25.000	353.000
2	39.587	356.495
3	54.060	360.438
4	68.404	364.826
5	82.606	369.653
6	96.652	374.915
7	110.530	380.608
8	124.226	386.726
9	137.727	393.262
10	151.020	400.212
11	164.092	407.568
12	176.932	415.323
13	189.526	423.470
14	201.864	432.002

15	213.933	440.909
16	225.721	450.184
17	237.219	459.818
18	248.413	469.802
19	248.622	470.000

DEFLECTION ANGLE & SEGMENT DATA FOR SPECIFIED SURFACE(Excluding Last Segment)

Angle/Segment No.	Deflection(Deg)	Segment Length(ft)
1	1.77	15.00
2	1.77	15.00
3	1.76	15.00
4	1.77	15.00
5	1.77	15.00
6	1.77	15.00
7	1.76	15.00
8	1.77	15.00
9	1.77	15.00
10	1.76	15.00
11	1.77	15.00
12	1.77	15.00
13	1.76	15.00
14	1.77	15.00
15	1.76	15.00
16	1.77	15.00

Circle Center At X = -81.109(ft) ; Y = 828.028(ft); and Radius = 486.735(ft)

Theta (deg) (ki=1.0)	FOS (Moment) (Equil.)	FOS (Force) (Equil.)	Lambda
13.30	1.359	1.251	0.236
19.95	1.328	1.256	0.363
37.90	1.093	1.274	0.778
26.76	1.268	1.262	0.504
23.52	1.303	1.259	0.435
28.53	1.242	1.264	0.544
26.29	1.274	1.262	0.494
26.62	1.270	1.262	0.501
27.40	1.259	1.263	0.518
27.17	1.263	1.263	0.513

((Modified Bishop FS for Specified Surface = 1.263))
Factor Of Safety For The Preceding Specified Surface = 1.263
Theta (ki = 1.0) = 27.17 Deg   Lambda = 0.513
Factor Of Safety Is Calculated By GLE (Spencer`s) Method (0-2)
Forces from Reinforcement, Piers/Piles, Applied Forces, and Soil Nails
(if applicable) have been applied to the slice base(s)
on which they intersect.
Selected ki function = Bi-linear
Selected Lambda Coefficient = 1.00
Tension Crack Water Force = 0.00(lbs)
Specified Tension Crack Water Force Factor = 0.000
Depth of Tension Crack (zo) at Center of Last Slice = 0.099(ft)

*** Line of Thrust and Slice Force Data ***

Slice No.	X Coord.	Y Coord.	L/H	Side Force (lbs)	ki	Force Angle (Deg)	Vert. Shear Force(lbs)
1	39.59	358.49	0.457	2313.	1.000	27.17	1187.2
2	54.06	363.65	0.391	5865.	1.000	27.17	3009.5
3	68.40	369.10	0.369	9908.	1.000	27.17	5084.7
4	82.61	374.81	0.358	13863.	1.000	27.17	7113.8
5	96.65	380.81	0.353	17283.	1.000	27.17	8869.0
6	110.53	387.07	0.349	19856.	1.000	27.17	10189.3
7	124.23	393.61	0.348	21386.	1.000	27.17	10974.7
8	137.73	400.40	0.348	21789.	1.000	27.17	11181.3
9	151.02	407.47	0.350	21067.	1.000	27.17	10810.7
10	164.09	414.81	0.354	19316.	1.000	27.17	9912.4
11	176.93	422.43	0.363	16708.	1.000	27.17	8573.8
12	189.53	430.40	0.380	13475.	1.000	27.17	6914.7
13	201.86	438.83	0.417	9910.	1.000	27.17	5085.3
14	213.93	447.20	0.451	6220.	0.776	21.70	1920.1
15	225.72	454.87	0.425	3074.	0.512	14.72	413.6

