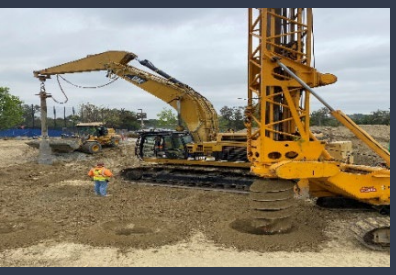
APPENDIX E1

GEOTECHNICAL REPORT

GEOTECHNICAL INVESTIGATION

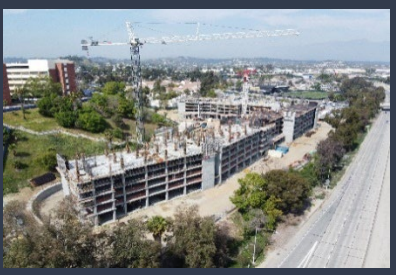
LAGUNA BEACH HIGH SCHOOL POOL MODERNIZATION
670 PARK AVENUE
LAGUNA BEACH, CALIFORNIA



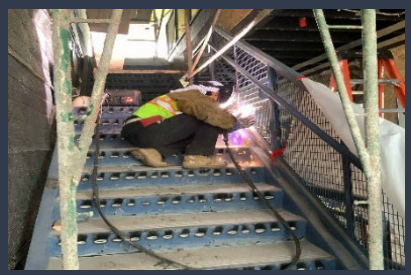


LAGUNA BEACH UNIFIED SCHOOL DISTRICT
550 BLUMONT STREET
LAGUNA BEACH, CALIFORNIA

Date: September 16th, 2024
MTGL Project No.: 2520A02
MTGL Log No.: 24-0280

















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September 16, 2024

MTGL Project No.: 2520A02
MTGL Log No.: 24-0280
MTGL Branch: Anaheim

Ryan Zajda, Director of Facilities
Laguna Beach Unified School District
550 Blumont Street
Laguna Beach, California 92651

Subject: **GEOTECHNICAL INVESTIGATION**
Laguna Beach High School Pool Replacement Project
670 Park Avenue
Laguna Beach, California

Dear Mr. Zajda,

MTGL Inc. is pleased to present this report describing the results of our geotechnical investigation for the subject project. With your authorization, we have performed this work in general accordance with our proposal dated February 21st, 2024. Based on the results of our investigation, we consider the planned developments feasible from a geotechnical perspective, provided the recommendations of this report are followed.

We appreciate this opportunity to be of continued service and look forward to providing additional consulting services during the planning and construction of the project. Should you have any questions regarding this report, please do not hesitate to contact us.

Respectfully submitted,
MTG$_L$, Inc.





Isaac Chun, P.E., G.E.
Vice President

Greg Wilson, P.G., C.E.G.
Senior Engineering Geologist

Trevor Carter, E.I.T.
Staff Engineer

EXECUTIVE SUMMARY

In accordance with your request and authorization, we have completed our geotechnical investigation for the subject site located at 670 Park Avenue in Laguna Beach, California. We understand that the project currently involves demolishing the main pool facility building, pump house, pool, and pool deck, as well as the parking lot. The proposed design includes a new a two-story pool building, pool and pool deck, bleachers, parking lot, restrooms and showers, equipment storage areas, shade structures with solar panels, scoreboard, and light poles. The proposed two-story pool building is expected to cover approximately 6,670 square feet, with the foundation level around 3,400 square feet. The approximate square footage of the new pool will be on the order of 11,100 square feet. The purpose of our work is to provide conclusions and recommendations regarding the geotechnical aspects of the project.

Our subsurface investigation was performed between August 15th and 16th, 2024 and consisted of drilling six exploratory soil borings within areas of proposed improvements. Six borings (B-1, B-2, P-1, and HA-1 through HA-3) were drilled to depths ranging from approximately 8½ to 51½ feet below ground surface (BGS) using hand tools, and a truck mounted drilling rig equipped with an 8 inch hollow stem auger. One shallow boring was converted to a borehole percolation test to evaluate infiltration feasibility. An MTGL engineer logged the borings and collected samples of the encountered materials for geotechnical laboratory testing. Selected samples were tested in our laboratory to evaluate their engineering properties.

As encountered in our borings, the site is underlain by undocumented fill (Qf), colluvium (Qoc), old paralic deposits (Qop) and Topanga Formation (Tt). Undocumented fill was encountered in each of the borings, excluding Boring P-1, beneath the existing pavement section and extended to depths ranging from approximately 2 to 7 feet BGS. As encountered, the fill generally consisted of various shades of brown, loose to medium dense, fine to coarse grained, silty to clayey sand, and stiff, sandy clay, with variable amounts of gravel, cobbles, rootlets, and debris. Colluvium was encountered in each of the borings, excluding Boring HA-3, either beneath the exiting pavement section or beneath the fill and extended to depths ranging from approximately 5 to 13 feet BGS. As encountered, the colluvium generally consisted of various shades of brown, loose to dense, fine to medium grained, silty to clayey sand. Old paralic deposits were encountered beneath the fill and/or colluvium in each of the borings and extended to the maximum depth explored, excluding Boring B-2. As encountered, the old paralic deposits generally consisted of various shades of brown, gray and olive, medium dense to very dense, silty to clayey sands, and very stiff to hard clays. Topanga Formation was encountered beneath the old paralic deposits in Boring B-2 at a depth of approximately 40 feet BGS and extended to the maximum depth explored. As encountered, the Topanga Formation generally consisted of light gray to olive, moderately cemented, fine to medium grained, silty sandstone.

The main geotechnical considerations affecting the project are the potential for cut/fill transitions below the proposed pool, the presence of potentially compressible and expansive undocumented fill soils, and the presence of existing retaining walls near the proposed two-story building and pool. To reduce the potential for settlement, the existing undocumented fill and colluvium should be excavated in their entirety below buildings, pools, and other settlement sensitive improvements. Areas to receive proposed structures and the new pool area should be graded in a manner that removes any existing cut/fill transitions and reduces the potential for new cut/fill transitions to develop during grading.

To reduce the potential for expansive heave, shallow foundations, concrete slabs-on-grade, pool shells, pavements, and hardscapes, should be underlain by at least 2 feet of material with an expansion index of 50 or less. Additionally, retaining walls and fill abutting the proposed pool shell should consist of granular, free draining, non-expansive materials.

In general, conventional shallow spread footings bearing entirely on compacted fill may be used for the support of the proposed structures. Proposed pole-type structures may be supported on Cast-In-Drilled-Hole (CIDH) deep foundations. Special care should be taken by the contractor to protect the existing retaining walls and foundation elements along the perimeter of the site and near proposed improvements during the demolition of existing improvements and construction of new improvements.

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1.00 INTRODUCTION

In accordance with your request and authorization, MTGL, Inc. has completed a geotechnical investigation for the subject site. The following report presents a summary of our findings, conclusions and recommendations based on our field investigation, laboratory testing, and engineering analysis.

1.01 SITE DESCRIPTION

The project site is located within the central portion of the city of Laguna Beach, California. The site is comprised of one parcel, APN No. 641-363-10, at the address of 670 Park Avenue, which contains the existing parking lot area, pool and pool deck area, and tennis court area. The site is bounded by Manzanita Drive and single-family residences to the west and north, tennis courts and single-family residences to the east, and Park Avenue to the south. Currently, the existing improvements at the site, which will be replaced as part of the project, generally include a parking lot located in the southwestern part of the site, a pumphouse building situated between the parking lot and the existing pool, a pool house/restroom building to the south of the pool, the existing pool and pool deck, concrete hardscape, and storage areas. Existing improvements at the site that are anticipated to remain in place during the pool replacement project consist of various single and multi-tiered retaining walls around the perimeter of the existing pool area and parking lot, a school district-owned building north of the pool area, and tennis courts and retaining walls to the east of the pool area.

Topography at the parking lot and pool deck areas is relatively flat and gently descends towards the southwest. However, the perimeter of site is generally bounded on the north, west, and east by ascending retaining walls. Specifically, the northern perimeter of the existing parking lot area is bound by a tiered retaining wall that ascends approximately 10 to 15 feet in height to a private residence toward the north. The western portion of the existing pool area is bound by an approximate 7 foot high retaining wall that ascends to the backyard of a private residence toward the west. The northern perimeter of the existing pool area is bound by an existing retaining wall that ascends approximately 12 to 15 feet in height to the district-owned building toward the north. Additionally, a staircase at the northeastern perimeter of the pool deck area ascends approximately 20 feet to the district-owned annex building and Manzanita Drive toward the north. The eastern perimeter of the pool area is bound by an approximately 5 feet high retaining wall ascending to the tennis court area toward the east. Multi-tiered retaining walls and retaining structures are located north of the existing tennis courts, ascending approximately 20 to 30 feet in height toward Manzanita Drive to the north, however, these improvements are anticipated to remain in place during the pool replacement project.

Elevations at the pool deck and parking lot areas range from approximately 182½ feet above mean sea level (MSL) at the northeastern portion of the pool deck to approximately 163 feet MSL at the southwestern portion of the parking lot (Google Earth, 2024). Based on our review of historic aerial photographs, the site was developed to its existing configuration between 1993 and 1994. The approximate site location is shown on the Site Location Map (Figure 1). The topography within the vicinity of the site is shown on the Site Topography Map (Figure 2).

1.02 PLANNED CONSTRUCTION

Based on our review of the provided conceptual plan, project summary, and conversations with the project development team, we understand this project currently proposes demolishing the existing main pool facility building, pump house, pool, and pool deck, as well as the parking lot. The proposed design includes a new a two-story pool building, pool and pool deck, bleachers, parking lot, restrooms and showers, equipment storage areas, shade structures with solar panels, scoreboard, and light poles. Associated civil improvements may include new utilities, sidewalks, driveways, shallow storm water best management practices (BMPs), and other auxiliary features. The total square footage of the new pool building will be approximately 6,667 square feet, with the foundation level having a footprint on the order of 3,400 square feet. Proposed surface elevations of the new pool deck and parking lot areas are anticipated to be within approximately 18 inches of existing surface elevations.

1.03 SCOPE OF WORK

We conducted this investigation in general conformance with the scope of work presented in our proposal No. P-24-062. The scope of our geotechnical services included the following:

- Reviewing readily available literature and maps to obtain background information of regional geology, site development, seismicity, and groundwater.
- Preparing applications for and obtaining well drilling permits from the Orange County Environmental Health Division (EHD).
- Marking out boring locations on the site and contacting Underground Service Alert (USA) to locate onsite utility lines.
- Utilizing a private utility locator service to clear borings of potential underground utility conflicts.
- Drilling, logging, and sampling of six exploratory borings using hand tools and an 8 inch hollow stem auger drill rig to a maximum depth of 51½ feet below ground surface (BGS).
- Converting one shallow boring to a borehole percolation test.
- Performing one borehole percolation test.
- Performing laboratory testing of select samples.

- Performing geotechnical engineering review of compiled data and performing geotechnical engineering analyses.
- Preparing this report summarizing our findings and presenting our conclusions and recommendations for the proposed construction.

1.03.1 FIELD INVESTIGATION

Prior to performing our field investigation, a site reconnaissance was performed by an MTGL engineer to observe the existing surface conditions, mark the proposed boring locations, evaluate each location with respect to obvious subsurface structures and determine access for the drilling rig. USA was subsequently notified of the marked locations for utility clearance as required by law. In addition, a private utility locator, Advantage Geophysics, was subcontracted to locate and mark potential utility conflicts. A well drilling permit was obtained from the Orange County EHD prior to commencement of drilling activities.

Our subsurface investigation was performed between August 15th and 16th, 2024, and consisted of drilling six exploratory soil borings within areas of proposed improvements. The six borings (B-1, B-2, P-1, and HA-1 through HA-3) were drilled to depths ranging from approximately 8½ to 51½ feet BGS using hand tools and a truck mounted drilling rig equipped with an 8 inch hollow stem auger. See Appendix B for further discussion of the field exploration and logs of test borings. Figure 3 attached presents a Subsurface Exploration Map.

MTGL engineers logged the borings and collected samples of the encountered materials for geotechnical laboratory testing. Representative disturbed bulk soil samples were obtained from the borings in the upper five feet. In-situ testing, including driving Modified California (CAL) and Standard Penetration Test (SPT) samplers at selected depth intervals, was performed and relatively undisturbed samples were obtained. Samplers were driven into the bottom of the boring with successive drops of a 140-pound weight falling 30 inches. The energy-corrected number of blows per foot required to drive the CAL and SPT samplers are shown on the boring logs in the N_{60} column (Appendix B). A conversion factor of 0.65 was used to normalize N_{60} values obtained by Modified California samplers (Burmister, 1948). SPTs were performed in general accordance with the American Society for Testing and Materials (ASTM) D1586 standard test method.

Soil samples were inspected and described in general conformance with the Unified Soil Classification System (USCS). Samples were sealed and packaged for transportation to our geotechnical laboratory. After completion of drilling, the borings were backfilled with cement slurry in accordance with EHD requirements and patched where appropriate.

1.03.2 PERCOLATION TESTING

MTGL conducted a preliminary infiltration feasibility study at the site on August 16th, 2024. One shallow boring (P-1) was drilled to a depth of approximately 8½ feet BGS using hand tools and converted to a borehole percolation test to evaluate infiltration feasibility. The boring was prepared and tested by an MTGL engineer in general accordance with methods outlined in the Orange County BMP Design Manual and Technical Guidance Document (Santa Ana Regional Water Quality Control Board, 2013). To prevent caving, a 3-inch diameter perforated pipe with an annular gravel pack was placed in the borehole. The test results of the percolation test are summarized in Section 4.11 Preliminary Infiltration Feasibility and in Appendix F.

1.03.3 LABORATORY TESTING

Laboratory testing was performed on select samples to verify the field classification of the recovered samples and evaluate the geotechnical properties of the subsurface materials. Laboratory tests were performed in general conformance with applicable ASTM or State of California Department of Transportation (Caltrans) standard methods. The results of our laboratory tests are presented in Appendix C.

1.03.4 GEOTECHNICAL ANALYSIS AND REPORT PREPARATION

This report was prepared to present our geotechnical conclusions and recommendations based on the findings of the background review, field exploration, laboratory testing, and geotechnical analysis of the compiled data.

2.00 FINDINGS

The following sections present our findings of our office research, field investigation, and laboratory testing.

2.01 REVIEW OF HISTORIC AERIAL PHOTOGRAPHS

Our review of historic aerial photographs and historic topographic maps indicates that construction of the existing pool facility to be replaced was completed between 1992 and 1994. Prior to 1992, the site was occupied by two buildings that were constructed before 1938. The site remained relatively unchanged between 1938 and 1992, with the exception of the construction of the tennis courts to the east of the existing pool facility and the district-owned annex building to the north of the existing pool facility between 1972 and 1980.

2.02 GEOLOGIC SETTING

The site is located within the Peninsular Ranges Geomorphic Province of California, which stretches from the Los Angeles basin to the tip of Baja California in Mexico. This province is characterized as a series of northwest trending mountain ranges separated by subparallel fault zones, and a coastal plain of subdued landforms. The mountain ranges are underlain primarily by Mesozoic metamorphic rocks that were intruded by plutonic rocks of the southern California batholith, while the coastal plain is underlain by subsequently deposited marine and non-marine sedimentary formations. The site is located within the coastal plain portion of the province. Based on geologic mapping (Morton & Miller, 2006), the site is underlain by undocumented fill, colluvium, old paralic deposits, and Topanga Formation. Figure 4 presents a map of regional geology within the vicinity of the site.

2.03 SUBSURFACE CONDITIONS

As encountered in our borings, the site is underlain by undocumented fill (Qf), colluvium (Qoc) old paralic deposits (Qop), and Topanga Formation (Tt). Materials encountered in our borings are generally consistent with the mapped geologic units presented by Morton and Miller, (2006). Figures 5A and 5B present Geologic Cross Sections A-A' and B-B', respectively. Descriptions of the materials encountered in the borings are presented below. Additional descriptions of materials encountered are presented on the boring logs in Appendix B.

Undocumented Fill (Qf) – Undocumented fill was encountered in each of the borings (excluding Boring P-1) beneath the pavement section and extended to depths ranging from approximately 2 to 7 feet BGS. As encountered, the fill generally consisted of various shades of brown, loose to medium dense, fine to coarse grained, silty to clayey sand, and stiff, sandy clay, with variable amounts of gravel, cobbles, rootlets, and debris.

No documentation regarding the placement of the existing fill underlying the site was available at the time of this report. Therefore, the fill is considered undocumented and unsuitable for the support of proposed structures.

Colluvium (Qoc) – Colluvium was encountered in each of the borings (excluding Boring HA-3) either beneath the pavement section or beneath the fill and extended to depths ranging from approximately 5 to 13 feet BGS. As encountered, the colluvium generally consisted of various shades of brown, loose to dense, fine to medium grained, silty to clayey sand.

Old paralic deposits (Qop) – Late to middle Pleistocene-age old paralic deposits were encountered beneath the fill and colluvium in each of the borings. These materials were observed to extend to the total depths explored in each of the borings, excluding Boring B-2, in which the deposits were observed to extend to a depth of approximately 40 feet BGS. As encountered, the old paralic deposits generally consisted of various shades of brown, gray and olive, medium dense to very dense, silty to clayey sands, and very stiff to hard clays. The deposits encountered were fine to coarse grained, oxidized, laminated, micaceous, and contained variable amounts of manganese nodules and calcium carbonate nodules.

Topanga Formation (Tt) – Miocene-age Topanga Formation was encountered beneath the old paralic deposits in Boring B-2 at a depth of approximately 40 feet BGS and extended to the maximum depth explored. As encountered, the Topanga Formation generally consisted of light gray to olive, moderately cemented, fine to medium grained, silty sandstone. The deposits encountered were oxidized and contained laminations.

A summary of boring locations and pertinent data for each boring are presented in the table below.

SUMMARY OF SUBSURFACE SOIL CONDITIONS

Boring No.	Drilled Depth (Feet)	Latitude (Degrees)	Longitude (Degrees)	Surface Conditions*	Existing Ground Elevation (Feet)**	Approximate Thickness of Fill (Feet)	Groundwater Depth Below Ground Surface (Feet)
B-1	21½	33.54229	-117.77630	AC	168½	2	Not Encountered
B-2	51½	33.54218	-117.77625	AC	171½	2½	Not Encountered
P-1	8½	33.54218	-117.77653	AC	165	Not Encountered	Not Encountered
HA-1	15	33.54257	-117.77580	PCC	182	6	Not Encountered
HA-2	15½	33.54253	-117.77606	PCC	182	7	Not Encountered
HA-3	10	33.54218	-117.77578	Soil	181½	6	Not Encountered

*PCC – Portland Cement Concrete, AT – Artificial Turf, AC – Asphalt Concrete, DG – Decomposed Granite
**Elevation data based on survey by JL Surveying (2022)

2.04 SURFACE AND GROUNDWATER CONDITIONS

Groundwater was not encountered in the borings. No areas of ponding or standing water were present at the time of our study. Further, no springs or areas of natural seepage were found. Additionally, no visible seepage was observed within the borehole after approximately one hour after drilling.

Based on our review of readily available data, historic groundwater in the immediate site vicinity is estimated to be at a depth greater than approximately 30 feet BGS (CGS, 1997). We do not anticipate groundwater to be encountered during the proposed construction. However, changes in rainfall, irrigation and local drainage may produce seepage or perched groundwater conditions below the site. These conditions are generally difficult to predict and typically mitigated if and when they occur.

2.05 GEOLOGIC HAZARDS

Geologic hazards are summarized and discussed with respect to the site and proposed development below.

2.05.1 STRONG GROUND MOTION AND MAPPED SEISMIC DESIGN PARAMETERS

A geologic hazard likely to affect the project is ground shaking as a result of movement along an active fault zone in the vicinity of the subject site (USGS, 2024). Based on the average field standard penetration resistance ($\bar{N}$) measured in Boring B-2, the site may be classified as Site Class D. The mapped site coefficients and maximum considered earthquake (MCER) spectral response acceleration parameters for this site class and location in accordance with the 2022 CBC are presented below:

2022 CALIFORNIA BUILDING CODE – MAPPED SITE COEFFICIENTS

Site Coordinates	
Latitude	**Longitude**
33.5423°	-117.7762°
Site Coefficients and Spectral Response Acceleration Parameters	**Values**
Site Class	D
Site Coefficients, F_a	1.0
Site Coefficients, F_v	Null*
Mapped Spectral Response Acceleration at Short Period, S_s	1.319 g
Mapped Spectral Response Acceleration at 1-Second Period, S_1	0.468 g
Mapped Design Spectral Acceleration at Short Period, S_{DS}	0.879 g
Design Spectral Acceleration at 1-Second Period, S_{D1}	Null*
Peak Ground Acceleration, *PGA*	0.577 g
Site Modified Peak Ground Acceleration, PGA_m	0.635 g

* A site-specific seismic hazard analysis is required per Section 11.4.8 of ASCE 7-16 and should be performed for Site Class D sites with S_1 values greater than or equal to 0.2g unless exceptions are taken in which the values of S_{M1} and S_{D1} are

increased by 50%. If exceptions are taken, then an F_v value of 1.7 may be used in accordance with Table 11.4-2 of ASCE 7-16 Supplement 3 (per the 2022 CBC).

2.05.2 SITE SPECIFIC GROUND MOTION HAZARD ANALYSIS

For a site class D, a site-specific ground motion analysis is required to be performed in accordance with the requirements of 2022 CBC and ASCE 7-16. As part of the site-specific analysis, base ground motions were evaluated with both a Probabilistic Seismic Hazard Analysis (PSHA) and a Deterministic Seismic Hazard Analysis (DSHA) to characterize earthquake ground shaking that may occur at the site during future seismic events.

The PSHA is based on an assessment of the recurrence of earthquakes on potential seismic sources in the region and on ground motion prediction models of different seismic sources in the region. The United States Geological Survey (USGS) unified hazard analysis tool was used to develop a seismic hazard curve and the USGS risk targeted ground motion calculator was used to analyze ground motions for corresponding periods. Maximum directional scale factors were applied to the results.

