APPENDIX E – Geotechnical Reports

Preliminary Geological and Other Hazards Evaluation

Avalon K-12 School Modernization
200 Falls Canyon Road
Avalon, California

Chambers Group
600 W. Broadway, Suite 250 | Glendale, California 91204

May 30, 2019 | Project No. 211093001

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

Geophysics | Engineering Geology | Laboratory Testing | Industrial Hygiene | Occupational Safety | Air Quality | GIS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

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Preliminary Geological and Other Hazards Evaluation

Avalon K-12 School Modernization
200 Falls Canyon Road
Avalon, California

Ms. Meghan Gibson
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May 30, 2019 | Project No. 211093001

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1 INTRODUCTION

Ninyo & Moore has performed a preliminary geological and other hazards evaluation for the proposed Avalon K-12 School Modernization project in the city of Avalon (Figure 1) for the Long Beach Unified School District (LBUSD). The purpose of our study was to provide a preliminary evaluation of the potential geological hazards and potential hazards from power lines and pipelines, and other sources in compliance with the California Environmental Quality Act and the California Department of Education. This report is intended to meet the requirements of the California Education Code regarding school site selection.

2 SCOPE OF SERVICES

Our scope of services for this geological hazards assessment included the following:

- Review of background information, including readily available geotechnical reports, geologic maps, fault maps, landslide maps, flood inundation maps, and aerial photographs.
- Performance of a geologic reconnaissance of the site and surrounding areas.
- Performance of a pipeline risk evaluation in accordance with Education Code Section 17213(a)
- Performance of an above-ground water or fuel storage tank analysis in accordance with California Code of Regulations 14010(h).
- Preparation of this report presenting our preliminary findings and conclusions regarding potential geological and other hazards.

3 SITE DESCRIPTION

The site is located at 200 Falls Canyon Road, within the city of Avalon, approximately 2,700 feet southwest of the boat harbor. The Avalon K-12 campus accommodates students from Kindergarten to Grade 12 and was originally built in 1924 with the last set of buildings being added in 1996. The school is located along the southern side of Falls Canyon Road, along the base of a narrow natural ravine (Falls Canyon) with a steep natural hillside to the south, a condominium development along the ridge to the north, City of Avalon (City) warehouses on the west, and the Catalina Island Golf Course on the east. Elevations range from approximately 70 feet above mean sea level (msl) on the eastern edge of the school to approximately 135 feet above msl on the western edge. The site is approximately 11.5 acres in size and encompasses seven permanent buildings, one modular building, and 22 relocatable/portable buildings. The seven permanent buildings total are approximately 49,000 square feet in size and encompass two 2-story buildings and five single-story buildings. The modular/relocatable buildings add approximately 23,000 square feet to the site.

4 PROPOSED PROJECT DESCRIPTION

The project will include two phases of work. The first phase will include implementation of a Remedial Action Plan (RAP) at the site. The RAP has been developed in support of the development of a removal action workplan, which will involve removal/abatement of hazardous materials in designated hotspot areas up to five feet below the existing ground surface and transport the contaminated soils offsite to designated hazardous waste disposal sites.

The second phase of the project will include:

- Removal/abatement of hazardous materials found within the buildings.
- Removal of up to five feet below the existing ground surface of contaminated soil in delineated hotspot areas, concrete, asphalt, and earth excavation to allow installation of the upgraded HVAC units or subsurface utilities, replace degraded asphalt/concrete or Americans with Disabilities (ADA) pathways.
- Excavation for the installation of a new synthetic-turf athletic field.

Once the hazardous and non-hazardous soil arrives at the Port of Los Angeles (POLA), non-hazardous soil would be transported to the Simi Valley Landfill, which is located at 2801 Madera Road, Simi Valley, California, approximately 62 miles from the POLA. Non-Resource Conservation and Recovery Act (RCRA) and RCRA soils would be transported to Waste Management Kettleman Hills, located at 5251 Old Skyline Road Kettleman City, California, approximately 200 miles from the POLA, or other appropriately licensed waste disposal facilities.

5 GEOLOGY

The project site is located within the Continental Borderland Geomorphic Province of southern California. The Continental Borderlands Province varies in width from approximately 60 to 150 miles and is composed of elevated blocks and ridges, with islands above the marine datum and deep, often enclosed submarine basins. The seaward edge, known as the Patton Escarpment, extends from Point Conception to Punta Banda in Baja California. Maximum relief within the Continental Borderland is roughly 8,500 feet (Norris and Webb, 1990). Santa Catalina Island is within the Catalina terrane of the California continental borderland. This terrane makes up most of the inner part of the Continental Borderland Province and it is characterized by Miocene and younger sedimentary and volcanic rocks that lie directly on Catalina Schist (Vedder, et al., 1993).

Active northwest-trending fault zones in the region include the Palos Verdes, Coronado Bank, and Newport-Inglewood fault zones. The northern boundary of the province is formed by the Transverse Ranges Southern Boundary fault system which includes the active Malibu, Santa Monica, Hollywood, and Raymond faults. The active San Andreas fault zone is located northeast of the province within the Colorado Desert Geomorphic Province. The predominant major tectonic activity associated with these and other faults within this regional tectonic framework is right-lateral, strike-slip movement (Norris and Webb, 1990).

Santa Catalina Island forms a ridge crest within the Catalina terrane. The island consists of Mesozoic metamorphic basement complexes (Catalina schist terrane of blueschist, greenschist and amphibolite facies) on the roughly northwestern half of the island and the Miocene Catalina Pluton, a lower Miocene quartz diorite stock on the roughly southeastern half of the island (Rowland, 1984; Bound-Sanders, et al., 1987) (Figure 3). Based on our review and site observations, the site is underlain by stream alluvium and quartz diorite bedrock. Alluvial soils typically consist of gravely and silty sand. The bedrock was observed in road cuts and other exposures on or near the site to consist of moderately to highly weathered, fine to medium-grain porphyritic quartz diorite.

6 FAULTING, SEISMICITY, AND GEOLOGIC HAZARDS

The site is not located within a State of California Earthquake Fault Zone (formerly known as Alquist-Priolo Special Studies Zone). However, the site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion in the project area is considered significant during the design life of the proposed structure. The numerous faults in southern California include active, potentially active, and inactive faults. As defined by the California Geological Survey (CGS), active faults are faults that have ruptured within Holocene time, or within approximately the last 11,000 years. Potentially active faults are those that show evidence of movement during Quaternary time (approximately the last 1.6 million years) but for which evidence of Holocene movement has not been established. Inactive faults have not ruptured in the last approximately 1.6 million years. The approximate locations of major faults in the site vicinity and their geographic relationship to the site are shown on Figure 4. Historical earthquakes with a magnitude of 6.5 or greater, or that caused significant loss of life and property within approximately 62 miles (100 kilometers) of the subject site were obtained from the CGS Regional Geologic Hazards and Mapping Program website (CGS, 2017) and are presented in Table 1.

Table 1 – Historical Earthquakes

Date	Name, Location, or Region Affected	Approximate Earthquake Epicenter to Site Distance in miles (km)	Earthquake Magnitude
November 22, 1800	San Diego/San Juan Capistrano	69.5 (43.2)	6.4
March 11, 1933	Long Beach	50.6 (31.5)	6.4
October 1, 1987	Whittier Narrows	84.6 (52.6)	6.0
January 17, 1994	Northridge	98.8 (61.4)	6.7

Note:
CGS, 2017.

In addition to the mapped faults shown on Figure 4, the San Joaquin Hills blind thrust fault is located approximately 33.7 miles from the site and the Puente Hills blind thrust fault is located approximately 40.7 miles from the site (United States Geological Survey [USGS], 2008). Blind thrust faults are low-angle faults at depth that do not break the surface and are, therefore, not shown on Figure 4. Although blind thrust faults do not have a surface trace, they can be capable of generating damaging earthquakes and are included in Table 2.

Table 2 lists selected principal known active faults within approximately 50 miles of the site that may affect the project and the maximum moment magnitude (M_{max}) as published by the USGS (USGS, 2008). The approximate fault-to-site distances were calculated using the USGS web-based program (USGS, 2008).

Table 2 – Principal Active Faults

Fault	Approximate Fault-to-Site Distance [1] miles (kilometers)	Maximum Moment Magnitude [1] (M_{max})
Palos Verdes	17.0 (27.8)	7.7
Coronado Bank	24.3 (39.1)	7.4
Newport-Inglewood	29.7 (47.8)	7.5
San Joaquin Hills Blind Thrust	33.7 (54.2)	7.1
Puente Hills Blind Thrust	40.7 (65.6)	7.0
Elsinore	47.8 (76.9)	7.8
Santa Monica	48.4 (77.9)	7.4
Anacapa-Dume	48.6 (79.5)	7.2
Malibu Coast	48.9 (80.0)	7.0

Notes:
[1] USGS, 2008

In general, seismic hazards that could impact the project include ground surface rupture, strong ground motion, liquefaction, and dynamic compaction of dry soils. These potential hazards are discussed in the following sections.

6.1 Surface Fault Rupture

Based on our review of the referenced literature and our site reconnaissance, no active faults are known to cross the project site. Therefore, the probability of damage from surface ground rupture is considered to be low. However, lurching or cracking of the ground surface as a result of nearby seismic events is possible.

6.2 Ground Motion

The 2016 California Building Code (CBC) specifies that the Risk-Targeted, Maximum Considered Earthquake (MCE_R) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. The MCE_R ground motion response accelerations are based on the spectral response accelerations for 5 percent damping in the direction of maximum horizontal response and incorporate a target risk for structural collapse equivalent to 1 percent in

50 years with deterministic limits for near-source effects. The horizontal peak ground acceleration (PGA) that corresponds to the MCE_R for the site was calculated as 0.33g using the USGS (USGS, 2017) seismic design tool (web-based).

The 2016 CBC specifies that the potential for liquefaction and soil strength loss be evaluated, where applicable, for the mapped Maximum Considered Earthquake Geometric Mean (MCE_G) PGA (PGA_M) with adjustment for site class effects in accordance with the American Society of Civil Engineers 7-10 Standard. The MCE_G PGA is based on the geometric mean PGA with a 2 percent probability of exceedance in 50 years. The mapped MCE_G PGA with adjustment for site class effects (PGA_M) was calculated as 0.39g using the USGS (USGS, 2017) seismic design tool.

6.3 Liquefaction and Seismically Induced Settlement

Liquefaction is the phenomenon in which loosely deposited granular soils and cohesionless fine-grained soils located below the water table undergo rapid loss of shear strength due to excess pore pressure generation when subjected to strong earthquake-induced ground shaking. Sufficient ground shaking duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure. This causes the soil to behave as a fluid for a short period of time. Liquefaction is known generally to occur in saturated or near-saturated cohesionless soils at depths shallower than 50 feet below the ground surface. Factors known to influence liquefaction potential include composition and thickness of soil layers, grain size, relative density, groundwater level, degree of saturation, and both intensity and duration of ground shaking.

The depth to groundwater in the vicinity of the site is reported to be deeper than roughly 50 feet below the existing ground surface (Santa Catalina Island Company, 2015). Based on the nature of the underlying bedrock materials and the reported groundwater levels, the potential for dynamic settlement due to liquefaction is considered to be low. However, the potential for liquefaction should be further evaluated by a geotechnical evaluation, including subsurface and laboratory evaluation, prior to final design.

6.4 Dynamic Compaction of Dry Soils

Relatively dry soils (e.g., soils above the groundwater table) with low density or softer consistency tend to undergo dynamic compaction during a seismic event. Earthquake shaking often induces significant cyclic shear strain in a soil mass, which responds to the vibration by undergoing volumetric changes. Volumetric changes in dry soils take place primarily through changes in the void ratio (usually contraction in loose or normally consolidated, soft soils and dilation in dense or overconsolidated, stiff soils) and secondarily through particle reorientation. Such volumetric changes are generally non-recoverable. Based on the nature of the underlying bedrock materials, the potential for dynamic compaction of dry soils is considered to be low.

6.5 Landsliding

There are no mapped landslides on site or in the vicinity, and the site is not mapped as having the potential for seismic-induced landslides. Based on this information and the location of the site, large scale landsliding is not considered to be a potential hazard at the site. However, steep slopes along the southern side of the project site expose weathered and fractured bedrock materials and, in some areas, may be subject to small to moderate sized rock fall-type surficial slope failures. These slopes should be observed, mapped, and further evaluated if development is proposed adjacent to steep slopes with exposed rock.

6.6 Tsunamis and Seiches

Tsunamis are long wavelength, seismic, sea waves (long compared to ocean depth) generated by the sudden movements of the ocean floor during submarine earthquakes, landslides, or volcanic activity. Seiches are waves generated in a large, enclosed body of water. Based on the location and elevation of the site, damage due to tsunamis or seiches is not a design consideration.

6.7 Flood Hazards

Based on review of the Federal Emergency Management (FEMA) website, the site is not mapped as lying within the 500-year floodplain (FEMA, 2008). Based on this review, the potential for flooding of the site is considered low.

7 OTHER HAZARDS BASED ON CALIFORNIA EDUCATION CODE REQUIREMENTS AND DISTRICT BOARD RESOLUTIONS

Ninyo & Moore has evaluated these conditions with respect to the site and they are discussed in the following sections.

7.1 High-Voltage Electrical Transmission Lines

In accordance with Title 5, Section 14010 of the California Code of Regulations, the property line of a new school site should be at least the following distances from the edge of respective power line easements: (1) 100 feet for a 50-133 kilovolt (kV) line, (2) 150 feet for a 220-230 kV line, and (3) 350 feet for a 500-550 kV line.

Ninyo & Moore requested information from Southern California Edison (SCE) on May 21, 2019, regarding overhead and underground electrical lines with the specified distances from the site. According to Mr. John Long with SCE, the highest voltage of electrical lines on the island is 12kV.

Review of GoogleEarth images suggests there are pole-mounted electrical lines located adjacent to the southern site boundary, as well pole-mounted electrical lines located within about 215 feet of the southern property boundary.

7.2 Underground Pipelines

Ninyo & Moore requested information regarding underground petroleum and natural gas lines located within 1,500 feet of the site from SCE, City of Avalon Fire Department (AFD), City of Avalon Public Works Department (APWD) (May 20, 2019), Office of the State Fire Marshal (SFM, May 21, 2019), and the California Public Utilities Commission (CPUC) (May 20, 2019). Ninyo & Moore also reviewed the National Pipeline Mapping System (NPMS, 2019).

According to Mr. Long with SCE, underground propane/natural gas pipelines are located within 1,500 feet of the site. These lines are located along Falls Canyon Road and Avalon Canyon Road and have a maximum allowable operating pressure (MAOP) of 10 pounds per square inch (psi).

The AFD and APWD had no information regarding underground pipelines located within 1,500 feet of the site. According to the SFM, “there are no pipelines jurisdictional to the State Fire Marshal in the area.” Review of the NPMS on-line Public Viewer did not identify underground petroleum or natural gas pipelines within 1,500 feet of the site.

According to the CPUC website, “The CPUC regulates the transportation rates and terms of service of pipeline companies that transport petroleum products owned by other companies. The CPUC does not … have safety jurisdiction over petroleum pipelines.”

7.3 Natural Gas Pipelines

Ninyo & Moore reviewed the Southern California Gas Company (SoCalGas) Gas Transmission and High Pressure Distribution Pipeline Interactive Map for natural gas transmission or high-pressure distribution pipelines located within 1,500 feet of the site. According to the map, no natural gas transmission or high-pressure distribution pipelines are located within 1,500 feet of the site.

7.4 Petroleum Pipelines

According to the NPMS, hazardous liquid pipelines are not within 1,500 feet of the site.

7.5 Water Pipelines

Ninyo & Moore requested information from the APWD on May 20, 2019 regarding high-pressure water pipelines within 1,500 feet of the site. According to Mr. Jordan Monroe with the APWD, the City owns and operates a saltwater wastewater system used for toilet, urinal flushing, and street wash-downs. Saltwater pipelines are located along Falls Canyon Road adjacent to the north of the site. The pipeline also crosses the northern portion of the site. According to Mr. Monroe, the saltwater pipelines are operated at a pressure of about 80 psi.

7.6 Railroad Tracks

According to the USGS, Santa Catalina East topographic map, railroad tracks are not present within 1,500 feet of the site.

7.7 Airports

According to Google Earth, the Avalon Airport and its nearest runway are approximately 6.5 miles northwest of the site.

7.8 Reservoirs/Water Storage Tanks

Large water tanks/reservoirs are not located on the site. Ninyo & Moore requested information from the APWD on May 20, 2019 regarding large water tanks/reservoirs within 1,500 feet of the site. According to Mr. Jordan Monroe with the APWD, a 420,000-gallon capacity potable water storage reservoir, which is owned and operated by SCE, is located approximately 330 feet north of the site. SCE also operates three potable water storage tanks located approximately 950 feet northwest of the site. According to Mr. Long with SCE, the combined storage capacity of the three water storage tanks is approximately 325,000 gallons. Section 17213(a) of the California Education Code states that the governing board of a school district should evaluate if the school site "contains one or more pipelines, situated underground or aboveground, which carry hazardous substances, acutely hazardous materials, or hazardous wastes, unless the pipeline is a natural gas pipeline which is used only to supply natural gas to that school or neighborhood."

8 CONCLUSIONS

Based on the results of our limited geological and other hazards evaluation, the following preliminary conclusions are provided for the proposed Avalon K-12 School project:

- The site is underlain by alluvium and quartz diorite bedrock.
- The subject site is not located within a State of California Earthquake Fault (Alquist-Priolo Special Studies) Zone.

- Active faults have not been mapped on or adjacent to the site. The closest known active fault, the Palos Verdes, is located approximately 17 miles northwest of the site.
- The site (like the majority of southern California) is located in Seismic Zone 4 according to the 2016 CBC. Accordingly, the potential for relatively strong seismic accelerations should be considered in the design of proposed Modernization.
- The site is not located in an area considered susceptible to large scale landslides. However, some slopes along the southern edge of the school were observed to expose weathered and fractured bedrock and may be subject to small to moderate sized rock falls. These slopes should be observed, mapped, and further evaluated if development is proposed adjacent to exposed rock slopes.
- The site is not located in an area considered susceptible to liquefaction, dynamic compaction, flood hazards, and inundation. The potential for liquefaction and dynamic compaction of soils should be further evaluated prior to final design.
- The potential hazards from high voltage power lines, natural gas and water pipelines, railroad tracks, airports, reservoir/water storage tanks, and lead in drinking water are not considered significant.

9 RECOMMENDATIONS

Prior to the design and construction of proposed improvements at the site, a detailed geotechnical evaluation, including subsurface exploration and laboratory testing, should be performed. The purpose of the geotechnical evaluation would be to 1) further evaluate the subsurface conditions, including liquefaction potential, at the site, 2) provide site-specific data regarding potential geologic and geotechnical constraints, and 3) provide information pertaining to the engineering characteristics of earth materials with regard to the proposed Modernization. Recommendations for earthwork, foundations, pavements, and other pertinent geotechnical design considerations may be formulated from the detailed geotechnical evaluation. In addition, pre-demolition ACM and LBP surveys should be conducted, as well as an investigation for suspected termiticides and lead in shallow soil around building foundations.

10 LIMITATIONS

The field evaluation and analyses presented in this report have been conducted in accordance with current engineering practice and the standard of care exercised by reputable geotechnical and environmental consultants performing similar tasks in this area. No warranty, implied or expressed, is made regarding the conclusions, recommendations, and professional opinions expressed in this report. Variations may exist and conditions not observed or described in this report may be encountered. Our preliminary conclusions and recommendations area based on an analysis of the observed conditions and the referenced background information.

The purpose of this study was to evaluate geological and other conditions within the project site and to provide a reconnaissance report to assist in the preparation of site selection documents for the project. A comprehensive geotechnical evaluation, including subsurface exploration and laboratory testing, should be performed prior to design and construction of structural Modernization.

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FIGURES

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

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FIGURE 1

SITE LOCATION

AVALON K-12 SCHOOL
AVALON, CALIFORNIA

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Geotechnical & Environmental Sciences Consultants

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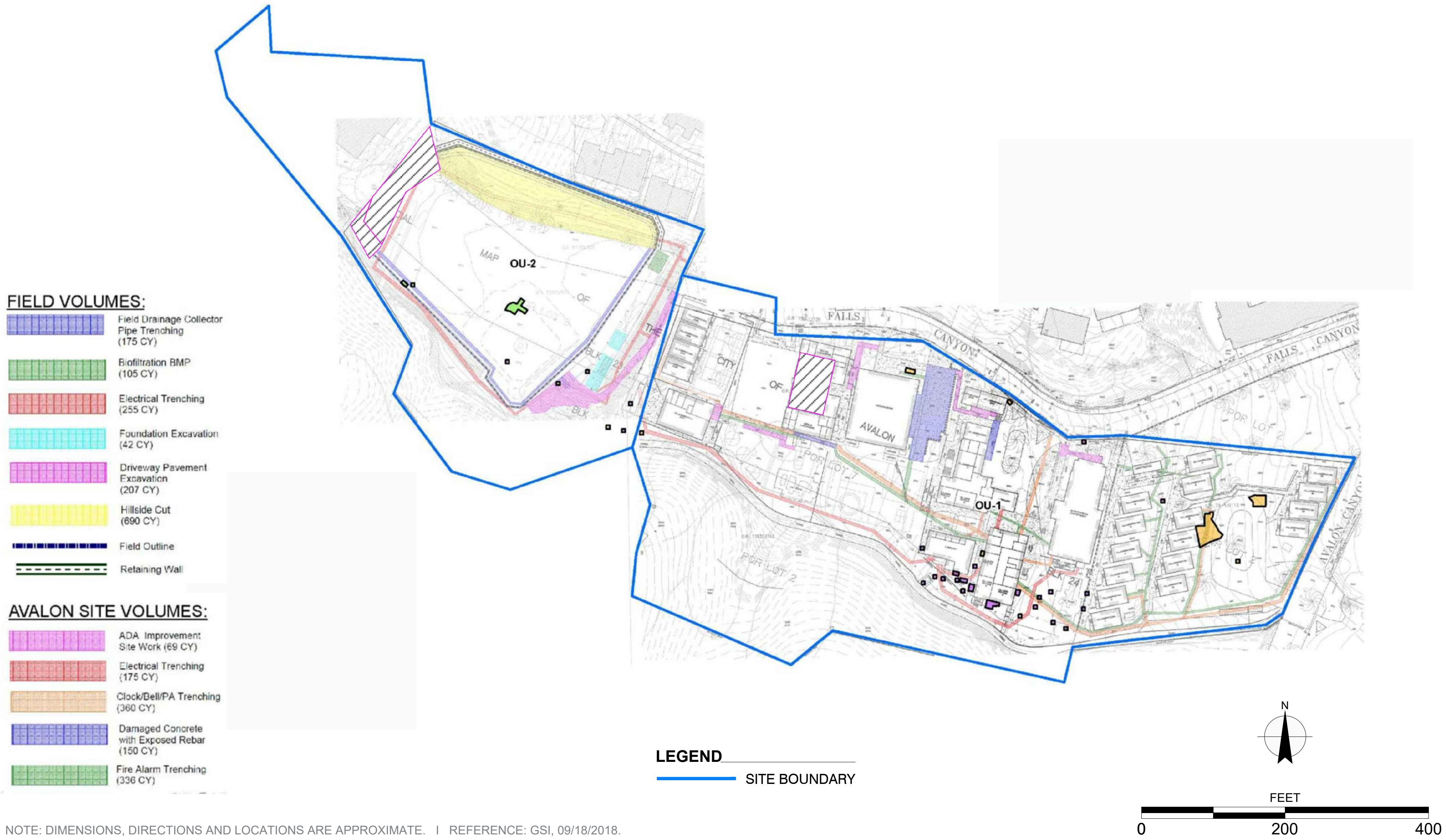

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: GSI, 09/18/2018.

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FIGURE 2

SITE PLAN

AVALON K-12 SCHOOL
AVALON, CALIFORNIA

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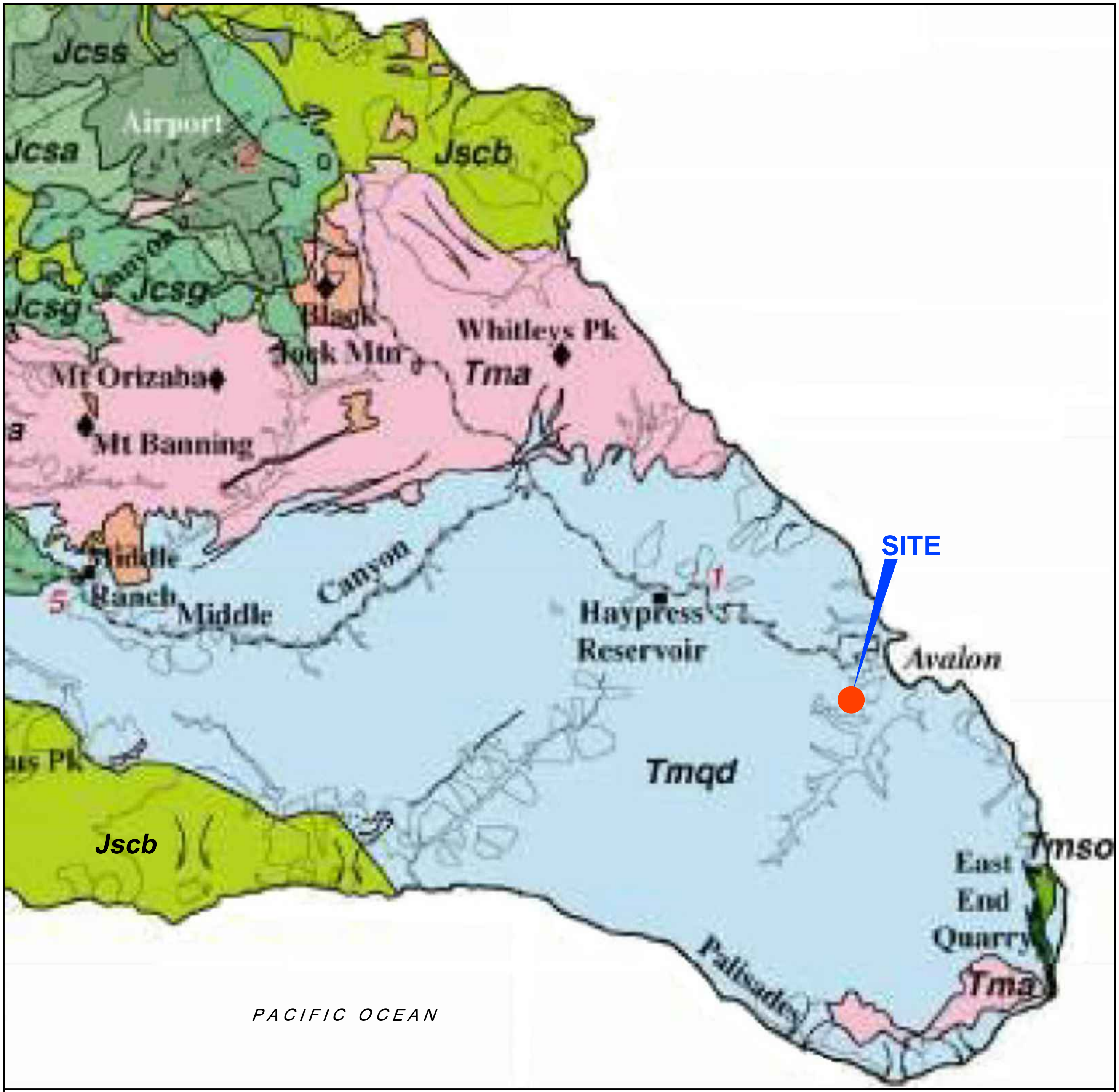

LEGEND

Tmqd	CATALINA ISLAND PLUTON, QUARTZ DIORITE
Jscb	CATALINA SCHIST, BLUESCHIST
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NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: BOHANNON & REISS, 1975..