16	237.22	461.29	0.194	749.	0.255	7.45	25.0
17	242.00	457.57	-1.100	56.	0.148	4.35	0.6
18	248.41	491.17	107.938	-21.	0.005	0.14	0.0
19	248.62	3335.75	0.000	0.	0.000	0.00	0.0

Table 1 - Individual data on the 19 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	14.6	3824.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	14.5	10941.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	14.3	17046.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	14.2	22140.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	14.0	26230.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
6	13.9	29331.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0
7	13.7	31457.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
8	13.5	32638.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
9	13.3	32901.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	13.1	32283.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
11	12.8	30831.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
12	12.6	28587.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
13	12.3	25610.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0
14	12.1	21953.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
15	11.8	17679.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
16	11.5	12861.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0
17	4.8	3878.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0
18	6.4	2353.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
19	0.2	2.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0

- TABLE 2 - Base Stress Data on the 19 Slices -

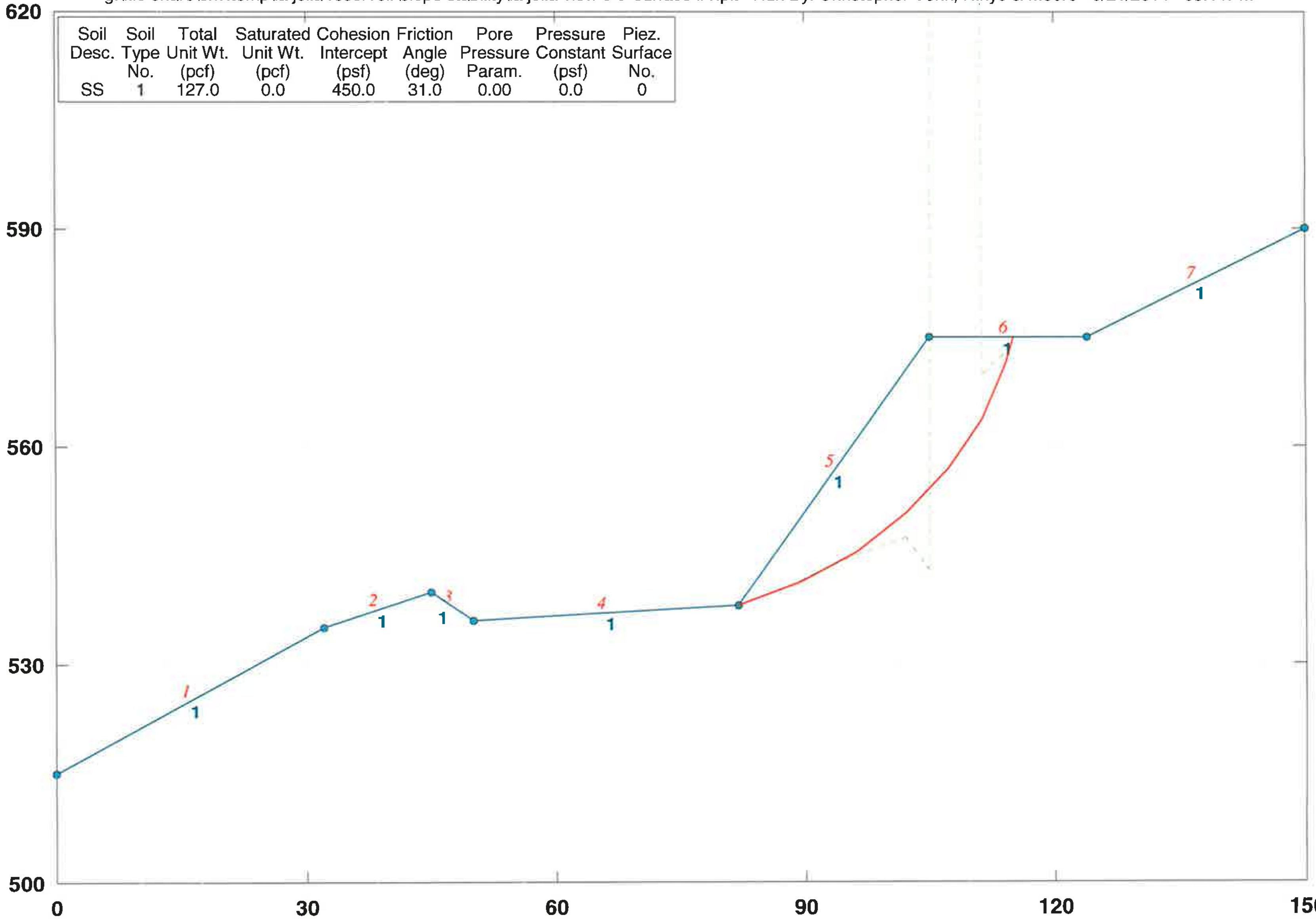
Slice No. *	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Total Normal Stress (psf)	Total Vert. Stress (psf)	Mobilized Shear Stress (psf)
1	13.47	32.29	15.00	284.46	254.98	209.26
2	15.24	46.82	15.00	752.69	729.43	423.35
3	17.01	61.23	15.00	1134.23	1136.40	597.80
4	18.77	75.50	15.00	1436.00	1476.03	735.78
5	20.54	89.63	15.00	1663.93	1748.76	840.00
6	22.30	103.59	15.00	1823.60	1955.37	913.01
7	24.07	117.38	15.00	1920.29	2097.14	957.22
8	25.83	130.98	15.00	1959.08	2175.88	974.95
9	27.60	144.37	15.00	1944.11	2193.38	968.11
10	29.37	157.56	15.00	1880.16	2152.26	938.87
11	31.13	170.51	15.00	1771.46	2055.43	889.16
12	32.90	183.23	15.00	1621.78	1905.89	820.73
13	34.66	195.70	15.00	1435.24	1707.27	735.43
14	36.43	207.90	15.00	1178.53	1463.55	618.06
15	38.20	219.83	15.00	962.42	1178.71	519.25
16	39.96	231.47	15.00	717.75	857.40	407.38
17	41.73	239.61	6.41	512.33	605.48	313.45
18	41.73	245.21	8.59	210.00	273.85	175.21
19	43.45	248.52	0.29	0.00	8.62	59.13