The DSHA is represented by the 84th percentile of the spectral accelerations for different periods using Pacific Earthquake Engineering Research Center's (PEER) Next Generation Attenuation West-2, Ground Motion Prediction Equations (NGA West 2 GMPE) tool. Fault parameters including the magnitude and width required for the NGA West 2 GMPE tool were obtained from the USGS Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3) model. After applying maximum directional scale factors appropriate for each period, the maximum directional deterministic model specific to the site was developed.
Based on the PSHA and DSHA models, the Site-Specific Risk-Targeted Maximum Considered Earthquake (MCER) was taken as the lesser of the spectral response accelerations from the PSHA and DSHA. The design response spectrum and design acceleration parameters were calculated in accordance with the procedures of ASCE 7-16. The site coefficients and maximum considered earthquake spectral response acceleration parameters are presented below. Tabulated values and graphical plots are included in Appendix E.

ASCE 7-16 SITE SPECIFIC SEISMIC PARAMETERS

Site Coefficients and Spectral Response Acceleration Parameters	Values
Site Class	D
Site Coefficient, F_a	1.0
Site Specific Site Coefficient, F_v	2.5
Site Specific Spectral Response Acceleration at Short Period, S_s	1.547 g
Site Specific Spectral Response Acceleration at 1-Second Period, S_1	0.526 g
Site Specific Design Spectral Acceleration at Short Period, S_{DS}	1.031 g
Site Specific Design Spectral Acceleration at 1-Second Period, S_{D1}	0.877 g
Site Specific Peak Ground Acceleration, PGA	0.628 g

2.05.3 HISTORIC EARTHQUAKES

The site is located in a seismically active area, as is the case throughout the majority of southern California. At this time, it is not possible to state with certainty when or where future large-magnitude earthquakes will occur, or what the magnitude or intensity of these events will be. Historic earthquakes that have occurred within approximately 60 miles (100 kilometers) of the site are summarized in the table below.

The nearest of these historic earthquakes was the magnitude 6.4 Long Beach earthquake of March 11th, 1933. Its epicenter was located approximately 15 miles northwest of the project site. The causative fault (Newport-Inglewood-Rose Canyon fault zone) is confirmed by the USGS. Our research of regional geologic and seismic data did not reveal any known instances of ground failure associated with regional seismic activity within the project site as a result of the earthquake.

HISTORIC SEISMICITY

Site Coordinates			
Latitude		**Longitude**	
33.5423°		-117.7762°	
Date	**Event/Location**	**Magnitude (M_W)**	**Epicentral Distance From Site (Miles)**
November 22nd, 1800	Catalina	6.3	45
July 11th, 1855	San Gabriel	6.0	43
December 8th, 1812	Wrightwood	7.5	58
December 16th, 1858	San Bernardino	6.0	51
July 30th, 1894	Lytle Creek	6.2	54
July 22nd, 1899	Cajon Pass	6.4	55
December 25th, 1899	San Jacinto	6.7	49
April 21st, 1918	Hemet	6.7	50
July 23rd, 1923	San Bernardino	6.2	49
March 11th, 1933	Long Beach	6.4	15

2.05.4 ACTIVE FAULTING AND FAULT-RUPTURE HAZARD

The closest known active fault is the Newport-Inglewood-Rose Canyon Fault zone (Dana Point Section) located approximately 2½ miles southwest of the project site (USGS, 2024). Other regional faults capable of generating seismic hazards include the Palos Verdes, Coronado Bank, San Diego Trough, Catalina Ridge, and San Clemente fault zones to the southwest and the San Joaquin Hills Thrust, Elsinore, San Jacinto, and San Andreas fault zones to the northeast. The site is not mapped as being located within an Alquist-Priolo Earthquake Fault Zone. Based on our review of the referenced fault data bases and geologic

maps, no active faults are known to underlie or project toward the site; therefore, the probability of fault rupture at the site is considered low. The Temple Hills Fault is mapped as underlying the northern perimeter of the site; however, the Temple Hills Fault is not considered to be a Holocene-active fault by USGS.

2.05.5 LIQUEFACTION AND DYNAMIC SETTLEMENT

Liquefaction occurs when loose, saturated sands and silts are subjected to strong ground shaking. The soils lose shear strength and behave like a liquid, resulting in ground surface settlements. Additional effects of liquefaction include possible lateral spreading and liquefaction-induced bearing failure during an earthquake. Due to the relatively dense materials underlying the site and the lack of a shallow groundwater table, the potential for liquefaction to affect the site is considered low. Additionally, the site is not mapped as being located within a zone of required investigation for liquefaction hazard (CGS, 1998) (see Figure 7).

2.05.5.1 BEARING FAILURE

When liquefaction occurs, the soil can completely lose its shear strength and lose its capacity to support structures resulting in a foundation bearing failure. Lightweight structures which are embedded into liquefiable soil and extend below the groundwater table contain large void spaces which may “float” or lift up and out of the ground surface during or after an earthquake. Based on the low potential for liquefaction to affect the site, the potential for bearing capacity failure due to liquefaction is considered low.

2.05.5.2 LATERAL SPREADING (LATERAL DISPLACEMENT)

Lateral spreading is a condition where a relatively stiff block of soil moves laterally toward a free face or slope on a liquefied zone of subsurface soil. Lateral spreads generally develop along gentle slopes and move toward a free face such as an incised river channel. Lateral spreads can cause significant horizontal movement causing fissures, and scarps to develop at the surface. Lateral spreads have been observed to disrupt foundations located across a failure, to rupture sewers, pipelines and other utilities and compress or buckle structures at the toe of the spread. Due to the site being relatively flat, and the low potential for liquefaction to affect the site, the potential for lateral spreading is considered negligible.

2.05.5.3 LIFELINE HAZARDS

Liquefaction, lateral spreading, and seismically induced settlement of structures may also pose problems for streets and lifelines. Specifically, natural gas pipelines may

break and catch fire during an earthquake and water lines may break preventing firefighters from accessing water. Therefore, consideration should be given to providing isolated and flexible connections for gas and water utility lines as a preventive measure.

2.05.6 TSUNAMIS, SEICHES, AND FLOODING

The site is not mapped as being located within an area mapped by the California Geological Survey as subject to inundation by tsunami (State of California, 2021). Therefore, the potential for a tsunami to affect the site is considered low.

Seiches are periodic oscillations in large bodies of water such as lakes, harbors, bays, or reservoirs. The site is not located adjacent to any confined bodies of water; therefore, the potential for a seiche to affect the site is considered low.

The site is not mapped as being located within a flood hazard area as identified by FEMA flood hazard maps. According to the Flood Insurance Rate Map (FIRM), the site is mapped as being located within an area designated as Zone X (FEMA, 2019). Zone X is defined as an area of minimal flood hazard.

2.05.7 LANDSLIDES AND SLOPE STABILITY

As described in Section 1.01 of this report, the perimeter of site is generally bound on the north, west, and east by ascending retaining walls that are anticipated to remain in place during the pool replacement project. Adequate setbacks from the face of wall and foundation elements at the locations described in Section 1.01 should be achieved at new building foundation bearing depths and pool excavation bottoms to mitigate potential slope and retaining wall instability.

Based on our review of available documentation, no landslides have been mapped within the project boundaries. No evidence of landslides was observed during our site reconnaissance and subsurface investigation. Additionally, the site is not mapped as being located within a zone of required investigation for landslide hazard (CGS, 1998) (see Figure 7). Therefore, the potential for landslides to affect the site is considered low.

2.05.8 SUBSIDENCE

The site is not mapped as being located in an area of known subsidence associated with fluid withdrawal (groundwater or petroleum) (USGS, 2024); therefore, the potential for subsidence due to the extraction of fluids is considered negligible.

2.05.9 HYDRO-CONSOLIDATION

Hydro-consolidation can occur in recently deposited sediments (less than 10,000 years old) that were deposited in a semi-arid environment. Examples of such sediments are aeolian sands, alluvial fan deposits, and mudflow sediments deposited during flash floods. The pore spaces between the particle grains can re-adjust when inundated by groundwater causing the material to consolidate. Loose and uncompacted fill materials can also be susceptible to collapse upon wetting. The relatively dense formational materials underlying the site are not considered susceptible to hydro-consolidation.

3.00 CONCLUSIONS

3.01 GENERAL CONCLUSIONS

Based on our geotechnical review of the planned construction, it is our opinion that the site is suitable for the proposed construction provided our conclusions and recommendations are taken into consideration during design, are incorporated into the project's plans and specifications, and are implemented during grading and construction. The recommendations presented herein may need to be updated once final plans are developed.

The main geotechnical considerations affecting the project are the potential for cut/fill transitions below the proposed pool, the presence of potentially compressible and expansive undocumented fill soils, and the presence of existing retaining walls near the proposed two-story building and pool. To reduce the potential for settlement, the existing undocumented fill and colluvium should be excavated in their entirety below buildings, pools, and other settlement sensitive improvements. Areas to receive proposed structures and the new pool area should be graded in a manner that removes any existing cut/fill transitions and reduces the potential for new cut/fill transitions to develop during grading.

To reduce the potential for expansive heave, shallow foundations, concrete slabs-on-grade, pool shells, pavements, and hardscapes, should be underlain by at least 2 feet of material with an expansion index of 50 or less. Additionally, retaining walls and fill abutting the proposed pool shell should consist of granular, free draining, non-expansive materials.

In general, conventional shallow spread footings bearing entirely on compacted fill may be used for the support of the proposed structures. Proposed pole-type structures may be supported on Cast-In-Drilled-Hole (CIDH) deep foundations. Special care should be taken by the contractor to protect the existing retaining walls and foundation elements along the perimeter of the site and near proposed improvements during the demolition of existing improvements and construction of new improvements.

4.00 RECOMMENDATIONS

The following recommendations are provided to address the geotechnical aspects of this project and are considered minimum. They may be superseded by more conservative requirements of the architect, structural engineer, building code, or governing agencies. In addition to the recommendations in this section, additional general earthwork and grading specifications are included in Appendix D.

4.01 EARTHWORK

4.01.1 SITE PREPARATION AND CLEARING

Site preparation should begin with the removal of existing structures or improvements, surface vegetation, trash, and debris. Abandoned underground improvements such as utility pipes and tanks should be removed from the site and capped or rerouted at the project perimeter. Resulting excavations should be backfilled and compacted in accordance with the recommendations provided in this report.

Removal of underground tanks is subject to state law as regulated by the County, City, and/or Fire Department. If storage tanks containing hazardous or unknown substances are encountered, the proper authorities must be notified prior to any attempts at removing such objects. If water wells are encountered during construction, they should be exposed and capped in accordance with the requirements of the regulating agencies.

4.01.2 EXCAVATION CHARACTERISTICS

Based on the materials encountered during our investigation, it is anticipated that excavations can be achieved with conventional heavy duty earthwork equipment in good working order. Difficult excavations should be anticipated in the existing parking lot where very dense and potential cemented aggregate base layers were encountered below the pavements. Additionally, difficult excavations may occur in very dense and/or cemented zones of the old paralic deposits and Topanga formation. Rock breakers, carbide tipped augers, or carbide/diamond tipped coring equipment may be required to excavate/drill very dense materials. Gravel and cobbles should also be anticipated. Contract documents should specify that the contractor mobilize equipment capable of excavating and compacting strongly cemented materials. Excavation sidewall instabilities or raveling may also develop in zones of low cohesion materials.

4.01.3 REMOVALS AND OVER-EXCAVATIONS

Recommendations for over-excavation of the existing materials are provided for the building areas supported on shallow foundations, pool areas, and non-structural areas. Structural

plans were not available at the time of our investigation. Therefore, once formal plans are prepared and available for review, this office should review these plans from a geotechnical viewpoint, comment on any changes, and revise the recommendations of this report, if necessary.

4.01.3.1 STRUCTURES SUPPORTED ON CONVENTIONAL FOUNDATIONS

Existing undocumented fill and colluvium should be excavated in their entirety beneath the proposed buildings and other settlement sensitive structures to expose competent old paralic deposits. Over-excavations on the order of approximately 5 to 6 feet are anticipated below the proposed building areas. In addition, over-excavations should extend a minimum depth of 2 feet below proposed foundation bottoms. Over-excavations should extend a minimum lateral distance of 2 feet outside the planned structures and settlement-sensitive improvements, or up to existing improvements, whichever is less. Additionally, the proposed structures should not be underlain by cut/fill transitions.

4.01.3.2 SWIMMING POOL

Existing undocumented fill and colluvium below the proposed pool should be excavated in their entirety to expose competent old paralic deposits. Over-excavations should extend a minimum depth of 2 feet below the bottom of the proposed pool shell/foundation. Given the difference in size and depth between the existing and proposed pools, there is potential for cut/fill transitions to develop during future demolition and earthwork operations. To reduce the potential for differential settlement, the over-excavation for the new pool should provide a uniform thickness of compacted fill below new pool bottom. Over-excavations on the order of approximately 13 feet are anticipated in the swimming pool area. The horizontal limits of excavation should extend a minimum distance of 2 feet outside the new pool perimeter.

4.01.3.3 NON-STRUCTURAL AREAS

Non-structural areas such as pavements, sidewalks, and other miscellaneous flatwork areas will require a minimum depth of 1 foot of removal and re-compaction below the lowest adjacent grade or bottom of bearing elevation. Excavation for hardscape areas should extend a minimum distance of 2 feet outside the hardscape limits.

The exposed soils beneath over-excavation and in cut areas not otherwise requiring over-excavation should be scarified to a minimum depth of 8 inches, moisture conditioned and compacted to a minimum of 90% relative compaction.

The above recommendations are based on the assumption that soils encountered during field exploration are representative of soils throughout the site. Removal and over-excavation depths must be verified, and adjusted if necessary, at the time of grading.

4.01.4 FILL MATERIALS

Removed and/or over-excavated soils, except for expansive soils, roots, debris, and rocks greater than 4 inches, may be used as compacted fill. Fill materials within the upper 12 inches of the new pool bottom should not contain rocks, debris, or fragments of material greater than 3 inches in largest dimension. Concrete slabs, foundations, and pool bottoms should be underlain by at least 2 feet of material with an expansion index of 20 or less as determined by ASTM D4829. We anticipate that most of the predominantly granular materials onsite will meet this expansion index criteria. Onsite clays, such as those encountered in the upper portions of the existing parking lot, are not anticipated to meet the expansion index criteria.

Prior to placing fill, exposed surfaces at the bottom of the excavations should be scarified to a depth of 8 inches, moisture conditioned to near optimum moisture content, and compacted to at least 90% relative compaction. Fill should be placed in horizontal lifts at a thickness appropriate for the equipment spreading, mixing, and compacting the material, but generally should not exceed 8 inches in loose thickness. Fill should be moisture conditioned to near optimum moisture content and compacted to at least 90% relative compaction. The upper 2 feet of fill beneath the new pool bottom should be moisture conditioned to near optimum moisture content and compacted to at least 95% relative compaction. Fill should be benched into sloping ground inclined steeper than 5:1 (horizontal to vertical). The maximum dry density and optimum moisture content for evaluating relative compaction should be determined in accordance with ASTM D1557.

Utility trench backfill beneath structures, pavements and hardscape should be compacted to at least 90% relative compaction. The upper 12 inches of subgrade beneath pavements should be compacted to at least 95%.

4.01.5 EXPANSIVE SOILS

Laboratory test results indicate onsite soils have an expansion index of 61, classified as having a medium expansion potential. To reduce the potential for expansive heave, soils with an expansion index greater than 20 should be removed from the upper 2 feet of

planned structures, pools, and hardscape areas. Predominantly granular material having an expansion index of 20 or less should be used as replacement fill. We anticipate that most of the predominantly granular materials onsite will meet this expansion index criteria. Onsite clays, such as those encountered in the upper portions of the existing parking lot, are not anticipated to meet the expansion index criteria.

4.01.6 IMPORTED SOILS

Imported fill material, if needed, should consist of predominantly granular soils, free of organic matter and rocks greater than 4 inches, with a very low expansion potential (i.e., an expansion index [EI] of 20 or less). Import fill material should be considered non-corrosive as defined by the Caltrans (2018) corrosion guidelines. Non-corrosive soils typically possess an electrical resistivity of more than 1,100 ohm-centimeters (ohm-cm), a chloride content less than 500 parts per million (ppm), less than 0.15% sulfates, and a pH more than 5.5. Materials for use as fill should be evaluated by MTGL's representative prior to filling or importing. MTGL should be afforded sufficient time to visually access, sample, and test proposed import materials prior to delivery on site. To reduce the potential of importing contaminated materials to the site, prior to delivery, soil materials obtained from offsite sources should be sampled and tested in accordance with governing standards of practice. Soils that exhibit a known risk to human health, the environment, or both, should not be imported to the site.

4.01.7 OVERSIZED MATERIALS

Excavations may generate oversized material. Oversized material is defined as rocks or cemented clasts greater than 4 inches in largest dimension. Oversized material should be broken down to no greater than 4 inches in largest dimension for use in engineered fill, used as landscape material, or disposed of off-site. Fill materials within the upper 12 inches of the new pool bottom should not contain rocks, debris, or fragments of material greater than 3 inches in largest dimension.

4.01.8 TEMPORARY EXCAVATIONS

Temporary excavations 4 feet deep or less can generally be made vertically. Deeper temporary excavations should be laid back no steeper than 1½:1 (horizontal: vertical), up to a maximum depth of 20 feet. The faces of temporary slopes should be inspected daily by the Contractor's Competent Person before personnel are allowed to enter the excavation. Zones of potential instability, sloughing, or raveling should be brought to the attention of the engineer and corrective action implemented before personnel begin working in the excavation. Excavated soils should not be stockpiled behind temporary excavations within a distance equal to the depth of the excavation. MTGL should be notified if other surcharge

loads are anticipated so that lateral load criteria can be developed for the specific situation. If temporary slopes are to be maintained during the rainy season, berms and other runoff diverting measures are recommended along the tops of slopes to prevent runoff water from entering the excavation, eroding the slope faces, or causing seepage issues.

Temporary slopes and excavations should be made in conformance with applicable OSHA standards and requirements. Based on the results of our investigation, the subsurface materials can predominately be categorized as Type C soil.

Slopes steeper than those described above will require shoring. Additionally, temporary excavations that extend below a plane inclined at 1½:1 (horizontal:vertical) downward from the outside bottom edge of existing structures or improvements will require shoring. Soldier piles and lagging, internally braced shoring or trench boxes could be used. If trench boxes are used, the soil immediately adjacent to the trench box is not directly supported. Ground surface deformations immediately adjacent to the pit or trench could be greater where trench boxes are used compared to other methods of shoring that actively resist earth pressures.

4.01.9 EXCAVATIONS NEAR EXISTING STRUCTURES

Temporary excavations near existing structures should be performed with special care. Trench excavations, utility installation, and over-excavations for remedial grading should not be allowed within a 2:1 (horizontal: vertical) projection from 9 inches above the bottom of an existing or proposed foundation, as described in 2022 CBC 1809A.14, unless such case is reviewed by the project's geotechnical engineer. Temporary excavations above such projections are anticipated to be acceptable provided they conform to the recommendations in Section 4.01.8. Instances where an excavation is planned to extend below the projections described above should be reviewed on a case-by-case basis by the project's geotechnical engineer.

Where remedial grading operations may compromise the support of existing foundations, excavations may take place by making a series of adjacent slot-cut excavations perpendicular to the adjacent structure in a sequential A-B method. Subsequent sections (B sections) should not be excavated until the remedial earthwork of the adjacent sections (A sections) are complete. The applicability of slot-cuts will depend on the conditions encountered (such as the depth of excavation, the lateral distance from the adjacent structure, the depth of the adjacent foundations, etc.). In general, the length of slot-cuts should be limited to 4 feet or 1/3 of the adjacent footing's width, whichever is less. Cuts deeper than 5 feet should be reviewed by MTGL prior to excavation. More stringent requirements may be necessary

based on the observed site conditions at the time of excavation. The excavations should be no deeper than necessary to complete the work and should be completed as soon as possible. In no case should slot-cuts remain open for more than 24 hours. Backfill of slot-cut excavations should be compacted to a minimum of 95% relative compaction as determined by ASTM test method D1557.

4.01.10 TEMPORARY SHORING

For design of cantilevered shoring, an active soil pressure equal to a fluid weighing 40 pcf can be used for level retained ground or 60 pcf for 2:1 (horizontal:vertical) sloping ground. The surcharge loads on shoring from traffic and construction equipment adjacent to the excavation can be modeled by assuming an additional 2 feet of soil behind the shoring.

For design of soldier piles, an allowable passive pressure of 350 psf per foot of embedment over 2.5 times the pile diameter or the spacing of the piles, whichever is less, up to a maximum of 4,500 psf can be used for soil above the groundwater level. An allowable passive pressure of 175 psf per foot of embedment over 2.5 times the pile diameter or the spacing of the piles, whichever is less, up to a maximum of 2,250 psf can be used for soil below the groundwater level. Hydrostatic pressure should be applied below the groundwater level.

Soldier piles should be spaced at least three pile diameters, center to center. Continuous lagging will be required throughout. The soldier piles should be designed for the full-anticipated lateral pressure; however, the pressure on the lagging will be less due to arching in the soils. For design of lagging, the earth pressures can be limited to a maximum value of 400 psf.

Installation of soldier piles below groundwater (or dewatered soil) will require special construction techniques and equipment, such as temporary casing and/or drilling slurry. Other installation methods may be available. If soldier piles are anticipated to extend to a depth greater than approximately 30 feet, contract documents should specify that the contractor mobilize equipment capable of installing piles below groundwater (or dewatered soil) in order to reduce the potential for claims of delays or extra work to occur.

Piles should be filled with concrete immediately after drilling. The concrete should be pumped to the bottom of the drilled holes using the tremie method. If casing is used, the casing should be removed as the concrete is placed, keeping the level of the concrete at least 5 feet above the bottom of the casing at all times.

4.01.11 SLOPES

All permanent slopes should be constructed no steeper than 2:1 (horizontal: vertical). Faces of fill slopes should be compacted either by rolling with a sheepsfoot roller or other suitable equipment or by overfilling and cutting back to design grade. Fills should be benched into sloping ground when inclined steeper than 5:1 (horizontal: vertical). It is our opinion that cut slopes constructed no steeper than 2:1 (horizontal: vertical) will possess an adequate factor of safety. An engineering geologist should observe all cut slopes during grading to ascertain that no unforeseen adverse geologic conditions are encountered that require revised recommendations.

Where buildings are planned near the top of a slope steeper than 3:1 (horizontal to vertical), special foundations or stabilization measures are recommended. Footings located adjacent to or within existing slopes should be deepened to a depth such that a minimum horizontal distance of H/3 (where H is the height of the slope) exists between the lower outside footing edge and the face of the slope.