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FIGURE 3

REGIONAL GEOLOGY

AVALON K-12 SCHOOL
AVALON, CALIFORNIA

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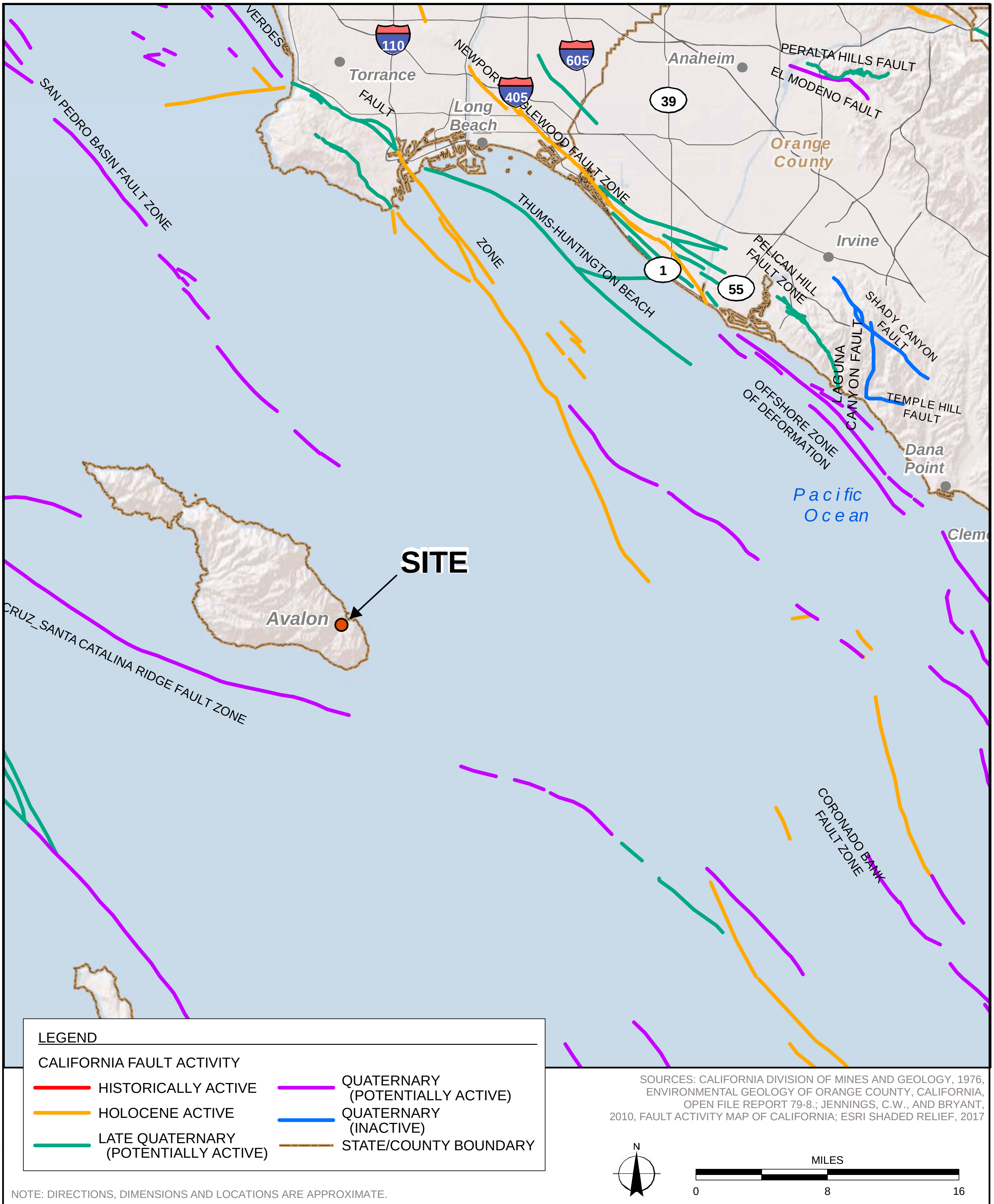
Geotechnical & Environmental Sciences Consultants



SOURCES: CALIFORNIA DIVISION OF MINES AND GEOLOGY, 1976, ENVIRONMENTAL GEOLOGY OF ORANGE COUNTY, CALIFORNIA, OPEN FILE REPORT 79-8.; JENNINGS, C.W., AND BRYANT, 2010, FAULT ACTIVITY MAP OF CALIFORNIA; ESRI SHADED RELIEF, 2017

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

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FIGURE 4

FAULT LOCATIONS

AVALON K-12 SCHOOL
AVALON, CALIFORNIA

211093001 | 5/19

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Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

October 24, 2018

Project No 12181.001
LBUSD Site/Task: 411-005

Long Beach Unified School District
Facilities Development & Planning
2425 Webster Avenue
Long Beach, California 90810

Attention: Ms. Talitha Crain

Subject: Limited Geotechnical Evaluation of Soil/Bedrock Conditions on Slope North of Ballfield, Avalon K-12 School Located at 200 Falls Canyon Road City of Avalon, Santa Catalina Island, California

INTRODUCTION

Leighton Consulting Inc. (Leighton) is pleased to present this letter report to the Long Beach Unified School District (District) summarizing the geotechnical suitability of soil/bedrock conditions in the slope north of the ballfield at the Avalon K-12 School located at 200 Falls Canyon Road in the City of Avalon, Santa Catalina Island, California (Figure 1, *Site Location Map*).

PURPOSE

Leighton understands the District is in the planning stages of a significant modernization project currently scheduled to be implemented at the Avalon school in the summer of 2020. As part of this modernization, the northern limits of the ballfield are planned to be extended to provide more space for the field. In order to accomplish this, a volume of soil and bedrock currently estimated at 690 cubic yards will be removed from the slope north of the ballfield. In addition, a large retaining wall will be engineered and constructed along the slope to support the northern side of the canyon adjacent to the ballfield. The ballfield, slope and location of proposed retaining wall are shown on Figure 2, *Site Plan*.

Once the soil/bedrock has been removed from the slope in preparation for construction of the retaining wall, the District intends to use the material generated as backfill for other areas of the campus where needed during the modernization project. The purpose of this evaluation is to preliminarily determine the geotechnical suitability of the material to be used as backfill during upcoming school modernization activities.

GEOTECHNICAL SCOPE OF WORK

On October 5, 2018, Leighton Professional Geologists (PGs) and Certified Engineering Geologists (CEGs) visited the school site to accomplish the following geotechnical tasks:

- Geologically map the slope north of the ballfield and surrounding bedrock in the nearby vicinity for geologic evaluation;
- Collect a representative bulk sample to assess the geotechnical suitability of the material to be used as backfill.
- Perform geotechnical laboratory testing on bulk soil samples collected during our field reconnaissance to determine general engineering properties of the near-surface soils that exist on the slope north of the ballfield.

GEOLOGIC SETTING

The site is located in what is known as the California Continental Borderland Geomorphic Province. This area is typified by elongated northwest and west-trending seafloor ridges and basins. The style of deformation within the province relates to the large-scale transform tectonism and volcanism which was initiated 20-24 million years ago coincident with subduction of the Farallon plate of the west coast of California. Since mid-Pliocene, uplift within the province has elevated portions of the borderland 6,000 feet or more. In most recent geologic time, erosion, deposition of alluvium/talus and man’s activities have formed the present landscape.

Rocks present within the Borderland Province reflect a history of an active continental margin. The oldest bedrock unit exposed on Catalina Island is the Catalina Schist, thought to form the basement complex of much of the inner borderland (Gath, 1985). These Mesozoic metamorphic rocks are similar in lithology to those of the Franciscan Complex along the California coast. The predominate earth unit mapped at the site is quartz diorite of the Catalina Pluton (Bailey, 1940). This early-Miocene pluton is a medium to fine-grained groundmass with larger crystals of altered plagioclase and hornblende. In the vicinity of the site, this unit is gray to gray-green in color and very dense to hard. Along fractures and on exposed surfaces the unit is iron-stained to reddish-orange brown. Phenocrysts within the fine-grained groundmass have been chemically altered on exposed surfaces. This unit forms the steep natural and man-made cut slopes in the vicinity of the site.

FINDINGS AND CONCLUSIONS

Bedrock exposures of Quartz diorite were not observed during our field reconnaissance on slope north of the ballfield. Bedrock is expected to be relatively weathered near surface and mantled with talus and artificial fill likely associated with the historical use of the ballfield and development of the hilltop residential housing located north of the ballfield. Talus is debris which accumulates on slopes as a result of erosion, rockfalls and the downward movement of material from slopes under the influence of gravity. We anticipate that quartz diorite bedrock exists at depth within the slope north of the ballfield in the vicinity of the planned retaining wall. Confirmation of the thickness of talus and fill debris on the slope face should be confirmed to aid in future retaining wall design.

Talus on slope ranges in size from 4 to 14 inches in long dimension

Quartz diorite bedrock outcrops were observed in natural slopes south of the ballfield and in road cuts above the cemetery north of the ballfield as shown in the adjacent photo. Structural geologic data in the form of joint sets was collected from the road cut exposure of quartz diorite bedrock. The geologic data collected is presented in the following table:

Note nearly right angles of joint sets in outcrop

Joint Sets		
1	N15°E, 69°W	N66°W, 70°S
2	N3°W, 80°W	N66°W, 75°S
3	N5°E, 85°E	N35°E, 14°W
4	N35°E, 54°W	N35°W, 70°N
5	EW, 78°S	N29°E, 49°N
6	N36°W, 84°N	N50°E, 69°N
7	N12°E, 72°N	N64°W, 53°S

Engineering Properties

Based on our reconnaissance, the near surface soils mantling the slope consist predominantly of dark yellowish brown poorly graded gravel with clay and sand (GP-GC). Some oversize rock debris as observed ranges in size from approximately 4-inches to 14-inches in long dimension with localized corestones exceeding 14-inches. Geotechnical engineering properties determined to be relevant for evaluating reuse of the soil on the basis of field and laboratory testing are discussed below.

Expansive Soil Characteristics: Expansive soils contain significant amounts of clay particles that swell considerably when wetted and shrink when dried. Foundations constructed on these soils are subject to uplifting forces caused by the swelling. Without proper mitigation measures, heaving and cracking of both building foundations and slabs-on-grade could result.

Expansion Index (EI) was performed to identify the expansion potential of the near surface onsite soils. The bulk soil sample collected during our field reconnaissance was taken from existing near surface soils on the slope north of the ballfield. Results of the Expansion Index testing indicate an EI of 10, which is characterized as very low expansion potential.

Corrosivity: To evaluate corrosion potential of the site soils, we tested a near surface bulk soil sample for soluble sulfate content, soluble chloride content, pH and resistivity. Results of these tests are summarized below:

Results of Corrosivity Testing

Sample Number	Sample Depth (feet)	Sulfate (mg/kg)	Chloride (mg/kg)	pH	Minimum Resistivity (ohm-cm)
B1	0 to 2	781	61	7.85	**1,660**

Note: mg/kg = milligrams per kilogram, or parts-per-million (ppm)

These results are discussed as follows:

- **Sulfate Exposure:** Based on our previous experience and Table 19.3.1.1 of ACI 318-14, in our opinion, sulfate exposure should be considered “moderate” for native gravelly sands sampled at the site. Based on Table 19.3.2.1 of ACI 318-14, for this exposure, there would be no restrictions on cement type (“cementitious material”) nor water/cement ratio; an ƒc’ (28-day compressive strength) of at-least (≥) 2,500 pounds-per-square-inch (psi) is required at a minimum for structural concrete.

- **Ferrous Corrosivity:** As shown above, minimum soil resistivity of 6,400 ohm-centimeters was measured in our laboratory test. In our opinion, based on resistivity correlation presented above, it appears for site slope soils that corrosion potential to buried steel may be characterized as "severely corrosive" at the site. As standard design concepts, ferrous pipe buried in moist to wet site earth materials should be avoided by using high-density polyethylene (HDPE) or other non-ferrous pipe when possible. Or ferrous pipe can be protected by polyethylene bags, tap or coatings, di-electric fittings or other means to separate the pipe from on-site earth materials.

Fill Placement and Compaction: Onsite soils free of organics, debris and oversized material (greater-than 3-inches in largest dimension) are suitable for use as compacted structural fill as part of the campus modernization project. Some oversize material should be expected during excavation of the north slope and will require crushing to 3-inch minus or be disposed of offsite. All structural fill must be free of hazardous materials.

All fill soil should be placed in thin, loose lifts, moisture-conditioned, as necessary, to within 3 percent above optimum moisture content, and compacted to a minimum **90% relative compaction** as determined by ASTM D 1557 standard test method (modified Proctor compaction curve).

CONCLUSIONS

Based upon the results of this limited geotechnical evaluation, the soil expected to be generated from cutting of the slope north of the ballfield is considered geotechnically suitable for re-use as fill during the upcoming modernization project. A concurrent environmental evaluation of this soil/bedrock is presented under separate cover.

CLOSURE

Leighton appreciates this opportunity to continue to serve your environmental and geotechnical needs. If you have any questions regarding this letter report please call us at your convenience at **(866)** ***LEIGHTON***, direct at the phone extension or e-mail address listed below.

CERTIFIED ENGINEERING GEOLOGIST
JOE A.
ROE
No. 2456
STATE OF CALIFORNIA

Respectfully submitted,

LEIGHTON CONSULTING, INC.

Joe Roe

Joe A. Roe, PG, CEG
Principal Geologist
Ext. 4263; jroe@leightongroup.com

JMP/JAR/lr

Attachments: References
Figure 1 – Site Location Map
Figure 2 – Site Plan
Geotechnical Laboratory Test Results

Distribution: (1) Addressee

REFERENCES

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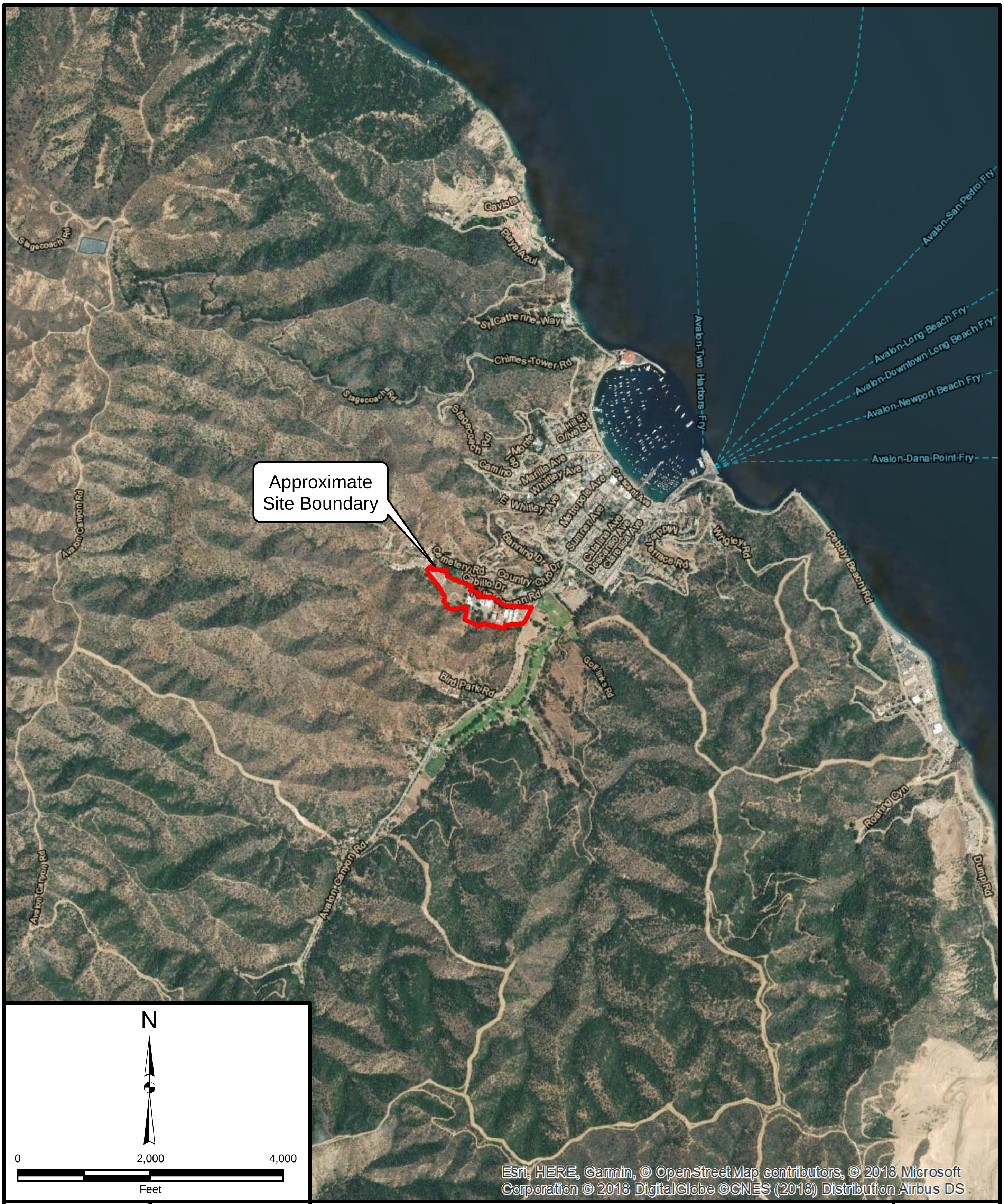


Esri, HERE, Garmin, © OpenStreetMap contributors, © 2018 Microsoft Corporation © 2018 DigitalGlobe ©CNES (2018) Distribution Airbus DS

Project: 12181.001

Eng/Geol: KRL/JAR

Scale:1 " = 2,000 '

Date: October 2018

Base Map: ESRI ArcGIS Online 2018
Thematic Information: Leighton
Author: Leighton Geomatics (btran)

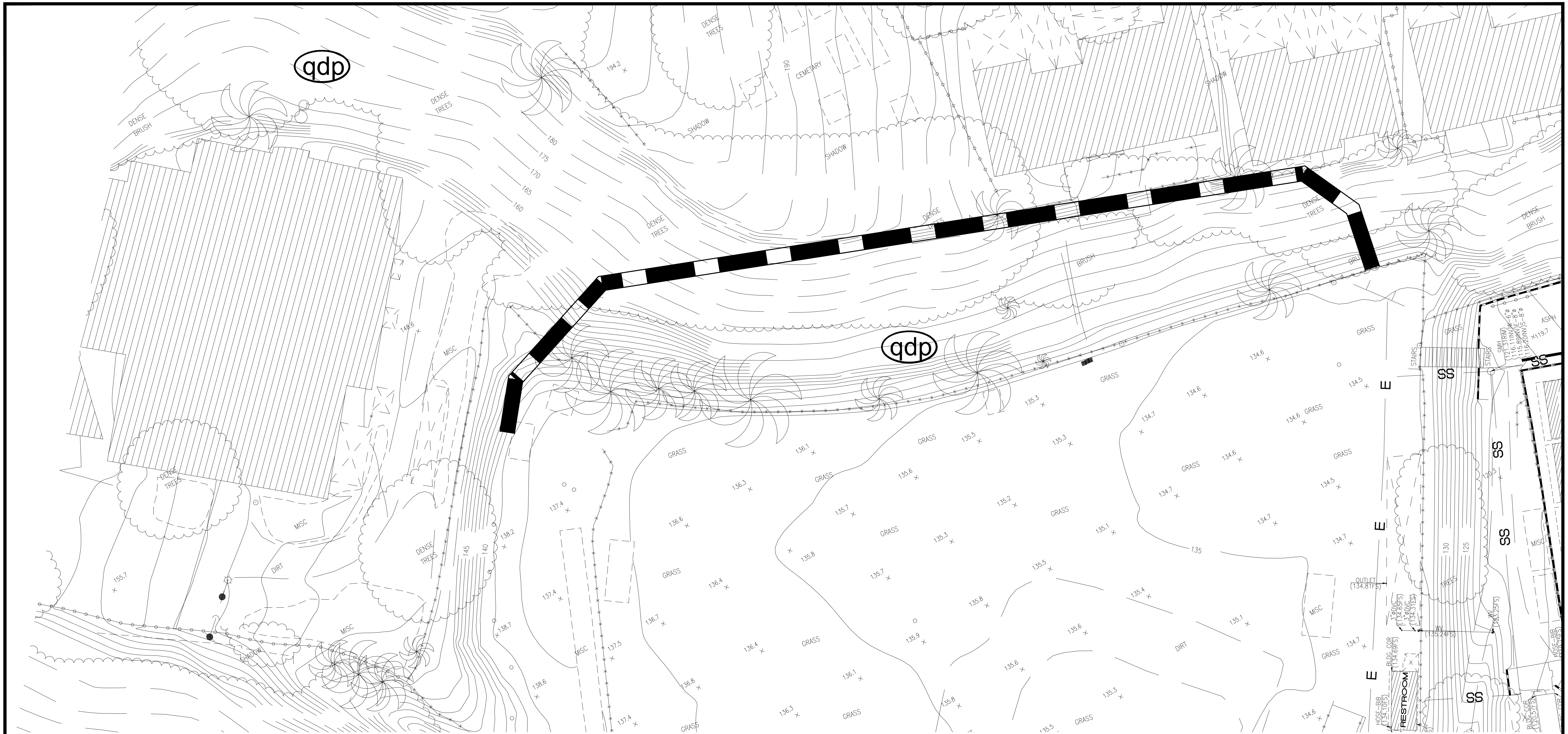
SITE LOCATION MAP

Avalon K-12 School
200 Falls Canyon Road
City of Avalon, Santa Catalina Island, California

Figure 1

Leighton

Map Saved as V:\Drafting\12181\001\Maps\12181-001_F01_SLM_2018-10-23.mxd on 10/23/2018 3:49:56 PM



LEGEND

qdp Tertiary Age, Quartz Dioritz Porphyry, Fine to Coarse Grained, Granite, Circled Where Buried Below Slopewash

▬▭▬▭▬ Approximate Location of Proposed Retaining Wall

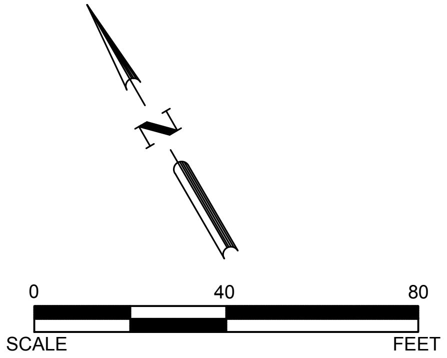


Figure 2	**SITE PLAN** Avalon K-12 School 200 Falls Canyon Road City of Avalon, Santa Catalina Island, California	
Leighton	Proj: 12181.001	Eng/Geol: KRL/JAR
	Scale: 1"=40'	Date: October 2018

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TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name: Avalon

Tested By : A. Santos Date: 10/17/18

Project No. : 12181.001

Data Input By: J. Ward Date: 10/23/18

Location	North Slope			
Sample No.	B1			
Sample Depth (ft)	0-1			
Soil Identification:	Dark yellowish brown (GP-GC)s			
Wet Weight of Soil + Container (g)	128.08			
Dry Weight of Soil + Container (g)	126.70			
Weight of Container (g)	57.22			
Moisture Content (%)	1.99			
Weight of Soaked Soil (g)	100.30			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	308			
Crucible No.	11			
Furnace Temperature (°C)	860			
Time In / Time Out	11:20/12:05			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	22.1661			
Wt. of Crucible (g)	22.1475			
Wt. of Residue (g) (A)	0.0186			
PPM of Sulfate (A) x 41150	765.39			
PPM of Sulfate, Dry Weight Basis	**781**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	0.8			
PPM of Chloride (C -0.2) * 100 * 30 / B	60			
PPM of Chloride, Dry Wt. Basis	**61**			

pH TEST, DOT California Test 643

pH Value	**7.85**			
Temperature °C	20.2			

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SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	Avalon	Tested By :	A. Santos	Date:	10/18/18
Project No. :	12181.001	Data Input By:	J. Ward	Date:	10/23/18
Location :	North Slope	Depth (ft.) :	0-1		
Sample No. :	B1				

Soil Identification:* Dark yellowish brown (GP-GC)s

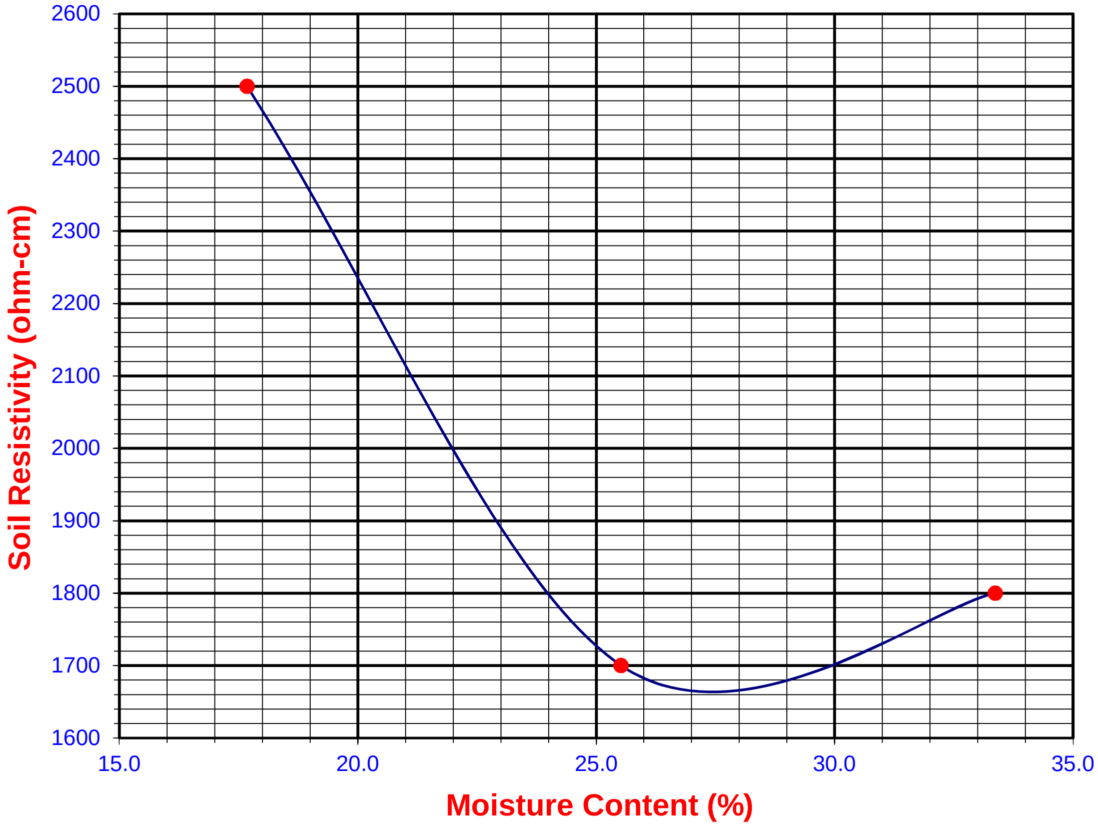
*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	17.68	2500	2500
2	30	25.52	1700	1700
3	40	33.37	1800	1800
4				
5				

Moisture Content (%) (MCi)	1.99
Wet Wt. of Soil + Cont. (g)	128.08
Dry Wt. of Soil + Cont. (g)	126.70
Wt. of Container (g)	57.22
Container No.	
Initial Soil Wt. (g) (Wt)	130.00
Box Constant	1.000

MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
1660	**27.5**	**781**	**61**	**7.85**	**20.2**



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EXPANSION INDEX of SOILS

ASTM D 4829

Project Name: Avalon
Tested By: S. Felter
Date: 10/17/18
Project No.: 12181.001
Checked By: J. Ward
Date: 10/23/18
Location: North Slope
Depth (ft.): 0-1
Sample No.: B1
Soil Identification: Dark yellowish brown poorly-graded gravel with clay and sand (GP-GC)s

Dry Wt. of Soil + Cont. (g)	1000.00
Wt. of Container No. (g)	0.00
Dry Wt. of Soil (g)	1000.00
Weight Soil Retained on #4 Sieve	0.00
Percent Passing # 4	100.00

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0095
Wt. Comp. Soil + Mold (g)	597.00	426.94
Wt. of Mold (g)	196.60	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	O	O
Wet Wt. of Soil + Cont. (g)	794.00	623.54
Dry Wt. of Soil + Cont. (g)	720.50	559.94
Wt. of Container (g)	0.00	196.60
Moisture Content (%)	10.20	17.50
Wet Density (pcf)	120.8	127.6
Dry Density (pcf)	109.6	108.6
Void Ratio	0.538	0.553
Total Porosity	0.350	0.356
Pore Volume (cc)	72.4	74.4
Degree of Saturation (%) [S_{meas}]	**51.2**	85.5

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
10/17/18	12:50	1.0	0	0.0320
10/17/18	13:00	1.0	10	0.0320
Add Distilled Water to the Specimen				
10/17/18	13:15	1.0	15	0.0360
10/18/18	6:30	1.0	1050	0.0415
10/18/18	7:37	1.0	1117	0.0415

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**10**

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PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS

ASTM D 6913

Project Name: Avalon
Project No.: 12181.001
Location: North Slope
Sample No.: B1
Soil Identification: Dark yellowish brown poorly-graded gravel with clay and sand (GP-GC)s

Tested By: ACS/OHF Date: 10/16/18
Checked By: J. Ward Date: 10/23/18
Depth (feet): 0-1

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	N/A	PH	Wt. of Air-Dry Soil + Cont.(g)	0.0	0.0
Wt. Air-Dried Soil + Cont.(g)	16400.9	726.4	Wt. of Dry Soil + Cont. (g)	0.0	0.0
Wt. of Container (g)	0.0	202.6	Wt. of Container No.______(g)	1.0	1.0
Dry Wt. of Soil (g)	16400.9	523.8	Moisture Content (%)	0.0	0.0