**** END OF GSTABL7 OUTPUT ****

La Jolla View Reservior Project C-C' La Jolla, California

g:\file share\ckv.temp\la jolla reservoir\slope stability\la jolla view c-c' surface #1.plt   Run By: Christopher Vonk, Ninyo & Moore   5/21/2014   03:11PM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
SS	1	127.0	0.0	450.0	31.0	0.00	0.0	0



GSTABL7 v.2 FSmin=1.294
Factor Of Safety Is Calculated By GLE (Spencer`s) Method (0-1)

GSTABL7

*** **GSTABL7** ***

** GSTABL7 by Dr. Garry H. Gregory, Ph.D.,P.E.,D.GE **
** Original Version 1.0, January 1996; Current Ver. 2.005.2, Jan. 2011 **

****

SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

****

Analysis Run Date: 5/21/2014
Time of Run: 03:11PM
Run By: Christopher Vonk, Ninyo & Moore
Input Data Filename: G:\File Share\CKV.temp\La Jolla Reservoir\Slope Stability\la jolla view c-c' Surface #1.in
Output Filename: G:\File Share\CKV.temp\La Jolla Reservoir\Slope Stability\la jolla view c-c' Surface #1.OUT
Unit System: English
Plotted Output Filename: G:\File Share\CKV.temp\La Jolla Reservoir\Slope Stability\la jolla view c-c' Surface #1.PLT
PROBLEM DESCRIPTION: La Jolla View Reservior Project C-C'
La Jolla, California

BOUNDARY COORDINATES
7 Top Boundaries
7 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	515.00	32.00	535.00	1
2	32.00	535.00	45.00	540.00	1
3	45.00	540.00	50.00	536.00	1
4	50.00	536.00	82.00	538.00	1
5	82.00	538.00	105.00	575.00	1
6	105.00	575.00	124.00	575.00	1
7	124.00	575.00	150.00	590.00	1