All slopes are susceptible to surficial slope failure and erosion. Water should not be allowed to flow over the top of slope. Additionally, slopes should be planted with vegetation that will reduce the potential for erosion.

4.01.12 TEMPORARY DEWATERING

Groundwater seepage may occur locally due to local irrigation or following heavy rain. Temporary dewatering can be accomplished by sloping the excavation bottom to a sump and pumping from the sump. A layer of gravel about 6 inches thick placed in the bottom of the excavation will facilitate groundwater flow and can be used as a working platform.

A specialty dewatering contractor should be contacted for additional project-specific dewatering recommendations.

4.02 FOUNDATIONS

In general, conventional shallow spread footings or mat foundations bearing entirely on compacted fill may be used for the support of the proposed structures. Proposed pole-type structures may be supported on Cast-In-Drilled-Hole (CIDH) deep foundations. Special care should be taken by the contractor to protect the existing retaining walls and foundation elements along the perimeter of the site during demolition of existing improvements and construction of new improvements.

Our recommendations are only minimum criteria based on geotechnical factors and should not be considered a structural design, or to preclude more restrictive criteria of governing agencies or by

the structural engineer. The foundation system should be designed by the project's structural engineer, incorporating the geotechnical parameters described herein and the requirements of applicable building codes.

The foundation recommendations provided herein are considered generally consistent with methods typically used in southern California, however, other alternatives may be available. Based on the results of our geotechnical investigation, recommendations for various foundation systems are presented in the following sections.

4.02.1 CONVENTIONAL SHALLOW FOUNDATIONS

The planned structures can be supported on shallow conventional spread and/or continuous footings bearing entirely on compacted fill. Over-excavation and re-compaction of foundation subgrade should be performed as recommended in Section 4.01.3. An allowable bearing capacity of 2,000 psf may be used to design the foundations. The bearing capacity can be increased by 250 psf for each foot of depth below the minimums listed below, up to a maximum of 3,500 psf. The bearing value can be increased by ⅓ when considering the total of all loads, including wind or seismic forces. Footings located adjacent to or within slopes should be extended to a depth such that a minimum horizontal distance of 7 feet or H/3 (where H is the height of the slope), which ever is greater, exists between the lower outside footing edge and the face of the slope.

The recommended minimum footing width and embedment depth below the lowest adjacent grade are as follows:

CONVENTIONAL SHALLOW FOUNDATIONS

Foundation Type	Minimum Width	Minimum Depth
Continuous (Interior)	12 inches	18 inches
Continuous (Perimeter)	12 inches	24 inches
Spread Footings	24 inches	24 inches

Lateral loads will be resisted by friction between the bottoms of footings and passive pressure on the faces of footings and other structural elements below grade. An allowable coefficient of friction of 0.35 can be used. Passive pressure can be computed using an allowable lateral pressure of 350 psf per foot of depth below the ground surface for level ground conditions. When combining passive pressure and friction for lateral resistance, the passive pressure component should be reduced by one-third.

The passive pressure can be increased by one-third when considering the total of all loads, including wind or seismic forces. The upper 1 foot of soil should not be relied on for passive support unless the ground is covered with pavements or slabs.

4.02.2 CAST IN DRILLED HOLE (CIDH) POLES FOUNDATIONS

Based on our review of preliminary design schematics provided by the district, we understand that pole type foundations are proposed for the support of canopies, light poles, and signs. For preliminary design purposes, individual drilled, cast-in-place concrete piers may be designed using a unit skin friction of 250 psf. This value assumes a minimum pile depth of 10 feet and minimum pile diameter of 30 inches. The upper 2 feet of all piers should be neglected when evaluating allowable axial capacities. Total downward capacities estimated may be increased by one-third for short-term transient loads. Uplift resistance can be taken as the weight of the pole and foundation plus 2/3 of the downward axial capacity.

Lateral resistance on piles may be evaluated using Section 1806A of the 2022 CBC for a Class 4 material. Accordingly, a lateral soil bearing of 150 pounds per square foot, per foot, of embedment may be used to verify required embedment depth. This value can be doubled, provided the pole can tolerate at least a ½ inch deflection at the ground surface due to short term loading, per Section 1806A.3.4 of the 2022 CBC. The upper 18 inches of soil should be neglected in non-paved areas. Piles should be spaced at least 3 diameters apart, center to center.

These values are for isolated single piles. A group action reduction in capacity would apply for closely spaced piles. These recommendations also assume that the foundations will be embedded against firm and intact soils.

During excavation, layers of cohesionless soils may be encountered; therefore, caving of the pile shafts should be anticipated. Special care should be taken during construction to stabilize the sides of the CIDH piles, as required. The actual required depths must be field verified by the geotechnical engineer. The adequacy of the CIDH piles will depend heavily on construction methods and procedures. Large zones of disturbance around the CIDH piles can lead to lower skin friction due to excessive stress relief around the length of the piles. The piles should be constructed by qualified contractors experienced in this type of construction and monitored on a full-time basis by the geotechnical consultant.

Contract documents should specify that the contractor mobilize equipment capable of penetrating hard, cemented material to reduce the potential for claims, delays, or extra work to arise.

Foundation plans are not available at this time. On a preliminary basis, we estimate that post-construction single pile foundation settlements will be less than 1 inch. Once foundation plans are available, MTGL should review the plans and revise these recommendations, as necessary.

4.02.3 SETTLEMENT CONSIDERATIONS

Foundations should be designed for the anticipated settlements. Static settlement of an individual foundation member will vary depending on the plan dimensions of the foundation and the actual load supported.

We estimate maximum static settlement of foundations designed and constructed in accordance with the recommendations presented to be on the order of 1 inch. Differential settlement between similarly loaded and adjacent footings are expected to be less than ½ inch across 40 feet, provided footings are founded on similar materials. Static settlement of all foundations is expected to occur rapidly and should be essentially complete shortly after initial application of the loads.

4.03 INTERIOR SLABS ON GRADE

The project's structural engineer should design concrete slabs-on-grades for buildings. However, it is recommended that interior slabs be at least 5 inches thick and reinforced with at least No. 4 bars at 18 inches on center each way. The concrete should have a 28-day compressive strength of at least 3,000 psi, a water to cement ratio of 0.50 or less, and a slump of 4 inches or less.

Slabs should be provided with weakened plane joints. Joints should be placed in accordance with the American Concrete Institute (ACI) guidelines. The project's architect should select the final joint patterns. A 1 inch maximum size aggregate mix is recommended for concrete slabs. The corrosion potential of on-site soils with respect to reinforced concrete will need to be taken into account in concrete mix design. Coarse and fine aggregate in concrete should conform to the "Greenbook" Standard Specifications for Public Works Construction.

Moisture protection should be installed beneath slabs where moisture sensitive floor coverings will be used. The project's architect should review the tolerable moisture transmission rate of the proposed floor covering and specify an appropriate moisture protection system. Typically, a plastic vapor barrier (15-mil Stegowrap or equivalent) is used and should comply with ASTM E1745. The

vapor barrier installation should comply with ASTM E1643 and ACI 302.1R. The floor covering manufacturer should be consulted to determine the volume of moisture vapor allowable and treatment needed to reduce moisture vapor emissions to acceptable limits for the particular type of floor covering installed.

4.04 HARDSCAPE

Hardscape and other exterior concrete slabs-on-grade not subject to vehicular loads should be underlain by at least 2 feet of material with an expansion index of 20 or less. Exterior slabs should be at least 4 inches thick and reinforced with at least No. 3 bars at 18 inches on center each way. Slabs should be provided with weakened plane joints. Joints should be placed in accordance with the American Concrete Institute (ACI) guidelines. The project's architect should select the final joint patterns. A 1 inch maximum size aggregate mix is recommended for concrete for exterior slabs. The corrosion potential of on-site soils with respect to reinforced concrete will need to be taken into account in concrete mix design. Coarse and fine aggregate in concrete should conform to the "Greenbook" Standard Specifications for Public Works Construction.

4.05 PREWETTING RECOMMENDATIONS

Prior to placing concrete slabs and flatwork, the underlying soils should be brought to within a minimum of 2% and a maximum of 4% above its optimum moisture content for a depth of 12 inches prior to the placement of concrete. The geotechnical consultant should perform in-situ moisture tests to verify that the appropriate moisture content has been achieved a maximum of 24 hours prior to the placement of concrete or moisture barriers.

Once the slab subgrade soil has been pre-wetted and compacted, the soil should not be allowed to dry prior to concrete placement. If the subgrade soil is dry, the moisture content of the soil should be restored prior to placement of concrete and re-tested.

Proper moisture conditioning and compaction of subgrade soils should be performed prior to placement of concrete. Even with proper site preparation, some soil moisture changes of the subgrade soils supporting the concrete flatwork due to edge effects (shrink/swell) may occur. Drying and/or wetting of subgrade soils adjacent to landscaped areas or open fields may increase the potential of shrink/swell effects beneath concrete flatwork areas. To help reduce edge effects, lateral cutoffs, such as inverted curbs are recommended. Control joints should be used to reduce the potential for flatwork panel cracks as a result of minor soil shrink/swell.

The recommendations are intended to reduce the potential for cracking of slabs; however, even with the incorporation of the recommendations presented herein, slabs may still exhibit some

cracking. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics.

4.06 CORROSIVITY

Soluble sulfate tests performed on sampled subgrade material suggest that concrete at the subject site will have a moderate exposure to water soluble sulfates in the soil. Accordingly recommend that the concrete be designed to resist a moderate exposure category (Class S1). Our recommendations for concrete exposed to sulfate-containing soils are presented below.

RECOMMENDATIONS FOR CONCRETE EXPOSED TO SULFATE CONTAINING SOILS

Sulfate Exposure Severity	Class	Water soluble sulfate (SO_4) in soil (% by wgt)	Sulfate (SO_4) in water (ppm)	Maximum Water to Cement Ratio by Weight	Minimum Compressive Strength (psi)	Cement Type	Calcium Chloride Admixture
Negligible	S0	0.00 - 0.10	0-150	---	2,500	---	No Restriction
Moderate	S1	0.10 - 0.20	150-1,500	0.50	4,000	II/V	No Restriction
Severe	S2	0.20 - 2.00	1,500-10,000	0.45	4,500	V	Not Permitted
Very Severe	S3	Over 2.00	Over 10,000	0.45	4,500	V Plus Pozzolan	Not Permitted

Corrosivity testing consisting of soils reactivity (pH) and resistivity (ohms-cm) were also tested on select soil samples. The test results indicate a soil reactivity of 7.7 and a resistivity of 965 ohms-cm. A neutral or non-corrosive soil has a reactivity value ranging from 5.5 to 8.4. Generally, soils that could be considered corrosive to metal have resistivities less than 3,000 ohms-cm. Soils with resistivity values of less than 1000 ohms-cm can be considered extremely corrosive.

Based on our test results, it is our opinion that the underlying soils at the site have a moderate to high corrosion potential. Protection of buried pipes utilizing coatings on all underground pipes; clean backfills and a cathodic protection system can be effective in controlling corrosion. A qualified corrosion engineer should be consulted to further assess the corrosive properties of the soil and provide mitigation measures appropriate to the improvements.

4.07 RETAINING STRUCTURES

Embedded structural walls and pool shells should be designed to resist the lateral earth pressures imposed by the adjacent soils. The magnitude of these earth pressures will depend on the amount of deformation that the wall or shell can yield under the load. If the wall can yield sufficiently to mobilize the full shear strength of the soils, it may be designed for the active condition. If the wall cannot yield under the applied load, then the shear strength of the soil cannot be mobilized, and

the earth pressures will be higher. These walls, such as basement walls and swimming pools, should be designed for the at rest condition. If a structure moves towards the retained soils, the resulting resistance developed by the soil will be the passive resistance.

For design purposes, the recommended equivalent fluid pressure for each case for walls is provided below. These values assume the walls or pool shells will be constructed above the static groundwater table and backfilled with non-expansive soils. Additionally, retaining wall backfill should be compacted to at least 90% relative compaction based on the maximum density defined by ASTM D1557. Soils adjacent to pools should also be compacted to at least 90% relative compaction based on the maximum density defined by ASTM D1557. Retaining structures should be designed to resist the following lateral earth pressures for static conditions.

- Passive Earth Pressure – Same as for shallow foundations.
- At rest lateral earth pressure - 60 pcf.
- Active Earth Pressures (Equivalent Fluid Weights):

Slope of Retained Material	Equivalent Fluid Weight (pcf)
Level	40
2:1 (H:V)	60

It is recommended that retaining wall footings be embedded at least 24 inches below the lowest adjacent finish grade. In addition, the wall footings should be designed and reinforced as required for structural considerations. The wall areas should be over excavated to a minimum depth of 2 feet below the bottom of the proposed footings. The required horizontal limits of the over-excavation area shall be a minimum distance of 2 feet. Soil retaining structures may be designed using a soil unit weight of 120 pcf.

Lateral resistance parameters provided above are ultimate values. Therefore, a suitable factor of safety should be applied to these values for design purposes. The appropriate factor of safety will depend on the design condition and should be determined by the project's structural engineer. These parameters do not include loading from adjacent structures. In addition to the above lateral forces due to retained earth, surcharge due to improvements, such as an adjacent structure or traffic loading, should be considered in the design of the retaining wall. Loads applied within a 1:1 projection from the surcharging structure on the stem of the wall should be considered in the design. If any super-imposed loads are anticipated, this office should be notified so that appropriate recommendations for earth pressures may be provided.

Retaining structures should be designed with effective drainage to prevent the accumulation of subsurface water behind the walls. Backdrains should be installed behind retaining walls exceeding 3 feet in height. Backdrains may consist of a 2 feet wide zone of ¾ inch crushed rock. The backdrain should be separated from the adjacent soils using a non-woven filter fabric, such as Mirafi 140N or equivalent. Weep holes should be provided, or a perforated pipe should be installed at the base of the back-drain and sloped to discharge to a suitable storm drain facility. As an alternative, a geo-composite drainage system such as Mira-drain 6000 or equivalent placed behind the wall and connected to a suitable storm drain facility can be used. The project's architect should provide waterproofing specifications and details. A typical detail for retaining wall backdrains is presented as Figure 8. Backdrains should outlet to suitable drainage devices. Additional construction considerations for pools are presented in Section 4.12 of this report.

4.08 SEISMIC EARTH PRESSURES

If required, the seismic earth pressure can be taken as equivalent to the pressure of a fluid weighing 22 pcf. This value is for level backfill and does not include a factor of safety. Appropriate factors of safety should be incorporated into the design. This pressure is in addition to the un-factored, static active earth pressure. The passive pressure and bearing capacity can be increased by ⅓ when determining the seismic stability of the wall.

4.09 PAVEMENT STRUCTURAL SECTIONS

Recommended pavement structural sections are based on the procedures outlined in "Design Procedures for Flexible Pavements" of the Highway Design Manual, California Department of Transportation. This procedure uses the principal that the pavement structural section must be of an adequate thickness to distribute the load from the design traffic (TI) to the subgrade soils in such a manner that the stresses from the applied loads do not exceed the strength of the soil (R value). Pavement sections were designed based on an R-Value of 10. As requested, structural pavement sections were designed based on Traffic Indexes of 4.0, 5.0 and 6.0. However, a minimum Traffic Index of 7 is recommended for the design of fire access lanes, if needed. The recommended structural sections are as follows:

ASPHALT PAVEMENT STRUCTURAL SECTIONS

Pavement Area	Traffic Index	Asphalt Thickness (inches)	Base Thickness (inches)
Light Traffic	4.0	3	6
Medium Traffic	5.0	3½	8
Heavy Traffic	6.0	4	12
Fire Access Lanes	7.0	5	12

PORTLAND CEMENT CONCRETE PAVEMENT STRUCTURAL SECTION

Pavement Area	Traffic Index	PCC Thickness (inches)	Base Thickness (inches)
Parking Areas & Driveways (min f'c = 4,500 psi)	5.0 – 6.0	6½	6
Fire Access Lanes (min f'c = 4,500 psi)	7.5 – 8.0	7½	6

The upper 12 inches of subgrade should be scarified, moisture conditioned to near optimum moisture content, and compacted to at least 95% relative compaction. Soft or yielding areas should be removed and replaced with compacted fill or aggregate base. Aggregate base and asphalt concrete should conform to the Caltrans Standard Specifications or the "Greenbook" and should be compacted to at least 95% relative compaction. Aggregate base should have an R-value of not less than 78. Materials and methods of construction should conform to good engineering practices.

4.10 UTILITY TRENCHES

Cal/OSHA construction safety orders should be observed during all underground work. Utility trench backfill within street right of way, utility easements, under or adjacent to sidewalks, driveways, or building pads should be observed and tested by the geotechnical consultant to verify proper compaction. Utility trenches which run parallel to footings and extend below a 1½:1 plane projected from the outside edge of the footing should be backfilled with structural fill soils and compacted to at least 90% of the ASTM D-1557 standard. The backfilled utility lines should also be capable of withstanding any surcharge loads imposed. Pea gravel backfill should not be used for these trenches. Trenches crossing perpendicular to foundations should be excavated and backfilled prior to the construction of the foundations. Pipes and trenches near existing footings or settlement sensitive improvements are subject to requirements set forth in the 2022 CBC Section 1809A.14 The excavations should be backfilled in the presence of the geotechnical engineer and tested to verify adequate compaction beneath the proposed footing.

4.10.1 THRUST BLOCKS

For level ground conditions, a passive earth pressure of 350 psf per foot of depth below the lowest adjacent final grade can be used to compute allowable thrust block resistance. A value of 175 psf per foot should be used below groundwater level, if encountered.

4.10.2 PIPELINE SUPPORT

It is anticipated that most of the materials along the pipeline alignments will provide adequate support for the pipes, although loose, soft, wet, and otherwise unsuitable materials may be encountered locally. Unsuitable materials encountered near trench

bottom levels should be evaluated during construction by the geotechnical engineer. Unsuitable materials should be removed from the full width of the trench. The bottoms of the excavations should be observed by the geotechnical consultant prior to placement of pipe bedding. The use of a stabilizing fabric such as Mirafi® HP 570 can be used to stabilize the bottom of the excavations, if needed.

4.10.3 MODULUS OF SOIL REACTION

A modulus of soil reaction (E') of 2,000 psi can be used to evaluate the deflection of buried flexible pipelines. This value assumes that granular bedding material is placed adjacent to the pipe and is compacted to at least 90% relative compaction.

4.10.4 BEDDING

Pipe bedding as specified in the "Greenbook" Standard Specifications for Public Works Construction can be used. Bedding material should consist of clean sand having a sand equivalent not less than 30 and should extend to at least 12 inches above the top of pipe. Alternative materials meeting the intent of the bedding specifications are also acceptable. Samples of materials proposed for use as bedding should be provided to the engineer for inspection and testing before the material is imported for use on the project. The on-site materials are not expected to meet "Greenbook" bedding specifications. The pipe bedding material should be placed over the full width of the trench. After placement of the pipe, the bedding should be brought up uniformly on both sides of the pipe to reduce the potential for unbalanced loads. No voids or uncompacted areas should be left beneath the pipe haunches. Ponding or jetting the pipe bedding should not be allowed.

4.10.5 BACKFILL

Excavated material free of organic debris and rocks greater than 6 inches in any dimension are generally expected to be suitable for use as backfill outside of the "pipe zone". Imported material should not contain rocks greater than 4 inches in any dimension or organic debris. Imported material should have an expansion index of 20 or less. MTGL should observe and, if appropriate, test proposed imported materials before they are delivered to the site. Backfill should be placed in lifts 8 inches or less in loose thickness, moisture conditioned to optimum moisture content or slightly above optimum moisture content, and compacted to at least 90% relative compaction. The upper 12 inches of soil beneath pavement subgrade should be compacted to at least 95% relative compaction.

4.11 INFILTRATION FEASIBILITY

MTGL conducted an infiltration feasibility study at the site on August 16th, 2024. At this time, no preliminary BMP exhibits were available for review. Additionally, the project is in the planning phase. Our fieldwork consisted of converting one shallow geotechnical boring to a percolation test hole. The boring was prepared and tested by an MTGL engineer in general accordance with methods outlined in the Orange County BMP Design Manual and Technical Guidance Document (Santa Ana Regional Water Quality Control Board, 2013). Appendix F presents percolation test data and results. A summary of our results is presented in the following table.

INFILTRATION RATE TEST RESULTS

Test Location	Test Depth (Feet)	Material Type at Test Depth (USCS Classification)	Adjusted Infiltration Rate* (Inches/Hour)
P-1	8½	**Old Paralic Deposits (Qop)**: CLAYEY SAND (SC)	0.01

*The adjusted infiltration rate is calculated by applying a factor of safety of 2 to the tested infiltration rate.

Based on our observations, the feasibility screening category is "No Infiltration." BMP facilities should be lined with an impermeable geomembrane to reduce the potential for water-related distress to adjacent structures or improvements. A subdrain should be installed at the bottom of the BMP facilities. Foundations should be set back at least 10 feet from BMP facilities, or the foundations deepened to a depth that extends below the bottom of the BMPs.

4.12 SWIMMING POOL

The recommended bearing material for the proposed swimming pool bottom is a minimum of 2 feet of compacted engineered fill as discussed in Section 4.01.3.2 of this report, except the fill should be compacted to 95% relative compaction as evaluated by ASTM D1557. If any footings are planned, the foundations may be designed in accordance with the conventional foundation recommendations section of this report, except the upper 2 feet of material beneath the footings should be compacted to 95% relative compaction as evaluated by ASTM D1557. The sidewalls of the pool will act as retaining walls and may be designed in accordance with the retaining wall design parameters presented in this report.

The following recommendations for swimming pools should be considered during the design and construction of the pools.

- Positive surface drainage should be provided around the pool and deck. Water should not be allowed to pond. Surface water should be collected and directed to drainage devices.
- Decking around the swimming pool should be cast free of the pool. Water stops should be provided between any bond beams and deck slabs.
- A hydrostatic relief valve should be provided.

- The pool should be designed for any possible surcharge from nearby structures which fall within a 1:1 (Horizontal: Vertical) plane from the surcharging structure.
- A qualified design engineer should prepare a specific plan and details for the proposed swimming pool.
- Prior to placement of the reinforcing steel, the pool excavation should be observed by the geotechnical engineer.
- Leakage from any appurtenant plumbing may create a groundwater condition that could have adverse effects on the underlying soil. All plumbing and pool features must be leak free and watertight.