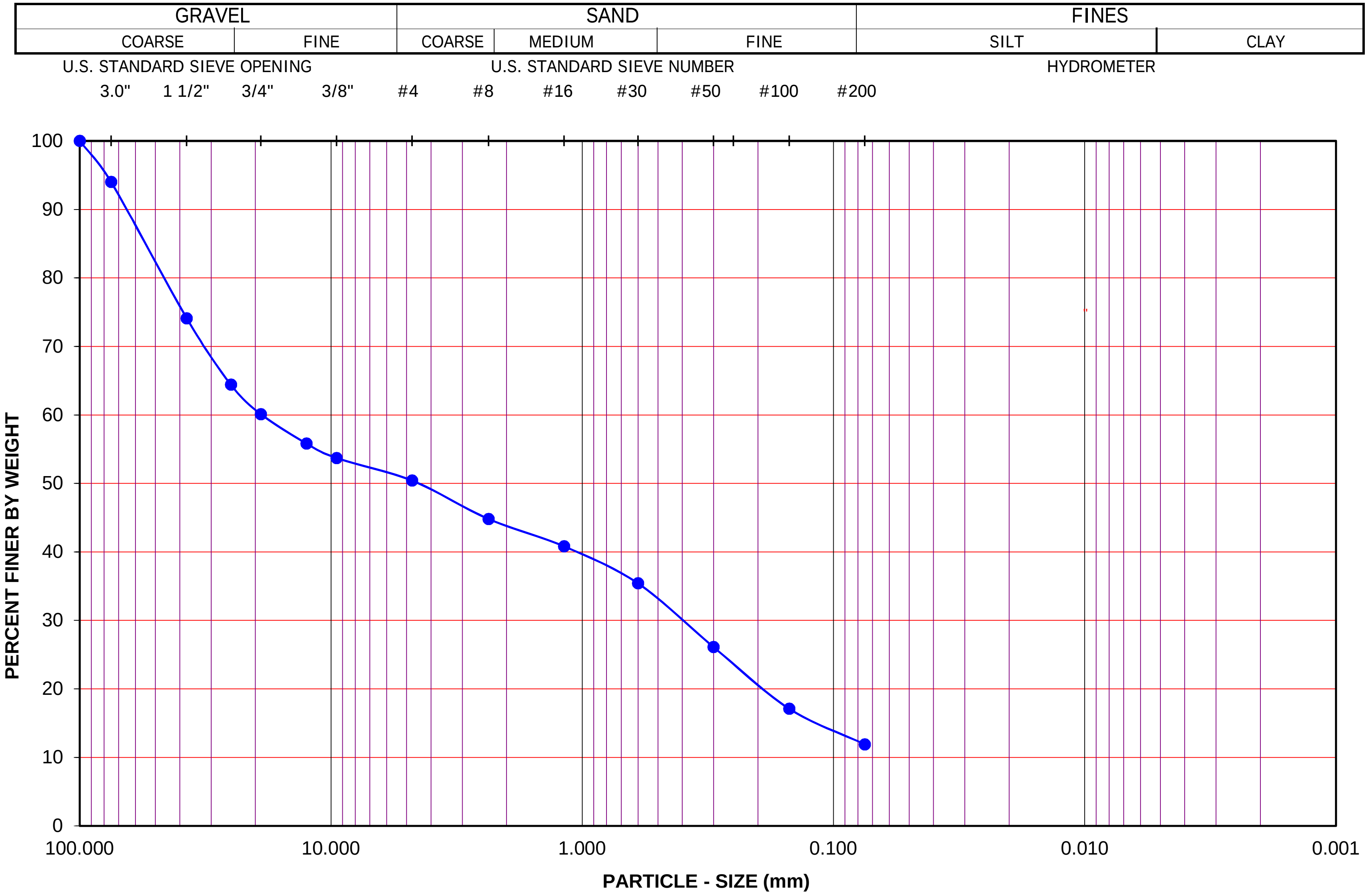
Passing #4 Material After Wet Sieve	Container No.	PH
	Wt. of Dry Soil + Container (g)	608.2
	Wt. of Container (g)	202.6
	Dry Wt. of Soil Retained on # 200 Sieve (g)	405.6

U. S. Sieve Size		Cumulative Weight of Dry Soil Retained (g)		Percent Passing (%)
	(mm.)	Whole Sample	Sample Passing #4	
4"	100.0	0.0		100.0
3"	75.0	989.4		94.0
1 1/2"	37.5	4241.0		74.1
1"	25.0	5837.2		64.4
3/4"	19.0	6546.4		60.1
1/2"	12.5	7251.8		55.8
3/8"	9.5	7585.9		53.7
#4	4.75	8130.1		50.4
#8	2.36		58.7	44.8
#16	1.18		99.4	40.8
#30	0.600		155.6	35.4
#50	0.300		252.8	26.1
#100	0.150		345.9	17.1
#200	0.075		400.0	11.9
PAN				

GRAVEL: **50 %**
SAND: **38 %**
FINES: **12 %**
GROUP SYMBOL: **(GP-GC)s**

Cu = D60/D10 = 351.85
Cc = $(D30)^2/(D60*D10)$ = 0.16

Remarks:



Project Name: Avalon

Project No.: 12181.001

Location: North Slope

Sample No.: B1

Depth (feet): 0-1

Soil Type : (GP-GC)s

Soil Identification: Dark yellowish brown poorly-graded gravel with clay and sand (GP-GC)s

GR:SA:FI : (%) 50 : 38 : 12

Oct-18

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GEOTECHNICAL EXPLORATION
AVALON K-12 SCHOOL SPORTS FIELD AND SITE IMPROVEMENTS
AVALON, SANTA CATALINA ISLAND
LOS ANGELES COUNTY, CALIFORNIA

Prepared for:

LONG BEACH UNIFIED SCHOOL DISTRICT
Facilities Planning & Development
2425 Webster Avenue
Long Beach, California 90810

Project No. 12396.001

September 23, 2019

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

September 23, 2019

Project No. 12396.001

Long Beach Unified School District
Facilities Planning & Development
2425 Webster Avenue
Long Beach, California 90810

Attention: Mr. Elston Soares

Subject: **Geotechnical Exploration**
Avalon K-12 School Sports Field and Site Improvements
Avalon, Santa Catalina Island
Los Angeles County, California

In accordance with the Geotechnical Engineering Services Agreement No. 10533.01 dated May 22, 2019, authorized on June 11, 2019, Leighton Consulting, Inc. performed a geotechnical exploration for the subject Avalon K-12 School sports field and site improvements project, located in Avalon, Santa Catalina Island, Los Angeles County, California.

The purpose of our exploration was to evaluate geotechnical conditions at the site for the planned improvements, and provide geotechnical recommendations for associated design and earthwork construction. The project is considered feasible from a geotechnical standpoint. The results of our exploration and conclusions and recommendations are presented in this report.

Assuming the North Slope will be retained by a soldier pile wall with tie-back anchors, we anticipate moderate to great resistance to be encountered during drilling for the soldier piles and tie-back anchors due to the dense terrace deposits containing gravel, cobbles, and boulders, and the possible presence of hard bedrock in the slope. Difficult to very difficult rippability of the earth materials within the North Slope should be expected during grading.

17781 Cowan ▪ Irvine, CA 92614-6009
949.250.1421 ▪ 949.250.1114 Fax

We appreciate this opportunity to be of service to LBUSD on this project. If you have any questions or if we can be of further service, please contact us at your convenience at ***866-LEIGHTON***, directly at the phone extensions or e-mail addresses listed below.

CERTIFIED ENGINEERING GEOLOGIST
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STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
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Respectfully submitted,

LEIGHTON CONSULTING, INC.

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Distribution: (1) addressee (via e-mail PDF)

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Appendices

1.0 INTRODUCTION

1.1 Site Location and Description

As depicted on Figure 1, *Site Location Map*, Avalon K-12 School is located at 200 Falls Canyon Road in the city of Avalon, California (latitude N33.3392° and longitude W118.3341°). The school campus is bordered by Falls Canyon Road, existing residential properties and a cemetery to the north, City of Avalon maintenance yard to the west, open space to the south, and Avalon Canyon Road to the east. The site for the proposed sports field and site improvements is located in the western portion of the school campus in the area of the existing ballfield. The existing ballfield is relatively flat at an elevation of roughly El. +134 feet mean sea level (msl), and surrounded by ascending slopes to the north and south, and by a descending slope to the east.

1.2 Proposed Improvements

Our understanding of the project is based on review of the *Site Demolition Plan* (Sheet C301) and *Precise Grading Plan* (Sheet C401) for the project prepared by Brandow & Johnston and dated January 18, 2019. The *Site Demolition Plan* (Sheet C301) was used as a base for our Plate 1, *Geotechnical Map*. The project primarily consists of the installation of a new synthetic turf athletic field. The exiting field will be expanded to the north by cutting into the toe of the existing slope and constructing a retaining wall up to 32 feet in height. Retaining walls, up to 13 ½ feet high behind home plate, and up to 4 feet high above the existing drainage ditch along the southern margins of the field, are also proposed. A new restroom/concession building is proposed on the east side of the field with a concrete side and stairs for accessibility. Two handicap parking spaces will be added at the base of the stairs with new asphalt concrete (AC) paving. An underground biofiltration unit for the synthetic turf field drainage system will be installed near the top of the proposed stairs. The overall layout of the existing and proposed site improvements is depicted on Plate 1, *Geotechnical Map*.

Since no specific retaining wall types have been proposed, we assumed a soldier pile wall with tie-back anchors for the North Slope, and conventional cantilever retaining walls for the other site retaining walls in developing our recommendations.

1.3 Purpose and Scope of Exploration

The purpose of our exploration was to: (1) evaluate geotechnical conditions within the general limits of proposed improvements, (2) identify significant geotechnical or geologic issues that may impact the proposed improvements, and (3) provide geotechnical recommendations for design and construction of the proposed improvements. Our exploration included the following tasks:

- **Research**: We reviewed available geotechnical literature, reports and aerial photographs relevant to this site, within our library and/or provided via electronic mail by you. Pertinent geotechnical documents reviewed are referenced at the end of this report text.

- **Field Exploration**: On June 26 and 27, 2019, and included six (6) hand-auger borings (designated HA-1 through HA-6), three (3) test pits and three (3) seismic refraction lines. In addition, percolation testing was performed in three (3) of the hand auger borings.

  Borings: The hand auger borings were drilled, logged and sampled to depths ranging from approximately 1.8 feet to 5 feet below ground surface (bgs). Upon completion of drilling, sampling and logging, borings HA-1, HA-2 and HA-5 were immediately backfilled with soil cuttings. Upon completion of drilling, sampling and logging, borings HA-3, HA-4 and HA-6 were converted to temporary percolation test wells for subsequent percolation testing.

  Percolation Testing: In-situ percolation testing was performed in borings HA-3, HA-4 and HA-6 in general accordance with the County of Los Angeles Department of Public Works (LADPW) *Guidelines for Geotechnical Investigation* and Reporting, *Low Impact Development Stormwater Infiltration* (LADPW, 2017). Refer to the discussion of infiltration rate presented in Section 2.3.1, *Infiltration*. Upon completion of the percolation testing, the well casing was removed from each boring and the borings were backfilled with soil cuttings.

  Test Pits: The test pits were excavated with a track mounted mini-excavator, logged and sampled to depths ranging from approximately 3 feet to 8 feet bgs. Upon completion of excavation, sampling and logging, the test pits were backfilled with soil cuttings.

  Seismic Refraction Lines: The seismic P-wave refraction lines were performed at the site by a subcontracted geophysicist (Southwest Geophysics, LLC). The seismic lines ranged in length from approximately 50 to 100 feet, and were located on the slope north of the existing ballfield.

Approximate locations of hand auger borings, test pits and seismic refraction lines are shown on Plate 1, *Geotechnical Map*. Logs of the borings and test pits, percolation test data and results of the seismic refraction survey are presented in Appendix A, *Field Exploration*.

- **Geotechnical Laboratory Testing**: Geotechnical laboratory tests were performed on selected driven and bulk soil samples obtained during our field exploration. This laboratory testing program was designed to evaluate engineering characteristics of site soils. A description of test procedures and results are presented in Appendix B, *Geotechnical Laboratory Testing*.

- **Engineering and Geologic Analysis**: Data obtained from field explorations and geotechnical laboratory testing was evaluated and analyzed to develop geotechnical conclusions and provide recommendations in accordance with the California Geological Survey (CGS) Note 48 (October 2013 version). Our subsurface interpretations are provided on Figure 2, *Geotechnical Cross Sections A–A' and B-B'*.

- **Report Preparation**: Results of our geologic hazards review and geotechnical exploration are summarized in this report, including our findings, conclusions and preliminary geotechnical design recommendations.

This report does not address the potential for encountering hazardous materials in site soils or groundwater. Important information about limitations of geotechnical reports in general, is presented in Appendix D, *Geoprofessional Business Association (GBA) Important Information About This Geotechnical Report*.

1.4 Previous Study

Leighton performed a limited geotechnical evaluation of the soil and bedrock conditions on the slope north of the ballfield at the site, which included site reconnaissance, geologic mapping and limited geotechnical laboratory testing (Leighton, 2018). The purpose of the study was to preliminarily determine the geotechnical suitability of the material generated from the north slope to be used as backfill during upcoming school modernization activities.

2.0 FINDINGS

2.1 Regional Geologic Setting

The site is located in what is known as the California Continental Borderland Geomorphic Province. This area is typified by elongated northwest and west-trending seafloor ridges and basins. The style of deformation within the province relates to the large-scale transform tectonism and volcanism which was initiated 20 to 24 million years ago coincident with subduction of the Farallon plate off the west coast of California. Since mid-Pliocene, uplift within the province has elevated portions of the borderland 6,000 feet or more. In most recent geologic time, erosion, deposition of alluvium/talus and man's activities have formed the present landscape.

Rocks present within the Borderland Province reflect a history of an active continental margin. The oldest bedrock unit exposed on Catalina Island is the Catalina Schist, thought to form the basement complex of much of the inner borderland (Gath, 1985). These Mesozoic metamorphic rocks are similar in lithology to those of the Franciscan Complex along the California coast. The predominate earth unit mapped at the site and vicinity is quartz diorite of the Catalina Pluton (Bailey, 1940). This early-Miocene pluton is a medium to fine-grained groundmass with larger crystals of altered plagioclase and hornblende. In the vicinity of the site, this unit is gray to gray-green in color and very dense to hard. Along fractures and on exposed surfaces the unit is iron-stained to reddish-orange brown. Phenocrysts within the fine-grained groundmass have been chemically altered on exposed surfaces. This unit forms the steep natural and man-made cut slopes in the vicinity of the site.

The site is located in the lower portion of the Falls Canyon watershed, which is generally an east-west trending canyon that terminates at Avalon Canyon approximately one-half mile southwest of Avalon Bay. Regional geologic maps covering the project site and vicinity, indicate the existing ballfield is underlain by artificial fill (af) overlying Quaternary-age alluvial deposits (Qal). The slope located immediately north of the existing ballfield is mapped to be underlain by Quaternary-age terrace deposits (Qtf), and the slopes to the south of the existing ballfield are mapped to be underlain by Tertiary-age intrusive rocks (Ti). Localized accumulations of surficial deposits such as Quaternary-age slope wash (Qsw), Quaternary-age colluvium (Qcol) and Quaternary-age terrace deposits (Qtf) are also mapped on the slopes south of the existing ballfield. The surficial geologic units mapped in the vicinity of the project site are shown on Figure 3, *Regional Geology Map* (reference unavailable).

2.2 Subsurface Soil and Bedrock Conditions

Based on review of available literature (see references) and our subsurface explorations, the existing ballfield is generally underlain by undocumented artificial fill (af) material, which overlies the Quaternary-age alluvial deposits (Qal) at depth. In addition, the slope located immediately north of the existing ballfield is mapped to be underlain by Quaternary-age terrace deposits (Qtf), which likely overlies Tertiary-age intrusive rocks (Ti) at depth. The slopes located immediately south of the existing ballfield are generally mapped to be underlain by Tertiary-age intrusive rocks (Ti). A description of the geologic units encountered during our exploration and anticipated at depth is provided below.

Artificial Fill, undocumented (af)

Undocumented artificial fill was encountered in all of our exploratory borings (HA-1 through HA-6) and in two of the test pits (TP-2 and TP-3). Our soil borings and test pits did not penetrate the full depth of the undocumented artificial fill. However, based on our understanding of the site, the depth of existing artificial fill within the existing ballfield is expected to vary in thickness up to roughly 20 feet thick in the central-eastern portion of the ballfield. The fill material as encountered primarily consists of light brown to dark brown, slightly moist to very moist, silty sand with variable amounts of gravel, cobbles and occasional boulders. The existing fill materials encountered at the site is likely associated with past grading during construction of the existing site improvements. It is our understanding that a portion of the School Campus was graded around approximately 1960 to construct the ballfield. However, we are not aware of any fill-placement documentation. Therefore, the fill is considered undocumented and should not be utilized to support the proposed improvements in its current condition without limited removal and recompaction as recommended in this report. Localized thicker accumulations of the fill materials may be encountered during future earthwork construction.

Quaternary Alluvium (Qal)

Quaternary alluvial deposits were not encountered in our exploratory borings or test pits, but are expected to exist within Falls Canyon at depth beneath the surface mantle of fill materials within the majority of the existing ballfield area. In general, the alluvial deposits are anticipated to consist of variable amounts of sand, silt, clay, gravel and cobbles and are locally derived from the quartz diorite bedrock and reworked sediments from the Quaternary terrace deposits. The alluvial deposits may be up to 100 feet thick in some portions of the lower canyon.

Quaternary Terrace Deposits (Qtf)

Quaternary terrace deposits are mapped to comprise the slope and mesa north of the existing ballfield, and were encountered in test pit TP-1 in the northern slope. The terrace materials as encountered generally consist of yellow brown to brown, dry to slightly moist, dense, sand, gravel, cobbles and boulders.

Tertiary Intrusive Rocks, quartz diorite (Ti)

Bedrock exposures of Tertiary-age intrusive rocks, specifically quartz diorite, were not observed during our subsurface exploration at the site. However, we anticipate that quartz diorite bedrock may exist at depth within the slope north of the ballfield in the vicinity of the planned retaining wall and beneath the alluvium at depth beneath the ballfield. In addition, quartz diorite bedrock is exposed in the slopes located immediately south of the existing ballfield.

Based on the existing site conditions limiting access for conventional drilling equipment in the vicinity of the proposed retaining wall, the existence and depth of bedrock in the vicinity of the proposed retaining wall north of the ballfield could not be confirmed. However, the results of the seismic refraction survey, specifically seismic lines SL-1 and SL-3 performed perpendicular to the slope face suggest that bedrock may exist as close as 5 to 10 feet from the slope face based on the measured seismic P-wave velocities and the correlation to rippability of the material. It should be noted that bedrock was not encountered in either of the test pits excavated within the lower portions of the slope north of the ballfield. If possible, confirmation of the thickness of terrace deposits and fill debris on the slope face should be confirmed to aid in future retaining wall design.

As a part of our previous study (Leighton, 2018), quartz diorite bedrock outcrops were observed in natural slopes south of the ballfield and in road cuts above the cemetery north of the ballfield. Structural geologic data in the form of joint sets was collected from the road cut exposure of quartz diorite bedrock. The geologic data collected is presented in the following table:

Table 1 – Summary of Structural Geologic Data

Joint Sets		
1	N15°E, 69°W	N66°W, 70°S
2	N3°W, 80°W	N66°W, 75°S
3	N5°E, 85°E	N35°E, 14°W
4	N35°E, 54°W	N35°W, 70°N
5	EW, 78°S	N29°E, 49°N
6	N36°W, 84°N	N50°E, 69°N
7	N12°E, 72°N	N64°W, 53°S

The stratigraphy of the subsurface soils encountered in each soil boring and test pit is presented on the logs included in Appendix A. The general distribution of subsurface conditions beneath the site, extrapolated between boring and test pit locations and interpreted from the seismic refraction lines, is depicted on Figure 2, *Geotechnical Cross-Sections A-A' and B-B'*. The complete laboratory test results performed for this study are included in Appendix B.

2.2.1 Expansive Soil Characteristics

The results of our previous laboratory testing (Leighton, 2018) performed on a representative sample of near surface site soils obtained from the north slope, indicate the near-surface soils generally possess a very low expansion potential (Expansion Index [EI] = 10).

2.2.2 Soil Corrosivity

The results of our previous laboratory testing (Leighton, 2018) performed on a representative sample of near surface site soils obtained from the north slope, indicate the near-surface soils are severely corrosive to buried metals. A summary of previous laboratory test results for corrosivity are presented in Table 2 below.

Table 2 – Summary of Corrosivity Testing Results

(Sample B1 at 0-2 feet bgs; Leighton, 2018)

Sulfate (ppm)	Chloride (ppm)	Resistivity (ohm-cm)	pH
781	61	1,660	7.85

2.2.3 Shear Strength

Evaluation of the shear strength characteristics of the soils included laboratory direct shear testing. The results of testing are included in Appendix B as well as summary graphs that provide values of angle of internal friction (ø) and cohesion (c) for use in geotechnical analysis.

2.2.4 Excavation Characteristics and Rippability

Based on our subsurface explorations performed at the site and our experience in the vicinity of the site, we anticipate the onsite artificial fill and alluvial materials can generally be excavated using conventional excavation equipment in good operating condition.

However, as indicated in Section 1.3, a seismic refraction survey was completed in the slope area north of the ballfield. The seismic refraction survey was performed to evaluate the rippability of the material that underlies the site and to preliminarily identify the depth and presence of bedrock within the slope. Based on the seismic velocity profiles, bedrock with high seismic P-wave velocities exist at depth. The seismic velocities of the materials can be correlated to rippability or hardness, and the high seismic velocities measured in the seismic lines performed in the vicinity of the proposed retaining wall in the slope area north of the ballfield indicate difficult to very difficult rippability, and even the possibility for blasting to be required.

In areas where fill, alluvium or terrace materials are exposed within the planned excavation depths, layers that contain granular, unconsolidated soils with little or no cementation and few fines may be exposed. These materials are prone to cave in or collapse in unshored excavations. See Section 4.2, *Temporary Excavations* for additional information on soil type and excavation characteristics.

2.3 Groundwater

Groundwater was not encountered during our exploration. Based on review of available information, groundwater has not been encountered beneath the project site during previous investigations or pre-remedial confirmation soil sampling. During the installation of debris netting on the School Campus, Mission Geoscience, Inc. drilled to a maximum depth of 45 feet bgs and did not encounter groundwater. These observations are consistent with measured depths to groundwater of approximately 100 feet bgs in nearby Avalon Canyon wells. Groundwater was encountered in a nearby test boring (AV-BR-1) within Avalon Canyon at approximately 100 feet bgs (Oberlander and Dickey, 2015). The groundwater flow direction has not been established at the site; however, it is presumed to follow the topography and alluvial deposits constrained by the canyon and flow east out of Falls Canyon and into Avalon Canyon, which drains toward the Pacific Ocean.

Localized zones of perched water or elevated moisture in near-surface soils due to nearby landscaping and percolation of stormwater runoff, should be expected during grading and foundation excavation phases of construction.

2.3.1 Infiltration

Percolation testing was performed within borings HA-3, HA-4 and HA-6 to evaluate the infiltration characteristics of subsurface soils. The percolation tests were conducted in general accordance with the County of Los Angeles Department of Public Works (LADPW) *Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stormwater Infiltration* (LADPW, 2017). Results of the percolation testing are presented in Appendix A. The test locations are shown on Plate 1, *Geotechnical Map*.

A falling head boring percolation test was performed at test wells HA-3 and HA-6. The infiltration rate for these tests was calculated by dividing the discharge volume by the infiltration surface area over a period of time. The volume of discharge was calculated by adding the total volume of water that dropped within the PVC pipe and within the annulus, and incorporating a reduction factor to account for the porosity of the annulus material. The infiltration surface area was based on the average water height within the test well.

A constant-head test, or high flowrate test, was implemented at test well HA-4 due to the generally favorable percolation characteristics at this location. The infiltration rate was calculated by recording the approximate volume of water delivered to the test zone while maintaining a relatively constant height of water in the well over the testing period. A water source (garden hose from onsite water source) was used to deliver water to the well at a relatively constant rate. The measured infiltration rate was calculated according to the procedure for a high flowrate percolation test, by dividing the total volume of water by the total duration of the test, and dividing by the percolation surface area.

Results of the infiltration testing at HA-3 at a depth of 0 to 1.8 feet bgs indicate a measured infiltration rate of 0.66 inches per hour. Results of the infiltration testing at HA-4 at a depth of 3.9 to 5 feet bgs indicate a measured infiltration rate of 280 inches per hour. Results of the infiltration testing at HA-6 at a depth of 0.2 to 3.6 feet bgs indicate a measured infiltration rate of 1.56 inches per hour. The results of the percolation testing are presented in Appendix A.

Based on the results of the percolation tests, the infiltration characteristics of the site soils are highly variable, which can be expected since the near surface site soils at the ballfield consist of artificial fill. It should be noted that a minimum reduction factor of 2 is recommended to be applied to the measured infiltration rates for HA-3 and HA-6, and a minimum reduction factor of 3 is recommended to be applied to the measured infiltration rate for HA-4. A summary of the infiltration test results is presented in the table below.

Table 3 – Summary of Infiltration Test Results

Test Well	Depth of Test Zone (feet)	Measured Infiltration Rate (in./hr.)	Minimum Reduction Factor	Design Infiltration Rate (in./hr.)
HA-3	0 to 1.8	0.66	2	0.33
HA-4	3.9 to 5	280	3	46
HA-6	0.2 to 3.6	1.56	2	0.78

According to the LADPW Guideline, stormwater infiltration is not allowed in areas that might cause pollutant mobilization. The designer should refer to the LADPW Guidelines for more details.

2.4 Geologic/Seismic Hazards

In general, geologic and seismic hazards include surface fault rupture, seismic shaking, liquefaction, seismically induced settlement, lateral spreading, seismically induced landslides, flooding, seismically-induced flooding, seiches and tsunamis. The following sections discuss these hazards and their potential impacts at the project site.

2.4.1 Surface Fault Rupture

Our review of available in-house literature indicates that no known active faults have been mapped across the site, and the site is not located within a designated Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). Therefore, the potential for surface fault rupture at the site is expected to be low. As such, a surface fault rupture hazard evaluation is not mandated for this site.

The location of the closest active faults to the site was evaluated using the United States Geological Survey (USGS) Earthquake Hazards Program National Seismic Hazard Maps (USGS, 2008). The closest active faults to the site include the Palos Verdes fault, Coronado Bank fault and the Newport-Inglewood fault located approximately 17.3 miles, 24.4 miles and 29.7 miles from the site, respectively. Major regional faults with surface expression located in proximity to the site are shown on Figure 4, *Regional Fault and Historic Seismicity Map*.

2.4.2 Historical Seismicity

Although Southern California has been seismically active during the past 200 years, written accounts of only the strongest shocks survive the early part of this period. Early descriptions of earthquakes are rarely specific enough to allow an association with any particular fault zone. It is also not possible to precisely locate epicenters of earthquakes that have occurred prior to the twentieth century.

A search of historical earthquakes was performed using the computer program EQ Search (Blake, 2018) for the time period between 1800 and 2019. Within that time frame, 346 earthquakes were documented between magnitude 4.00 and 9.0 within a 62-mile (100-kilometer) radius of the site. Of these earthquakes, the closest was an earthquake located 10.8 miles (17.4 kilometers) from the site, and occurred on April 16, 1942 (Appendix C, *Seismicity Data*). Although not precisely located, the epicenter for this earthquake event is located to the east of the project site and registered magnitude 4.0 Mw and induced estimated peak ground acceleration (PGA) of 0.061g at the project site. The largest PGA at the site is estimated to have been roughly 0.100g from the magnitude 6.3 Mw earthquake that occurred in offshore of Huntington Beach and shook the region on March 11, 1933.

The largest earthquake recorded within the search radius was associated with a 7.0 Mw quake occurring on September 24, 1827 with an epicenter 59.5 miles (95.8 km) away from the project site. For a general view of recorded historical seismic activity see Figure 4, *Regional Fault and Historic Seismicity Map*.

Review of additional data available from the Center for Engineering Strong Motion Data (CESMD) website (http://strongmotioncenter.org/) indicates that the highest recorded ground acceleration was 0.007g for a station located approximately 0.4 mile southwest of the project site. The recorded ground acceleration was from the 3.9 Mw San Clemente Island earthquake that occurred on October 24, 2017.

2.4.3 Liquefaction Potential

Liquefaction is often used in reference to the loss of soil strength or stiffness in saturated sandy soils due to increasing pore-water pressure during severe ground shaking. For fine-grained soils (i.e., clays and plastic silts), the term cyclic softening is often used. In general, adverse effects of liquefaction or cyclic softening include excessive ground settlement, loss of bearing support for structural foundations, and seismically induced lateral ground deformations.

Based on review of available information, the project site is not located within a known area that has been identified as being potentially susceptible to liquefaction. In addition, since the depth to groundwater is anticipated to be greater than 50 feet bgs, the potential for liquefaction to occur at the site is considered low.

2.4.4 Seismically Induced Settlement

Strong ground motion during earthquakes tends to rearrange looser soils particles into a more compact arrangement, especially in granular soil deposits. The cumulative effects of soil particles rearrangement during earthquake ground shaking can result in vertical settlement. In general, a poorly graded granular deposit is more susceptible to settlement than a fine-grained or well-graded soil. Based on our evaluation of the site soils using P-wave velocities from seismic refraction, hazards from seismically induced settlement should be low.

2.4.5 Seismically Induced Lateral Ground Displacements

Depending on topography, modes of seismically induced lateral ground displacement, associated with soil liquefaction, consist of ground oscillation (ground slope less than 0.3 percent), lateral spread (0.3 to 5 percent ground slope), or flow failure (ground slope greater than 5 percent). Since the potential for liquefaction to occur at the site is low, the potential for seismically induced lateral ground displacement(s) to occur at the site is also low.