User Specified Y-Origin = 500.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
1 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	127.0	0.0	450.0	31.0	0.00	0.0	0

Trial Failure Surface Specified By 8 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	82.000	538.000
2	89.345	541.170
3	96.103	545.452
4	102.107	550.739
5	107.208	556.901
6	111.282	563.786
7	114.228	571.224
8	115.071	575.000

DEFLECTION ANGLE & SEGMENT DATA FOR SPECIFIED SURFACE(Excluding Last Segment)

Angle/Segment No.	Deflection(Deg)	Segment Length(ft)
1	9.02	8.00
2	9.01	8.00
3	9.02	8.00
4	9.00	8.00
5	9.01	8.00

Circle Center At X = 65.870(ft) ; Y = 585.696(ft); and Radius = 50.350(ft)

Theta (deg) (ki=1.0)	FOS (Moment) (Equil.)	FOS (Force) (Equil.)	Lambda

22.50	1.301	1.290	0.414
33.75	1.278	1.300	0.668
27.30	1.295	1.294	0.516

((Modified Bishop FS for Specified Surface = 1.292))
Factor Of Safety For The Preceding Specified Surface = 1.294
Theta (ki = 1.0) = 27.30 Deg   Lambda =   0.516
Factor Of Safety Is Calculated By GLE (Spencer`s) Method (0-1)
Forces from Reinforcement, Piers/Piles, Applied Forces, and Soil Nails
(if applicable) have been applied to the slice base(s)
on which they intersect.
Selected ki function = Constant (1.0)
Selected Lambda Coefficient = 1.00
Tension Crack Water Force =   0.00(lbs)
Specified Tension Crack Water Force Factor =   0.000
Depth of Tension Crack (zo) at Center of Last Slice =   1.888(ft)

*** Line of Thrust and Slice Force Data ***

Slice No.	X Coord.	Y Coord.	L/H	Side Force (lbs)	ki	Force Angle (Deg)	Vert. Shear Force(lbs)
1	89.35	541.48	0.036	3005.	1.000	27.30	1550.9
2	96.10	544.99	-0.030	4272.	1.000	27.30	2205.0
3	102.11	547.44	-0.168	3007.	1.000	27.30	1551.9
4	105.00	542.85	-0.548	1246.	1.000	27.30	642.9
5	107.21	2528.74	108.947	-8.	1.000	27.30	-4.0
6	111.28	569.82	0.538	-1793.	1.000	27.30	-925.4
7	114.23	572.95	0.458	-1176.	1.000	27.30	-606.9
8	115.07	587.75	0.000	-5.	1.000	27.30	-2.4

Table 1 - Individual data on the   8 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	7.3	4032.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	6.8	10248.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	6.0	13283.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	2.9	7416.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	2.2	5449.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
6	4.1	7583.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
7	2.9	2804.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0
8	0.8	202.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0

- TABLE 2 - Base Stress Data on the   8 Slices -

Slice No. *	Alpha (deg)	X-Coord. Slice Cntr (ft)	Base Leng. (ft)	Total Normal Stress (psf)	Total Vert. Stress (psf)	Mobilized Shear Stress (psf)
1	23.34	85.67	8.00	488.72	504.08	574.48
2	32.36	92.72	8.00	1068.07	1280.96	843.40
3	41.37	99.10	8.00	1284.63	1660.47	943.92
4	50.38	103.55	4.54	1194.67	1634.81	902.16
5	50.38	106.10	3.46	1145.43	1573.74	879.31
6	59.39	109.24	8.00	601.26	947.90	626.72
7	68.39	112.75	8.00	78.38	350.51	384.01
8	77.41	114.65	3.87	0.00	52.24	245.10

**** END OF GSTABL7 OUTPUT ****