4.13 CONSTRUCTION CONSIDERATIONS

4.13.1 MOISTURE-SENSITIVE SOILS AND WEATHER-RELATED CONCERNS

The upper soils encountered at this site may be sensitive to disturbances caused by construction traffic and to changes in moisture content. During wet weather periods, increases in the moisture content of the soil can cause significant reduction in the soil strength and its support capabilities. In addition, soils that become excessively wet may be slow to dry and therefore significantly delay the progress of the grading operations. Therefore, it will be advantageous to perform earthwork and foundation construction activities during the dry season. Much of the on-site soils may be susceptible to erosion during periods of inclement weather. As a result, the project's Civil Engineer/Architect and Grading Contractor should take appropriate precautions to reduce the potential for erosion during and after construction.

4.13.2 DRAINAGE AND GROUNDWATER CONSIDERATIONS

Based on our investigation, groundwater is expected to be below the anticipated depths of grading and the installation of subdrains is not expected to be necessary. However, variations in the ground water table may result from fluctuation in the ground surface topography, subsurface stratification, precipitation, irrigation, and other factors such as impermeable and/or cemented formational materials overlain by fill soils. In addition, during retaining wall excavations, seepage may be encountered. Therefore, we recommend that a representative of MTGL, Inc. be present during grading operations to evaluate areas of seepage. Drainage devices for reduction of water accumulation can be recommended should these conditions occur.

Water should not be allowed to collect in the foundation excavation, on floor slab areas, or on prepared subgrades of the construction area either during or after construction. Undercut or excavated areas should be sloped to facilitate removal of any collected rainwater, groundwater, or surface runoff. Positive site drainage should be provided to

reduce infiltration of surface water around the perimeter of the structure and beneath the floor slabs. The grades should be sloped away from the structure and surface drainage should be collected and discharged such that water is not permitted to infiltrate the backfill and floor slab areas.

4.13.3 SITE DRAINAGE

The site should be designed to provide for positive drainage away from structures in accordance with the building code and applicable local requirements. Unpaved areas should slope no less than 2% away from structure. Paved areas should slope no less than 1% away from structures. Concentrated roof and surface drainage from the site should be collected in engineered, non-erosive drainage devices and conducted to a safe point of discharge. The site drainage should be designed by a civil engineer.

4.14 PLAN REVIEW

MTGL should review the grading and foundation plans to verify that the intent of the recommendations presented in this report has been implemented and that revised recommendations are not necessary as a result of design changes after the date of this report.

5.00 GEOTECHNICAL OBSERVATION AND TESTING

The recommendations provided in this report are based on preliminary design information and subsurface conditions as interpreted from the investigation. Our preliminary conclusions and recommendations should be reviewed and verified during site grading and revised accordingly if exposed geotechnical conditions vary from our preliminary findings and interpretations. The geotechnical consultant should perform geotechnical observation and testing during the following phases of grading and construction:

- During site grading, remedial grading over-excavation, and slot-cut excavations.
- During foundation excavations and placement or deep and shallow foundations.
- Upon completion of retaining wall footing excavations, prior to placing concrete.
- During excavation and backfilling of public utility trenches; of private utility trenches, as directed.
- During processing and compaction of the subgrade for the access and parking areas and prior to construction of pavement sections.
- When any unusual or unexpected geotechnical conditions are encountered during any phase of construction.

6.00 LIMITATIONS

The findings, conclusions, and recommendations contained in this report are based on the site conditions as they existed at the time of our investigation, and further assume that the subsurface conditions encountered during our investigation are representative of conditions throughout the site. Should subsurface conditions be encountered during construction that are different from those described in this report, this office should be notified immediately so that our recommendations may be re-evaluated.

This report was prepared for the exclusive use and benefit of the owner, architect, and engineer for evaluating the design of the facilities as it relates to geotechnical aspects. It should be made available to prospective contractors for information on factual data only, and not as a warranty of subsurface conditions included in this report.

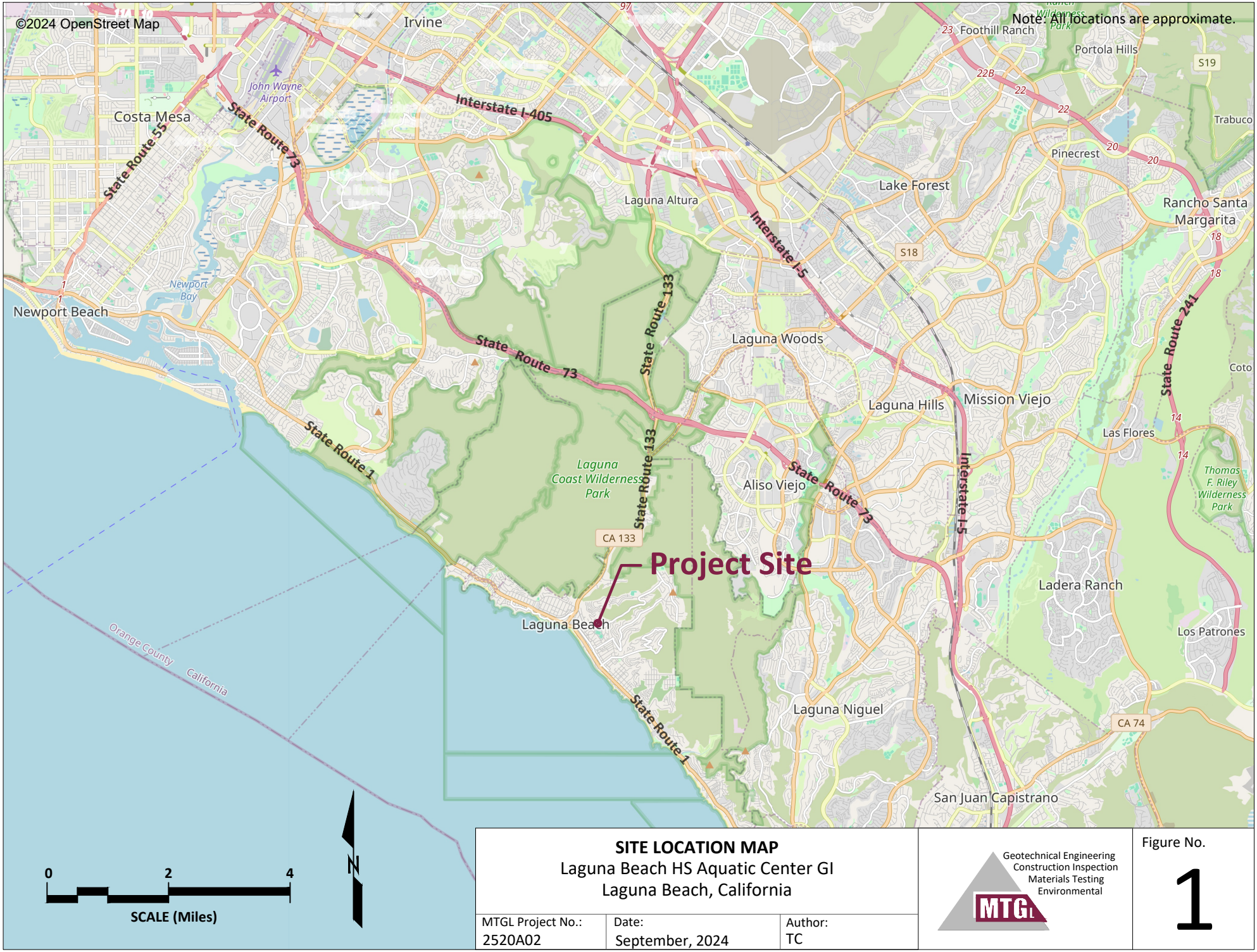
Our investigation was performed using the standard of care and level of skill ordinarily exercised under similar circumstances by reputable soil engineers and geologists currently practicing in this or similar localities. No warranty, express or implied, is made as to the conclusions and professional advice included in this report.

This firm does not practice or consult in the field of safety engineering. We do not direct the Contractor's operations, and we are not responsible for their actions. The contractor will be solely and completely responsible for working conditions on the job site, including the safety of all persons and property during performance of the work. This responsibility will apply continuously and will not be limited to our normal hours of operation.

The findings of this report are considered valid as of the present date. However, changes in the conditions of a site can occur with the passage of time, whether they are due to natural events or to human activities on this or adjacent sites. In addition, changes in applicable or appropriate codes and standards may occur, whether they result from legislation or the broadening of knowledge.

Accordingly, this report may become invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and revision as changed conditions are identified.

FIGURES



SITE LOCATION MAP
Laguna Beach HS Aquatic Center GI
Laguna Beach, California

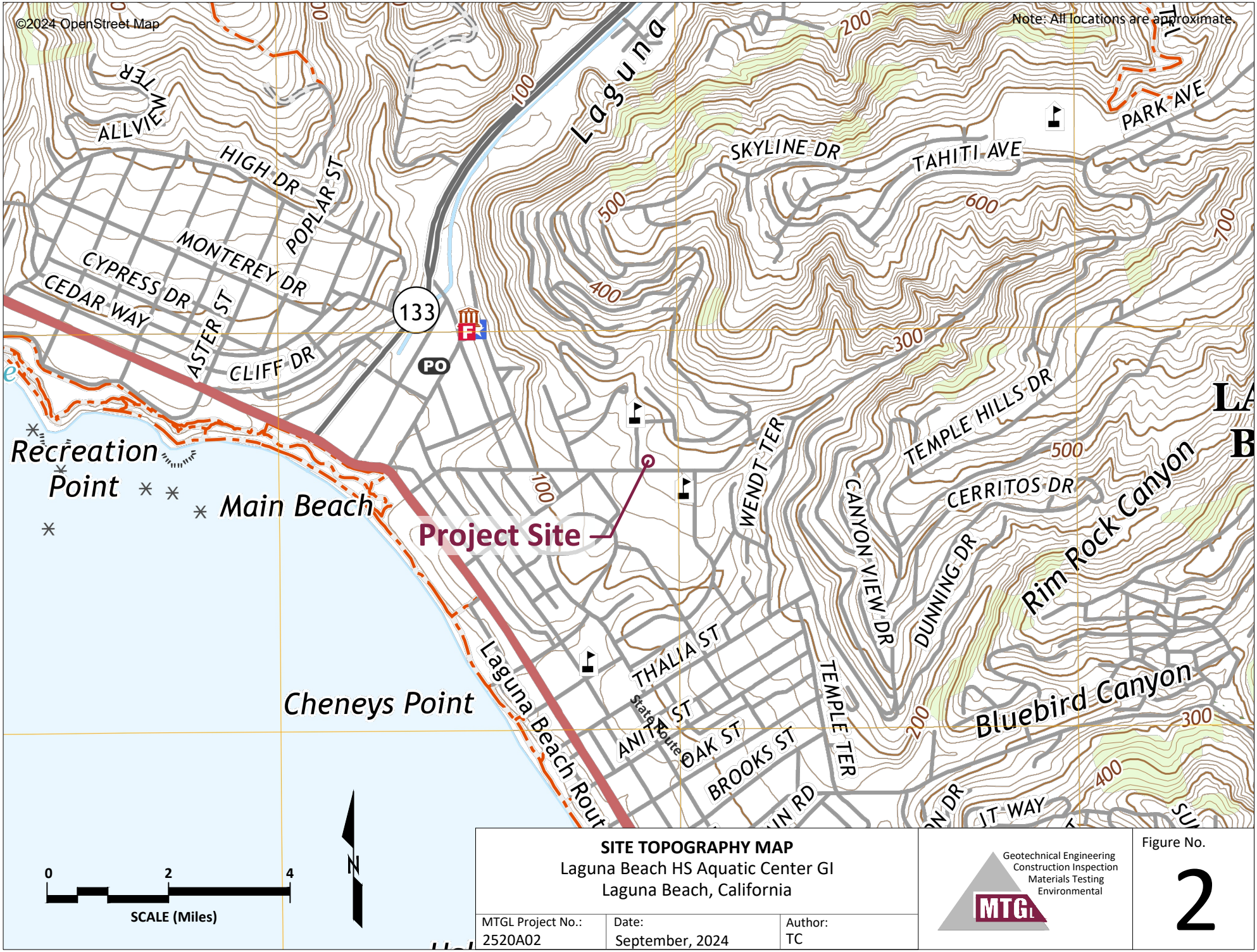
MTGL Project No.:	Date:	Author:
2520A02	September, 2024	TC

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Figure No.

1



SITE TOPOGRAPHY MAP
Laguna Beach HS Aquatic Center GI
Laguna Beach, California

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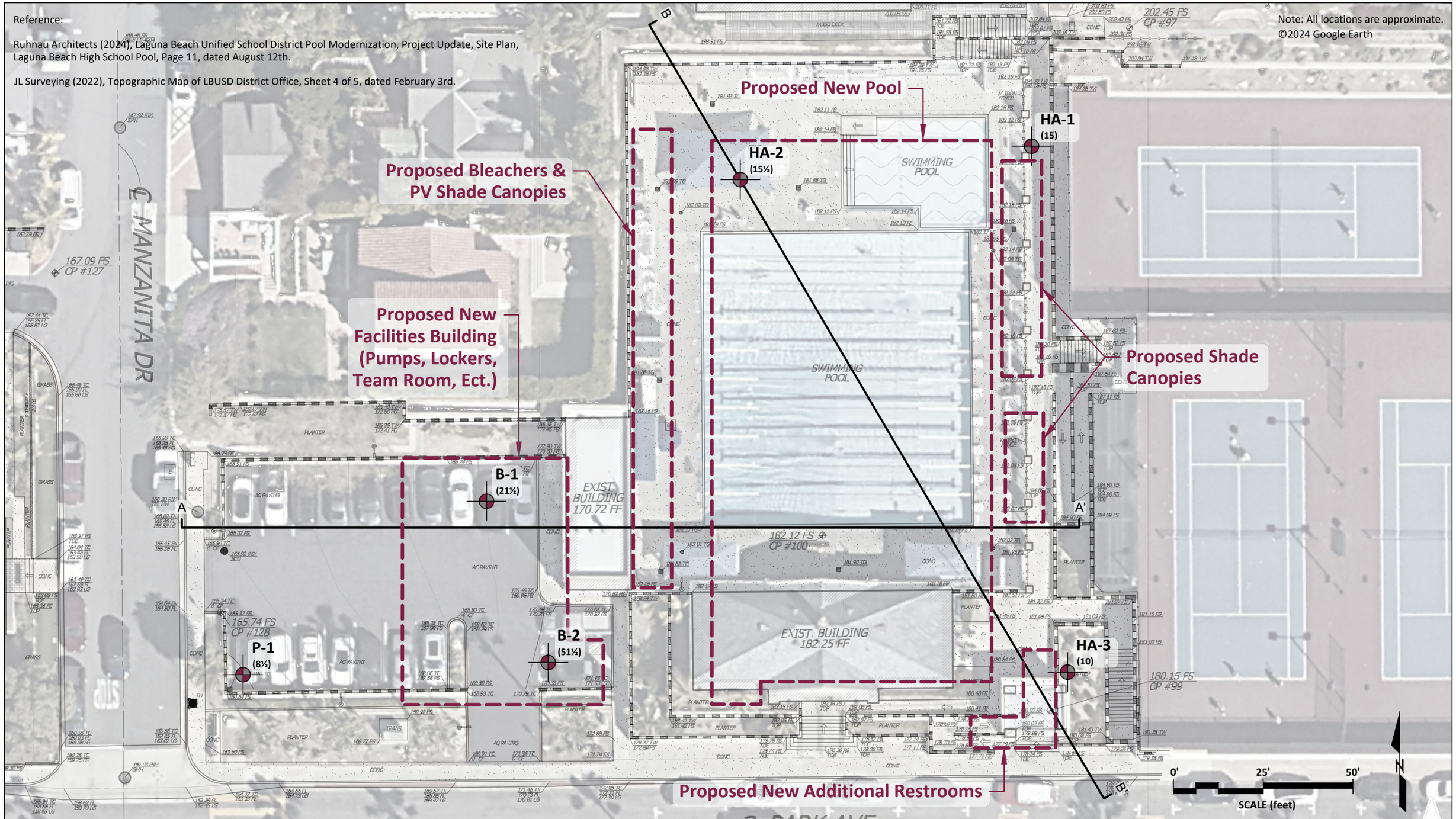
Figure No.
2

Reference:

Ruhnau Architects (2024), Laguna Beach Unified School District Pool Modernization, Project Update, Site Plan, Laguna Beach High School Pool, Page 11, dated August 12th.

JL Surveying (2022), Topographic Map of LBUSD District Office, Sheet 4 of 5, dated February 3rd.

Note: All locations are approximate.
©2024 Google Earth



LEGEND

B-2 (51½) | P-1 (8½) | HA-3 (10) - Location of Boring
Hollow-Stem Auger | Percolation Test | Hand Auger
(Depth in Feet)

B B' - Geologic Cross Section

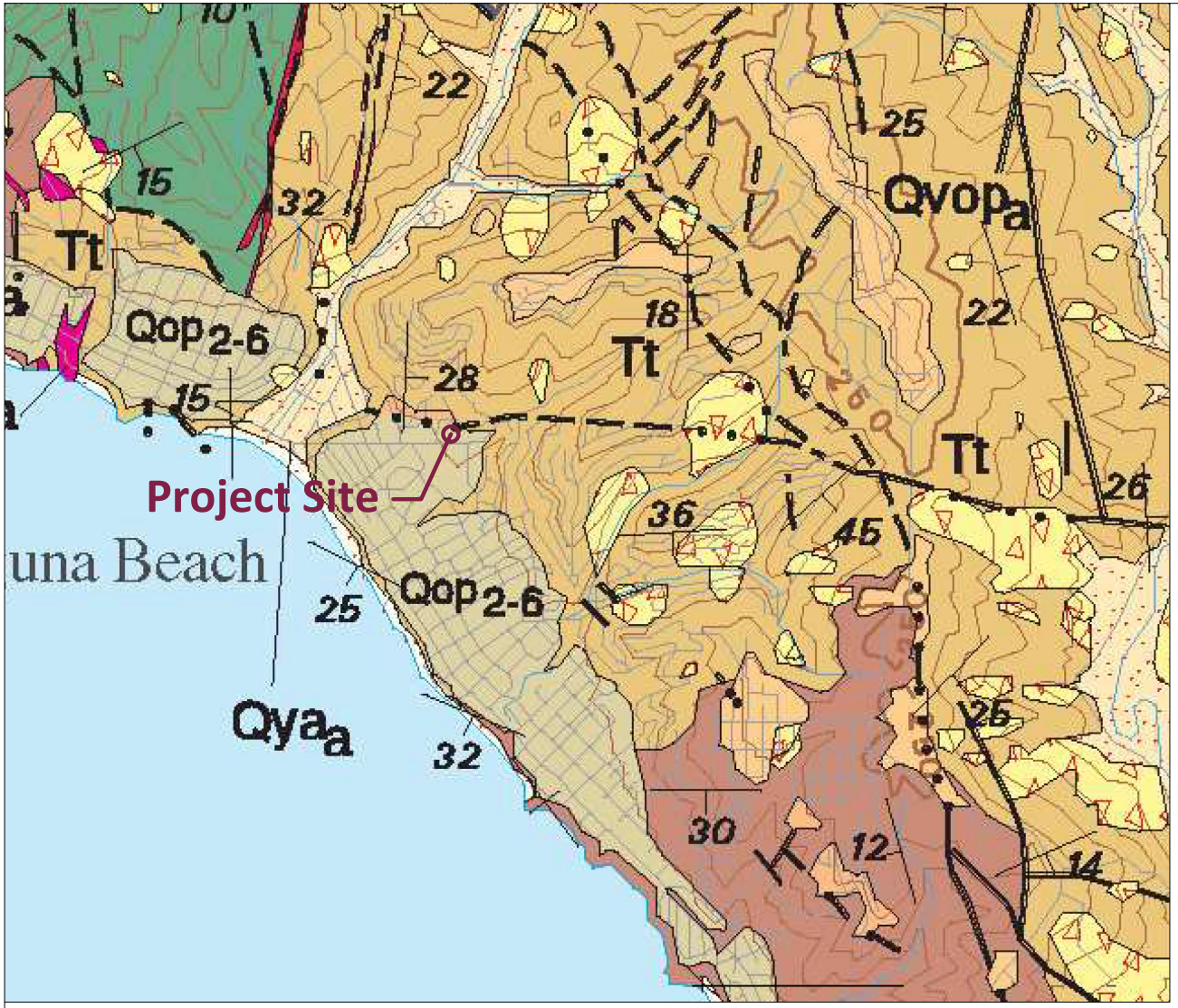
- - - Approximate Limits of Proposed Improvements

SUBSURFACE EXPLORATION MAP
Laguna Beach HS Aquatic Center GI
Laguna Beach, California

MTGL Project No.:	Date:	Author:
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Figure No.
3



EXPLANATION:

Qya_a — **Young axial-channel deposits -**Slightly to moderately consolidated silt, sand, and gravel deposits.

Qop_{2-6} — **Old paralic deposits, Units 2-6, undivided** - Paralic deposits consisting of poorly sorted, moderately permeable, reddish-brown, interfingered strandline, beach, estuarine and colluvial deposits composed of silt, sand and cobbles. These deposits rest on the 22-55m terraces.

Tt — **Topanga Formation (middle Miocene) -**Marine sandstone, siltstone, and shale.

Reference: Morton, D.M., and Miller, F.K. (2006), Geologic map of the San Bernardino and Santa Ana 30' x 60' Quadrangles, California, Scale: 1:100,000, Open-File Report OF-2006-1217, Sheet 1 of 4, Version 1.0.

0 ½ 1
SCALE (Mile)

Note: All locations are approximate.