2.4.6 Slope Stability and Seismically Induced Landslides

The site is not located within a known area that has been identified as being potentially susceptible to seismically induced landslides. However, the existing slope located north of the ballfield is at an inclination of approximately 1.3:1 (horizontal:vertical). We evaluated the stability of the existing North Slope for static and pseudostatic conditions, and the results indicate the slope is generally stable. Results of our analyses are presented in Appendix D, Slope Stability Analysis. Proposed slopes, if any, should be engineered and constructed at a gradient of 2:1 (horizontal:vertical) or flatter.

2.4.7 Flooding

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2008), the project site is located within a flood hazard area identified as “Zone X”, which is defined as an area of minimal flood hazard. Regionally, storm runoff flow is generally directed to the southeast towards Avalon Canyon. As shown on Figure 5, *Flood Hazard Zone Map*, the site is not located within a flood hazard zone.

Earthquake-induced flooding can be caused by failure of dams or other water-retaining structures as a result of earthquakes. The project site is not located within a known flood impact zone from failure of a regional dam or reservoir.

2.4.8 Seiches and Tsunamis

Seiches are large waves generated in very large enclosed bodies of water or partially enclosed arms of the sea in response to ground shaking. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. Based on the inland location and topographic configuration of the site, the potential for tsunami inundation of the site is not considered a hazard.

2.4.9 Subsidence

Subsidence is defined as a sinking of the Earth's surface in response to geologic or man-induced causes. Subsurface solution of limestone during cave formation may lead to a series of subsidence features at the ground surface, which, collectively, are termed karst topography. Since the site is not underlain by limestone, the potential for subsidence to affect the site due to this condition is not a consideration for the project.

2.4.10 Methane

Based on review of State of California Division of Oil and Gas Geothermal Resources (DOGGR) records, the project site is not located within a known methane hazard zone or documented oil field (DOGGR, 2019). Based on these findings, the potential for methane hazard at the site appears to be low.

3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 Conclusions

Based upon our geotechnical exploration and evaluation, the design and construction of the proposed improvements is considered feasible from a geotechnical engineering perspective. We anticipate moderate to great resistance to be encountered during drilling for the soldier piles and tie-back anchors for the proposed retaining wall on the North Slope due to the high P-wave velocities obtained from the seismic refraction. Difficult to very difficult rippability of the earth materials within the North Slope should be expected during grading.

Proposed shallow foundations, concrete slab-on-grades, hardscape, and pavements should be supported on a layer of new engineered fill at least 1 foot thick.

This site is **not** located within a currently designated Alquist-Priolo Earthquake Fault Zone for surface fault rupture. However, as is the case for most of Southern California, strong ground shaking has and will occur at this site.

3.2 Earthwork

Earthwork is generally expected to include site grading, subgrade preparation for proposed shallow spread footings and slabs-on-grade. Proposed shallow spread footings, slabs-on-grades, hardscape, and pavements should be supported on at least 1 foot of new engineered fill. We anticipate the majority of the on-site soils will be suitable for use as engineered fill.

3.2.1 Earthwork Observation and Testing

Leighton Consulting, Inc. should observe and test all grading and earthwork, to check that the site is properly prepared, the selected fill materials are satisfactory, and that placement and compaction of fills has been performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is essential. A bulk sample of any imported soil or aggregate material should be submitted to the Leighton Consulting, Inc. geotechnical laboratory at least two working days in advance of earth material placement and compaction. Project plans and specifications should incorporate recommendations contained in the text of this report.

Variations in site conditions are possible and may be encountered during construction. To confirm correlation between soil data obtained during our field and laboratory testing and actual subsurface conditions encountered during construction, and to observe conformance with approved plans and specifications, it is essential that we are retained to perform continuous or intermittent review during earthwork, excavation and foundation construction phases. Therefore, conclusions and recommendations presented in this report are contingent upon us performing construction observation services.

3.2.2 Surface Drainage

Water should not be allowed to pond or accumulate anywhere except in detention basins set back at least 25 feet from structures. Pad drainage should be designed to collect and direct surface water away from structures to approved drainage facilities. Hardscape drains should be installed and drain to storm water disposal systems. Drainage patterns and drainpipes approved at the time of fine grading should be maintained throughout the life of proposed structures. Irrigation and/or percolation should not be allowed for at least 25 feet horizontally around any proposed building.

3.2.3 Site Preparation and Overexcavation

Prior to grading, all areas to receive structural fill, engineered structures, and pavements should be cleared of surface and subsurface obstructions, including any existing debris, over-sized material greater than 8 inches in maximum dimension, loose, compressible, or unsuitable soils, and stripped of vegetation. Removed vegetation and debris should be properly disposed off-site.

All areas of proposed slabs-on-grades, hardscape, pavements, and shallow foundations should be overexcavated a minimum of 1 foot below the proposed subgrade elevations. The overexcavation should extend laterally a minimum of 2 feet beyond the limits of the proposed improvements. The exposed excavation bottoms should be observed by Leighton Consulting, Inc. to check that competent soils are exposed prior to processing of the bottoms. All overexcavation bottoms should be processed by scarification to a minimum depth of 6 inches, moisture conditioned to 0 to 2 percent above optimum moisture content, and recompacted to at least 90 percent relative compaction based on ASTM Test Method D1557.

The subgrade for the proposed synthetic turf field should be prepared in accordance with the manufacturer's specifications, or at the very least, the subgrade should be proof rolled by heavy, rubber-tired construction to identify any loose or soft areas. The proof rolling should be observed by Leighton. Any soft or loose soil should be processed and compacted.

If overly wet, soft, and unstable soils are encountered at the subgrade elevations or bottoms of excavations, the wet soil should be scarified and processed to dry. Alternatively, the wet soil may be excavated and replaced with a layer of crushed rock that is completely wrapped with a non-woven geotextile such as Mirafi N140 (or approved equal). The gradation of the crushed rock and the thickness of the crushed-rock layer should be evaluated during construction.

3.2.4 Engineered Fill

Onsite soils free of organics, debris and oversized material (greater-than 8-inches in largest dimension) are suitable for use as compacted structural fill. However, any soil to be placed as fill, whether onsite or imported material, should be first viewed by Leighton Consulting, Inc., and then tested if and as necessary, prior to approval for use as compacted fill. All structural fill should be free of hazardous materials.

All fill soil should be placed in thin, loose lifts not exceeding approximately 6 inches, moisture-conditioned, as necessary, to 0 to 2 percent above optimum moisture content, and compacted to a minimum 90% relative compaction as determined by ASTM D 1557 standard test method (modified Proctor compaction curve).

3.2.5 Pipeline Backfilling

Pipeline trenches should be backfilled with compacted fill in accordance with this report, and applicable Standard Specifications for Public Works Construction (Greenbook), 2015 Edition standards.

3.3 Seismic Design Parameters

To accommodate effects of ground shaking produced by regional seismic events, seismic design can, at the discretion of the designing Structural Engineer, be performed in accordance with the 2016 Edition of the California Building Code

(CBC). Table 4, *2016 CBC Seismic Parameters* (below), lists seismic design parameters based on the 2016 CBC methodology:

Table 4 - 2016 CBC Seismic Design Parameters

Categorization/Coefficients	
Site Longitude (decimal degrees) West	-118.3341
Site Latitude (decimal degrees) North	33.3392
Site Class Definition	D
Mapped Spectral Response Acceleration at 0.2s Period, S_s	0.867 g
Mapped Spectral Response Acceleration at 1s Period, S_1	0.332 g
Short Period Site Coefficient at 0.2s Period, F_a	1.0
Long Period Site Coefficient at 1s Period, F_v	1.5
Site Modified Peak Ground Acceleration, PGA_M	0.589g
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	1.000 g
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	0.577 g
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	0.666 g
Design Spectral Response Acceleration at 1s Period, S_{D1}	0.384 g
Seismic Design Category (S_1<0.75)	D

3.4 Shallow Foundations

The proposed structural elements may be supported by shallow foundations consisting of strip footings, isolated square footings, or both. Overexcavation and recompaction of subgrade soils for shallow foundations should be performed as detailed in Section 3.2.

3.4.1 Minimum Embedment and Width

Strip and square footings should be at least 12 inches wide. The footings should have a minimum embedment of 12 inches below the lowest adjacent grade.

3.4.2 Allowable Bearing Pressure

An allowable net bearing pressure of 2,000 pounds-per-square-foot (psf) may be used for design of footings. This value is based on the minimum embedment depth and width above. The allowable bearing pressure may be increased by 500 psf for every additional foot of width, and 1,000 psf for every additional foot of embedment beyond the minimum dimensions. The maximum allowable bearing pressure should not exceed 4,000 psf. The allowable bearing pressure are for total dead load and sustained live loads, and can be increased by one-third when considering short-duration wind or seismic loads. Footing reinforcement should be designed by the structural engineer.

3.4.3 Lateral Load Resistance

Soil resistance available to withstand lateral loads on a shallow foundation is a function of the frictional resistance along the base of the footing and the passive resistance that may develop as the face of the structure tends to move into the soil. The frictional resistance between the base of the foundation and the subgrade soil may be computed using a coefficient of friction of 0.40. The passive resistance may be computed using an equivalent fluid pressure of 350 pounds-per-cubic-foot (pcf), assuming there is constant contact between the footing and undisturbed soil. These friction and passive values are ultimate values. For spread footings and slabs-on-grade bearing on engineered fill, full friction and passive resistance can be combined to resist lateral loads; although some lateral displacement is required to mobilize full passive resistance.

3.4.4 Settlement Estimates

For the recommended allowable bearing pressure, total settlement due to static loading is expected to be less than 1. Differential settlement is expected to be less than approximately ½ inch. The estimated differential settlement is assumed to be over a horizontal distance of 30 feet between two similarly loaded footings.

3.5 Stability of Slopes

We performed slope stability analyses using the computer program SLIDE (version 8.023) by Rocscience, Inc. Both static and pseudostatic conditions were considered. SLIDE calculates the factor of safety against instability of a slope by the method of slices according to simplified Janbu, Bishop or Spencer methods. The input parameters for the program include slope geometry, soil stratigraphy, soil unit weight, and soil shear strength. A uniform surcharge pressure of 300 psf (Section A-A') and 250 psf (Section B-B') were also used as input parameters. The program allows a number of random failure surfaces to be generated during search routines to assist in estimating the critical (least) factor of safety for the modeled slope conditions.

In the pseudostatic analysis, the seismic force is modeled as a horizontal force equivalent to the weight of the sliding mass times a horizontal seismic coefficient expressed as a percent of gravity (g). In effect, the dynamic effects are replaced by a static force, and the approach therefore is termed the pseudostatic method of analysis. A horizontal seismic coefficient of 0.15 was used in the analyses.

Our static and pseudostatic analyses for the global stability of the approximately 1.5:1 (H:V) eastern slope (Section B-B') are shown in the figures presented in Appendix D, Slope Stability Analysis. The minimum factors of safety obtained from the analyses are presented in Table 5 below. Generally, factors of safety greater than or equal to 1.5 and 1.1 are acceptable for static and pseudostatic conditions, respectively.

Limit equilibrium analyses performed for the proposed retaining wall on the North Slope (Section A-A') evaluated the required reaction forces for the static and pseudostatic conditions and aided our development of lateral earth pressures. The results are also presented in Appendix D.

Table 5 – Slope Stability Analysis Results for Section B-B'

Condition	Estimated Minimum Factor of Safety
Static	2.38
Pseudostatic	1.83

3.5.1 <u>Grading of Slopes</u>

Cut and fill slope inclinations should not exceed 2:1 (H:V). Fill slopes should be constructed by overfilling and trimming back to provide a firm, well compacted slope face. Fill slopes should have a keyway at the toe of the slope and be benched into suitable soil. Keyway and benching recommendation details are presented in Standard Details A of Appendix E, General Earthwork and Grading Specifications. Engineered fill should be placed and compacted according to the recommendations in Section 3.2.5 of this report. Prior to placing engineered fill, the exposed surface of keyway and benching excavations should be observed by Leighton

Consulting, Inc. Excavations may have to be deepened if unsuitable earth materials are exposed.

3.5.2 Slope Erosion and Drainage

The soils at the site are considered to be moderately erodible. Therefore, slopes should be inspected periodically for erosion and repaired immediately if erosion is detected. To minimize erosion, all disturbed areas should be planted with erosion-resistant vegetation suited to the area. As an alternative, jute netting or geotextile erosion control mats can be installed per the manufacturer's recommendations.

Paved interceptor drains should be provided along the tops of slopes where tributary area flows toward the slope, and should be cleaned before the start of each rainy season. The interceptor drains should be sloped to a suitable drainage device and disposed off-site.

3.6 Cantilever Retaining Wall Design

3.6.1 Design Static Lateral (Horizontal) Earth Pressures

We recommend that retaining walls be backfilled with non-expansive sands (EI≤30). For drained retaining walls with level sand backfill, the following parameters may be used for retaining wall design:

Table 6 - Retaining Wall Design Earth Pressures

Retaining Wall Condition	Equivalent Fluid Pressure (pounds-per-cubic-foot)*
Active, Level Backfill	37
Active, 2:1 Backfill	60
Active, 1.5:1 Backfill	86
Passive Resistance	390

*Only for properly drained backfill

Cantilever walls that are designed to yield at least 0.001H, where H is equal to the wall height, may be designed using the active condition. Passive pressure is used to compute soil resistance to lateral structural movement.

Total depth of retained earth for design of walls and for uplift resistance should be measured as the vertical height of the stem below the ground surface at the wall face for stem design, or measured at the heel of the footing for overturning and sliding. A total unit weight of 120 pounds-per-cubic-foot (pcf) may be assumed to calculate weight of compacted fill soil over wall footings, if properly compacted and drained.

3.6.2 Retaining Wall Surcharges

In addition to the above lateral forces due to retained earth, surcharge due to above grade loads on the wall backfill, such as traffic, should be considered in design of retaining walls. Vertical surcharge loads behind a retaining wall on or in backfill within a 1:1 (horizontal:vertical) plane projection up and out from the retaining wall toe, should be considered as lateral and vertical surcharge. Unrestrained (cantilever) retaining walls should be designed to resist one-third of these surcharge loads applied as a uniform horizontal pressure on the wall.

In areas where autos and pickup trucks will drive, we suggest assuming a uniform vertical surcharge of 300 psf, which would result in active and at-rest horizontal surcharges of 85 psf and 135 psf, respectively. This should be doubled in areas of heavy construction traffic (such as concrete trucks, heavy equipment delivery-trucks, etc.). If crane outrigger loads or other point load sources are applied as wall surcharge, this will require additional analyses based on load source and location relative to the wall.

3.7 Tied-Back Retaining Wall Design

Tied-back retaining walls should be designed to resist a trapezoidal distribution of lateral earth pressure. The lateral pressure in the upper 0.2H and lower 0.2H portions of the wall height, H, should transition linearly to 410H, maximum, and decrease linearly from 410H to zero, respectively. The maximum lateral pressure is 410H in psf where H is height of the wall in feet. Any surcharge should be applied in accordance with the previous section of this report, Section 3.6.2.

3.7.1 Permanent Soldier Piles

Permanent soldier piles can be used where there is limited space for spread footings and temporary backcuts. Soldier piles typically consist of steel H-beams set in predrilled holes and backfilled with structural concrete below the proposed lowest excavation level; then with lean-mix concrete (or structural concrete) for the exposed wall segment to the top-of-wall ground surface. Some form of lagging or facing concrete between soldier piles is expected to be required. Soldier piles may be assumed to have a passive resistance below the lowest adjacent excavation (bottom of footings) of 780 pounds-per-square-foot (psf), per foot of embedment of the soldier pile encased in concrete in firm contact with undisturbed terrace deposits. This passive pressure should not exceed 15,000 psf, and is based on the assumption that soldier piles will be spaced at least three diameters on center.

The downward component of a tie-back anchor load transferred to the soldier pile may be supported by frictional resistance between the soldier piles and the retained earth, and the skin friction of the pile shaft below finished excavation grade. The coefficient of friction between the soldier piles and the retained earth may be taken as 0.40 times the horizontal component of anchor load. The allowable downward capacity of a soldier pile below the excavated level may be estimated using an allowable skin friction of 64 psf per foot embedment below the bottom of the finished excavation.

Cast in drilled shaft soldier piles should be constructed in accordance with Section 205-3.3.2 of the Standard Specifications for Public Works Construction (Greenbook), 2015 Edition. Drilled shafts should be filled with concrete on the same day drilled, and under no circumstances should shafts be left open overnight. If water is encountered in shafts, then it should be pumped out, and a “tremie” or concrete pump hose used to place concrete to the bottom of the shaft.

The contractor may choose to evaluate potential for difficult drilling conditions and caving of soldier pile shafts by drilling pilot holes with the intended production drilling equipment. Different equipment may result in different drilled shaft conditions. Casing should be available on site during drilling for rapid use if caving conditions are encountered.

3.7.2 Permanent Wall Surfaces (Lagging) Between Soldier Piles

For temporary construction, wood lagging is typically placed between soldier piles to reduce caving and pop-outs. However, wood lagging is rarely used for permanent earth retaining structures, since wood in contact with soil lacks long-term durability. Pre-cast-concrete panes can also be used. But permanent wall surfaces between soldier piles typically consist of pneumatically-sprayed (air-placed gunite or shotcrete) concrete applied to previously-placed steel reinforcement connected to soldier piles. The pneumatically-sprayed (gunite or shotcrete) concrete section has been renamed “Air-Placed Concrete” in Section 303-2 of the 2015 Edition of the *Standard Specifications for Public Works Construction* (Greenbook). For proper corrosion resistance, all reinforcing steel should have at least 3-inches of concrete cover where concrete is cast/sprayed against earth materials.

Earth pressures between soldier piles spaced less-than-or-equal-to (≤) 8 feet on-center are modest to negligible due to soil arching between soldier piles. Negligible static earth load should be assumed within undisturbed bedrock (Ti) between soldier piles spaced no-more-than (≤) 8 feet on-center (assuming 24-inch-diameter soldier piles resulting in 6-feet of exposed earth material, or less, between soldier piles concrete). However, in terrace deposits, active earth pressures should be considered capped at 400 psf.

3.7.3 Permanent Retaining Wall Tie-Back Anchors

At least one row of tieback anchors or other bracing will likely be required for excavations exceeding approximately 15 feet in height. Un-bonded portions of tieback anchors should extend to the assumed failure plane. The failure plane is an imaginary line extending up from the bottom of the excavation at a minimum inclination of 30^o from the vertical. Anchors should be installed starting at no more than 8 feet below the surface of the adjacent retained soils and should be at inclinations between 15^o and 45^o down from the horizontal. Locations and angles of anchors may need to be adjusted to clear existing foundations and utilities. Bond resistance on tieback anchors may be assumed to be 60H pounds-per-square-foot (psf), where H is the average depth of the tieback anchor portion (beyond the failure plane) in feet, but not to exceed 900 psf in terrace deposits. Tie-back anchor design skin friction values can be increased for pressure-injected (grouted) anchors, at the risk of the design/builder, as a function of the means and methods of anchor installation (proprietary or otherwise).

All of the production anchors should be tested to at least 150% of the design load; the total deflection during the tests should not exceed 12 inches. The rate of creep under the 150% test should not exceed 0.1 inch over a 15 minute period for the anchor to be approved for the design loading.

After a satisfactory test, each production anchor should be locked-off at the design load. The lock-off load should be verified by rechecking the load in the anchor. If the lock-off load varies by more than 10% from the design load, the load should be reset until the anchor is locked-off within 10% of the design load.

To reduce chances of caving during tie-back testing, the portion of the anchor shafts within the failure wedge may need to be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill may contain a small amount of cement to allow the sand to be placed by pumping.

Tieback anchors typically extend beneath adjacent off-site properties. Written approval by the owners of these properties should be obtained prior to preparing shoring plans. Tieback anchors should be designed to have a sufficient inclination and depth to avoid existing subterranean structures and/or underground utilities. The shoring engineer, prior to preparing shoring plans, should be provided with an updated plan showing locations and elevations of structures and utilities that could be affected.

Since permanent tie-backs may be proposed, a robust corrosion resistant design is essential. All threadbars, strands and conventional reinforcing steel must have at least three-inches of concrete cover. Care should be taken to make sure strands and/or threadbars are centered in concrete tie-back anchors. Mechanical strand/wedge and DYWIDAG bar connections should be epoxy coated and/or encased in grease caps. Combinations of epoxy coating and galvanized steel can also be used.

3.7.4 Retaining Wall Incremental Seismic Lateral Loads

For retaining walls less-than (<) 12 feet in height with level retained ground behind the wall, incremental seismic loads need not be considered. However, for walls more than 12 feet in height or with ascending sloped retained ground, an incremental seismic load of 172 pounds per foot (uniform distribution) should be used for retaining wall design.

3.7.5 Retaining Wall Drainage

Adequate drainage should be provided for soldier pile walls consisting of weep holes between soldier piles, which drain geocomposite “chimney” drains behind reinforced concrete facing. Miradrain, Enkadrain or similar drainage geocomposites should be placed halfway between soldier piles for the full wall height except the top one foot, as a “chimney” drain, before air-placed concrete is placed or other lagging/surfacing. One chimney drain product can be found here:

http://www.conteches.com/Products/Erosion-Control/Geocomposites/C-Drain-Geocomposite

3.7.6 Construction-Phase Deflection Monitoring

Soldier piles should be monitored weekly for line and grade, surveyed by a California licensed Professional Land Surveyor (PLS) until all earthwork and wall construction has been completed. Survey results must be sent to Leighton Consulting, Inc., weekly, preferably by e-mail. If total horizontal deflection inward (towards the excavation) exceeds one-inch, then excavation adjacent to excessively-deflecting soldier pile(s) should be halted immediately, and shoring design at that location should be reevaluated by the shoring designer, owner and Leighton Consulting, Inc. Any movement more than one inch will require remedial shoring at the location of excessive deflection, to prevent additional movement prior to further construction in that area.

3.8 Concrete Slab-On-Grade

Concrete slabs-on-grade should be designed by the structural engineer in accordance with 2016 CBC requirements for soils with a low expansion potential. More stringent requirements may be required by the structural engineer and/or architect; however, slabs-on-grade should have the following minimum recommended components:

- **Subgrade:** Floor slabs-on-grade and adjacent concrete flatwork should be underlain by at least 2 feet of relatively non-expansive soil (EI<60). Slab-on-grade subgrade soil should be moisture conditioned to or within 2% **over** optimum moisture content, to a minimum depth of 24 inches within building footprints, and compacted to 90% of the modified Proctor (ASTM D 1557) laboratory maximum density prior to placing either a moisture barrier, steel and/or concrete.

- **Moisture Barrier:** A moisture barrier consisting of at least 15-mil-thick Stego-wrap vapor barriers (see: http://www.stegoindustries.com/products/stego_wrap_vapor_barrier.php), or equivalent, should then be placed below slabs where moisture-sensitive floor coverings or equipment will be placed.

- **Reinforced Concrete**: A conventionally reinforced concrete slab-on-grade with a thickness of at least 4 inches should be placed in pedestrian areas without heavy loads. Reinforcing steel should be designed by the structural engineer, but as a minimum should be No. 3 rebar placed at 18-inches on-center, each direction (perpendicularly), mid-depth in the slab. A modulus of subgrade reaction (k) as a linear spring constant, of 75 pounds-per-square-inch per inch deflection (pci) can be used for design of heavily loaded slabs-on-grade, assuming a linear response up to deflections on the order of ¾-inch.

- **Slab-On-Grade Control Joints**: Slab-on-grade crack control joint locations and spacing should be designed by the project structural engineer. However, consideration should be given to potential for differential-vertical-offset at control joints, due to subgrade expansion/shrinkage. Where possible, slabs-on-grade should be allowed to "float" on the subgrade to allow for differential vertical expansion/shrinkage of the subgrade. Interior full-depth joints at wall and column interfaces are recommended to allow the slab-on-grade to "float" unrestrained by vertical structural components. However, doweling is recommended at other joints in open areas of rooms to avoid trip hazards.

Minor cracking of concrete after curing due to expansion, drying and shrinkage is normal and should be expected. However, cracking is often aggravated by a high water-to-cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. The use of low-slump concrete or low water/cement ratios can reduce the potential for shrinkage cracking.

3.9 Sulfate Attack and Ferrous Corrosion Protection

Water-soluble sulfates in soil can react adversely with concrete. As referenced in the 2016 California Building Code (CBC), Section 1904A, concrete subject to exposure to sulfates shall comply with requirements set forth in ACI 318. Based on results of laboratory testing, concrete structures in contact with the onsite soil will have "negligible" exposure to water-soluble sulfates in the soil. Therefore, common Type II Portland cement may be used for concrete construction in contact with site soils. Import fill soils should be tested for corrosivity and sulfate attack before import to the site.

One bulk soil sample was tested for corrosivity (Leighton, 2018). Results indicated a minimum electrical resistivity of 1,660 ohm-cm. Based on these test results, onsite soil is considered severely corrosive to ferrous metals. Therefore, ferrous pipe buried in moist to wet site earth materials should be avoided by using high-density polyethylene (HDPE), polyvinyl chloride (PVC) and/or other non-ferrous pipe when possible. Ferrous pipe can also be protected by polyethylene bags, tap or coatings, di-electric fittings or other means to separate the pipe from on-site soils.

3.10 Preliminary Pavement Sections

Based on design procedures outlined in the current Caltrans *Highway Design Manual* and an assumed design R-value of 35 for silty sand subgrade, preliminary flexible pavement sections were calculated for the Traffic Indices (TIs) tabulated, and are listed below. The pavement subgrade should be observed by the geotechnical engineer or his representative during construction to see if the assumed R-value is valid for the soils exposed. An R-value test may be needed if the exposed subgrade soil significantly differs from the assumed, and if so, the preliminary pavement sections will need to be revised.

Table 7 - Hot Mixed Asphalt (HMA) Pavement Sections

Assumed Traffic Index	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
4.0 (automobile parking)	3	3.5
5.0 (driveways and truck traffic)	3	5
6.0 (roadways and heavy truck traffic)	3.5	6.5

For fire truck (60,000-pound “apparatus”) lanes, asphalt pavements designed for a TI=6.0 are recommended. However, note that undistributed apparatus outrigger loads could cause local asphalt pavement punching damage. When possible, outrigger loads should be distributed over asphalt pavements with planks and plywood. Otherwise, areas where outrigger loads are anticipated could be paved with 8-inch-thick concrete as described below.

Portland cement concrete pavement sections were calculated in accordance with procedures developed by the Portland Cement Association. Concrete paving sections for three Traffic Indices (TIs) are presented below:

Table 8 - Portland Cement Concrete Pavement Sections

Assumed Traffic Index	PC Concrete (inches)	Base Course (inches)
4.0 (automobile parking)	6	4
5.0 (driveways and truck traffic)	6.5	
6.0 (roadways and heavy truck traffic)	7	

We have assumed that this Portland cement concrete will have a compressive strength of at least 3,000 pounds-per-square-inch (psi). Prior to placement of aggregate base, subgrade soils should be scarified to a minimum depth of 6 inches, moisture-conditioned, as necessary, and recompacted to a minimum of 90 percent relative compaction, determined in accordance with ASTM D 1557 modified Proctor laboratory maximum density. Aggregate base should be placed in thin lifts; moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction. Field observation and periodic testing, as needed during placement of base course materials, should be undertaken to ensure that requirements of Caltrans' *Standard Specifications* (2015) and Special Provisions are fulfilled. Consideration should be given to reinforce concrete pavements where large outrigger point loads are anticipated.

Adequate drainage (both surface and subsurface) should be provided such that the subgrade soils and aggregate base materials are not allowed to become wet. All pavement construction should be performed in accordance with the Caltrans *Standard Specifications* (2015). Recommended structural pavement materials should conform to the specified provisions in the Caltrans *Standard Specifications* (2015) including grading and quality requirements, shown below:

- **Asphalt Concrete (Hot Mixed Asphalt)** for pavement should be Type A and should conform to Section 39 of the *Standard Specifications*. Asphalt concrete specimens should be tested for surface abrasion in accordance with CT-360.
- **Portland Cement Concrete (PCC)** pavement should conform to Section 40 of the *Standard Specifications*. PCC pavement materials (pavement, structures, minor concrete) should conform to Section 90 of the *Standard Specifications*.

- **Class II Aggregate Base (AB)** should conform to Section 26 of the *Standard Specifications*.

Traffic Indices (TIs) used in our pavement design are considered reasonable values for typical parking lot areas, and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Irrigation adjacent to pavements, without a deep curb or other cutoff to separate landscaping from the paving, will result in premature pavement failure. Traffic parameters used for design were selected based on engineering judgment and not on information furnished to us such as an equivalent wheel-load analysis or a traffic study.