N

REGIONAL GEOLOGY MAP
Laguna Beach HS Aquatic Center GI
Laguna Beach, California

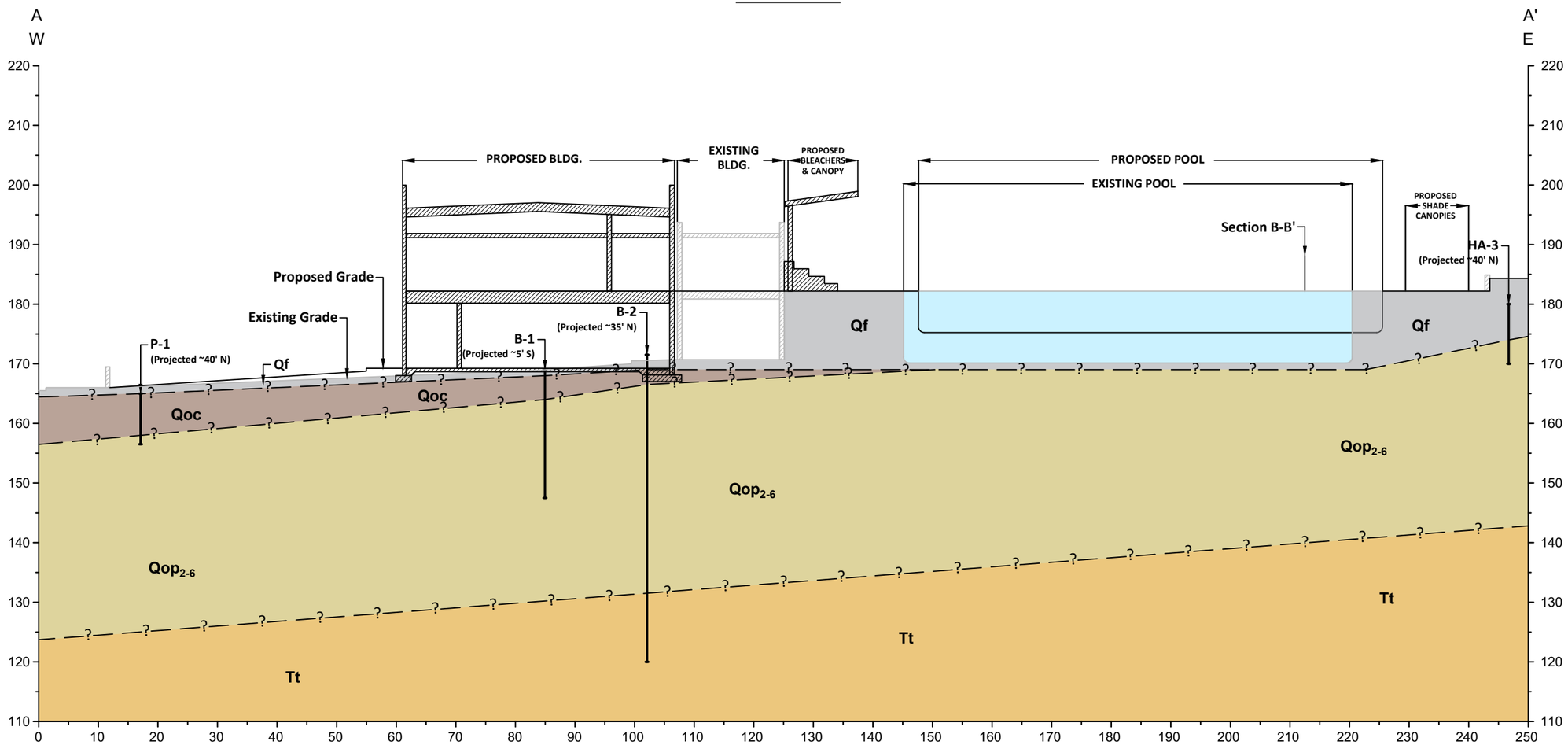
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Figure No.
4

Note: All locations and elevations are approximate.

SECTION A-A'



Reference:

Ruhnau Architects (2024), Laguna Beach Unified School District Pool Modernization, Project Update, Site Plan, Laguna Beach High School Pool, Page 11, dated August 12th.

JL Surveying (2022), Topographic Map of LBUSD District Office, Sheet 4 of 5, dated February 3rd.

0' 20' 40'

SCALE (feet)
1:1 Horizontal to Vertical

LEGEND

B-2 | P-1 | HA-3 — Location of Boring

—?— Geologic Contact, Queried Where Uncertain or Inferred

Qf — Fill

Qoc — Colluvium

Qop_{2-6} — Old paralic deposits Unit 2-6, undivided

Tt — Topanga Formation

GEOLOGIC CROSS SECTION
Laguna Beach HS Aquatic Center GI
Laguna Beach, California

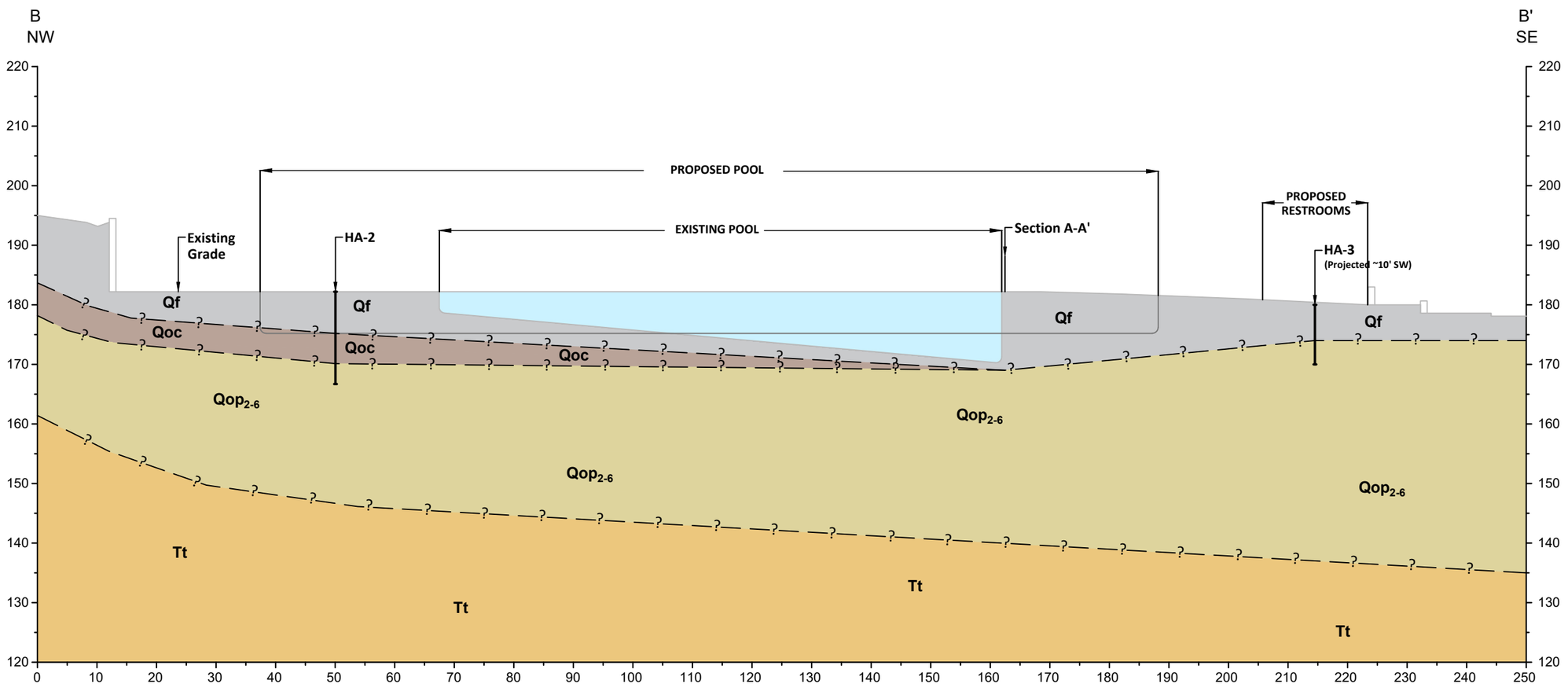
MTGL Project No.:	Date:	Author:
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Figure No.
5A

Note: All locations and elevations are approximate.

SECTION B-B'



Reference:

Ruhnau Architects (2024), Laguna Beach Unified School District Pool Modernization, Project Update, Site Plan, Laguna Beach High School Pool, Page 11, dated August 12th.

JL Surveying (2022), Topographic Map of LBUSD District Office, Sheet 4 of 5, dated February 3rd.

0' 20' 40'

SCALE (feet)
1:1 Horizontal to Vertical

LEGEND

HA-3 — Location of Boring

—?— Geologic Contact, Queried Where Uncertain or Inferred

Qf — Fill

Qoc — Colluvium

Qop_{2-6} — Old paralic deposits Unit 2-6, undivided

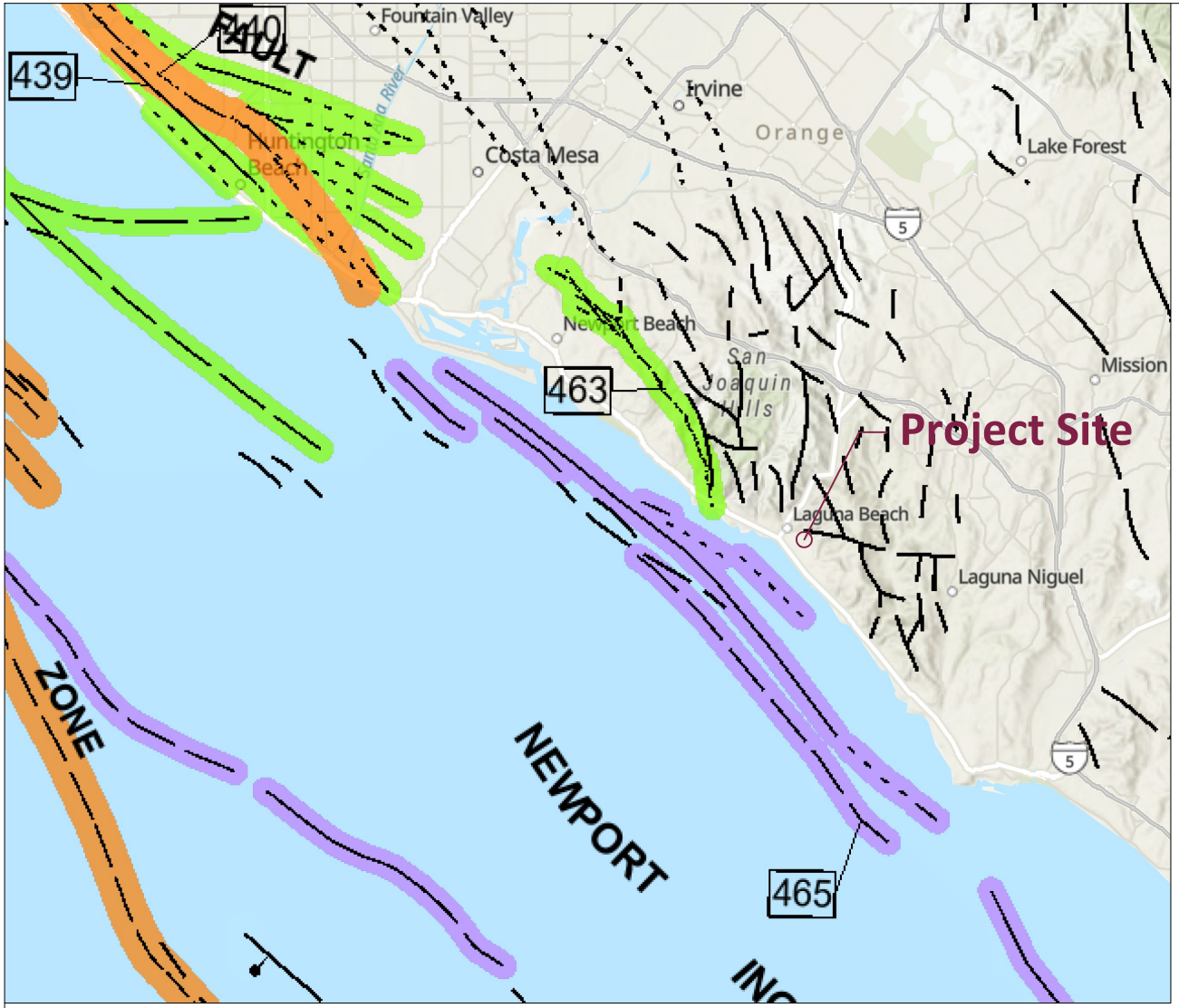
Tt — Topanga Formation

GEOLOGIC CROSS SECTION
Laguna Beach HS Aquatic Center GI
Laguna Beach, California

MTGL Project No.:	Date:	Author:
2520A02	September, 2024	TC

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Figure No.
5B



EXPLANATION:

Holocene fault displacement (during past 11,700 years) without historic record.

Late Quaternary fault displacement (during past 700,000 years).

Quaternary fault (age undifferentiated).

Pre-Quaternary fault (older that 1.6 million years) or fault without recognized Quaternary displacement.

ADDITIONAL FAULT SYMBOLS

Bar and ball on downthrown side (relative or apparent).

Arrows along fault indicate relative or apparent direction of lateral movement.

Arrow on fault indicates direction of dip.

JENNINGS ID NUMBER:

439	Newport-Inglewood, South Branch Fault
440	Newport-Inglewood, North Branch Fault
463	Pelican Hills Fault
465	Newport-Inglewood-Rose Canyon fault zone, Oceanside section

Reference:
California Geological Survey, 2010, Fault Activity Map, C.W. Jennings, W.A. Bryant, accessed on July 25, 2024, at https://gis.conservation.ca.gov/server/admin/CGS/FaultActivityMapCA/MapServer

Fault traces on land are indicated by solid lines where well located, by dashed lines where approximately located or inferred, and by dotted lines where concealed by younger rocks or by lakes or bays. Fault traces are queried where continuation or existence is uncertain. All offshore faults based on seismic reflection profile records are shown as solid lines where well defined, dashed where inferred, queried where uncertain.

0 4 8

SCALE (Miles)

Note: All locations are approximate.

N

REGIONAL FAULT MAP

Laguna Beach HS Aquatic Center GI
Laguna Beach, California

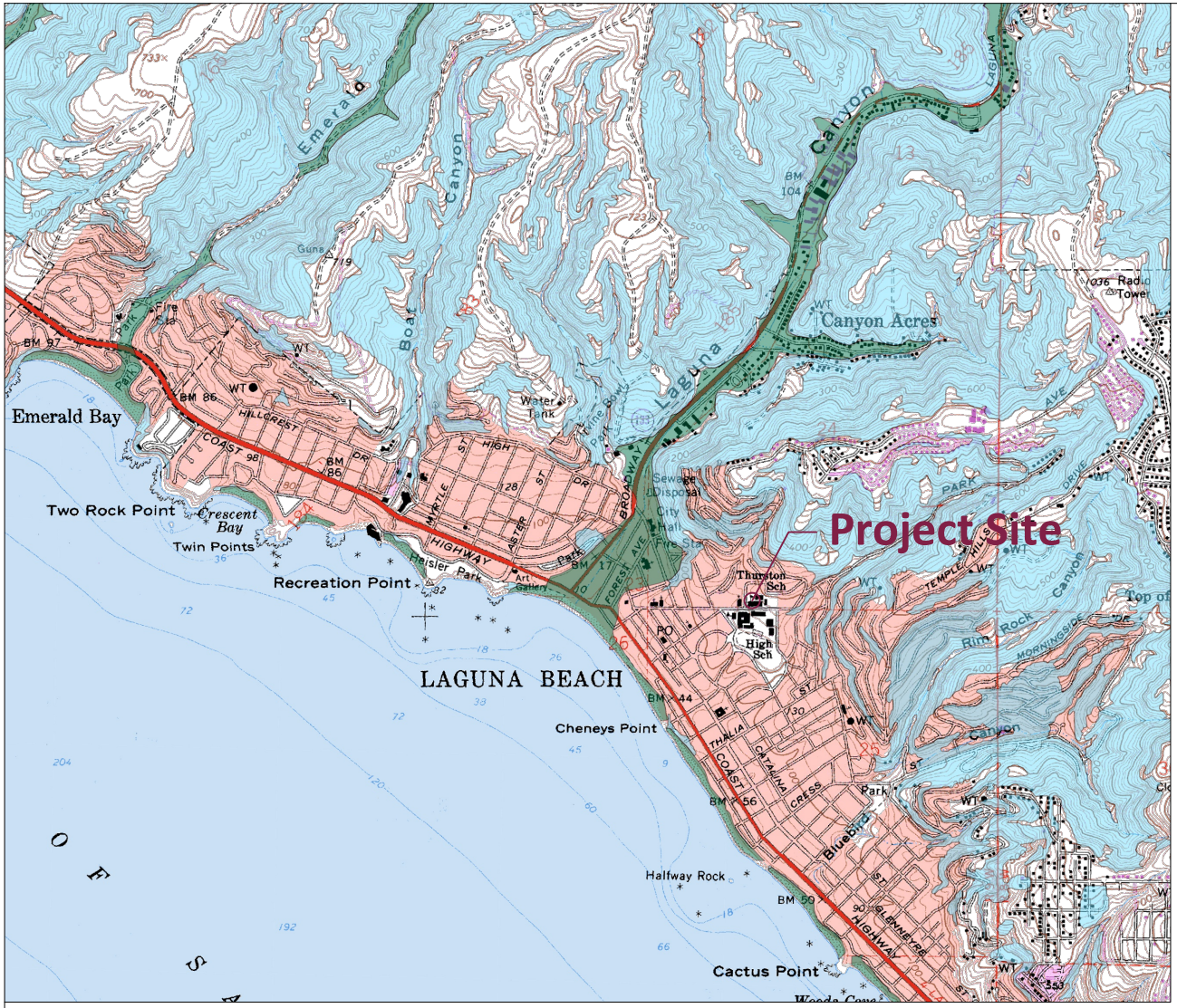
MTGL Project No.:	Date:	Author:
2520A02	September, 2024	TC

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Figure No.

6



EXPLANATION:

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

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

This Map Shows Seismic Hazard Zones
Alquist-Priolo Earthquake Fault Zones Have Not Been Prepared
For The Laguna Beach Quadrangle

This map shows the location of Seismic Hazard Zones, referred to here as Earthquake Zones of Required Investigation. The Geographic Information System (GIS) digital files of these regulatory zones released by the California Geological Survey (CGS) are the "Official Maps." GIS files are available at the GGS website

Reference:
California Geological Survey, Earthquake Zones of Required Investigation, Seismic Hazard Zones Laguna Beach Quadrangle, accessed from CGS Information Warehouse: Regulatory Maps on August 29th, 2024, at https://maps.conservation.ca.gov/cgs/informationwarehouse/regulatorymaps/

0 ½ 1
SCALE (Miles)

Note: All locations are approximate.

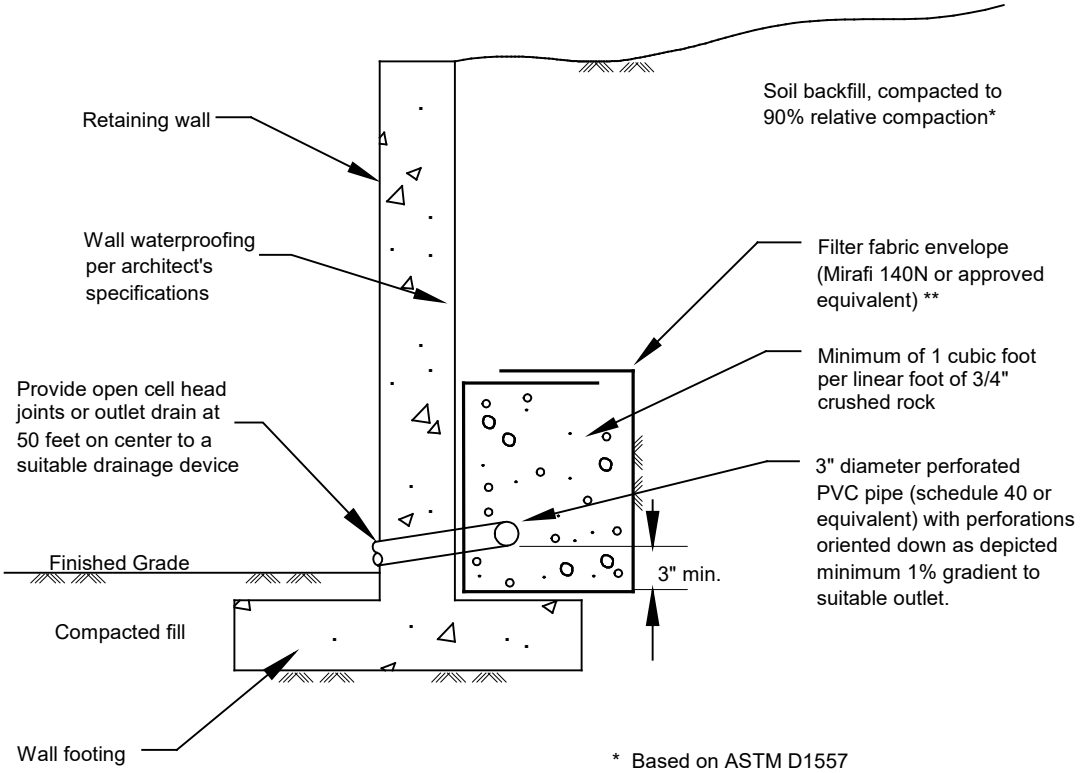
N

EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP
Laguna Beach HS Aquatic Center GI
Laguna Beach, California

MTGL Project No.:	Date:	Author:
2520A02	September, 2024	TC

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MTGL

Figure No.
7



* Based on ASTM D1557

** If class 2 permeable material (See gradation to left) is used in place of 3/4" - 1 1/2" gravel. Filter fabric may be deleted. Class 2 permeable material compacted to 90% relative compaction. *

SPECIFICATIONS FOR CLASS 2 PERMEABLE MATERIAL (CAL TRANS SPECIFICATIONS)

Sieve Size	% Passing
1"	100
3/4"	90-100
3/8"	40-100
No.4	25-40
No.8	18-33
No.30	5-15
No.50	0-7
No.200	0-3

RETAINING WALL DRAINAGE DETAIL

Figure 8

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

American Society of Civil Engineers, 2017, ASCE 7-16 Minimum Design Loads and Associated Criteria for Building and Other Structures.

Burmister, DM (1948). "Importance & Practical Use of Relative Density in Soil Mechanics," Proc., ASTM, 48, 1249-1268.

California Building Standards Commission, 2022, California Building Code, Title 24, California Code of Regulations.

California Geological Survey, 1997, Seismic Hazard Zone Report for the Laguna Beach 7.5-Minute Quadrangle, Orange County, California, SHZR No. 013.

California Geological Survey, 1998, Earthquake Zones of Required Investigation - Seismic Hazard Zones Map, Laguna Beach 7.5-Minute Quadrangle, Orange County, California, Scale 1:24,000.

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APPENDIX B

FIELD EXPLORATION PROGRAM

APPENDIX B

FIELD EXPLORATION PROGRAM

The subsurface conditions for this Geotechnical Investigation were explored by excavating six exploratory borings with an 8 inch diameter hollow stem auger and hand tools. Driven samples were obtained by SPT or a Modified California Split Spoon Sampler. The approximate locations of the borings are shown on the Subsurface Exploration Map (Figure 3). The field exploration was performed under the supervision of our engineer and geologist who maintained a continuous log of the subsurface soils encountered and obtained samples for laboratory testing.

Subsurface conditions are summarized on the accompanying Logs of Borings. The logs contain descriptive information and interpretation of subsurface conditions based on the obtained samples. The stratum indicated on these logs represents the approximate boundary between earth units, however, transitions may be gradual. The logs show subsurface conditions at the dates and locations indicated and may not be representative of subsurface conditions at other locations and times.

Identification of the soils encountered during the subsurface exploration was made using the field identification procedure of the Unified Soils Classification System (ASTM D2488). A legend indicating the symbols and definitions used in this classification system and a legend defining the terms used in describing the relative compaction, consistency or firmness of the soil are attached in this appendix. Bag samples of the major earth units were obtained for laboratory inspection and testing, and the in-place density of the various strata encountered in the exploration was determined.

The exploratory borings were backfilled in general accordance with DEH requirements and patched where appropriate.

UNIFIED SOIL CLASSIFICATION SYSTEM					
No. 200 U.S. Standard Sieve is the smallest particle visible	Coarse-grained soils >1/2 of materials is larger than #200 sieve	GRAVELS are more than half of coarse fraction larger than #4 sieve	Clean Gravels (less than 5% fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
			Gravels with fines	GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		SANDS are more than half of coarse fraction larger than #4 sieve	Clean Sands (less than 5% fines)	GM	Silty Gravels, poorly graded gravel-sand-silt mixtures
			Sands with fines	GC	Clayey Gravels, poorly graded gravel-sand-clay mixtures
	Fine-grained Soils >1/2 of materials is smaller than #200 sieve	SILTS AND CLAYS Liquid Limit Less than 50		SW	Well-graded sands, gravelly sands, little or no fines
				SP	Poorly graded sands, gravelly sands, little or no fines
				SM	Silty Sands, poorly graded sands-gravel-clay mixtures
				SC	Clayey Sands, poorly graded sand-gravel-silt mixtures
				ML	Inorganic clays of low to med plasticity, gravelly, sandy, silty, or lean clays
		SILTS AND CLAYS Liquid Limit Greater than 50		CL	Inorganic clays of low to med plasticity, gravelly, sandy, silty, or lean clays
				OL	Organic silts and clays of low plasticity
				MH	Inorganic silts, micaceous or diatomaceous fine sands or silts
				CH	Inorganic clays of high plasticity, fat clays
				OH	Organic silts and clays of medium to high plasticity
Highly Organic Soils				PT	Peat, humus swamp soils with high organic content

GRAIN SIZE					SIZE PROPORTION
Description		Sieve Size	Grain Size	Approximate Size	Trace – Less than 5%
Boulders		>12”	>12”	Larger than basketball-sized	Few – 5% to 10%
Cobbles		3”- 12”	3”- 12”	Fist-sized to basketball-sized	Little – 15% to 20%
Gravel	Coarse	¾”- 3”	¾”- 3”	Thumb-sized	Some – 30% to 45%
	Fine	#4 - ¾”	0.19” - 0.75”	Peat-sized to thumb-sized	Mostly – 50% to 100%
Sand	Coarse	#10 - #4	0.079” - 0.19”	Rock salt-sized to pea-sized	**MOISTURE CONTENT**
	Medium	#40 - #10	0.017” - 0.079”	Sugar-sized to rock salt-sized	Dry – Absence of moisture
	Fine	#200 - #40	0.0029” - 0.017”	Flour-sized to sugar-sized	Moist – Damp but not visible
Fines		Passing #200	<0.0029”	Flour-sized or smaller	Wet – Visible free water

Project: **Laguna Beach HS Aquatic Center GI**

Project Location: **670 Park Ave, Laguna Beach, CA 92651**

Project Number: **2520A02**

Log of Boring B-1

Sheet 1 of 1

Date(s) Drilled **8/15/2024**	Logged By **THC**	Checked By **GSW**
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch HSA**	Total Depth of Borehole **21½ Feet BGS**
Drill Rig Type **CME-75**	Drilling Contractor **Baja Exploration**	Approximate Surface Elevation **168½ Feet MSL**
Groundwater Level and Date Measured **Not Encountered**	Sampling Method(s) **Bulk, Modified California, SPT**	Hammer Data **140lb/30-inch drop (ETR: 71.9%)**
Borehole Backfill **Auger Cuttings**	Location **See Boring Location Map**	

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance	N60	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Lab Testing	REMARKS
	0				Asphalt Base		Approximately 3½ inches of Asphalt Concrete over 8 inches of Aggregate Base (Possibly Cement Treated Base).				
					SC		**Fill (Qf):** CLAYEY SAND (SC), loose to medium dense, brown to light brown, moist, fine to medium grained, trace debris.				
					SC		**Colluvium (Qoc):** CLAYEY SAND (SC), loose to medium dense, light brown, moist, fine to medium grained, trace manganese staining, trace white precipitates.			PD MAX DS	
	5		4 6 7	16	SC		**Old paralic deposits (Qop):** CLAYEY SAND (SC), medium dense, light brown to brown, moist, fine to medium grained, trace manganese staining.				
			11 19 23	33			Dense, light brown to olive to brown.				
	10		8 11 17	34			Trace white precipitates.				
			8 18 32	39	CL		SANDY CLAY (CL), hard, olive to light gray, moist, fine to medium grained sands, white precipitate nodules.			WA	
	15		15 15 11	31	SM		SILTY SAND (SM), dense, light brown, moist, fine to medium grained, trace clay.				
			19 18 31	38	SC		CLAYEY SAND (SC), dense, light brown to olive, moist, fine to medium grained, trace manganese staining.				
	20		9 12 18	36			Thin interbeds of SILTY SAND (SM), and SANDY CLAY (CL).				
							Boring Terminated at 21½ Feet BGS.				

Project: **Laguna Beach HS Aquatic Center GI** Project Location: **670 Park Ave, Laguna Beach, CA 92651** Project Number: **2520A02**	Log of Boring B-2 **Sheet 1 of 3**

Date(s) Drilled **8/15/2024**	Logged By **THC**	Checked By **GSW**
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch HSA**	Total Depth of Borehole **51½ Feet BGS**
Drill Rig Type **CME-75**	Drilling Contractor **Baja Exploration**	Approximate Surface Elevation **171½ Feet MSL**
Groundwater Level and Date Measured **Not Encountered**	Sampling Method(s) **Bulk, Modified California, SPT**	Hammer Data **140lb/30-inch drop (ETR: 71.9%)**
Borehole Backfill **Auger Cuttings**	Location **See Boring Location Map**	

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance	N60	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Lab Testing	REMARKS
	0				Asphalt Base		Approximately 4 inches of Asphalt Concrete over 8½ inches of Aggregate Base (Possibly Cement Treated Base).				
					CL		**Fill (Qf):** SANDY CLAY (CL), medium stiff, light brown to grayish brown, moist, fine to medium grained sands, trace debris and gravel.			RV EI COR	
					SC		**Colluvium (Qoc):** CLAYEY SAND (SC), medium dense, light brown, moist, fine to medium grained, trace manganese staining, trace white precipitates.				
							Brown, decreased fines content.				
	5				SC		**Old paralic deposits (Qop):** CLAYEY SAND (SC), dense, brown, moist, fine to medium grained, trace manganese staining, very thin interbeds of SILTY SAND (SM).	11.0	120.4	DS	
			12 19 25	34							
							Medium dense.				
			6 7 7	17							
	10						Brown to light brown.	10.3	114.6		
			13 16 20	28							
							Light brown to olive brown, fine grained, trace calcium carbonate nodules.				
			8 9 13	26							
	15						Very dense, increased fines content.				
			15 35 45	>50						WA	
					CL-ML		SANDY SILTY CLAY (CL-ML), very stiff, olive, moist, fine grained sands.				
			6 8 13	25						AL	
	20						Hard, thin interbeds of light brown, fine grained CLAYEY SAND (SC) and SILTY SAND (SM), trace manganese staining, laminated, slightly micaceous.				
			12 18 24	33							

Project: **Laguna Beach HS Aquatic Center GI**
Project Location: **670 Park Ave, Laguna Beach, CA 92651**
Project Number: **2520A02**

Log of Boring B-2
Sheet 2 of 3

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance	N60	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Lab Testing	REMARKS
							Old paralic deposits (Qop): SANDY SILTY CLAY (CL-ML), hard, olive, moist, fine grained sands, thin interbeds of light brown, fine grained CLAYEY SAND (SC) and SILTY SAND (SM), trace manganese staining, laminated, slightly micaceous. *(Continued)*				
	25		5 9 12	25	SC		CLAYEY SAND (SC), medium dense, olive, moist, fine to medium grained, thin interbeds of SANDY CLAY (CL), trace rootlets, manganese staining, calcium carbonate nodules.				
	30		15 30 33	49			Dense, thin interbeds of SILTY CLAY (CL-ML), fine grained Poorly graded SAND with SILT (SP-SM), few rounded gravel, oxidized.				
	35		7 8 12	24	CL		CLAY (CL), very stiff, olive to grayish brown, moist, some silt, trace coarse grained sand, calcium carbonate nodules.				
	40		40 50/4"	>50			**Topanga Formation (Tt):** SILTY SANDSTONE, moderately cemented, light gray to olive, moist, fine to medium grained, trace clay, trace manganese staining. Breaks down to SILTY SAND (SM).				

Project: **Laguna Beach HS Aquatic Center GI**
Project Location: **670 Park Ave, Laguna Beach, CA 92651**
Project Number: **2520A02**

Log of Boring B-2
Sheet 3 of 3

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance	N60	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Lab Testing	REMARKS
	45		37 50/6"	>50			**Topanga Formation (Tt):** SILTY SANDSTONE, moderately cemented, light gray to olive, moist, fine to medium grained, trace clay, trace manganese staining. Breaks down to SILTY SAND (SM). *(Continued).* Fine grained, laminated, oxidized.				
	50		16 24 25	>50			Slight increase in fines content.				
							Boring Terminated at 51½ Feet BGS. No Groundwater Encountered. Boring Backfilled 8/15/2024 with Cement Bentonite Grout.				
	55										
	60										
	65										

Project: **Laguna Beach HS Aquatic Center GI** Project Location: **670 Park Ave, Laguna Beach, CA 92651** Project Number: **2520A02**	Log of Boring P-1 **Sheet 1 of 1**

Date(s) Drilled **8/15/2024**	Logged By **THC**	Checked By **GSW**
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **8-inch HSA**	Total Depth of Borehole **8½ Feet BGS**
Drill Rig Type **CME-75**	Drilling Contractor **Baja Exploration**	Approximate Surface Elevation **165 Feet MSL**
Groundwater Level and Date Measured **Not Encountered**	Sampling Method(s) **Bulk, Modified California, SPT**	Hammer Data **140lb/30-inch drop (ETR: 71.9%)**
Borehole Backfill **Auger Cuttings**	Location **See Boring Location Map**	

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance	N60	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Lab Testing	REMARKS
165	0				Asphalt Base		Approximately 3 inches of Asphalt Concrete over 9 inches of Aggregate Base (Possibly Cement Treated Base).				
					SC		**Colluvium (Qoc):** CLAYEY SAND (SC), loose to medium dense, brown, moist, fine to medium grained, trace manganese staining.				
							Medium dense, increasingly moist.				
160	5		4 5 6	13			Brown to light brown, increasingly moist.	17.0			
			8 13 15	22	SC		**Old paralic deposits (Qop):** CLAYEY SAND (SC), medium dense, brown, moist, fine to medium grained, trace manganese staining.			PD	
							Boring Terminated at 8½ Feet BGS. No Groundwater Encountered. Boring Converted to Borehole Percolation Test 8/15/204. Boring Backfilled 8/16/2024.				
155	10										
150	15										
145	20										

Project: **Laguna Beach HS Aquatic Center GI**

Project Location: **670 Park Ave, Laguna Beach, CA 92651**

Project Number: **2520A02**

Log of Boring HA-1

Sheet 1 of 1

Date(s) Drilled **8/15/2024**	Logged By **JR**	Checked By **GSW**
Drilling Method **Hand Auger**	Drill Bit Size/Type **4-inch**	Total Depth of Borehole **15 Feet BGS**
Drill Rig Type **Hand Auger**	Drilling Contractor **Strongarm Environmental Field Services, Inc.**	Approximate Surface Elevation **182 Feet MSL**
Groundwater Level and Date Measured **Not Encountered**	Sampling Method(s) **Bulk**	Hammer Data **N/A**
Borehole Backfill **Auger Cuttings**	Location **See Boring Location Map**	

Elevation (feet)	Depth (feet)	Material Type	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Lab Testing	REMARKS
182	0	Concrete	Approximately 8 inches of Portland Cement Concrete (PCC).				
		SC / SM	**Fill (Qf):** CLAYEY SAND (SC), medium dense, brown to grayish brown, moist, fine to coarse grained, trace debris.				
			SILTY SAND (SM), medium dense, light brown to yellowish brown, moist, fine to medium grained, trace clay.			PD	
177	5						
		SM	**Colluvium (Qoc):** SILTY SAND (SM), dense, yellowish brown, moist, fine to coarse grained, trace gravel, micaceous.				
			Medium dense, brown, trace gravel, micaceous.				
			Brown to light grayish brown.				
172	10						
		SC	**Old paralic deposits (Qop):** CLAYEY SAND (SC), medium dense, light grayish brown to olive brown, moist, fine to medium grained, trace silt, manganese staining, micaceous.				
167	15		Boring Terminated at Approximately 15 Feet BGS. No Groundwater Encountered. Boring Backfilled on 8/15/2024.				
162	20						

Project: **Laguna Beach HS Aquatic Center GI**

Project Location: **670 Park Ave, Laguna Beach, CA 92651**

Project Number: **2520A02**

Log of Boring HA-2

Sheet 1 of 1

Date(s) Drilled **8/15/2024**	Logged By **JR**	Checked By **GSW**
Drilling Method **Hand Auger**	Drill Bit Size/Type **4-inch**	Total Depth of Borehole **15½ Feet BGS**
Drill Rig Type **Hand Auger**	Drilling Contractor **Strongarm Environmental Field Services, Inc.**	Approximate Surface Elevation **182 Feet MSL**
Groundwater Level and Date Measured **Not Encountered**	Sampling Method(s) **Bulk**	Hammer Data **N/A**
Borehole Backfill **Auger Cuttings**	Location **See Boring Location Map**	

Elevation (feet)	Depth (feet)	Material Type	MATERIAL DESCRIPTION
182	0	Concrete	Approximately 8 inches of Portland Cement Concrete (PCC).
		SC	**Fill (Qf):** CLAYEY SAND (SC), medium dense, brown to grayish brown, moist, fine to coarse grained, trace gravel.
		SM	SILTY SAND (SM), medium dense, brown to grayish brown, moist, fine to coarse grained, micaceous, trace clay.
			Yellowish brown to light brown.
177	5	SC	CLAYEY SAND (SC), medium dense, light brown, moist, fine to medium grained, slightly micaceous.
		SM	**Colluvium (Qoc):** SILTY SAND (SM), medium dense, brown, moist, fine to medium grained, micaceous, trace gravel.
172	10		
		SM	**Old paralic deposits (Qop):** SILTY SAND (SM), medium dense, brown, moist, fine to medium grained.
167	15		Boring Terminated at 15½ Feet BGS. No Groundwater Encountered. Boring Backfilled on 8/15/2024.
162	20		

Sample Type | Sampling Resistance | N60 | Graphic Log | Water Content, % | Dry Unit Weight, pcf | Lab Testing | REMARKS

Project: **Laguna Beach HS Aquatic Center GI**

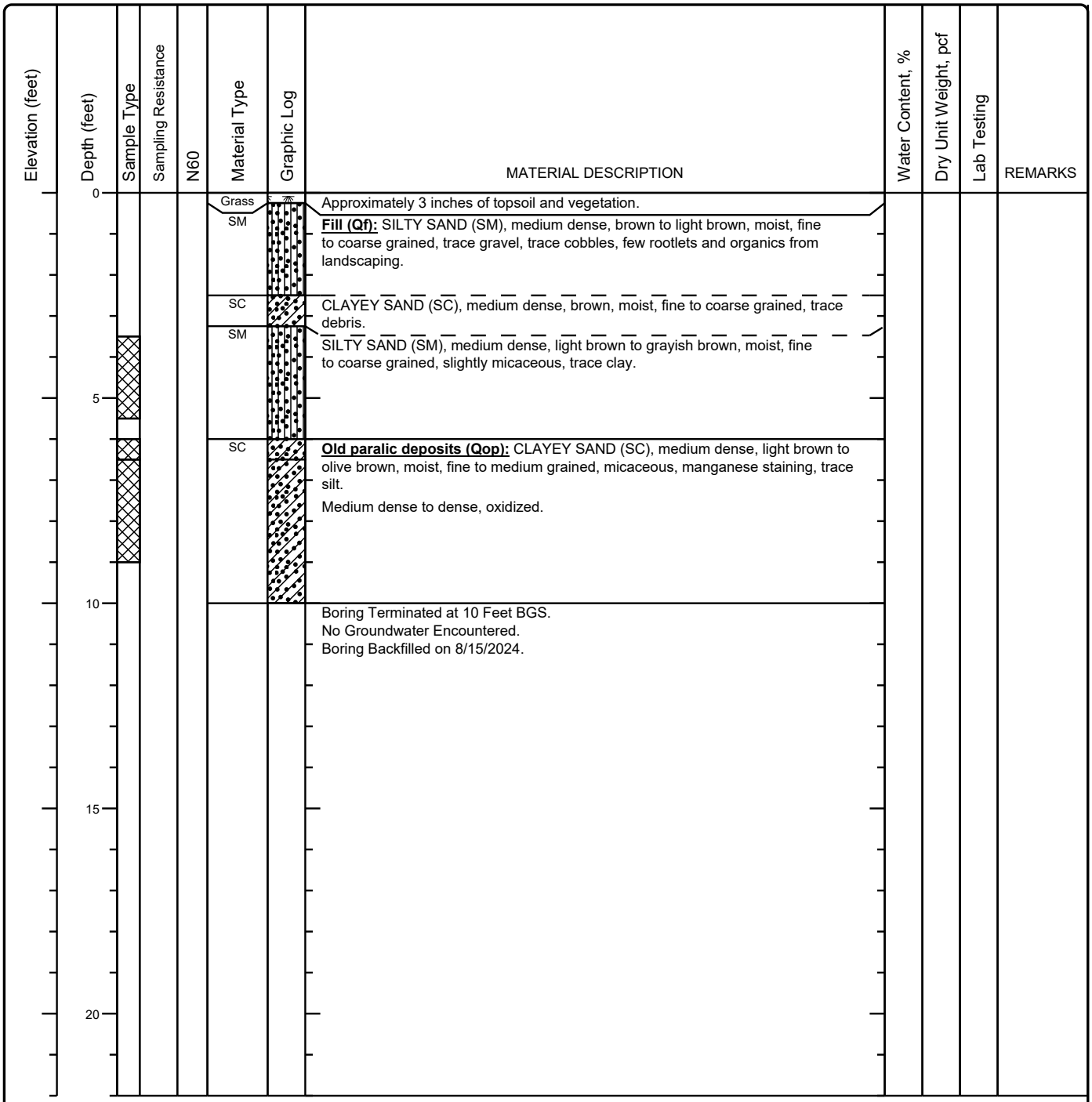
Project Location: **670 Park Ave, Laguna Beach, CA 92651**

Project Number: **2520A02**

Log of Boring HA-3

Sheet 1 of 1

Date(s) Drilled **8/15/2024**	Logged By **JR**	Checked By **GSW**
Drilling Method **Hand Auger**	Drill Bit Size/Type **4-inch**	Total Depth of Borehole **10 Feet BGS**
Drill Rig Type **Hand Auger**	Drilling Contractor **Strongarm Environmental Field Services, Inc.**	Approximate Surface Elevation **181½ Feet MSL**
Groundwater Level and Date Measured **Not Encountered**	Sampling Method(s) **Bulk**	Hammer Data **N/A**
Borehole Backfill **Auger Cuttings**	Location **See Boring Location Map**	



Project: **Laguna Beach HS Aquatic Center GI**
Project Location: **670 Park Ave, Laguna Beach, CA 92651**
Project Number: **2520A02**

Key to Log of Boring
Sheet 1 of 1

Elevation (feet)	Depth (feet)	Sample Type	Sampling Resistance	N60	Material Type	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Unit Weight, pcf	Lab Testing	REMARKS
1	2	3	4	5	6	7	8	9	10	11	12

COLUMN DESCRIPTIONS

1 Elevation (feet): Elevation (MSL, feet).
2 Depth (feet): Depth in feet below the ground surface.
3 Sample Type: Type of soil sample collected at the depth interval shown.
4 Sampling Resistance: Number of blows to advance driven sampler one foot (or distance shown) beyond seating interval using the hammer identified on the boring log.
5 N60: Energy corrected SPT blow count per foot.
6 Material Type: Type of material encountered.
7 Graphic Log: Graphic depiction of the subsurface material encountered.
8 MATERIAL DESCRIPTION: Description of material encountered. May include consistency, moisture, color, and other descriptive text.
9 Water Content, %: Water content of the soil sample, expressed as percentage of dry weight of sample.
10 Dry Unit Weight, pcf: Dry weight per unit volume of soil sample measured in laboratory, in pounds per cubic foot.
11 Lab Testing : Laboratory Testing
12 REMARKS : Comments and observations regarding drilling or sampling made by driller or field personnel.