4.0 CONSTRUCTION CONSIDERATIONS

4.1 Retaining Wall Construction

Based on the limited subsurface information that was obtained during our subsurface exploration directly at the planned retaining wall location and elevation due to access constraints, it is recommended that the contractor selected for construction perform their own subsurface exploration at the planned wall location to confirm subsurface conditions anticipated during construction. We anticipate that hard bedrock and difficult to very difficult excavation conditions will likely be encountered during grading and wall construction within the North Slope. It is important that a contractor with excavation experience in similar conditions should be consulted for the proper excavation methodology, equipment, and production rate based on the findings of this report that should be confirmed with additional subsurface exploration.

4.2 Temporary Excavations

All temporary excavations, including utility trenches, retaining wall excavations, and foundation excavations should be performed in accordance with project plans, specifications, and all OSHA requirements. Excavations 4 feet or deeper should be laid back or shored in accordance with OSHA requirements before personnel are allowed to enter.

No surcharge loads should be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater from the top of the cut, unless the cut is shored appropriately. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site foundation should be properly shored to maintain support of the adjacent structure.

Temporary excavations should be treated in accordance with the State of California version of OSHA excavation regulations, Construction Safety Orders for Excavation General Requirements, Article 6, Section 1541, effective October 1, 1995. The sides of excavations should be shored or sloped in accordance with OSHA regulations. OSHA allows the sides of unbraced excavations, up to a maximum height of 20 feet, to be cut to a ¾H:1V (horizontal:vertical) slope for Type A soils, 1H:1V for Type B soils, and 1½H:1V for Type C soils. Near-surface onsite sandy soils are to be considered Type C soils which are subject to collapse in shallow unbraced excavations (i.e. approximately 3-feet in vertical height).

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor shall be responsible for providing the “competent person” required by OSHA standards to evaluate soil conditions. Close coordination between the competent person and the geotechnical engineer should be maintained to facilitate construction while providing safe excavations.

4.3 Temporary Shoring

Temporary cantilever shoring can be designed based on the active equivalent fluid pressure of 37 pounds-per-cubic-foot (pcf) in alluvium. If excavations are braced at the top and at specific depth intervals, then braced earth pressure may be approximated by a uniform rectangular soil pressure distribution. This uniform pressure expressed in pounds-per-square-foot (psf), may be assumed to be 28 multiplied by H for design, where H is equal to the depth of the excavation being shored, in feet. These recommendations are valid only for trenches not exceeding 15 feet in depth at this site.

4.4 Geotechnical Services during Construction

Our geotechnical recommendations provided in this report are based on information available at the time the report was prepared and may change as plans are developed. Additional geotechnical exploration, testing and/or analysis may be required based on final plans. Leighton Consulting, Inc. should review site grading, foundation and shoring (if any) plans when available, to comment further on geotechnical aspects of this project and check to see general conformance of final project plans to recommendations presented in this report.

Leighton Consulting, Inc. should be retained to provide geotechnical observation and testing during excavation and all phases of earthwork. Our conclusions and recommendations should be reviewed and verified by us during construction and revised accordingly if geotechnical conditions encountered vary from our findings and interpretations. Geotechnical observation and testing should be provided:

- During all grading excavation and fill operations,
- During compaction of all fill materials,
- During drilling of foundation piles,
- After excavation of all footings and prior to placement of concrete,
- During utility trench backfilling and compaction,
- During pavement subgrade and base preparation, and/or
- If and when any unusual geotechnical conditions are encountered.

5.0 LIMITATIONS

This report was necessarily based in part upon data obtained from a limited number of observances, site visits, soil samples, tests, analyses, histories of occurrences, spaced subsurface explorations and limited information on historical events and observations. Such information is necessarily incomplete. The nature of many sites is such that differing characteristics can be experienced within small distances and under various climatic conditions. Changes in subsurface conditions can and do occur over time. This exploration was performed with the understanding that this subject site is proposed for development as described in Section 1.2 of this report. Please also refer to Appendix F, GBA's *Important Information About This Geotechnical Report*, presenting additional information and limitations regarding geotechnical engineering studies and reports.

Until reviewed and accepted by the California Geological Survey (CGS), this report may be subject to change. Changes may be required as part of the CGS review process. Leighton Consulting, Inc. assumes no risk or liability for consequential damages that may arise due to design work progressing before this report is reviewed and accepted by CGS.

This report was prepared for LBUSD based on their needs, directions and requirements at the time of our exploration, in accordance with generally accepted geotechnical engineering practices at this time in California for public schools. This report is not authorized for use by, and is not to be relied upon by, any party except LBUSD and their design and construction management team, with whom Leighton Consulting, Inc. has been contracted by for this work. Use of or reliance on this report by any other party is at that party's risk. Unauthorized use of or reliance on this report constitutes an agreement to defend and indemnify Leighton Consulting, Inc. from and against any liability which may arise as a result of such use or reliance, regardless of any fault, negligence, and/or strict liability of Leighton Consulting, Inc.

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Project: 12396.001	Eng/Geol: EC/JMP
Scale:1 " = 1,000 '	Date: September 2019

Base Map: ESRI ArcGIS Online 2019
Thematic Information: Leighton
Author: Leighton Geomatics (btran)

SITE LOCATION MAP

Avalon K-12 Sports Field and Site Improvements Avalon, California

Figure 1

Leighton

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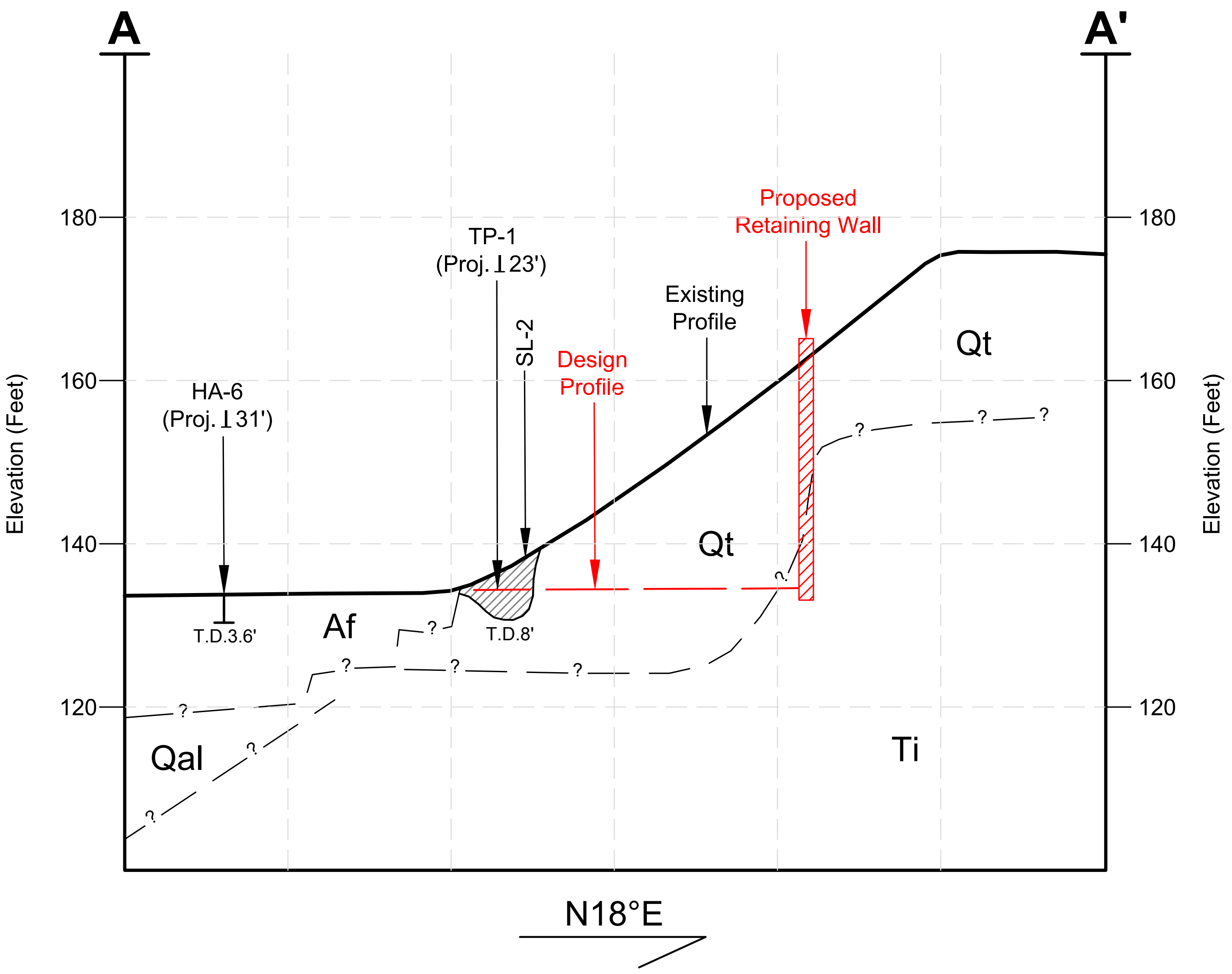

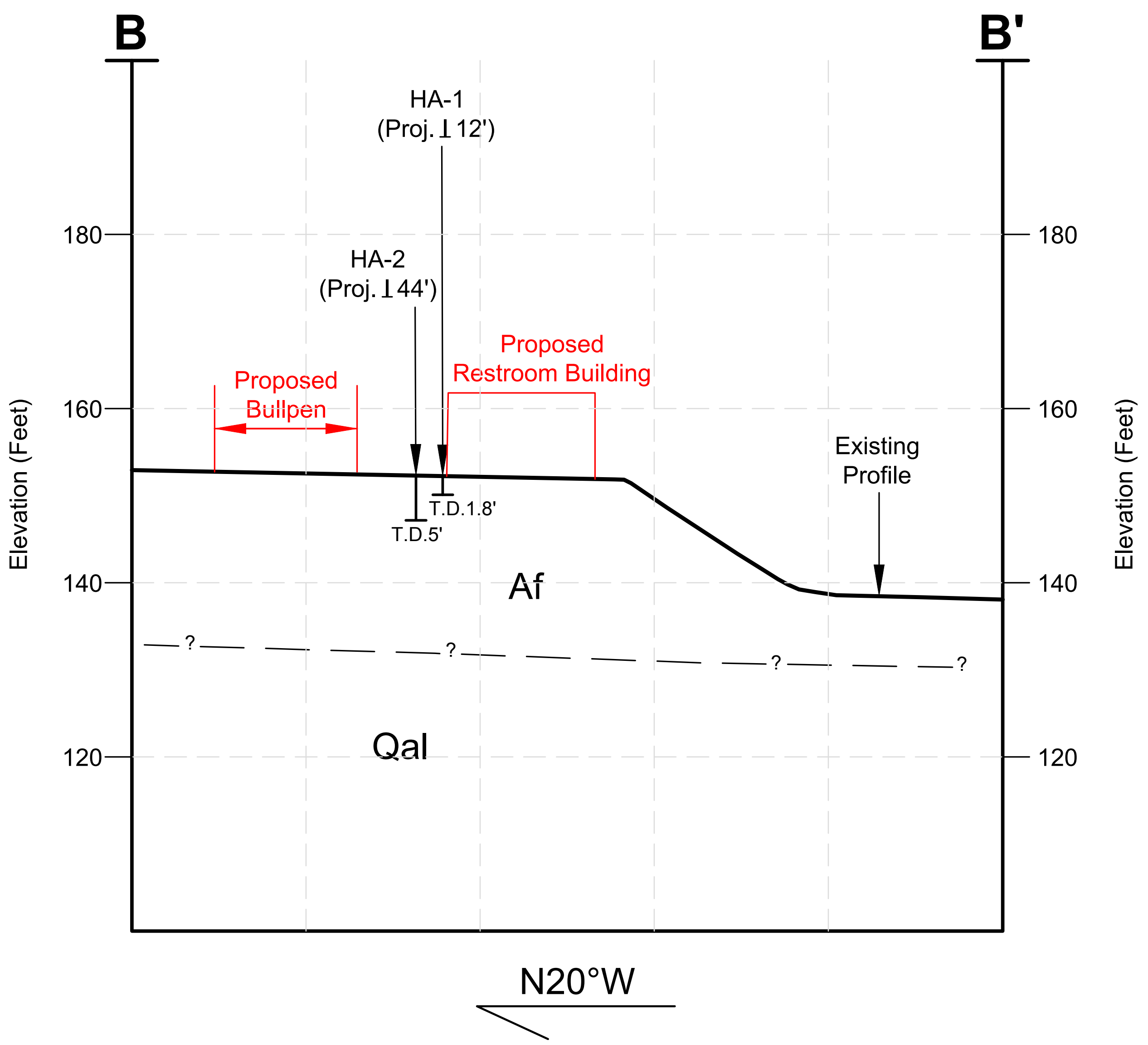

LEGEND

Af	ARTIFICIAL FILL, UNDOCUMENTED
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Qt	QUATERNARY AGE TERRACE DEPOSITS
Ti	TERTIARY-AGE INTRUSIVE ROCKS, QUARTZ DIORITE

Proj: 12396.001
Eng/Geol: EC/JMP
Scale: 1"=20'
Date: September 2019
Reference:

GEOTECHNICAL CROSS SECTIONS A-A' AND B-B'

Avalon K-12 Sports Field
and Site Improvements
Avalon, California

Figure 2

Leighton

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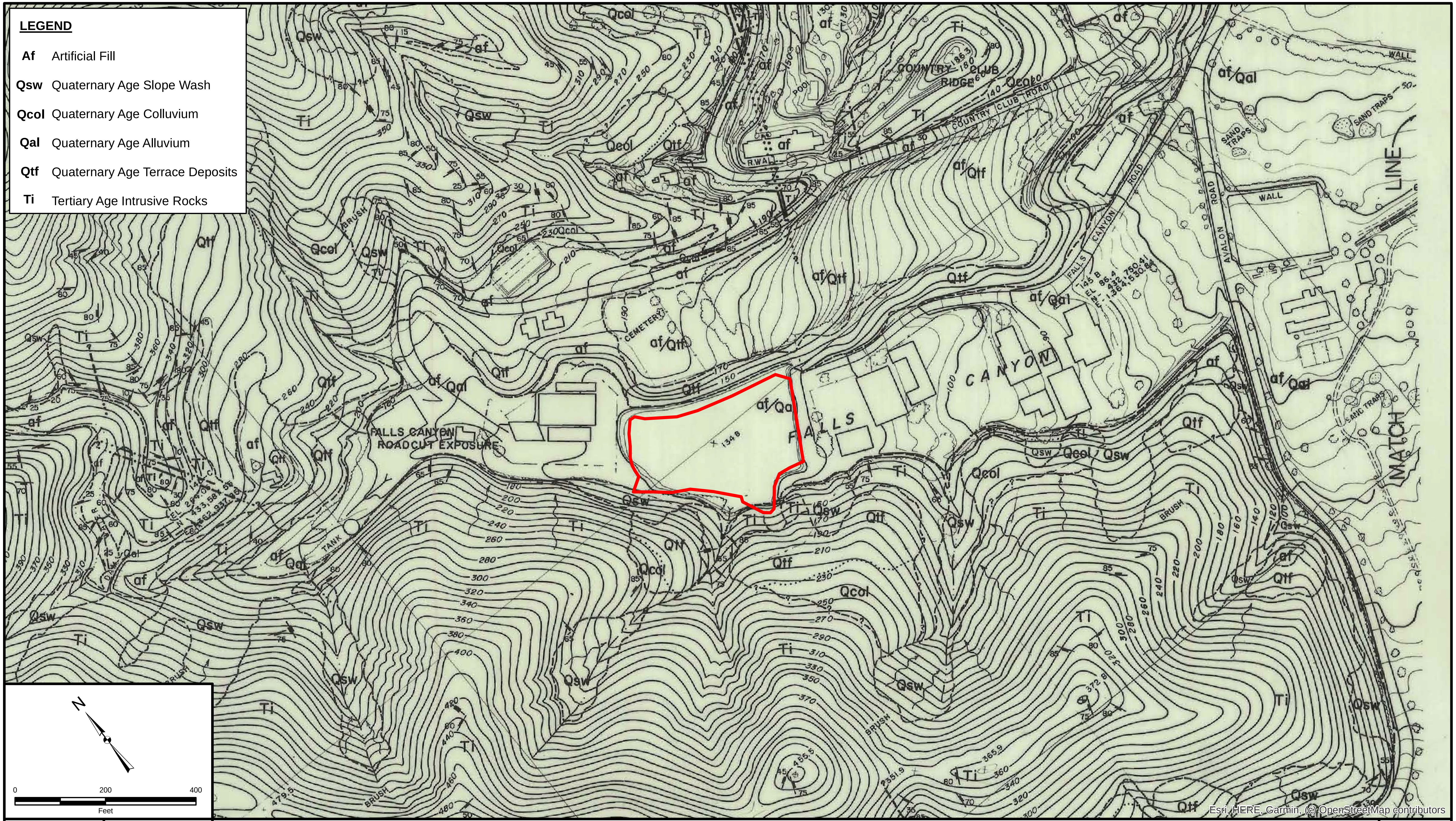

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Base Map: ESRI ArcGIS Online 2019
Thematic Information: Leighton
Author: Leighton Geomatics (btran)

REGIONAL GEOLOGY MAP

Avalon K-12 Sports Field and Site Improvements
Avalon, California

Figure 3

Leighton

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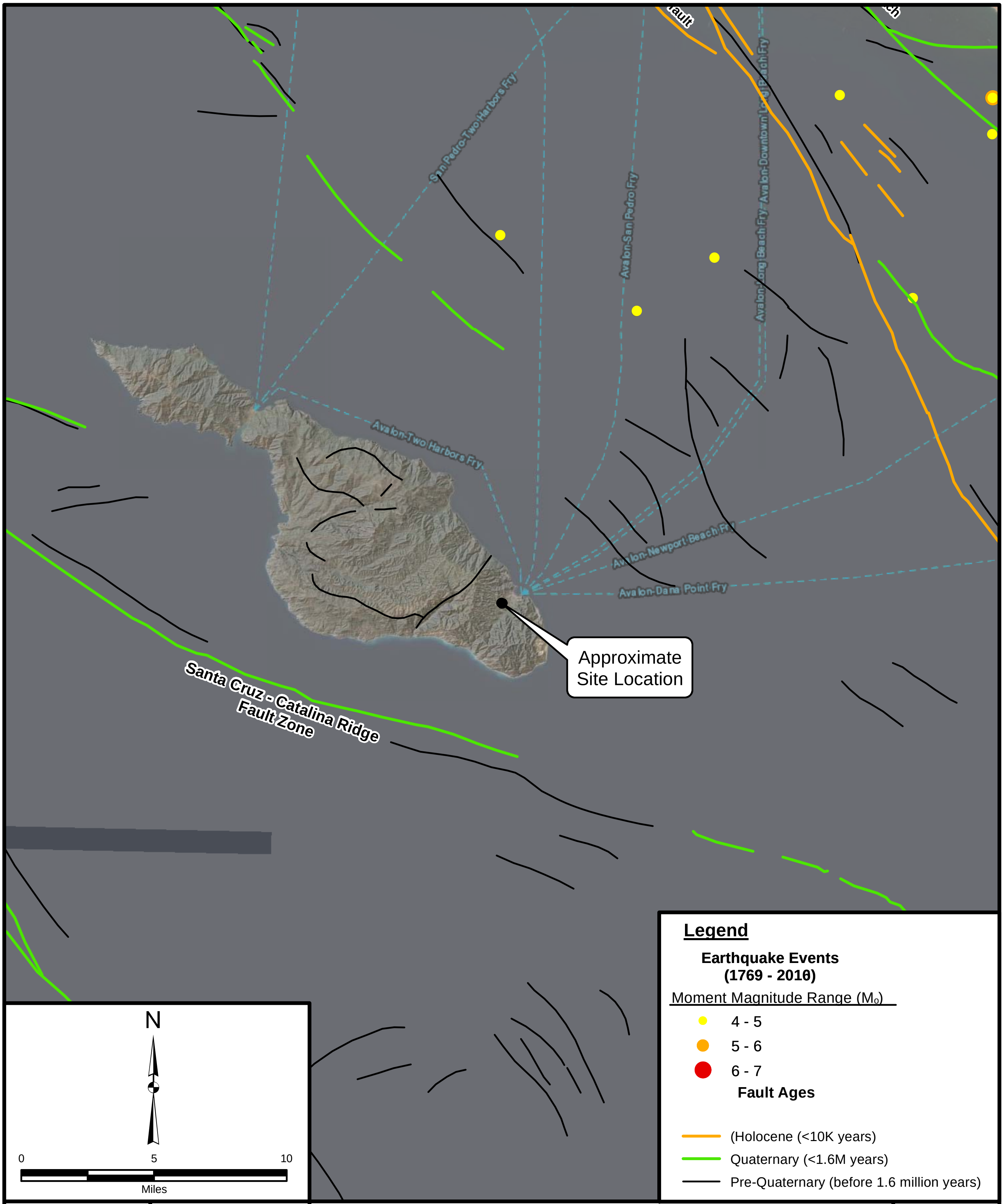


Project: 12396.001	Eng/Geol: EC/JMP
Scale:1 " = 5 miles	Date: September 2019

Base Map: ESRI ArcGIS Online 2019
Thematic Information: Leighton, Bryant, W. A. (compiler), 2005, Digital Database of Quaternary and Younger Faults from the Fault Activity Map of California, version 2.0: CGS, USGS, SCEC.
Author: Leighton Geomatics (btran)

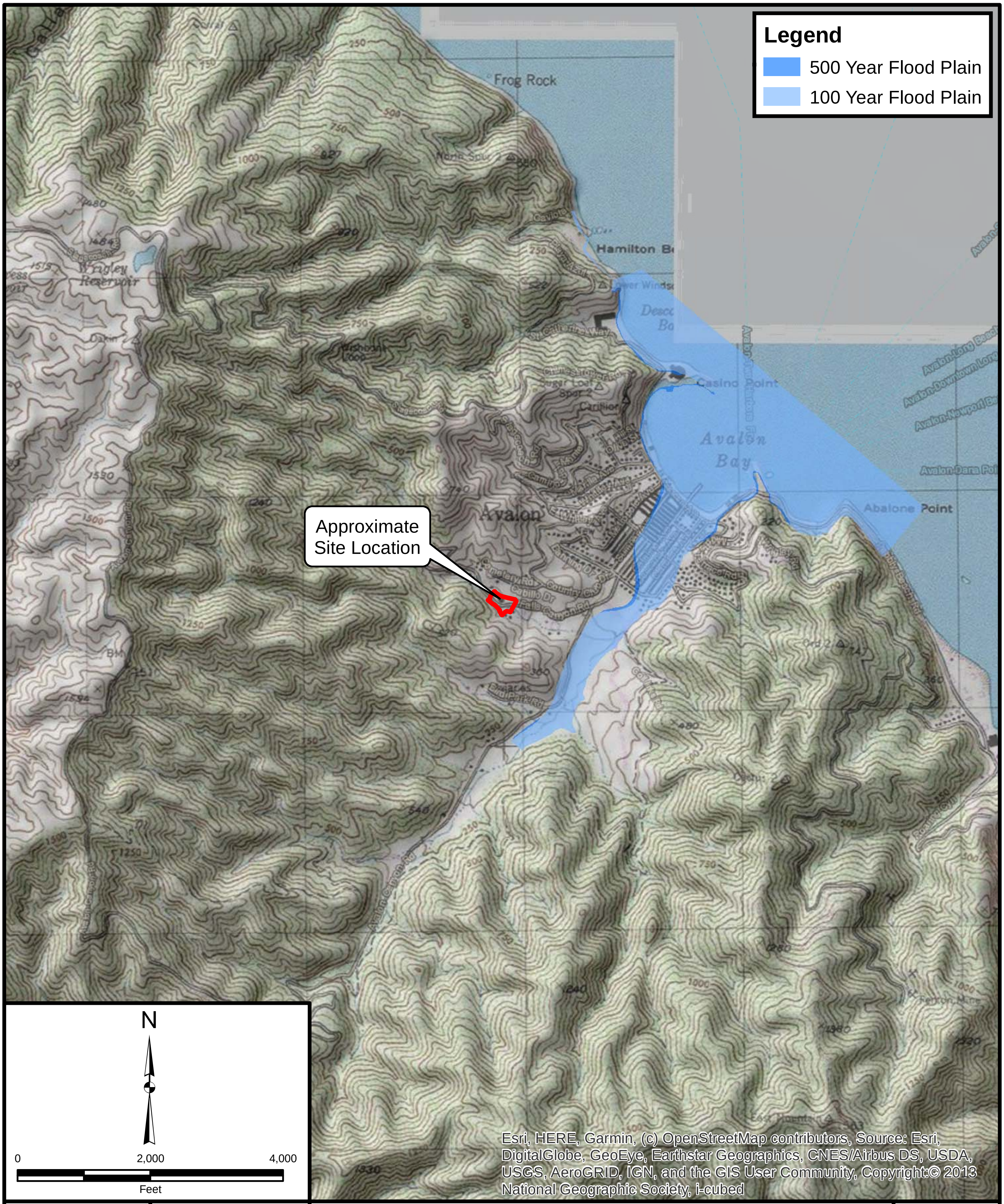
REGIONAL FAULT AND HISTORICAL SEISMICITY MAP

Avalon K-12 Sports Field
and Site Improvements
Avalon, California

Figure 4

Leighton

Map Saved as V:\Drafting\12396\001\Maps\12396-001_F04_RF&HSM_2019-09-12.mxd on 9/10/2019 2:28:47 PM



Esri, HERE, Garmin, (c) OpenStreetMap contributors, Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community, Copyright:© 2013 National Geographic Society, i-cubed

Project: 12396.001
Eng/Geol: EC/JMP
Scale:1 " = 2,000 '
Date: September 2019

Base Map: ESRI ArcGIS Online 2019
Thematic Information: Leighton, CA DWR, FEMA
Author: Leighton Geomatics (btran)

FLOOD HAZARD ZONE MAP

Avalon K-12 Sports Field
and Site Improvements
Avalon, California

Figure 5

Leighton

Map Saved as V:\Drafting\12396\001\Maps\12396-001_F05_FHZM_2019-09-12.mxd on 9/10/2019 5:28:23 PM

DEMOLITION NOTES:

① REMOVE EXISTING CONCRETE STEPS.
② REMOVE EXISTING RETAINING WALL.
③ REMOVE EXISTING CORRUGATED METAL PIPE.
④ REMOVE EXISTING FENCE.
⑤ REMOVE EXISTING BLEACHERS.
⑥ REMOVE EXISTING BUILDING.
⑦ DEMOLISH EXISTING CULVERT.
⑧ REMOVE EXISTING BASEBALL DIAMOND.
⑨ SCARIFY TOP 2" OF EXISTING FIELD.
⑩ REMOVE EXISTING TREE.
⑪ REMOVE EXISTING SCOREBOARD.
⑫ REMOVE EXISTING CONCRETE PAD.
⑬ REMOVE EXISTING POST.

SALVAGE NOTES:

A PROTECT EXISTING CATCH BASIN.
B PROTECT EXISTING POWER POLE.
C PROTECT EXISTING TREE.
D PROTECT EXISTING CONCRETE GUTTER.
E PROTECT EXISTING FENCE.

LEGEND

┌ ─ ─ ┐	LIMITS OF DEMOLITION
▨	EXISTING BUILDING OUTLINE

DEMOLITION PLAN

0' 10' 20' 40'
GRAPHIC SCALE : 1"=20'

NORTH

LEGEND

0 SL-3 50	APPROXIMATE LOCATION OF SEISMIC LINE
HA-5 ◕ T.D.1.9'	APPROXIMATE LOCATION OF HAND-AUGER BORING
HA-6 ⊕ T.D.3.6'	APPROXIMATE LOCATION OF HAND-AUGER BORING AND PERCOLATION TEST
TP-3 ▚ T.D.3'	APPROXIMATE LOCATION OF TEST PIT
B ├──┤ B'	GEOTECHNICAL CROSS SECTION

Plate-1	**GEOTECHNICAL MAP** Avalon K-12 Sports Field and Site Improvements Avalon, California	
Leighton	Proj: 12396.001	Eng/Geol: EC/JMP
	Scale: As Shown	Date: September 2019

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KEY MAP

NORTH

0' 50' 100' 200'
GRAPHIC SCALE : 1"=100'

AVALONFIELD_julmer-LBUSD_logo-15487848.jpg

Long Beach Unified School District

2425 Webster Avenue
Long Beach, CA 90810

LBUSD No.: 9154-1819-411
PTN No.: 64725-371

Facilities Management

AVALONFIELD_julmer-NAC-Architecture-Black-15487852.png

837 NORTH SPRING STREET
THIRD FLOOR
LOS ANGELES, CA 90012
Tel: 323.475.8075
www.nacarchitecture.com

BRANDOW & JOHNSTON
STRUCTURAL+CIVIL ENGINEERS (B&J HBK, INC.)
700 S FLOWER ST #1800, LOS ANGELES, CA 90017
T: (213) 596-4500 WWW.BJSCE.COM
EST. 1945

AVALONFIELD_julmer-P2SLOGO_TITLEBLOCK_VERTICAL-15487882.png

IDENTIFICATION STAMP
DIV. OF THE STATE ARCHITECT
FILE NO:
A#:
AC______ FLS______ SS______
DATE____________

200 FALLS CANYON ROAD, AVALON, CA 90704

AVALON K-12 SPORTS FIELD AND SITE IMPROVEMENTS

DESIGN DEVELOPMENT

AVALONFIELD_julmer-MeasureE_logo-15487854.tif

REV.	DATE	DESCRIPTION
	11/14	SCHEMATIC DESIGN

JOB No.: 161-18041

PROJ MGR: Author
DATE: 01/18/19

SITE DEMOLITION PLAN

C301

APPENDIX A

FIELD EXPLORATION

Our field exploration consisted of a surface reconnaissance and a subsurface exploration program including hand auger borings, test pits and seismic refraction lines. These subsurface exploration locations are plotted on Plate 1, *Geotechnical Map*, and described in more detail below:

Borings: On June 26 and 27, 2019, six (6) hand auger borings were drilled, logged and sampled to depths ranging from approximately 1.8 feet to 5 feet below ground surface (bgs). Encountered soils were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D 2488). Driven samples consisting of California ring-lined soil samples were obtained at selected intervals within the hand auger borings. Bulk soil samples were also collected from these borings. Boring logs are included as part of this appendix. The borings were backfilled immediately after drilling, logging and sampling the same day.

Percolation Tests: On June 26 and 27, 2019, three (3) percolation tests were performed in general accordance with the County of Los Angeles Department of Public Works (LADPW) *Guidelines for Geotechnical Investigation* and Reporting, *Low Impact Development Stormwater Infiltration* (LADPW, 2014). The results of the percolation testing are included as a part of this appendix. Upon completion of the percolation testing, the well casing was removed from each boring and the boring was backfilled with soil cuttings.

Test Pits: On June 27, 2019, three (3) test pits were excavated with a track mounted mini-excavator, logged and sampled to depths ranging from approximately 3 feet to 8 feet below ground surface (bgs). Encountered soils were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D 2488). Driven samples consisting of California ring-lined soil samples were obtained at the bottom of each test pit. Bulk soil samples were also collected from the test pits. Logs of the test pits are included as part of this appendix. The test pits were backfilled immediately after excavating, logging and sampling the same day.

Seismic Refraction Lines: On June 26, 2019, three (3) seismic P-wave refraction lines were performed at the site by a subcontracted geophysicist (Southwest Geophysics, LLC). The seismic lines ranged in length from approximately 50 to 100 feet, and were located on the slope north of the existing ballfield. The results of the seismic refraction survey are presented in a report prepared by Southwest Geophysics, LLC and dated July 15, 2019. A copy of the report is included as a part of this appendix.

Subsurface Variations and Limitations: The attached subsurface exploration logs and related information depict subsurface conditions only at the approximate locations indicated and at the particular date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these locations. Passage of time may result in altered subsurface conditions due to possible environmental changes. In addition, any lines of stratification depicted on these logs represent an approximate boundary between soil types, but these transitions can be gradual.

GEOTECHNICAL BORING LOG HA-1

Project No. 12396.001
Project Avalon Ballfield
Drilling Co. LCI
Drilling Method Hand Auger - Bulk/CAL
Location 200 Falls Canyon Road, Avalon, CA 90704

Date Drilled 6-27-19
Logged By EC, JP
Hole Diameter 5"
Ground Elevation N/A'
Sampled By EC, JP

Elevation Feet	Depth Feet	Graphic Log N S	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0			1				SM	@0': Silty SAND, light brown, slightly moist to moist, fine grained, some gravel, up to 1", rootlets	SA
				2		100	5			DS
									@1.6': Coarse gravel or cobble in sidewall of boring	
									Notes: Total Depth of Boring: 1.8 feet bgs No groundwater encountered Refusal due to sampler bouncing on gravel, cobble Boring backfilled with soil cuttings	
	5									

SAMPLE TYPES:

- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:

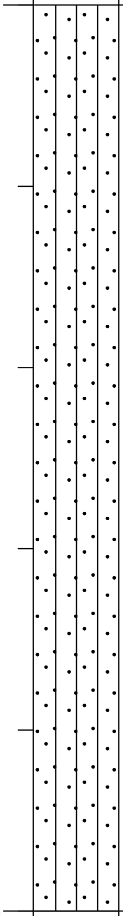
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG HA-2

Project No. 12396.001
Project Avalon Ballfield
Drilling Co. LCI
Drilling Method Hand Auger - N/A
Location 200 Falls Canyon Road, Avalon, CA 90704

Date Drilled 6-27-19
Logged By EC, JP
Hole Diameter 5"
Ground Elevation N/A'
Sampled By EC, JP

Elevation Feet	Depth Feet	Graphic Log N S	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0							SM	@0': Silty SAND, light brown, moist, fine grained, some fine and coarse gravel to 1.5"	
									@2': Increase in moisture, fine to medium grained	
									@3': Gravelly silty SAND, grey brown, very moist, fine to coarse grained, coarse gravel to 2"	
									@4': Becomes wet	
	5								**Notes:** Total Depth of Boring: 5 feet bgs No groundwater encountered Boring backfilled with soil cuttings	

SAMPLE TYPES:

B	BULK SAMPLE
C	CORE SAMPLE
G	GRAB SAMPLE
R	RING SAMPLE
S	SPLIT SPOON SAMPLE
T	TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING
AL	ATTERBERG LIMITS
CN	CONSOLIDATION
CO	COLLAPSE
CR	CORROSION
CU	UNDRAINED TRIAXIAL
DS	DIRECT SHEAR
EI	EXPANSION INDEX
H	HYDROMETER
MD	MAXIMUM DENSITY
PP	POCKET PENETROMETER
RV	R VALUE
SA	SIEVE ANALYSIS
SE	SAND EQUIVALENT
SG	SPECIFIC GRAVITY
UC	UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG HA-3

Project No.	12396.001	**Date Drilled**	6-27-19
Project	Avalon Ballfield	**Logged By**	EC, JP
Drilling Co.	LCI	**Hole Diameter**	5"
Drilling Method	Hand Auger - Bulk	**Ground Elevation**	N/A'
Location	200 Falls Canyon Road, Avalon, CA 90704	**Sampled By**	EC, JP

Elevation Feet	Depth Feet	Graphic Log N S	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0			1				SM	@0': Silty SAND, light brown, moist, fine grained, with fine and coarse gravel, angular, up to 2.75" gravel	
									Notes: Total Depth of Boring: 2 feet bgs No groundwater encountered refusal at 2', hand auger failed, sampler bouncing on coarse gravel, Borehole converted to percolation test hole	
	5									

SAMPLE TYPES:

B	BULK SAMPLE
C	CORE SAMPLE
G	GRAB SAMPLE
R	RING SAMPLE
S	SPLIT SPOON SAMPLE
T	TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

GEOTECHNICAL BORING LOG HA-4

Project No. 12396.001
Project Avalon Ballfield
Drilling Co. LCI
Drilling Method Hand Auger - CAL
Location 200 Falls Canyon Road, Avalon, CA 90704

Date Drilled 6-27-19
Logged By EC, JP
Hole Diameter 5"
Ground Elevation N/A'
Sampled By EC, JP

Elevation Feet	Depth Feet	Graphic Log N S	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0							CL-ML	@0': Sandy Silty CLAY, dark brown, very moist to wet, very fine sand	
								SM	@0.33': Silty SAND, light brown, moist to very moist, fine to coarse grained	
				1		105	11			
									@1.75': Concrete debris up to 5", cobbles up to 3.5"	
				2		90	6	SC	@4': Clayey SAND, dark brown, very moist, fine to medium grained, some gravel, up to 2", some carbonate veins, no cobbles	
									@4.67': Occasional Cobble to 3.5"	
	5									
									Notes: Total Depth of Boring: 5 feet bgs No groundwater encountered Borehole converted to percolation test hole	

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG HA-5

Project No. 12396.001
Project Avalon Ballfield
Drilling Co. LCI
Drilling Method Hand Auger - Bulk/CAL
Location 200 Falls Canyon Road, Avalon, CA 90704

Date Drilled 6-27-19
Logged By EC, JP
Hole Diameter 5"
Ground Elevation N/A'
Sampled By EC, JP

Elevation Feet	Depth Feet	Graphic Log N S	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0			1				SM	@0': Silty SAND, light gray brown, moist, fine grained, rootlets	SA, MD
				2		95	5			
									@1': Becomes yellow brown, some fine gravel, anguar to subangular up to 1.5"	
									@1.5': Cobble encountered, increase in gravel	
									Notes: Total Depth of Boring: 1.92 feet bgs due to refusal No groundwater encountered Backfilled with soil cuttings	
	5									

SAMPLE TYPES:

B	BULK SAMPLE
C	CORE SAMPLE
G	GRAB SAMPLE
R	RING SAMPLE
S	SPLIT SPOON SAMPLE
T	TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG HA-6

Project No. 12396.001
Project Avalon Ballfield
Drilling Co. LCI
Drilling Method Hand Auger - Bulk/CAL
Location 200 Falls Canyon Road, Avalon, CA 90704

Date Drilled 6-27-19
Logged By EC, JP
Hole Diameter 5"
Ground Elevation N/A'
Sampled By EC, JP

Elevation Feet	Depth Feet	Graphic Log N S	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0							SM	@0': Silty SAND, light brown, moist, fine grained	
				3						
				1		103	6			
									@1.5': Coarse gravel to 1.5", subangular to angular	
				2		102	8		@3': Dark yellow brown, fine to medium grained, fine and coarse gravel up to 2.5", trace clay	
									Notes: Total Depth of Boring: 3.67 feet bgs No groundwater encountered Borehole converted to percolation test hole Refusal at 3.67 feet	
	5									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING
AL	ATTERBERG LIMITS
CN	CONSOLIDATION
CO	COLLAPSE
CR	CORROSION
CU	UNDRAINED TRIAXIAL
DS	DIRECT SHEAR
EI	EXPANSION INDEX
H	HYDROMETER
MD	MAXIMUM DENSITY
PP	POCKET PENETROMETER
RV	R VALUE
SA	SIEVE ANALYSIS
SE	SAND EQUIVALENT
SG	SPECIFIC GRAVITY
UC	UNCONFINED COMPRESSIVE STRENGTH

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

Boring Percolation Test Data Sheet

Project Number:	12396.001	**Test Hole Number:**	HA-3	
Project Name:	Avalon Ballfield	**Date Excavated:**	6/26/2019	
Earth Description:	Fill	**Date Tested:**	6/26/2019	
Liquid Description:	Tap water	**Depth of boring (ft):**	1.8	
Tested By:	JMP	**Radius of boring (in):**	2.75	
Time Interval Standard		**Radius of casing (in):**	1	
Start Time for Pre-Soak:	6/26/19 10:31 AM	**Length of slotted of casing (ft):**		1.8
Start Time for Standard:	6/26/19 11:35 AM	**Depth to Initial Water Depth (ft):**		0
Standard Time Interval		**Porosity of Annulus Material, *n* :**		0.4
Between Readings, mins:	30	**Bentonite Plug at Bottom:**	No	

Percolation Data

Reading	Time	Time Interval, Δt (min.)	Initial/Final Depth to Water (ft.)	Initial/Final Water Height, H_0/H_f (in.)	Total Water Drop, Δd (in.)	Infiltration Rate (in./hr.)
P1	10:31	30	0.00	21.6	18.8	1.83
	11:01		1.57	2.8		
P2	11:03	30	0.00	21.6	15.1	1.29
	11:33		1.26	6.5		
1	11:35	30	0.00	21.6	13.2	1.06
	12:05		1.10	8.4		
2	12:06	30	0.00	21.6	12.0	0.93
	12:36		1.00	9.6		
3	12:37	30	0.00	21.6	11.3	0.86
	13:07		0.94	10.3		
4	13:09	30	0.00	21.6	10.7	0.80
	13:39		0.89	10.9		
5	13:40	30	0.00	21.6	9.6	0.70
	14:10		0.80	12.0		
6	14:11	30	0.00	21.6	9.4	0.67
	14:41		0.78	12.2		
7	14:43	30	0.00	21.6	9.1	0.65
	15:13		0.76	12.5		
8	15:16	30	0.00	21.6	9.2	0.66
	15:46		0.77	12.4		

Infiltration Rate (I) = Discharge Volume/Surface Area of Test Section/Time Interval

Measured Infiltration Rate, I (Average of Last 3 ReadingsLast Readings) = 0.66 in./hr.

Design Infiltration Rate

Reduction Factor from Test Procedure, RF_t = 2

Reduction Factor for Site Variability, # of Tests and Investigation, RF_v = 1

Reduction Factor for Long Term Siltation, Plugging and Maintenance, RF_t = 1

Reduction Factor, RF = RF_t x RF_v x RF_s = 2

Design Infiltration Rate = Measured Infiltration Rate / Reduction Factor (RF) = 0.33 in./hr.

Boring Percolation Test Data Sheet

Project Number:	12396.001	**Test Hole Number:**	HA-4
Project Name:	Avalon Ballfield	**Date Excavated:**	6/26/2019
Earth Description:	Fill	**Date Tested:**	6/26/2019
Liquid Description:	Tap water	**Depth of boring (ft):**	5
Tested By:	JMP	**Radius of boring, r (in):**	2.75
		Diameter of casing (in):	2
		Length of slotted of casing (ft):	5
		Depth to Initial Water Depth (ft):	3.95
		Porosity of Annulus Material, *n* :	0.4
		Bentonite Plug at Bottom:	No

High Flowrate (Constant Head) Field Percolation Data

Reading	Time	Time Interval, Δt (minutes)	Depth to Water (feet bgs)	Water Height, H (inches)	Cumulative Water Volume Delivered (gallons)
1	0:05	-	-	-	0.0
2	0:35	30	3.95	12.6	146.5
3	1:05	30	3.95	12.6	293.0
4	1:35	30	3.95	12.6	439.5
5	2:05	30	3.95	12.6	586.0
6	2:35	30	3.95	12.6	732.5
7	3:05	30	3.95	12.6	879.0
8	3:35	30	3.95	12.6	1025.5

High Flowrate Percolation Test Calculation

Total Volume of Water Delivered (gallons)	1025.5
Total Volume of Water Delivered (cubic inches)	236890.5
Average Water Height (inches)	12.6
Average Percolation Surface Area (cubic Inches)	241.5
Duration of Test (minutes)	210
Duration of Test (hours)	3.50

$$\text{Measured Infiltration Rate} = (\text{Total Volume})/(\text{Test Duration})/(\text{Surface Area})$$

Measured Infiltration Rate = 280.3 in./hr.

Design Infiltration Rate

Reduction Factor from Test Procedure, RF_t =	3
Reduction Factor for Site Variability, # of Tests and Investigation, RF_v =	2
Reduction Factor for Long Term Siltation, Plugging and Maintenance, RF_t =	1
Reduction Factor, RF = RF_t x RF_v x RF_s =	6

Design Infiltration Rate = Measured Infiltration Rate / Reduction Factor (RF) = 46.7 **in./hr.**

Boring Percolation Test Data Sheet

Project Number: 12396.001
Project Name: Avalon Ballfield
Earth Description: Fill
Liquid Description: Tap water
Tested By: JMP
Time Interval Standard
Start Time for Pre-Soak: 6/27/19 8:59 AM
Start Time for Standard: 6/27/19 9:56 AM
Standard Time Interval Between Readings, mins: 30

Test Hole Number: HA-6
Date Excavated: 6/26/2019
Date Tested: 6/26/2019
Depth of boring (ft): 3.66
Radius of boring (in): 2.75
Radius of casing (in): 1
Length of slotted of casing (ft): 3.66
Depth to Initial Water Depth (ft): 0.2
Porosity of Annulus Material, *n* : 0.4
Bentonite Plug at Bottom: No

Percolation Data

Reading	Time	Time Interval, Δt (min.)	Initial/Final Depth to Water (ft.)	Initial/Final Water Height, H_0/H_f (in.)	Total Water Drop, Δd (in.)	Infiltration Rate (in./hr.)
P1	8:59	25	0.20	41.5	34.8	2.16
	9:24		3.10	6.7		
P2	9:25	30	0.20	41.5	33.5	1.69
	9:55		2.99	8.0		
1	9:56	30	0.20	41.5	32.6	1.62
	10:26		2.92	8.9		
2	10:27	30	0.20	41.5	32.4	1.60
	10:57		2.90	9.1		
3	10:58	30	0.20	41.5	32.2	1.58
	11:28		2.88	9.4		
4	11:29	30	0.20	41.5	32.0	1.57
	11:59		2.87	9.5		
5	12:03	30	0.20	41.5	32.2	1.58
	12:33		2.88	9.4		
6	12:34	30	0.20	41.5	31.9	1.56
	13:04		2.86	9.6		
7	13:05	30	0.20	41.5	32.0	1.57
	13:35		2.87	9.5		
8	13:36	30	0.20	41.5	31.8	1.55
	14:06		2.85	9.7		

Infiltration Rate (I) = Discharge Volume/Surface Area of Test Section/Time Interval

Measured Infiltration Rate, I (Average of Last 3 ReadingsLast Readings) = 1.56 in./hr.

Design Infiltration Rate

Reduction Factor from Test Procedure, RF_t = 2
Reduction Factor for Site Variability, # of Tests and Investigation, RF_v = 1
Reduction Factor for Long Term Siltation, Plugging and Maintenance, RF_t = 1
Reduction Factor, RF = RF_t x RF_v x RF_s = 2
Design Infiltration Rate = Measured Infiltration Rate / Reduction Factor (RF) = 0.78 in./hr.

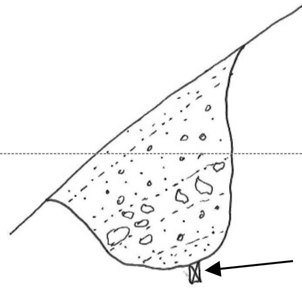
Log of Test Pit: TP-1

Project Name: LBUSD Avalon School
Project Number: 12396.001
Equipment: Backhoe 3-foot wide bucket
Logged by: JMP
Elevation: ~136 feet
Location/Grid: See Plate 1 – Geotechnical Map

Earth Material Description: This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Geologic Attitudes	Earth Materials Exposed On: June 27, 2019	Geologic Unit	Engineering Properties: Unified Soil Classification	Sample Number	Moisture (%)	Density (pcf)
	Quaternary Terrace Deposits (Qtf):					
	@0-10”: Gravelly SAND (SW), brown to yellow brown, dry, fine to coarse sand, fine to coarse angular gravel, rootlets, top soil	Qtf	SW			
	@10”-4.5’: Sandy GRAVEL (GW), yellow brown, dry to slightly moist, fine to coarse sand, fine to coarse angular gravel, some angular to subangular cobbles, slightly tilted, few rootlets		GW			
	@4.5’-5.5’: Cobbles/Boulders with sandy matrix (GP), angular to subangular, clasts of quartz diorite, appears slightly tilted, few rootlets		GP			
	@5.5’-7.5’: Sandy GRAVEL (GW), yellow brown, moist, fine to coarse sand, fine to coarse angular to subangular gravel, slightly tilted		GW			
	@7.5’-8’: Gravelly SAND (SW), dense, yellow brown to brown, moist, fine to coarse sand, fine rounded gravel		SW	1A/1B		

Graphical Representation: West wall **Scale:** 1 inch = 5 feet **Surface Slope:** ~37° **Trend:** ~N-S


1A (drive) @ 8’
1B (small bag) @ 8’

Total Depth = 8 feet, No groundwater encountered on 6/27/2019

***** This log is a part of a report by Leighton and should not be used as a stand-alone document.*****

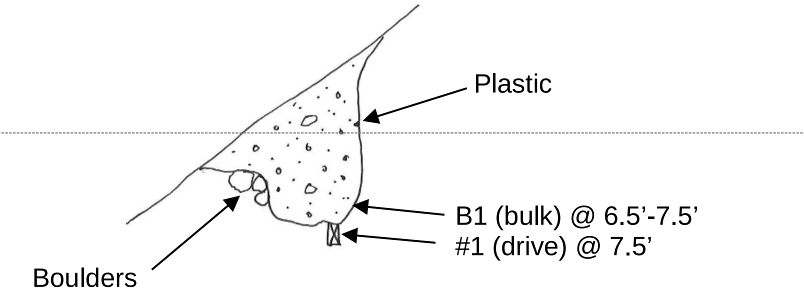
Log of Test Pit: TP-2

Project Name: LBUSD Avalon School
Logged by: JMP
Project Number: 12396.001
Elevation: ~136 feet
Equipment: Backhoe 3-foot wide bucket
Location/Grid: See Plate 1 – Geotechnical Map

Earth Material Description: This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Geologic Attitudes	Earth Materials Exposed On: June 27, 2019	Geologic Unit	Engineering Properties			
			Unified Soil Classification	Sample Number	Moisture (%)	Density (pcf)
	Artificial Fill (Af): @0-7.5': Gravelly Silty SAND (SW), brown, slightly moist to moist, fine to coarse sand, fine to coarse angular to subangular gravel, few cobbles and boulders, uniformly mixed, rootlets throughout, some small pieces of plastic at 3 feet deep	Af	SW	B1/#1		

Graphical Representation: West wall **Scale:** 1 inch = 5 feet **Surface Slope:** ~39° **Trend:** ~N-S



Total Depth = 7½ feet, No groundwater encountered on 6/27/2019

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Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

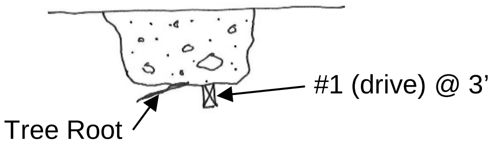
Log of Test Pit: TP-3

Project Name: LBUSD Avalon School
Logged by: JMP
Project Number: 12396.001
Elevation: ~132 feet
Equipment: Backhoe 3-foot wide bucket
Location/Grid: See Plate 1 – Geotechnical Map

Earth Material Description: This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Geologic Attitudes	Earth Materials Exposed On: June 27, 2019	Geologic Unit	Engineering Properties			
			Unified Soil Classification	Sample Number	Moisture (%)	Density (pcf)
	Artificial Fill (Af): @0-3': Silty Gravelly SAND with cobbles (SW), brown, slightly moist to moist, fine to coarse sand, fine to coarse angular to subangular gravel, angular to subangular cobbles up to 8 inches in dimension, one boulder. Large root at bottom of trench from adjacent peppertree.	Af	SW	#1		

Graphical Representation: South wall **Scale:** 1 inch = 5 feet **Surface Slope:** Flat **Trend:** ~E-W



Total Depth = 3 feet, No groundwater encountered on 6/27/2019

***** This log is a part of a report by Leighton and should not be used as a stand-alone document.*****

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

SEISMIC REFRACTION STUDY
AVALON BALL FIELD
AVALON, CATALINA ISLAND

PREPARED FOR:
Leighton Consulting, Inc.
17781 Cowan
Irvine, CA 92614

PREPARED BY:
Southwest Geophysics, LLC
6280 Riverdale Street, Suite 200
San Diego, CA 92120

July 15, 2019
Project No. 119318

SOUTHWEST
GEOPHYSICS LLC
YOUR SUBSURFACE SOLUTION

July 15, 2019
Project No. 119318

Mr. Edward Che, P.E., G.E.
Leighton Consulting, Inc.
17781 Cowan
Irvine, CA 92614

Subject: Seismic Refraction Study
Avalon Ball Field
Avalon, Catalina Island

Dear Mr. Che:

In accordance with your authorization, we have performed a seismic refraction evaluation pertaining to the Avalon Ball Field project located on Catalina Island in the city of Avalon, California. Specifically, our study consisted of performing three seismic P-wave refraction traverses at the project site. The purpose of our study was to develop subsurface velocity profiles of the areas evaluated, and to assess both the apparent rippability of the subsurface materials and depth to bedrock. Our field services were conducted on June 26th, 2019. This data report presents our methodology, equipment used, analysis, and results.

We appreciate the opportunity to be of service on this project. Should you have any questions please contact the undersigned at your convenience.

Sincerely,
SOUTHWEST GEOPHYSICS, LLC

Patrick Lehrman

Connor F. Shaw
Project Geologist/Geophysicist

Patrick Lehrman, P.G., P.Gp.
Principal Geologist/Geophysicist

CFS/CDD/PFL/pfl

Distribution: Addressee (electronic)

REGISTERED GEOPHYSICIST
PATRICK F. LEHRMANN
No. 1043
Exp. 1/31/20
STATE OF CALIFORNIA

TABLE OF CONTENTS

1. INTRODUCTION

In accordance with your authorization, we have performed a seismic refraction evaluation pertaining to the Avalon Ball Field project located on Catalina Island in Avalon, California (Figure 1). Specifically, our study consisted of performing three seismic P-wave refraction traverses at the project site. The purpose of our study was to develop subsurface velocity profiles of the areas evaluated, and to assess both the apparent rippability of the subsurface materials and depth to bedrock. Our field services were conducted on June 26th, 2019. This data report presents our methodology, equipment used, analysis, and results.

2. SCOPE OF SERVICES

Our scope of services included:

- Performance of three seismic P-wave refraction lines at the project site.
- Compilation and analysis of the data collected.
- Preparation of this data report presenting our results and conclusions.

3. SITE AND PROJECT DESCRIPTION

The project site is located approximately 1 mile west of the Catalina Express Ferry port (Figure 1). Access to the site is via Falls Canyon Road on the east side of the baseball field. The site is on an embankment that defines the northern boundary of a baseball field with Cabrillo Drive further to the north. Vegetation on the embankment consists of seasonal grass and brush, scattered trees, and some cacti. Several areas with unconsolidated fill material were also observed in the study area. Figures 2 and 3 depict the general site conditions in the areas of the seismic traverses.

Based on our discussions with you, it is our understanding that your office is conducting a geotechnical evaluation for a proposed retaining wall pertaining to the proposed extension of the baseball field at Avalon High School. We also understand that the results from our study may be used in the formulation of design and construction parameters for the project.

4. METHODOLOGY

A seismic P-wave (compression wave) refraction evaluation was conducted at the project site to evaluate the depth to bedrock and rippability characteristics of the subsurface materials and to develop subsurface velocity profiles of the areas evaluated. The seismic refraction method uses first-arrival times of refracted seismic waves to estimate the thicknesses and seismic velocities of subsurface layers. Seismic P-waves generated at the surface, using a hammer and plate, are refracted at boundaries separating materials of contrasting velocities. These refracted seismic waves are then detected by a series of surface vertical component 14-Hz geophones and recorded with a 24-channel Geometrics Geode seismograph. The travel times of the seismic P-waves are used in conjunction with the shot-to-geophone distances to obtain thickness and velocity information on the subsurface materials.

Three seismic lines (SL-1 through SL-3) were conducted in the study area. The general locations and lengths of the lines were selected by your office. Shot points (signal generation locations) were conducted along the lines at the ends, midpoint, and intermediate points between the ends and the midpoint.

The seismic refraction theory requires that subsurface velocities increase with depth. A layer having a velocity lower than that of the layer above will not generally be detectable by the seismic refraction method and, therefore, could lead to errors in the depth calculations of subsequent layers. In addition, lateral variations in velocity, such as those caused by core stones, intrusions or boulders can also result in the misinterpretation of the subsurface conditions. In general, the effective depth of evaluation for a seismic refraction traverse is approximately one-third to one-fifth the length of the spread.

In general, the seismic P-wave velocity of a material can be correlated to rippability (see Table 1 below), or to some degree "hardness." Table 1 is based on published information from the Caterpillar Performance Handbook (Caterpillar, 2011) as well as our experience with similar materials, and assumes that a Caterpillar D-9 dozer ripping with a single shank is used. We emphasize that the cutoffs in this classification scheme are approximate and that rock characteristics, such as fracture spacing and orientation, play a significant role in determining rock quality or rippability. The rippability of a mass is also dependent on the excavation equipment used and the skill and experience of the equipment operator.

Table 1 – Rippability Classification

Seismic P-wave Velocity	Rippability
0 to 2,000 feet/second	Easy
2,000 to 4,000 feet/second	Moderate
4,000 to 5,500 feet/second	Difficult, Possible Blasting
5,500 to 7,000 feet/second	Very Difficult, Probable Blasting
Greater than 7,000 feet/second	Blasting Generally Required

It should be noted that the rippability cutoffs presented in Table 1 are slightly more conservative than those published in the Caterpillar Performance Handbook. Accordingly, the above classification scheme should be used with discretion, and contractors should not be relieved of making their own independent evaluation of the rippability of the on-site materials prior to submitting their bids.

5. DATA ANALYSIS

The collected data were processed using SIPwin (Rimrock Geophysics, 2003), a seismic interpretation program, and analyzed using SeisOpt Pro (Optim, 2008). SeisOpt Pro uses first arrival picks and elevation data to produce subsurface velocity models through a nonlinear optimization technique called adaptive simulated annealing. The resulting velocity model provides a tomography image of the estimated geologic conditions. Both vertical and lateral velocity information is contained in the tomography model. Changes in layer velocity are revealed as gradients rather than discrete contacts, which typically are more representative of actual conditions.

6. RESULTS AND CONCLUSIONS

As previously indicated, three seismic traverses were conducted as part of our study. Figures 4a through 4c present the velocity models generated from our analysis. Based on the results it appears that the study areas are underlain by low velocity materials (i.e., topsoil, colluvium, etc.) in the near surface and granitic bedrock at depth. Distinct vertical and lateral velocity variations are evident in the models. Moreover, the degree of bedrock weathering and the depth to bedrock appears to be highly variable across the study areas.