FIELD AND LABORATORY TEST ABBREVIATIONS

COR: Corrosive Series Tests (CTM-643,417,422)
MAX: Maximum Dry Density and Optimum Moisture (ASTM D1557)
DS: Direct Shear Test (ASTM, D3080)
EI: Expansion Index, (ASTM, D4829)
AL: Atterberg Limits (ASTM, D4318)
PD: Particle-Size Distribution using Sieve Analysis (ASTM D6913)
WA: Wash sieve (percent passing No. 200 Sieve)

MATERIAL GRAPHIC SYMBOLS

Clayey SAND (SC)
Silty SAND (SM)
Lean CLAY, CLAY w/SAND, SANDY CLAY (CL)
SILTY CLAY (CL-ML)
Silty SANDSTONE, degrades to SILTY SAND (SM).

SURFACE GRAPHIC SYMBOLS

Portland Cement Concrete (PCC)
Asphaltic Concrete (AC)
Aggregate Base (AB)

TYPICAL SAMPLER GRAPHIC SYMBOLS

2-inch-OD unlined split spoon (SPT)
2.5-inch-OD Modified California
Bulk Sample

OTHER GRAPHIC SYMBOLS

Water level (at time of drilling, ATD)
Water level (after waiting, AW)
Minor change in material properties within a stratum
Inferred/gradational contact between strata
Queried contact between strata

GENERAL NOTES

1: Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive, and actual lithologic changes may be gradual. Field descriptions may have been modified to reflect results of lab tests.
2: Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times.

Figure B-1

APPENDIX C

LABORATORY TESTING PROCEDURES & RESULTS

APPENDIX C

LABORATORY TESTING PROCEDURES

1. Particle Size Analysis

   Particle size analysis on representative soil samples were determined using the standard test method of the ASTM D6913.

2. Atterberg Limits

   The liquid limit, plastic limit, and the plasticity index of the major soil types encountered were determined using the standard test methods of the ASTM D4318.

3. Percent Finer

   Determining the amount of material finer than No. 200 sieve in soils by washing was performed on soil samples using the standard test methods of the ASTM D1140.

4. Expansion Index

   Expansion index of materials encountered was determined using the standard test methods of the ASTM D4829.

5. Maximum Density

   Maximum density tests were performed on a representative bag sample of the near surface soils in accordance with ASTM D1557.

6. Direct Shear

   Direct Shear Tests were performed on in-place samples of site soils in accordance with ASTM D3080.

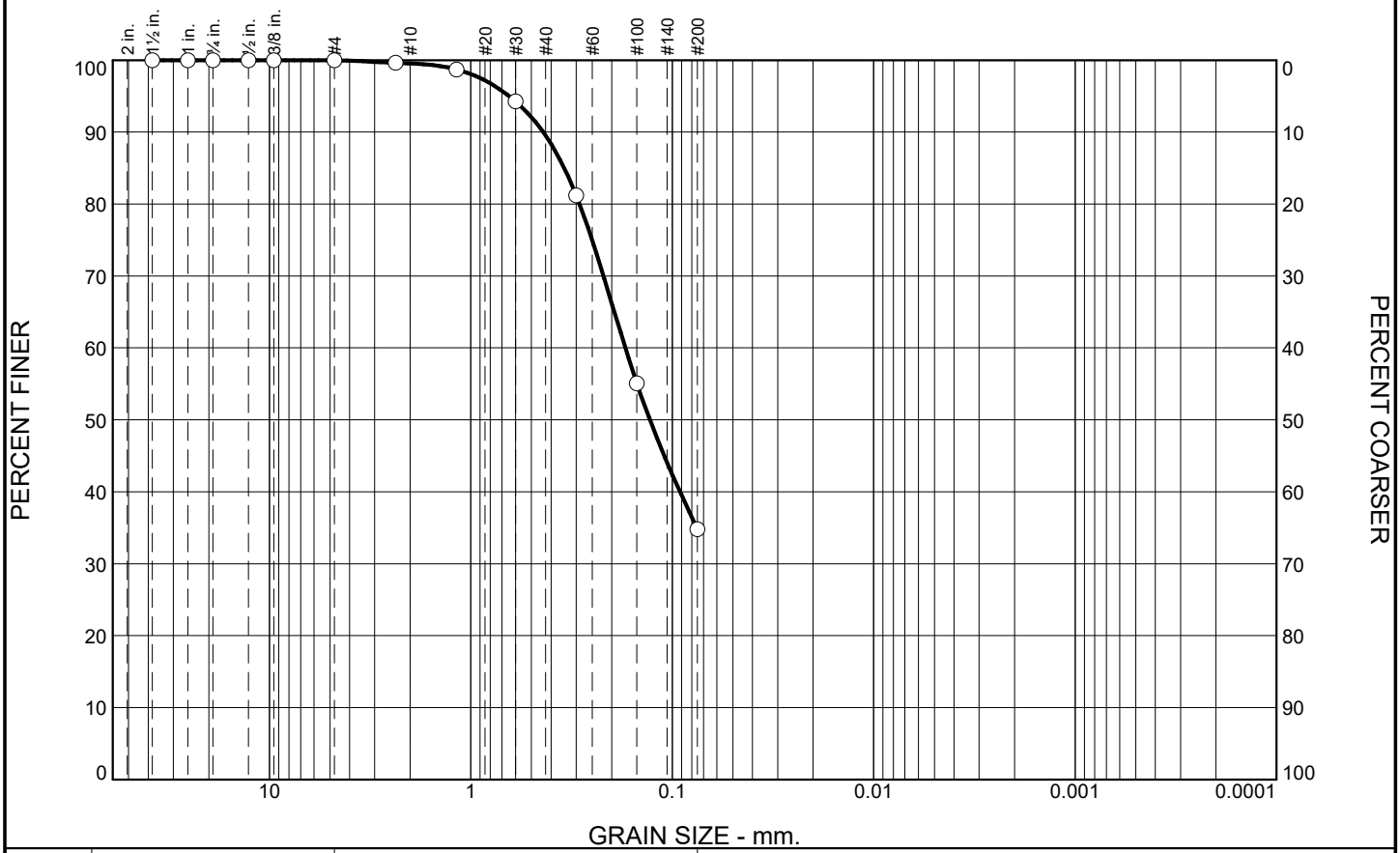
7. Resistance Value Testing

   R-Value testing was completed in substantial compliance with Caltrans Test Method 301. Graphical plots of our tests are included in this appendix.

8. Corrosion

   Chemical testing was performed on representative samples to determine the corrosion potential of the onsite soils. Testing consisted of pH, chlorides (CTM 422), soluble sulfates (CTM 417), and resistivity (CTM 643).

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.5	10.0	54.7	34.8	

SIEVE SIZE	PERCENT FINER
1-1/2"	100.0
1"	100.0
3/4"	100.0
1/2"	100.0
3/8"	100.0
#4	100.0
#8	99.6
#16	98.7
#30	94.3
#50	81.2
#100	55.1
#200	34.8

Material Description

Light Brown, CLAYEY SAND (SC)

Classification

USCS=SC

Location: B-1 at 2 to 5 Feet
Sample Number: 24-8319

Date: 08/15/24

MTGL, Inc.

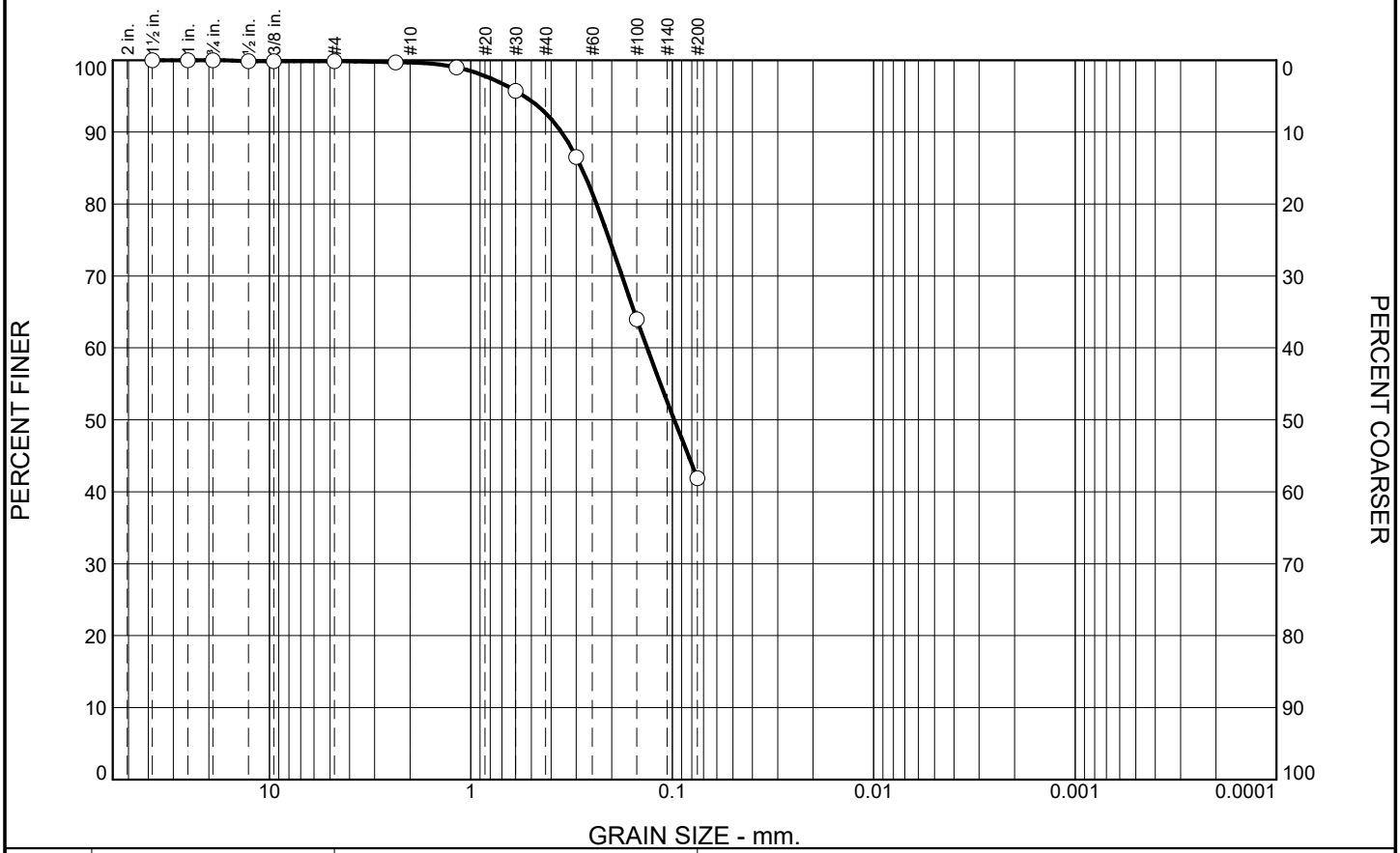
Anaheim, CA

Client: Laguna Beach Unified School District
Project: Laguna Beach HS Pool Modernization GI

Project No: 2520A02 **Figure** C-1

Tested By: J. ALVAREZ

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.1	0.2	7.1	50.7	41.9	

SIEVE SIZE	PERCENT FINER
1-1/2"	100.0
1"	100.0
3/4"	100.0
1/2"	99.9
3/8"	99.9
#4	99.9
#8	99.7
#16	99.0
#30	95.7
#50	86.5
#100	64.0
#200	41.9

Material Description

Light brown to yellowish brown, SILTY SAND (SM)

Classification

USCS=SM

Location: HA-1 at 1 to 3½ Feet
Sample Number: 24-8319

Date: 08/15/24

MTGL, Inc.

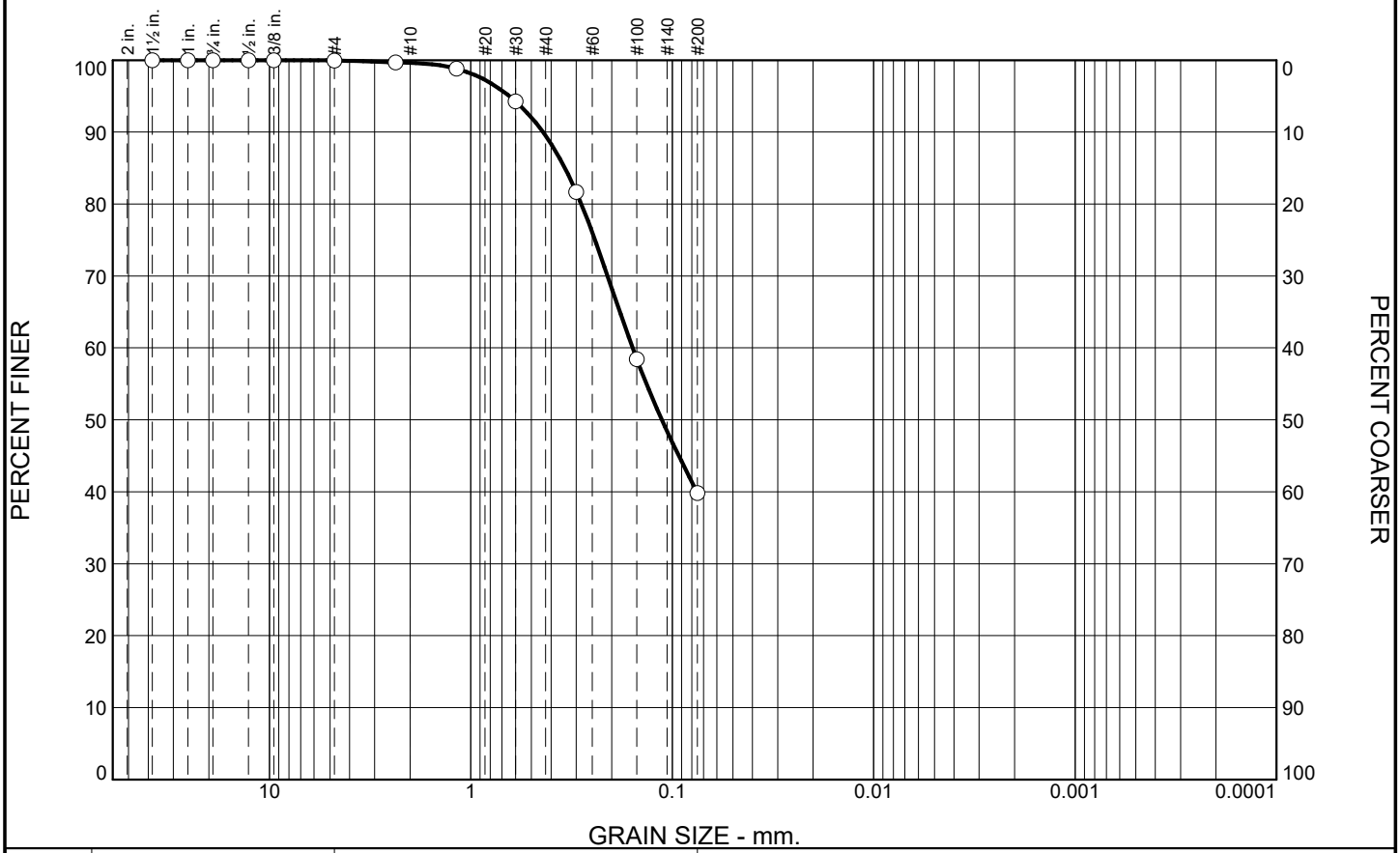
Anaheim, CA

Client: Laguna Beach Unified School District
Project: Laguna Beach HS Pool Modernization GI

Project No: 2520A02 **Figure** C-2

Tested By: J. ALVAREZ

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.4	10.1	49.7	39.8	

SIEVE SIZE	PERCENT FINER
1-1/2"	100.0
1"	100.0
3/4"	100.0
1/2"	100.0
3/8"	100.0
#4	100.0
#8	99.7
#16	98.8
#30	94.3
#50	81.7
#100	58.4
#200	39.8

Material Description

Brown, CLAYEY SAND (SC)

Classification

USCS=SC

Location: P-1 at 8 to 8½ Feet
Sample Number: 24-8319

Date: 08/15/24

MTGL, Inc.

Anaheim, CA

Client: Laguna Beach Unified School District
Project: Laguna Beach HS Pool Modernization GI

Project No: 2520A02 **Figure** C-3

Tested By: J. ALVAREZ

R-VALUE

CALIFORNIA TEST 301

SAMPLE	DESCRIPTION	R-VALUE
B-2 at 1 to 2½ Feet	SANDY CLAY (CL)	10

MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT

ASTM D1557

SAMPLE	DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)
B-1 at 2 to 5 Feet	CLAYEY SAND (SC)	124.9	9.7

EXPANSION INDEX

ASTM D4829

SAMPLE	DESCRIPTION	EXPANSION INDEX
B-2 at 1 to 2½ Feet	SANDY CLAY (CL)	61

Classification of Expansive Soil [1]

Expansion Index	Expansion Potential
1-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very High

1. ASTM - D4829

RESISTIVITY, pH, SOLUBLE CHLORIDE and SOLUBLE SULFATE

Resitivity (CT.643), Soluble Sulfates (CT.417) Soluble Chlorides (CT.442)

SAMPLE	pH	RESISTIVITY (Ω-cm)	SULFATE (ppm)	CHLORIDE (ppm)
B-2 at 1 to 2½ Feet	7.7	965	279	69

ATTERBERG LIMITS

ASTM D4318

SAMPLE	FINES CLASSIFICATION	Liquid Limit	Plastic Limit	Plastic Index
B-2 at 18 to 19 Feet	SANDY SILTY CLAY (CL-ML)	25	18	7

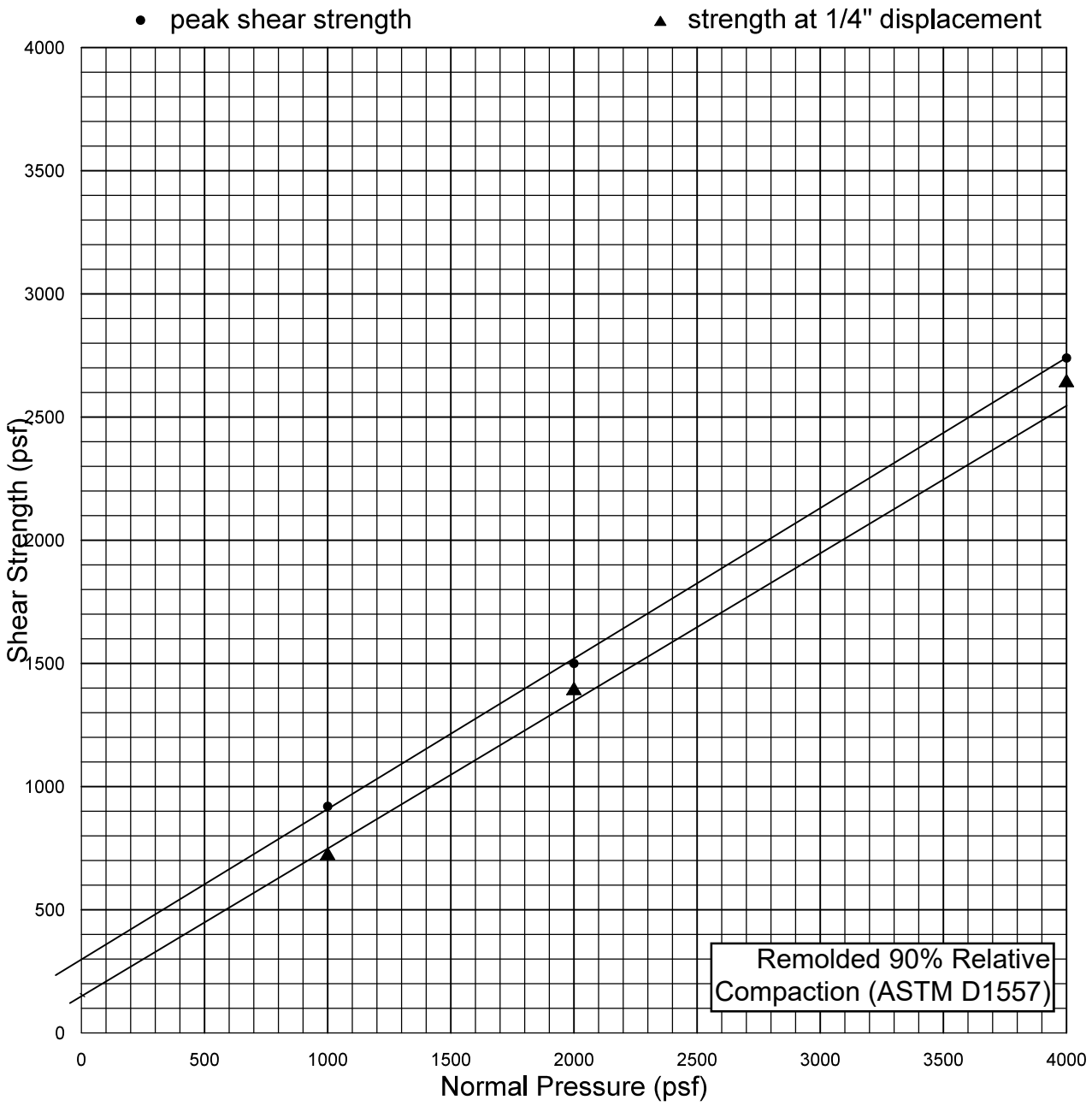
* NP indicates non plastic

PERCENT FINER

ASTM D1140

SAMPLE	DESCRIPTION	% Finer
B-1 at 13½ to 14 Feet	SANDY CLAY (CL)	53.3
B-2 at 16 to 16½ Feet	CLAYEY SAND (SC)	46.6

DIRECT SHEAR TEST - ASTM D-3080



Strain Rate: 0.0084 in. / min.