Based on the refraction results, variability in the excavatability (including depth of rippability) of the subsurface materials should be expected across the project area. A contractor with excavation experience in similar difficult conditions should be consulted for expert advice on excavation methodology, equipment and production rate.

7. LIMITATIONS

The field evaluation and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, express or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be present. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluations will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Southwest Geophysics, LLC should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

8. SELECTED REFERENCES

Caterpillar, Inc., 2011, Caterpillar Performance Handbook, Edition 41, Caterpillar, Inc., Peoria, Illinois.

Mooney, H.M., 1976, Handbook of Engineering Geophysics, dated February.

Optim, Inc., 2008, SeisOpt Pro, V-5.0.

Rimrock Geophysics, 2003, Seismic Refraction Interpretation Program (SIPwin), V-2.76.

Telford, W.M., Geldart, L.P., Sheriff, R.E., and Keys, D.A., 1976, Applied Geophysics, Cambridge University Press.

SITE LOCATION MAP

N

Avalon Ball Field
Avalon, Catalina Island

Project No.: 119318

Date: 07/19

SOUTHWEST GEOPHYSICS LLC

Figure 1

LINE LOCATION MAP

N

Avalon Ball Field
Avalon, Catalina Island

Project No.: 119318

Date: 07/18

SOUTHWEST GEOPHYSICS LLC

Figure 2

0 50 100

approximate scale in feet

SITE PHOTOGRAPHS

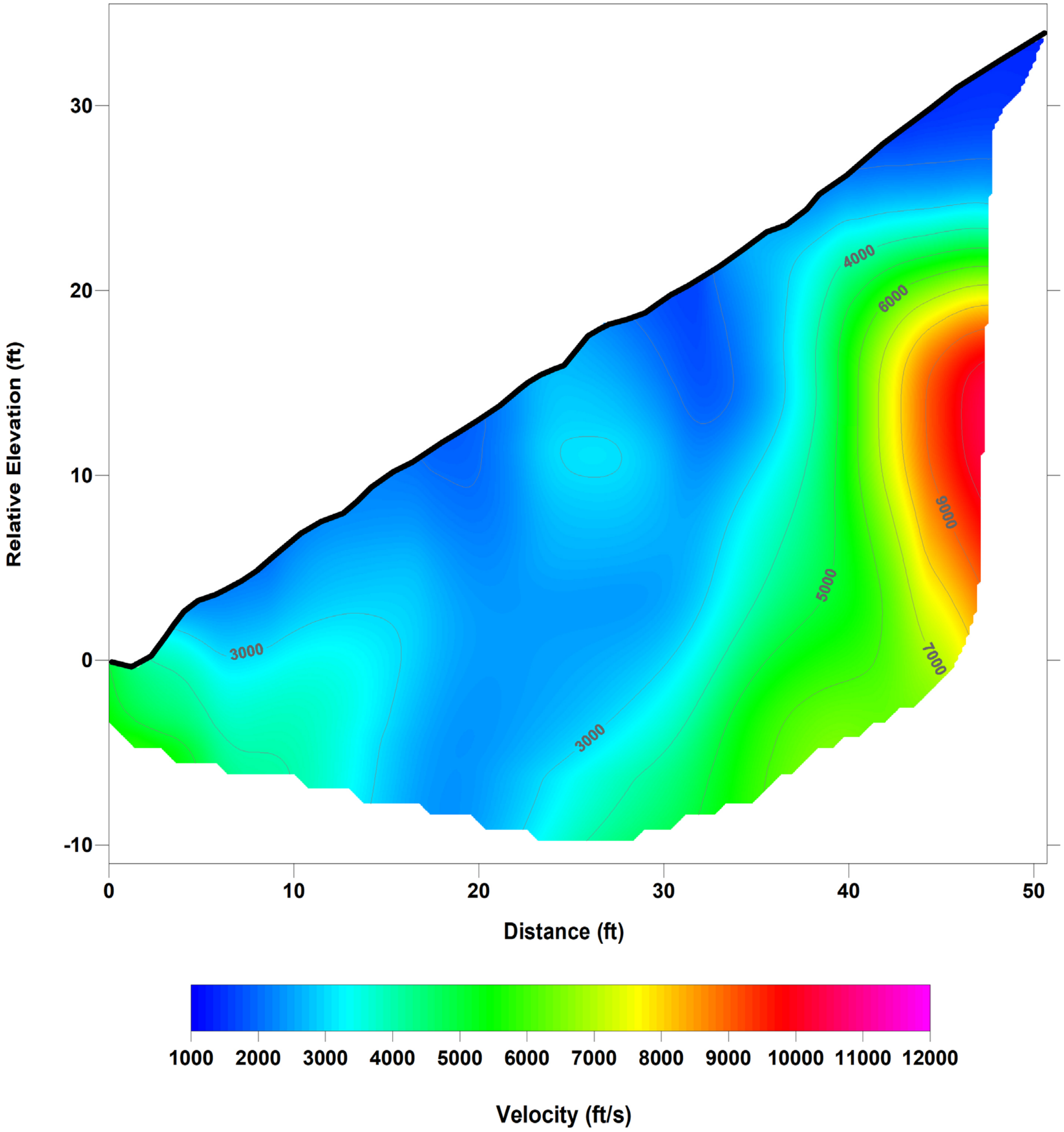
Avalon Ball Field
Avalon, Catalina Island

Project No.: 119318
Date: 07/19

SOUTHWEST GEOPHYSICS INC

Figure 3

TOMOGRAPHY MODEL



P-WAVE PROFILE SL-1

Avalon Ball Field
Avalon, Catalina Island

Project No.: 119318

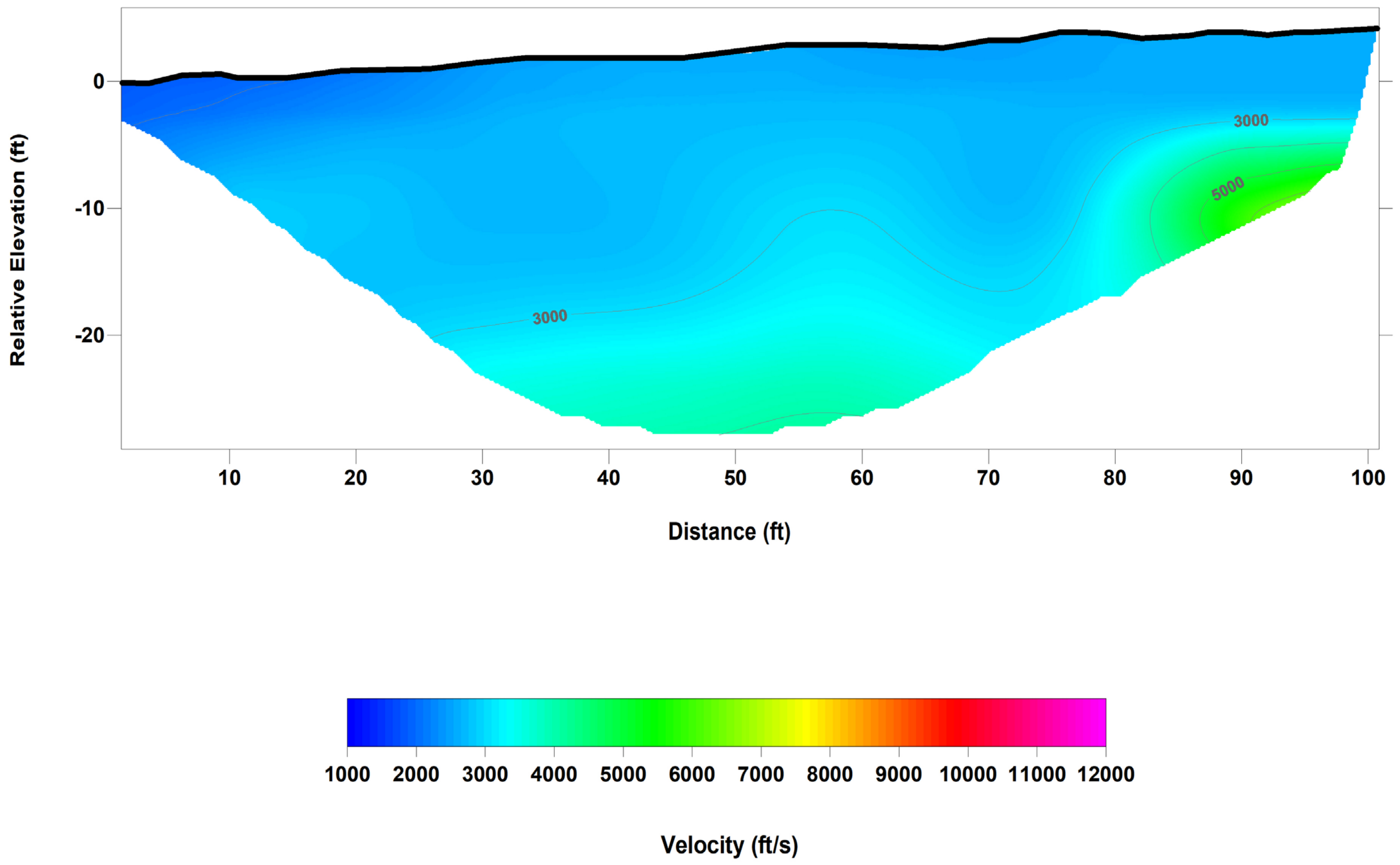
Date: 07/19

SOUTHWEST GEOPHYSICS LLC

Figure 4a

Note: Contour Interval = 1,000 feet per second

TOMOGRAPHY MODEL



P-WAVE PROFILE SL-2

Avalon Ball Field
Avalon, Catalina Island

Project No.: 119318

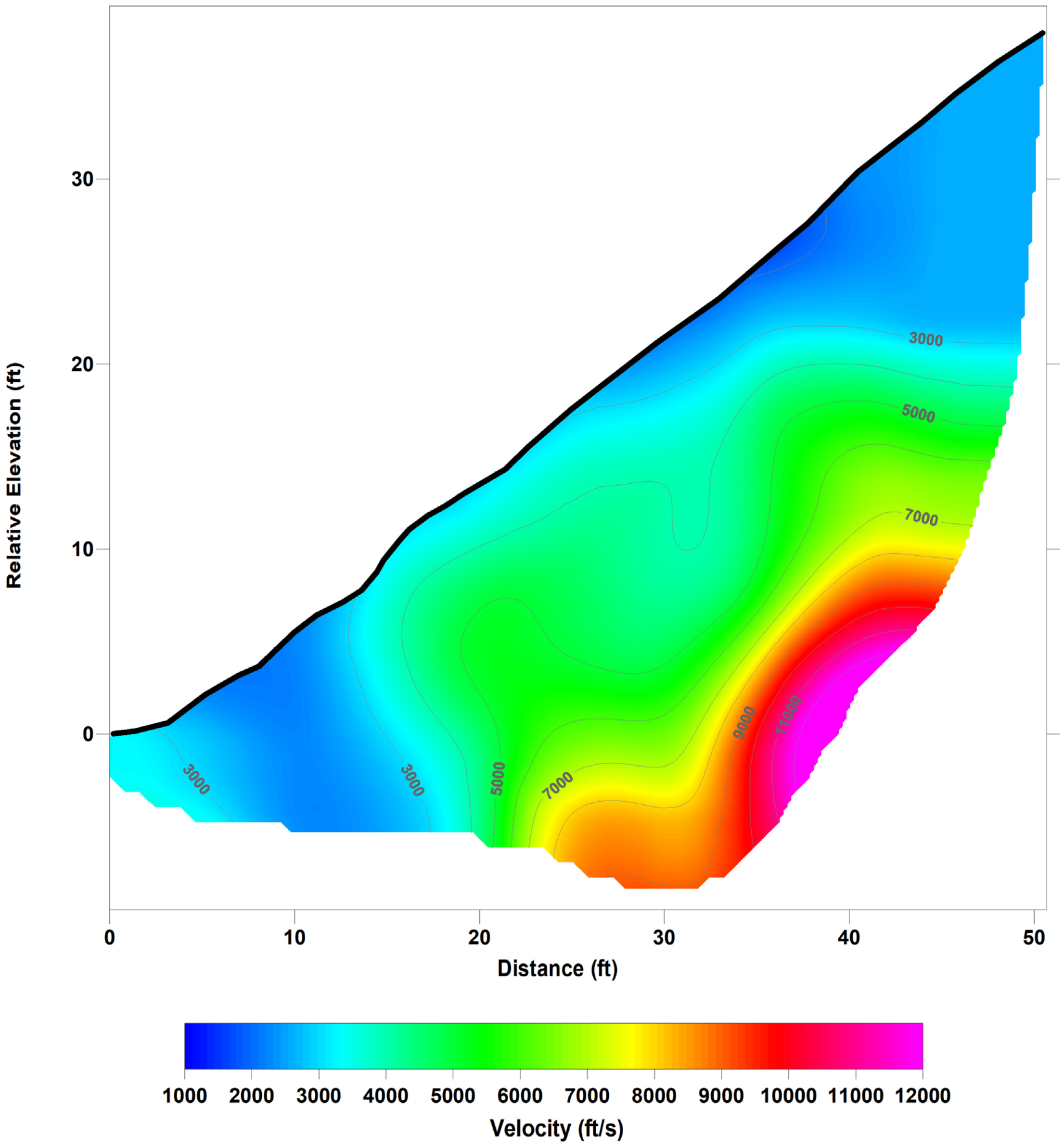
Date: 07/19

SOUTHWEST GEOPHYSICS LLC

Figure 4b

Note: Contour Interval = 1,000 feet per second

TOMOGRAPHY MODEL



P-WAVE PROFILE SL-3

Avalon Ball Field
Avalon, Catalina Island

Project No.: 119318

Date: 07/19

SOUTHWEST GEOPHYSICS LLC

Figure 4c

Note: Contour Interval = 1,000 feet per second

APPENDIX B

GEOTECHNICAL LABORATORY TESTING

Our geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of physical and mechanical properties of soils underlying proposed improvements, and to aid in verifying soil classification.

In-Situ Moisture and Density: As-sampled soil moisture content was measured (ASTM D 2216) on selected samples recovered from our borings. In addition, in place dry density was measured (ASTM D 2937) on selected driven soil samples. Results of these tests are shown on our logs at the appropriate sample depths in Appendix A.

Particle-Size Distribution
The grain-size distribution of a selected samples were evaluated by sieving. The test was performed in general accordance with ASTM D 6913. The results are presented in this appendix.

Direct Shear: Direct shear tests were performed on two drive samples and one remolded sample (90% relative compaction). Three different rings were cut from soil samples then inundated with water and sheared separately at three different normal loads to establish soil friction and cohesion parameters. Results of these tests are presented in this appendix on the *Direct Shear Test Results* sheets.

Modified Proctor Compaction Curve: Laboratory modified Proctor compaction curves (ASTM D 1557) were established for bulk soil samples to determine the modified Proctor laboratory maximum dry density and optimum moisture content. Results of these tests are presented on the following "*Modified Proctor Compaction Test*" sheets in this appendix.

Expansion Index (EI): An Expansion Index (EI) test was performed in accordance with the ASTM D 4829 Standard Test Method, for shallow bulk soil samples from the site. EI results are included in this appendix on the "*Expansion Index of Soils*" sheet.

Corrosivity Tests: To evaluate corrosion potential of subsurface soils at the site, we tested two bulk samples collected during our subsurface exploration for pH, electrical resistivity (CTM 532/643), soluble sulfate content (CTM 417 Part II) and soluble chloride content (CTM 422) testing. Results of these tests are enclosed at the end of this appendix.

Boring No.	TP-1	TP-2	TP-3					
Sample No.	1A	1	1					
Depth (ft.)	8.0	7.5	3.0					
Sample Type	Ring	Ring	Ring					
Soil Identification	Yellowish brown silty sand with gravel (SM)g	Yellowish brown clayey sand with gravel (SC)g	Olive brown silty sand with gravel (SM)g					
Pocket Penetrometer (tons/ft^2)	>4.50/3.25	1.50	>4.50/3.75					
Weight Soil + Rings / Tube (g)	919.9	853.9	894.4					
Weight of Rings / Tube (g)	222.0	222.0	222.0					
Average Length (in.)	5.00	5.00	5.00					
Average Diameter (in.)	2.415	2.415	2.415					
Wet. Wt. of Soil + Cont. (g)	183.3	511.8	658.1					
Dry Wt. of Soil + Cont. (g)	171.5	480.2	620.5					
Weight of Container (g)	59.2	37.3	108.4					
Container No.								
Wet Density	116.1	105.1	111.8					
Moisture Content (%)	**10.5**	**7.1**	**7.3**					
Dry Density (pcf)	**105.0**	**98.1**	**104.2**					
Degree of Saturation (%)	46.9	26.8	32.1					

Leighton

MOISTURE & DENSITY of SOILS

ASTM D 2216 & ASTM D 2937

Project Name: Avalon Ballpark
Project No.: 12396.001
Tested By: GB/RMM
Date: 07/05/19

Borehole	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Dry Density (pcf)	Satur-ation (%)	Void Ratio
HA-1	1.0							5.0	100.0		
HA-4	0.8							11.3	105.4		
HA-4	4.0							6.3	90.3		
HA-5	0.5							5.4	95.3		
HA-6	1.0							5.6	103.0		
HA-6	3.0							7.5	101.5		

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Summary of Laboratory Results

Project Name: Avalon Ballfield
Project Number: 12396.001
Date: 7/19/2019 11:28:01 AM
Figure No. 1

Leighton

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS

ASTM D 6913

Project Name: Avalon Ballfield

Project No.: 12396.001

Boring No.: HA-1

Sample No.: 1

Soil Identification: Yellowish brown silty sand with gravel (SM)g

Tested By: R. Manning Date: 07/10/19

Checked By: J. Ward Date: 07/23/19

Depth (feet): 0-1.8

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	16	11-9	Wt. of Air-Dry Soil + Cont.(g)	0.0	0.0
Wt. Air-Dried Soil + Cont.(g)	4958.4	739.2	Wt. of Dry Soil + Cont. (g)	0.0	0.0
Wt. of Container (g)	231.4	215.8	Wt. of Container No.______(g)	1.0	1.0
Dry Wt. of Soil (g)	4727.0	523.4	Moisture Content (%)	0.0	0.0

Passing #4 Material After Wet Sieve		
	Container No.	11-9
	Wt. of Dry Soil + Container (g)	559.7
	Wt. of Container (g)	215.8
	Dry Wt. of Soil Retained on # 200 Sieve (g)	343.9

U. S. Sieve Size		Cumulative Weight of Dry Soil Retained (g)		Percent Passing (%)
	(mm.)	Whole Sample	Sample Passing #4	
2"	50.0			
1 1/2"	37.5	0.0		100.0
1"	25.0	80.1		98.3
3/4"	19.0	152.6		96.8
1/2"	12.5	326.0		93.1
3/8"	9.5	481.6		89.8
#4	4.75	817.0		82.7
#8	2.36		31.0	77.8
#16	1.18		72.5	71.2
#30	0.600		126.5	62.7
#50	0.300		203.0	50.6
#100	0.150		282.2	38.1
#200	0.075		342.1	28.6
PAN				

GRAVEL: **17 %**

SAND: **54 %**

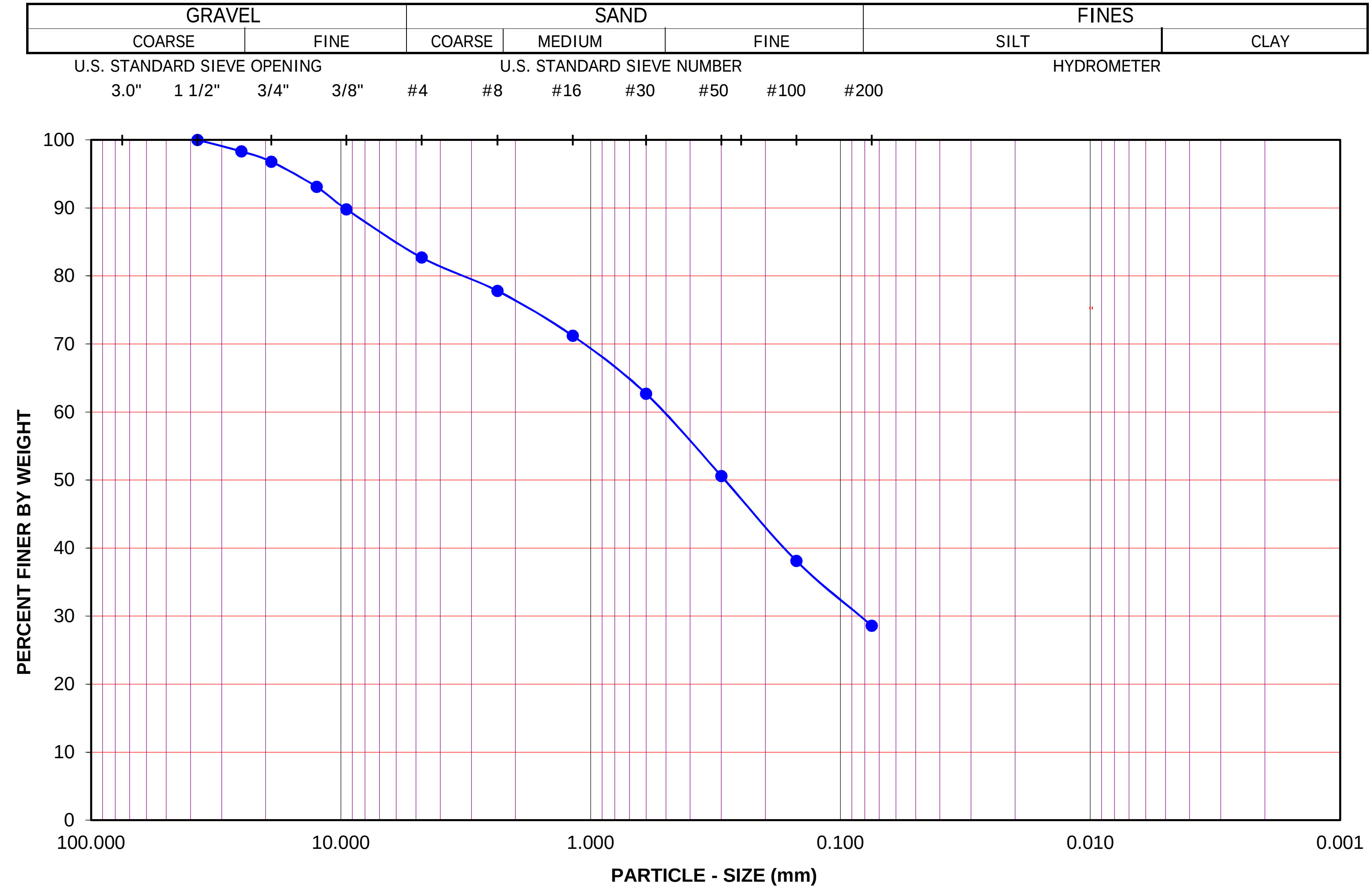
FINES: **29 %**

GROUP SYMBOL: **(SM)g**

Cu = D60/D10 = ______

Cc = $(D30)^2/(D60*D10)$ = ______

Remarks:



Project Name: Avalon Ballfield

Project No.: 12396.001

Boring No.: HA-1

Sample No.: 1

Depth (feet): 0-1.8

Soil Type : (SM)g

Soil Identification: Yellowish brown silty sand with gravel (SM)g

GR:SA:FI : (%) 17 : 54 : 29

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PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Jul-19

Leighton

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS
ASTM D 6913

Project Name: Avalon Ballfield
Project No.: 12396.001
Boring No.: HA-5
Sample No.: 1
Soil Identification: Yellowish brown silty, clayey sand with gravel (SC-SM)g

Tested By: R. Manning Date: 07/10/19
Checked By: J. Ward Date: 07/23/19
Depth (ft.): 0-23"

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	E-2	R-201	Wt. of Air-Dry Soil + Cont.(g)	0.0	0.0
Wt. Air-Dried Soil + Cont.(g)	4797.4	738.2	Wt. of Dry Soil + Cont. (g)	0.0	0.0
Wt. of Container (g)	246.6	219.7	Wt. of Container No.______(g)	1.0	1.0
Dry Wt. of Soil (g)	4550.8	518.5	Moisture Content (%)	0.0	0.0

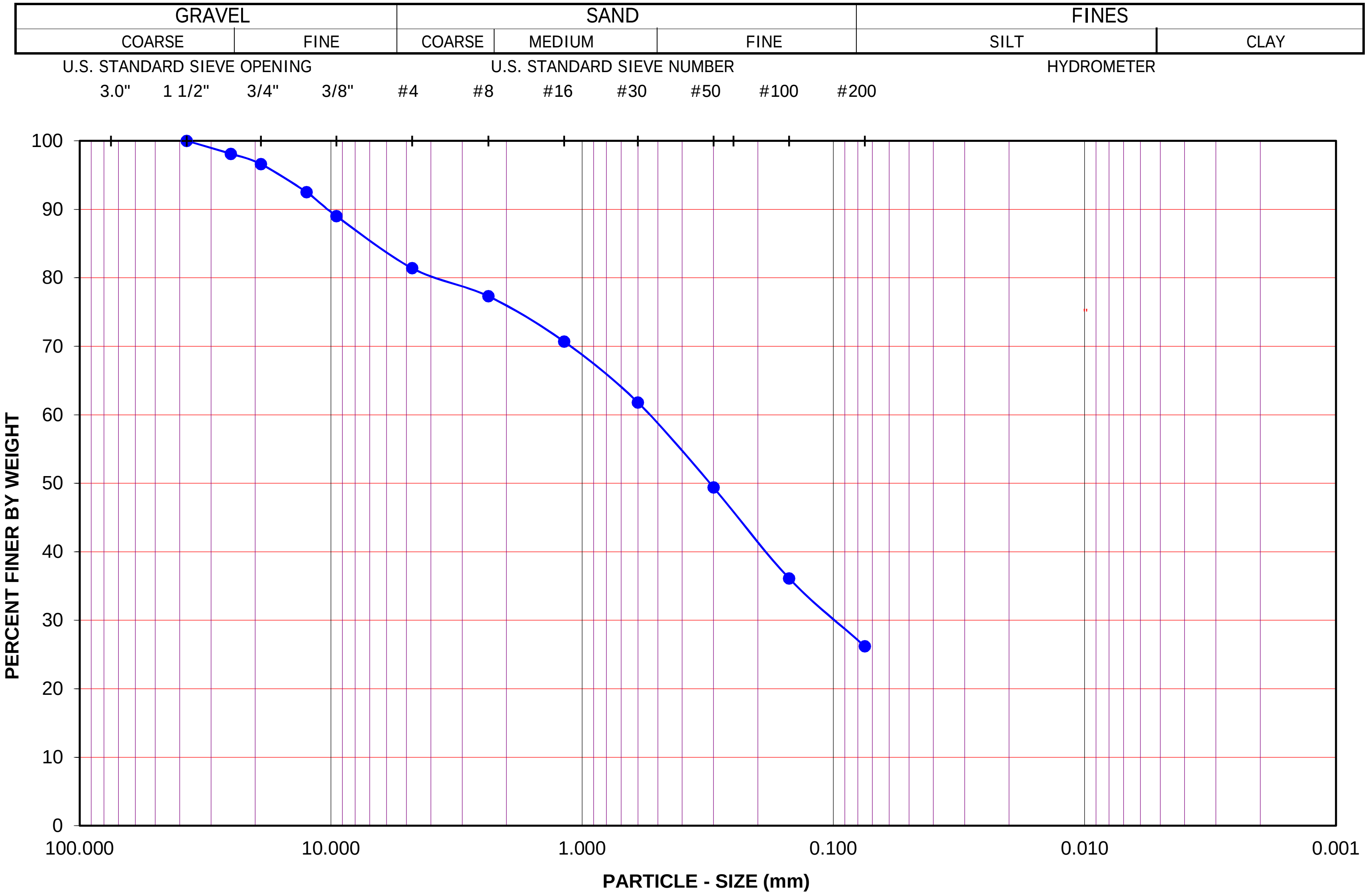
Passing #4 Material After Wet Sieve	Container No.	R-201
	Wt. of Dry Soil + Container (g)	574.2
	Wt. of Container (g)	219.7
	Dry Wt. of Soil Retained on # 200 Sieve (g)	354.5

U. S. Sieve Size		Cumulative Weight of Dry Soil Retained (g)		Percent Passing (%)
	(mm.)	Whole Sample	Sample Passing #4	
2"	50.0			
1 1/2"	37.5	0.0		100.0
1"	25.0	87.4		98.1
3/4"	19.0	154.9		96.6
1/2"	12.5	342.9		92.5
3/8"	9.5	499.3		89.0
#4	4.75	845.8		81.4
#8	2.36		26.0	77.3
#16	1.18		68.3	70.7
#30	0.600		124.7	61.8
#50	0.300		204.0	49.4
#100	0.150		288.4	36.1
#200	0.075		351.6	26.2
PAN				

GRAVEL: **19 %**
SAND: **55 %**
FINES: **26 %**
GROUP SYMBOL: **(SC-SM)g**

Cu = D60/D10 = ________
Cc = $(D30)^2/(D60*D10)$ = ________

Remarks: ____



Project Name: Avalon Ballfield

Project No.: 12396.001

Boring No.: HA-5

Sample No.: 1

Depth (ft.): 0-23"