Sample Location	Material Description	Dry Density (pcf)	Initial W.C (%)	Final W.C (%)
B-1 at 2 to 5 Feet	Recompacted Fill / Colluvium: CLAYEY SAND (SC)	112.4 (90% RC)	10.0	16.4

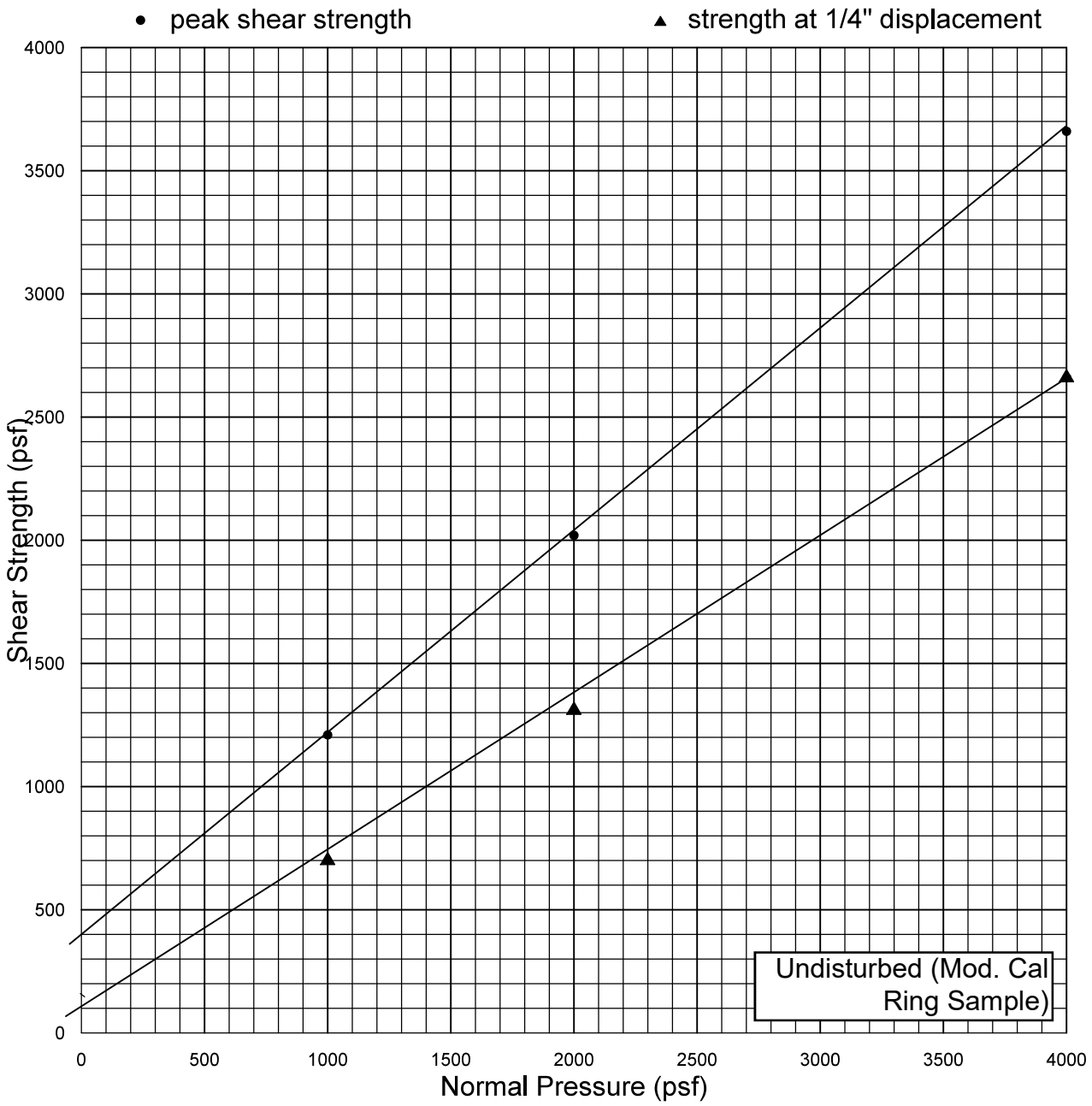
Normal Pressure (psf)	Peak Shear strength (psf)	Ultimate Shear Strength (psf)
1000	920 @ 0.0550"	720
2000	1500 @ 0.0605"	1390
4000	2640 @ 0.1405"	2640
	C = 300 psf	C = 150 psf
	ϕ = 31.5 deg.	ϕ = 31.5 deg.

Date: 08-28-2024

GeoLogic Associates

Figure C-5

DIRECT SHEAR TEST - ASTM D-3080



Strain Rate: 0.0084 in. / min.

Sample Location	Material Description	Dry Density (pcf)	Initial W.C (%)	Final W.C (%)
B-2 at 6 to 6½ Feet	Old Paralic Deposits: CLAYEY SAND (SC)	120.4	11.0	13.4

Normal Pressure (psf)	Peak Shear strength (psf)	Ultimate Shear Strength (psf)
1000	1210 @ 0.1050"	700
2000	2020 @ 0.1105"	1310
4000	3660 @ 0.1400"	2660
	C = 400 psf	C = 100 psf
	ϕ = 40 deg.	ϕ = 33.5 deg.

Date: 08-28-2024

GeoLogic Associates

APPENDIX D

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

APPENDIX D

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

GENERAL

These specifications present general procedures and requirements for grading and earthwork as shown on the approved grading plans, including preparation of areas to be filled, placement of fill, installation of subdrains, and excavations. The recommendations contained in the attached geotechnical report are a part of the earthwork and grading specifications and shall supersede the provisions contained herein in the case of conflict. Evaluations performed by the Consultant during the course of grading may result in new recommendations, which could supersede these specifications, or the recommendations of the geotechnical report.

EARTHWORK OBSERVATION AND TESTING

Prior to the start of grading, a qualified Geotechnical Consultant (Geotechnical Engineer and Engineering Geologist) shall be employed for the purpose of observing earthwork procedures and testing the fills for conformance with the recommendations of the geotechnical report and these specifications. It will be necessary that the Consultant provide adequate testing and observation so that he may determine that the work was accomplished as specified. It shall be the responsibility of the Contractor to assist the Consultant and keep them apprised of work schedules and changes so that he may schedule his personnel accordingly.

It shall be the sole responsibility of the Contractor to provide adequate equipment and methods to accomplish the work in accordance with applicable grading codes or agency ordinances, these specifications, and the approved grading plans.

Maximum dry density tests used to determine the degree of compaction will be performed in accordance with the American Society for Testing and Materials Test Method (ASTM) D1557.

PREPARATION OF AREAS TO BE FILLED

Clearing and Grubbing: All brush, vegetation and debris shall be removed or piled and otherwise disposed of.

Processing: The existing ground which is determined to be satisfactory for support of fill shall be scarified to a minimum depth of 8 inches. Existing ground, which is not satisfactory, shall be over-excavated as specified in the following section.

Over-excavation: Soft, dry, spongy, highly fractured, or otherwise unsuitable ground, extending to such a depth that surface processing cannot adequately improve the condition, shall be over-excavated down to firm ground, approved by the Consultant.

Moisture conditioning: Over-excavated and processed soils shall be watered, dried-back, blended, and mixed as required to have a relatively uniform moisture content near the optimum moisture content as determined by ASTM D1557.

Re-compaction: Over-excavated and processed soils, which have been mixed, and moisture conditioned uniformly shall be recompacted to a minimum relative compaction of 90% of ASTM D1557.

Benching: Where soils are placed on ground with slopes steeper than 5:1 (horizontal to vertical), the ground shall be stepped or benched. Benches shall be excavated in firm material for a minimum width of 4 feet.

FILL MATERIAL

General: Material to be placed as fill shall be free of organic matter and other deleterious substances and shall be approved by the Consultant.

Oversize: Oversized material defined as rock, or other irreducible material with a maximum dimension greater than 6 inches, shall not be buried or placed in fill, unless the location, material, and disposal methods are specifically approved by the Consultant. Oversize disposal operations shall be such that nesting of oversized material does not occur, and such that the oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 feet vertically of finish grade or within the range of future utilities or underground construction, unless specifically approved by the Consultant.

Import: If the importing of fill material is required for grading, the import material shall meet the general requirements.

FILL PLACEMENT AND COMPACTION

Fill Lifts: Approved fill material shall be placed in areas prepared to receive fill in near-horizontal layers not exceeding 8 inches in compacted thickness. The Consultant may approve thicker lifts if testing indicates the grading procedures are such that adequate compaction is being achieved with lifts of greater thickness. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to attain uniformity of material and moisture in each layer.

Fill Moisture: Fill layers at a moisture content less than optimum shall be watered and mixed, and wet fill layers shall be aerated by scarification or shall be blended with drier material. Moisture conditioning and mixing of fill layers shall continue until the fill material is at uniform moisture content at or near optimum.

Compaction of Fill: After each layer has been evenly spread, moisture conditioned, and mixed, it shall be uniformly compacted to not less than 90% of maximum dry density in accordance with ASTM D1557. Compaction equipment shall be adequately sized and shall be either specifically designed for soil compaction or of proven reliability, to efficiently achieve the specified degree of compaction.

Fill Slopes: Compacting on slopes shall be accomplished, in addition to normal compacting procedures, by backrolling of slopes with sheepsfoot rollers at frequent increments of 2 to 3 feet as the fill is placed, or by other methods producing satisfactory results. At the completion of grading, the relative compaction of the slope out to the slope face shall be at least 90% in accordance with ASTM D1557.

Compaction Testing: Field tests to check the fill moisture and degree of compaction will be performed by the consultant. The location and frequency of tests shall be at the consultant's discretion. In general, these tests will be taken at an interval not exceeding 2 feet in vertical rise, and/or 1,000 cubic yards of fill placed. In addition, on slope faces, at least one test shall be taken for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope.

SUBDRAIN INSTALLATION

Subdrain systems, if required, shall be installed in approved ground to conform to the approximate alignment and details shown on the plans or herein. The subdrain location or materials shall not be changed or modified without the approval of the Consultant. The Consultant, however, may recommend and, upon approval, direct changes in subdrain line, grade, or materials. All subdrains should be surveyed for line and grade after installation and sufficient time shall be allowed for the surveys, prior to commencement of fill over the subdrain.

EXCAVATION

Excavations and cut slopes will be examined during grading. If directed by the Consultant, further excavation or over-excavation and refilling of cut areas, and/or remedial grading of cut slopes shall be performed. Where fill over cut slopes are to be graded, unless otherwise approved, the cut portion of the slope shall be made and approved by the Consultant prior to placement of materials for construction of the fill portion of the slope.

APPENDIX E

SITE-SPECIFIC GROUND MOTION ANALYSIS RESULTS

SITE-SPECIFIC GROUND MOTION ANALYSIS (ASCE 7-16)

Project: Laguna Beach HS Aquatic Center
Client: LBSD
Job No: 2520A02

Latitude: 33.5423 deg
Longitude: -117.7762 deg
Vs_{30}: 260 (Assumed) m/s

Calculated By: THC
Checked By: IC
Date: August, 2024

Period T (sec)	PROBABILISTIC (RISK-TARGETED) GROUND MOTION ANALYSIS				DETERMINISTIC (84TH-PERCENTILE) GROUND MOTION ANALYSIS			CODE-BASED (LOWER LIMIT) ASCE 7-16 SECTION 11.4.6		SITE-SPECIFIC DESIGN RESPONSE		
	Uniform Hazard Ground Motion (g)	Risk Targeted Ground Motion (g)	Maximum Direction Scale Factor	Maximum Directional Probabilistic Sa (g)	84th Percentile Spectral Accelaration (g)	Maximum Direction Scale Factor	Maximum Directional Deterministic Sa (g)	Code Based S_a (g)	80% of Code Based S_a (g)	Design S_{aM} (g)	Design S_a (g)	T x S_a (T>1s)
PGA	0.628	0.595	1.1	0.655	0.893	1.1	0.983	0.352	0.281	0.655	0.436	---
0.10	1.061	1.027	1.1	1.130	1.448	1.1	1.593	0.757	0.606	1.130	0.753	---
0.20	1.440	1.391	1.1	1.530	1.891	1.1	2.080	0.879	0.703	1.530	1.020	---
0.30	1.600	1.528	1.125	1.719	2.303	1.125	2.590	0.879	0.703	1.719	1.146	---
0.50	1.528	1.436	1.175	1.687	2.493	1.175	2.929	0.879	0.703	1.687	1.125	---
0.75	1.226	1.149	1.2375	1.422	2.152	1.2375	2.663	0.762	0.610	1.422	0.948	---
1.00	1.001	0.933	1.3	1.213	1.760	1.3	2.288	0.572	0.457	1.213	0.809	0.809
2.00	0.524	0.487	1.35	0.657	0.878	1.35	1.185	0.286	0.229	0.657	0.438	0.877
3.00	0.330	0.307	1.4	0.430	0.471	1.4	0.659	0.191	0.152	0.430	0.287	0.860
4.00	0.228	0.212	1.45	0.307	0.319	1.45	0.462	0.143	0.114	0.307	0.205	0.820
5.00	0.170	0.157	1.5	0.236	0.227	1.5	0.340	0.114	0.091	0.236	0.157	0.785

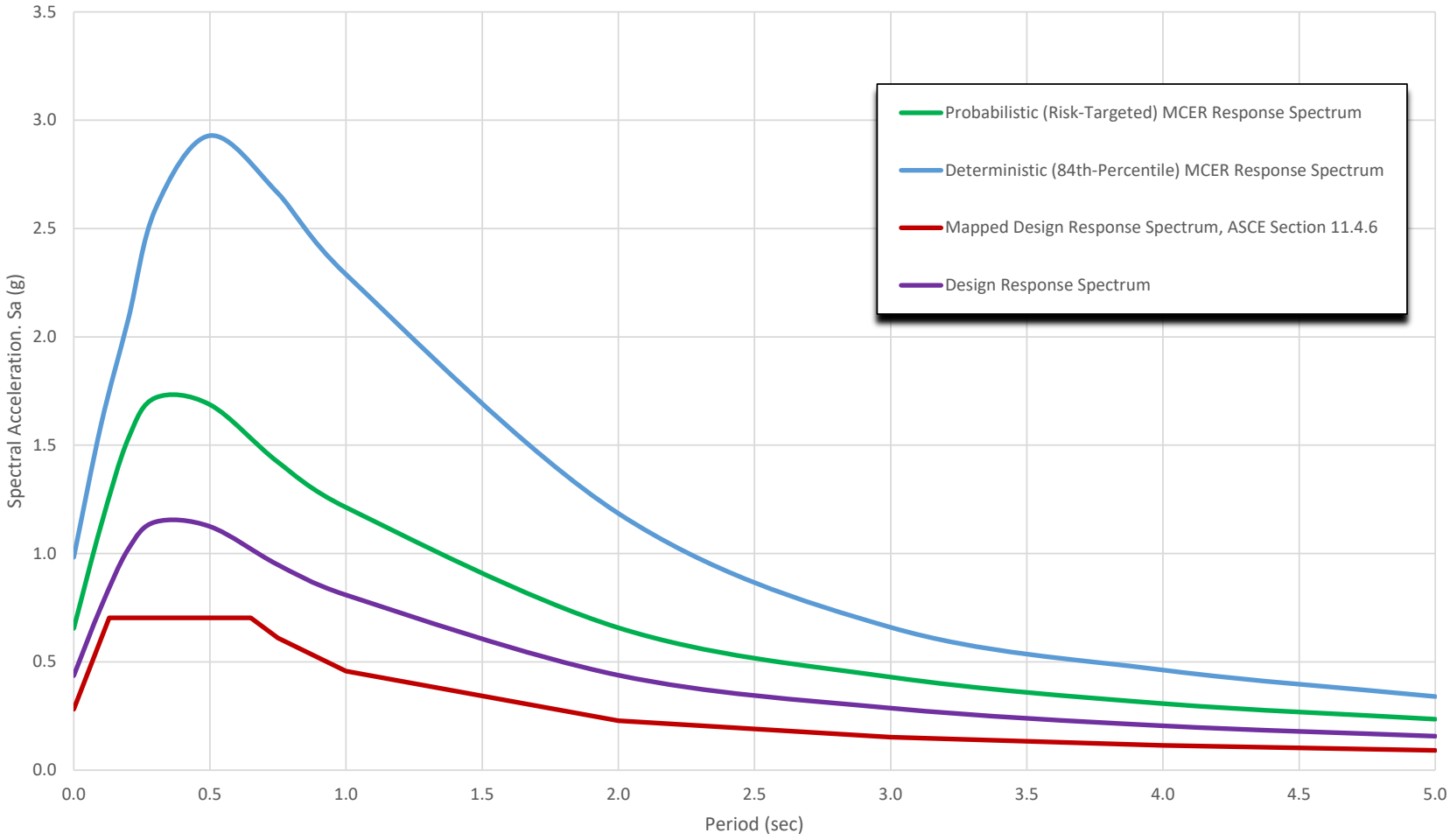
INPUT PARAMETERS - SEAOC (https://seismicmaps.org/)

Site Class=	D	
F_a=	1.0	Short Period Site Coefficient
S_S=	1.319	Mapped MCE_R, 5% Damped at T=0.2s
S_1=	0.468	Mapped MCE_R, 5% Damped at T=1s
S_{DS}=	0.879	Design, 5% Damped at Short Periods
S_{MS}=	1.319	The MCE_R, 5% Damped at Short Periods
T_L (sec)=	8.0	Long Period Transition (Sect 11.4.6)
F_{PGA} (g)=	1.1	Site Coefficient for PGA
PGA_M (g)=	0.635	
F_v=	1.832	Used Only for Calculation of T_o and T_s
S_{M1}=	0.857	
S_{D1}=	0.572	Design, 5% Damped at T=1s
T_o (sec)=	0.130	Defined in ASCE 7-16 Sect 11.4.6
T_S (sec)=	0.650	Defined in ASCE 7-16 Sect 11.4.6

SITE-SPECIFIC DESIGN PARAMETERS

S_{DS}=	**1.031**	90% of max S_a (ASCE 7-16 Sect 21.4)
S_{MS}=	**1.547**	MCE_R, 5% Damped, adjusted for Site Class
S_{D1}=	**0.877**	Design, 5% Damped, at T=1s (Sect 11.4.5)
S_{M1}=	**1.315**	MCE_R, 5% Damped, at T=1s, adjusted for Site
F_a=	**1.000**	Short Period Site Coefficient (7-16 Sect 21.3)
F_v=	**2.500**	Long Period Site Coefficient (7-16 Sect 21.3)
S_S=	**1.547**	MCE_R, 5% Damped at T=0.2s
S_1=	**0.526**	MCE_R, 5% Damped at T=1s
$PGA_{Probabilistic}$ (g)=	**0.628**	Peak Ground Acceleration, Probabilistic
$PGA_{Deterministic}$ (g)=	**0.893**	Peak Ground Acceleration, Deterministic
F_{PGA} (g)=	**1.1**	Site Coefficient for PGA, when PGA = 0.5g
0.5*F_{PGA} (g)=	**0.550**	OK (Check $PGA_{Deterministic}$ > 0.5 x F_{PGA})
0.8*PGA_M (g)=	**0.508**	PGA_M (g) (Determined from ASCE 7-16 Eq. 11.8-1)
Site Specific PGA (g) =	**0.628**	(Check $PGA_{Site\ Specific}$> 0.8 x PGA_M)

DETERMINISTIC (84TH-PERCENTILE) GROUND MOTION ANALYSIS											
	PGA	0.10	0.20	0.30	0.50	0.75	1.00	2.00	3.00	4.00	5.00
Newport-Inglewood (M=7.15)	0.711	1.073	1.464	1.723	1.784	1.518	1.304	0.724	0.471	0.319	0.227
San Joaquin Hills (M=7.02)	0.893	1.448	1.891	2.303	2.493	2.152	1.760	0.878	0.469	0.274	0.182
Palos Verdes (M=7.38)	0.314	0.527	0.737	0.805	0.747	0.590	0.477	0.255	0.168	0.120	0.089
84th Percentile Spectral Accelaration	**0.893**	**1.448**	**1.891**	**2.303**	**2.493**	**2.152**	**1.760**	**0.878**	**0.471**	**0.319**	**0.227**



MTGL

Laguna Beach HS Aquatic Center
Laguna Beach, California

By:	THC	Date:	August, 2024
Job Number:	2520A02	Figure:	E-3

APPENDIX F

INFILTRATION TEST RESULTS

PERCOLATION TEST DATA

Project Name:	**Laguna Beach HS Aquatic Center**	**Project No:**	**2520A02**

Test Information

Test Hole No.:	**P-1**	**Tested by:**	**T. Carter**	**Date:**	**8/16/2024**
Depth of Hole (in), D_T:	**103**	**USCS Soil Classification:**	**CLAYEY SAND (SC)**	**Gravel Pack:**	**Yes**
				Gravel Size:	**3/4 inch**

Test Hole Dimensions (in):	**Circle**	**Square / Rectangle**		**Perferated Pipe Diameter (in): 3**
	Diameter	**Length**	**Width**	
	8	**N/A**	**N/A**	

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in)	Final Depth to Water (in)	Change in Water Level (in)	Greater than or Equal to 6" (y/n)	Pass/Fail
1	12:35 PM	1:00 PM	25	80.6	80.8	0.2	No	Fail

Note: Sandy soil criteria test perfomred on 8/15/2024.

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurement sper hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Dt Time Interval (min)	D_o Initial Depth to Water (in)	D_f Final Depth to Water (in)	DD Change in Water Level (in)	Percolation Rate (min/in)	Tested Infiltration Rate (in/hr)	*Reliable Infiltration Rate (in/hr)
1	7:33 AM	8:03 AM	30	83.2	83.2	0.0	∞	0.00	0.00
2	8:03 AM	8:33 AM	30	83.2	83.4	0.2	125.00	0.04	0.02
3	8:33 AM	9:03 AM	30	83.4	83.6	0.1	250.00	0.02	0.01
4	9:05 AM	9:35 AM	30	80.6	80.8	0.2	125.00	0.04	0.02
5	9:35 AM	10:05 AM	30	80.8	80.9	0.1	250.00	0.02	0.01
6	10:06 AM	10:36 AM	30	80.9	81.0	0.1	250.00	0.02	0.01
7	10:36 AM	11:06 AM	30	81.0	81.3	0.2	125.00	0.04	0.02
8	11:06 AM	11:36 AM	30	81.3	81.4	0.1	250.00	0.02	0.01
9	11:39 AM	12:09 PM	30	79.24	79.24	0.0	∞	0.00	0.00
10	12:09 PM	12:39 PM	30	79.24	79.36	0.1	250.00	0.02	0.01
11	12:39 PM	1:09 PM	30	79.36	79.48	0.1	250.00	0.02	0.01
12	1:09 PM	1:39 PM	30	79.48	79.60	0.1	250.00	0.02	0.01

*The reliable infiltration rate is calculated by applying a factor of safety of 2 to the tested infiltration rate.

Remarks: Approximatly 2 inches of water remaining in test hole following presoak.

Porchet Method

$$I_t = \frac{\Delta H\,(60r)}{\Delta t\,(r + 2H_{avg})}$$

Where: ΔH = Change in height over time

Δt = Time interval

r = Test hole radius

H_{avg} = Average head over time interval

I_t = Tested infiltration rate