Soil Type : (SC-SM)g

Soil Identification: Yellowish brown silty, clayey sand with gravel (SC-SM)g

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PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) **19 : 55 : 26**

Jul-19

Leighton

DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name:	Avalon Ballfield	Tested By:	G. Bathala	Date:	07/11/19
Project No.:	12396.001	Checked By:	J. Ward	Date:	07/23/19
Boring No.:	HA-1	Sample Type:	Ring		
Sample No.:	2	Depth (ft.):	1-1.5		

Soil Identification: Olive brown silty, clayey sand (SC-SM)

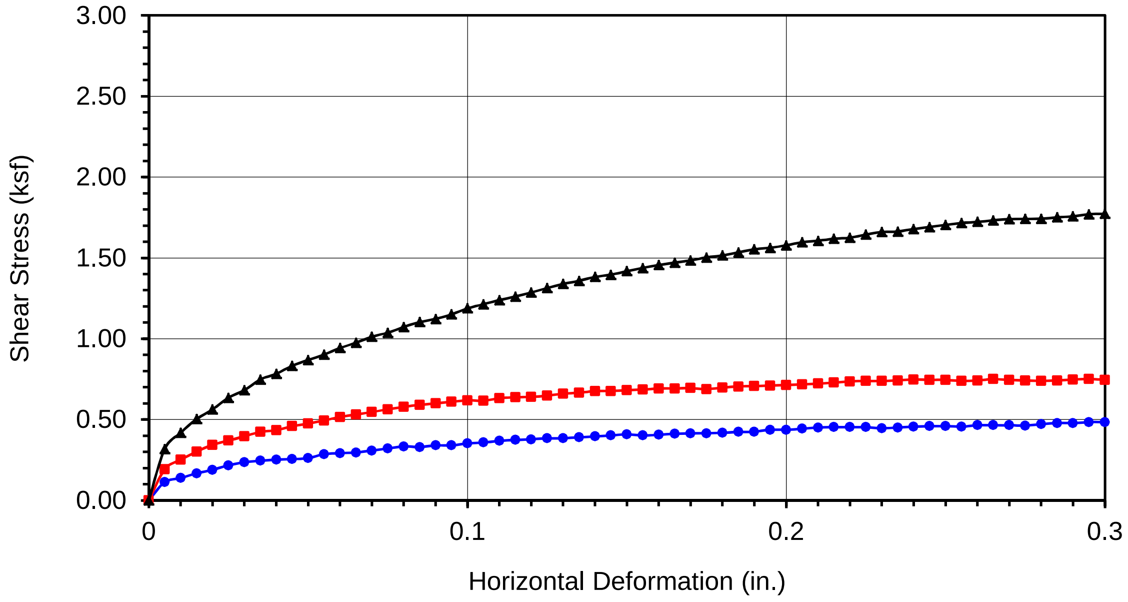
Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	163.92	168.52	170.56
Weight of Ring(gm):	42.79	45.83	45.64

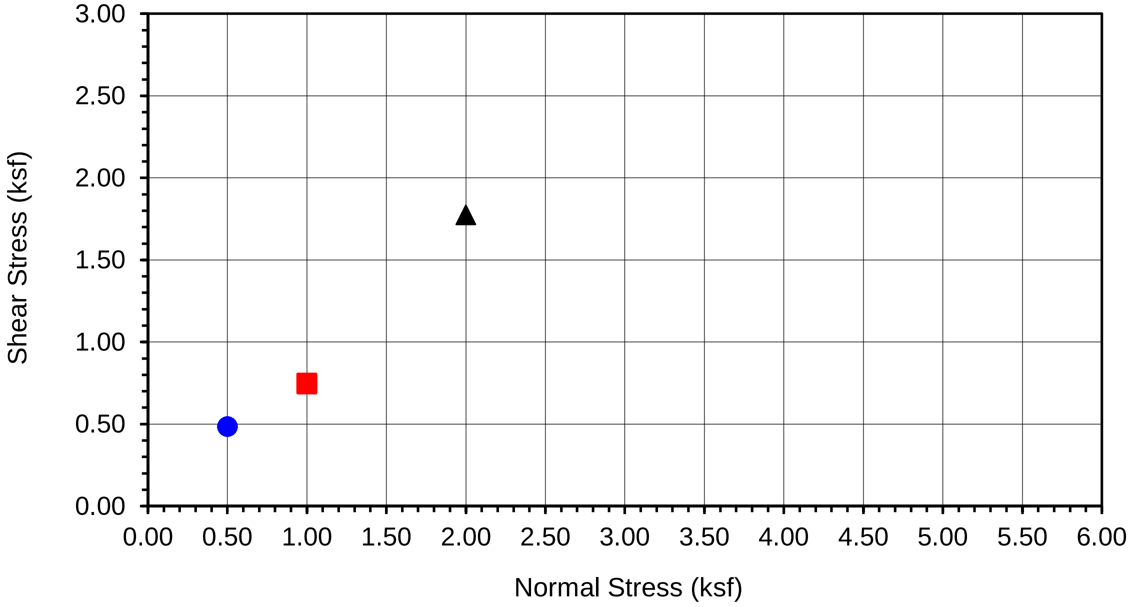
Before Shearing

Weight of Wet Sample+Cont.(gm):	192.27	192.27	192.27
Weight of Dry Sample+Cont.(gm):	184.98	184.98	184.98
Weight of Container(gm):	39.58	39.58	39.58
Vertical Rdg.(in): Initial	0.0000	0.2661	0.2473
Vertical Rdg.(in): Final	-0.0555	0.3401	0.3442

After Shearing

Weight of Wet Sample+Cont.(gm):	189.54	189.78	202.35
Weight of Dry Sample+Cont.(gm):	170.64	170.62	184.12
Weight of Container(gm):	58.19	57.12	67.93
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**HA-1**
Sample No.	**2**
Depth (ft)	**1-1.5**

Sample Type:

Ring

Soil Identification:
Olive brown silty, clayey sand (SC-SM)

Normal Stress (kip/ft^2)	0.500	1.000	2.000
Peak Shear Stress (kip/ft^2)	● 0.484	■ 0.751	▲ 1.773
Shear Stress @ End of Test (ksf)	○ 0.484	□ 0.745	△ 1.773
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	5.01	5.01	5.01
Dry Density (pcf)	95.9	97.2	98.9
Saturation (%)	17.9	18.4	19.2
Soil Height Before Shearing (in.)	0.9445	0.9260	0.9031
Final Moisture Content (%)	16.8	16.9	15.7

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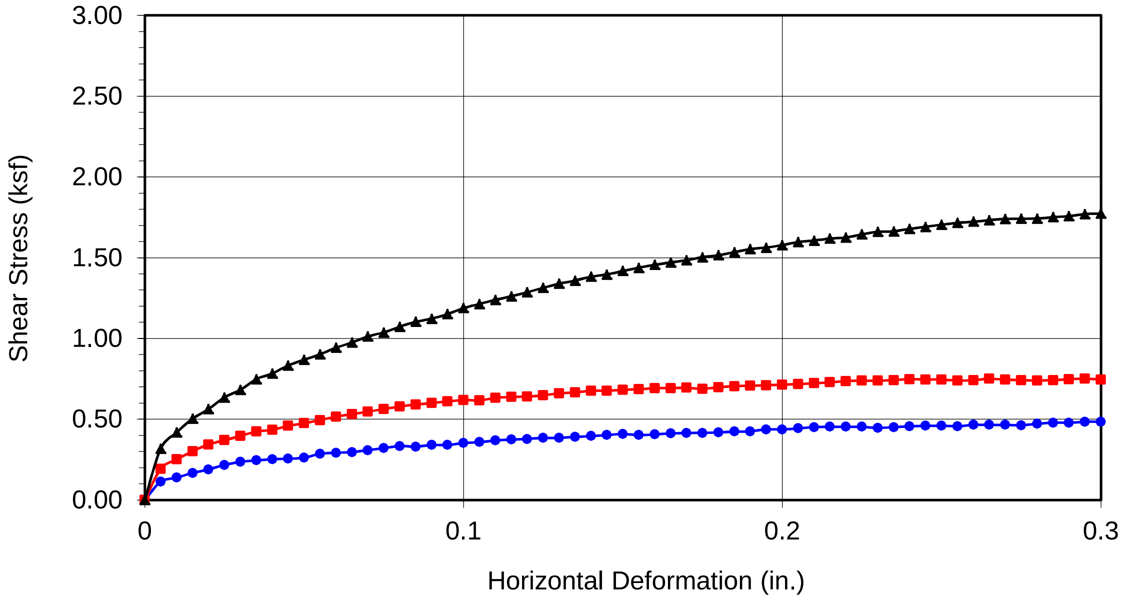
DIRECT SHEAR TEST RESULTS

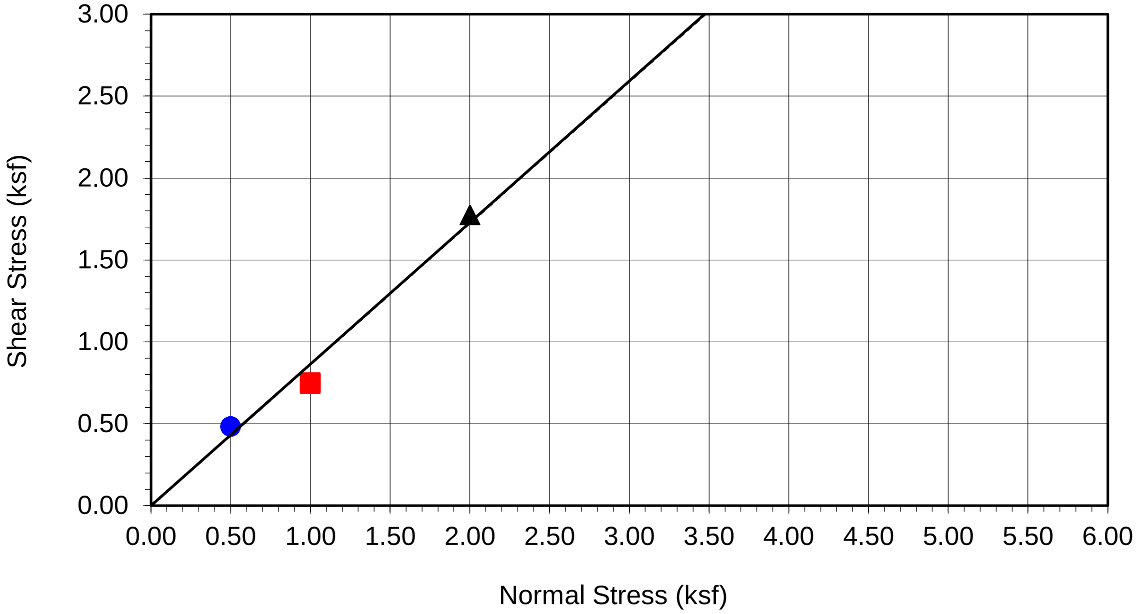
Consolidated Drained - ASTM D 3080

Project No.: 12396.001

Avalon Ballfield

07-19





Boring No.	**HA-1**
Sample No.	**2**
Depth (ft)	**1-1.5**
Sample Type:	Ring

Soil Identification:

Olive brown silty, clayey sand
(SC-SM)

Strength Parameters

	C (psf)	ϕ (°)
Peak	0	41
Ultimate	0	41

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.484	■ 0.751	▲ 1.773
Shear Stress @ End of Test (ksf)	○ 0.484	□ 0.745	△ 1.773
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	5.01	5.01	5.01
Dry Density (pcf)	95.9	97.2	98.9
Saturation (%)	17.9	18.4	19.2
Soil Height Before Shearing (in.)	0.9445	0.9260	0.9031
Final Moisture Content (%)	16.8	16.9	15.7

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DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 12396.001

Avalon Ballfield

07-19

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DIRECT SHEAR TEST

Consolidated Drained - ASTM D 3080

Project Name:	Avalon Ballfield	Tested By:	G. Bathala	Date:	07/10/19
Project No.:	12396.001	Checked By:	J. Ward	Date:	07/23/19
Boring No.:	TP-1	Sample Type:	Ring		
Sample No.:	1A	Depth (ft.):	8.0		

Soil Identification: Yellowish brown silty sand with gravel (SM)g

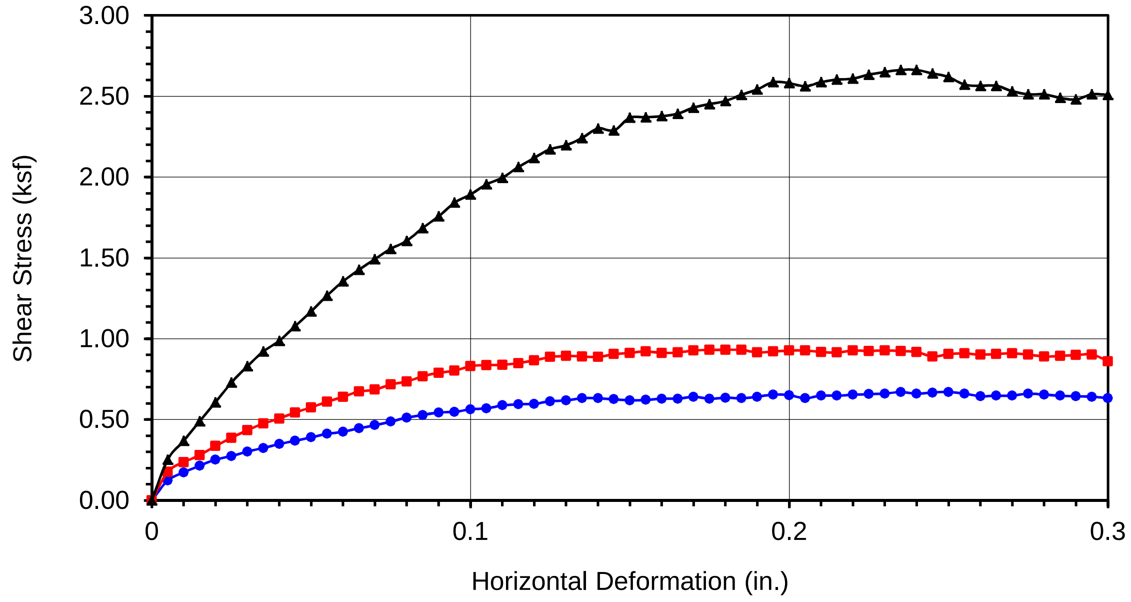
Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	177.34	183.29	185.32
Weight of Ring(gm):	43.31	45.05	43.51

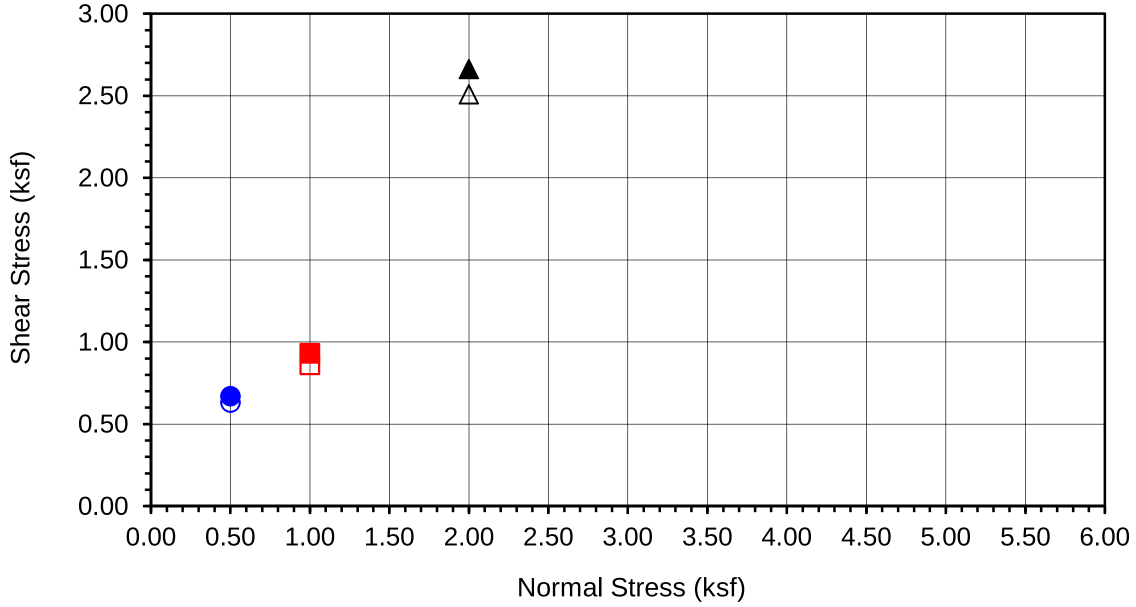
Before Shearing

Weight of Wet Sample+Cont.(gm):	183.30	183.30	183.30
Weight of Dry Sample+Cont.(gm):	171.54	171.54	171.54
Weight of Container(gm):	59.24	59.24	59.24
Vertical Rdg.(in): Initial	0.2477	0.0000	0.2497
Vertical Rdg.(in): Final	0.2619	-0.0086	0.2766

After Shearing

Weight of Wet Sample+Cont.(gm):	205.27	207.93	199.45
Weight of Dry Sample+Cont.(gm):	187.35	189.30	181.91
Weight of Container(gm):	69.53	67.93	57.12
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**TP-1**
Sample No.	**1A**
Depth (ft)	**8**

Sample Type:

Ring

Soil Identification:
Yellowish brown silty sand
with gravel (SM)g

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.670	■ 0.931	▲ 2.663
Shear Stress @ End of Test (ksf)	○ 0.632	□ 0.861	△ 2.509
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	10.47	10.47	10.47
Dry Density (pcf)	100.9	104.1	106.8
Saturation (%)	42.2	45.6	48.8
Soil Height Before Shearing (in.)	0.9858	0.9914	0.9731
Final Moisture Content (%)	15.2	15.3	14.1

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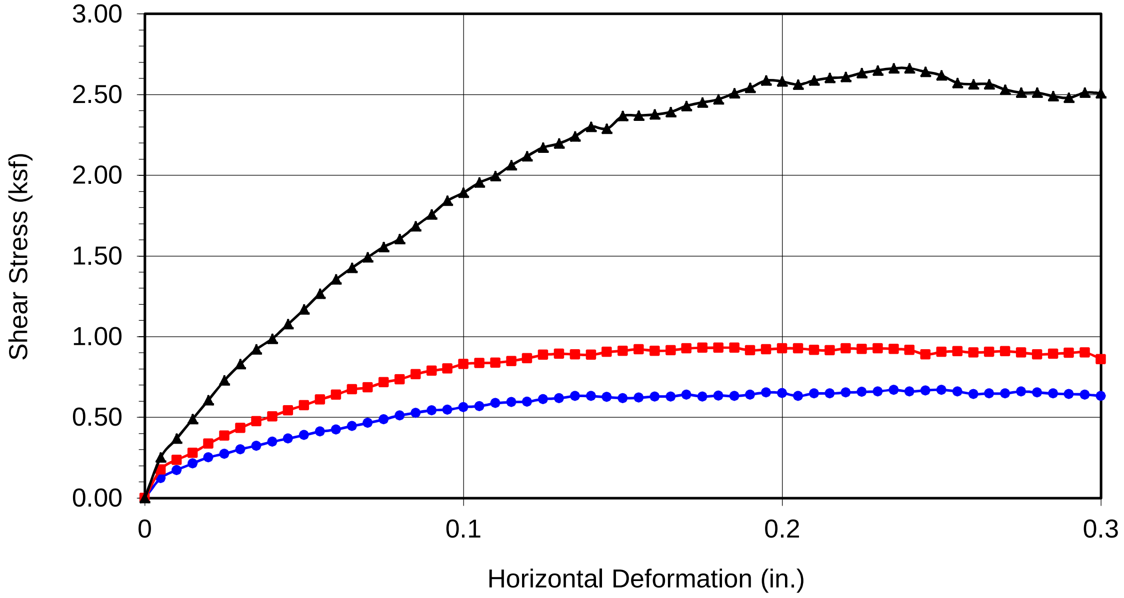
DIRECT SHEAR TEST RESULTS

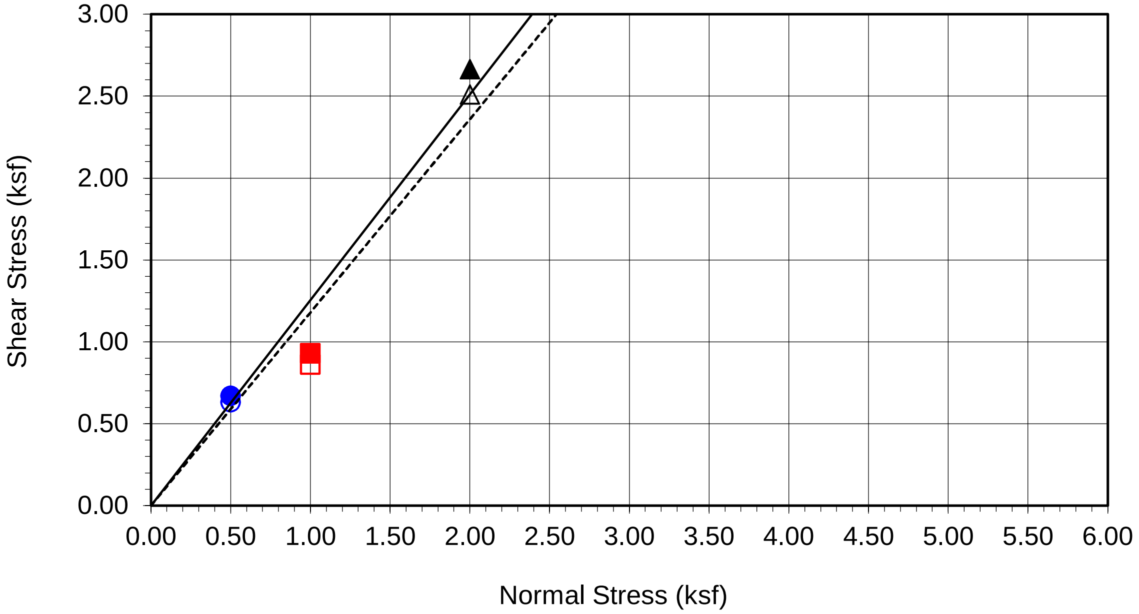
Consolidated Drained - ASTM D 3080

Project No.: 12396.001

Avalon Ballfield

07-19





Boring No.	**TP-1**
Sample No.	**1A**
Depth (ft)	**8**

Sample Type: Ring

Soil Identification:
Yellowish brown silty sand with gravel (SM)g

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	0	51
Ultimate	0	50

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.670	■ 0.931	▲ 2.663
Shear Stress @ End of Test (ksf)	○ 0.632	□ 0.861	△ 2.509
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	10.47	10.47	10.47
Dry Density (pcf)	100.9	104.1	106.8
Saturation (%)	42.2	45.6	48.8
Soil Height Before Shearing (in.)	0.9858	0.9914	0.9731
Final Moisture Content (%)	15.2	15.3	14.1

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DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 12396.001

Avalon Ballfield

07-19

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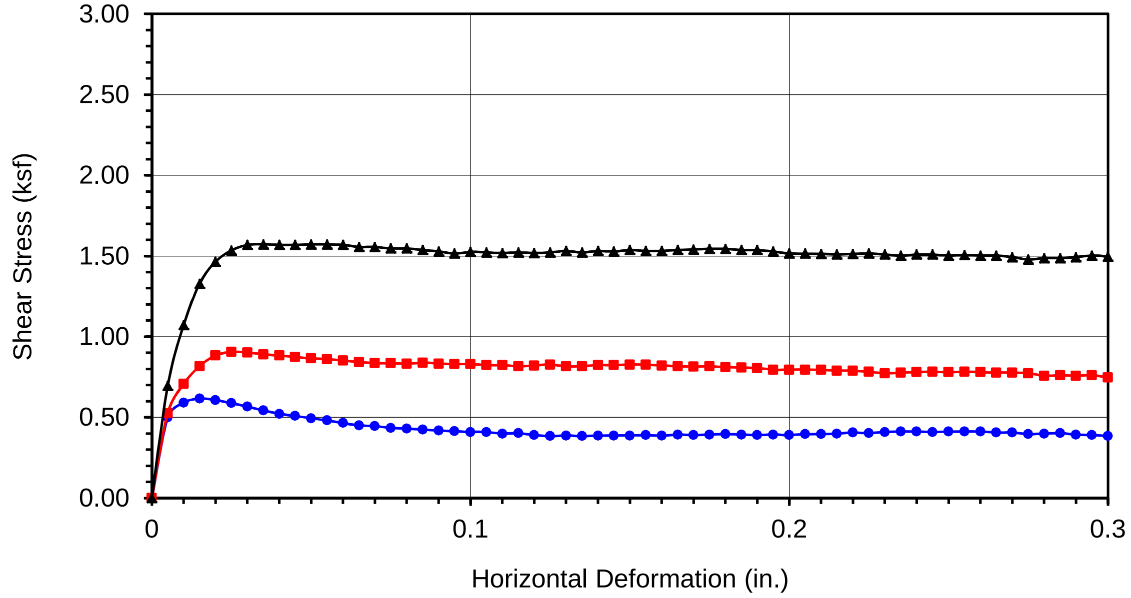
DIRECT SHEAR TEST

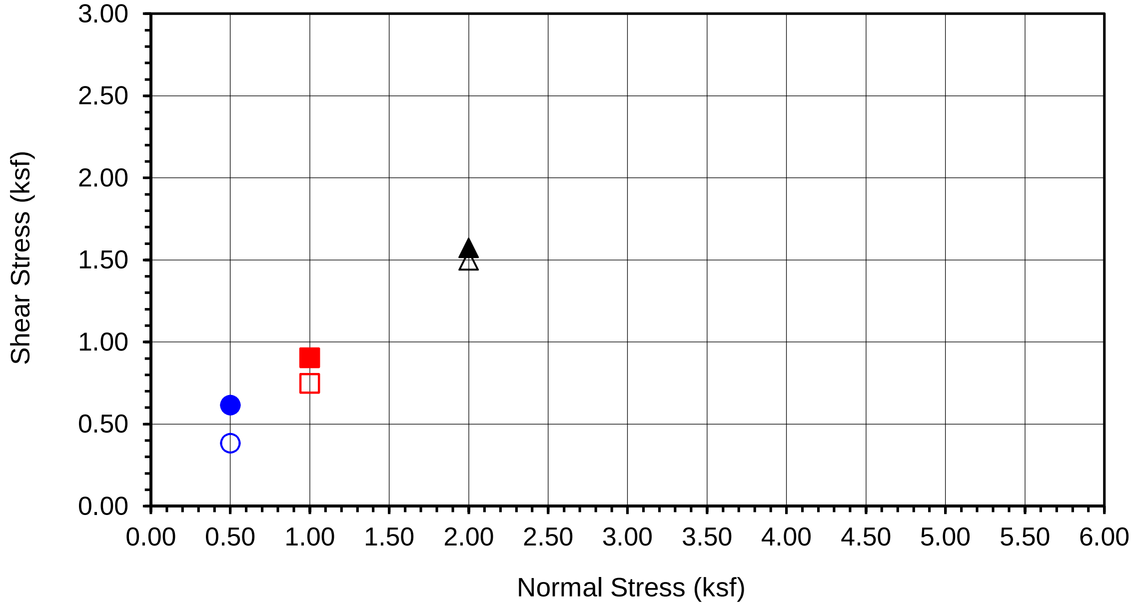
Consolidated Drained - ASTM D 3080

Project Name:	Avalon Ballfield	Tested By:	G. Bathala	Date:	07/10/19
Project No.:	12396.001	Checked By:	J. Ward	Date:	07/23/19
Boring No.:	TP-2	Sample Type:	90% Remold		
Sample No.:	B1	Depth (ft.):	6.5-7.5		

Soil Identification: Yellowish brown silty, clayey gravel with sand (GC-GM)s

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	193.04	194.84	192.66
Weight of Ring(gm):	43.45	45.20	42.98
Before Shearing			
Weight of Wet Sample+Cont.(gm):	146.91	146.91	146.91
Weight of Dry Sample+Cont.(gm):	139.23	139.23	139.23
Weight of Container(gm):	60.64	60.64	60.64
Vertical Rdg.(in): Initial	0.0000	0.2762	0.2642
Vertical Rdg.(in): Final	-0.0025	0.2823	0.2741
After Shearing			
Weight of Wet Sample+Cont.(gm):	215.50	233.52	195.66
Weight of Dry Sample+Cont.(gm):	193.37	212.10	174.95
Weight of Container(gm):	59.22	77.40	39.72
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**TP-2**
Sample No.	**B1**
Depth (ft)	**6.5-7.5**

Sample Type:

90% Remold

Soil Identification:
Yellowish brown silty, clayey gravel with sand (GC-GM)s

Normal Stress (kip/ft^2)	0.500	1.000	2.000
Peak Shear Stress (kip/ft^2)	● 0.616	■ 0.905	▲ 1.572
Shear Stress @ End of Test (ksf)	○ 0.384	□ 0.748	△ 1.496
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	9.77	9.77	9.77
Dry Density (pcf)	113.3	113.4	113.4
Saturation (%)	54.1	54.2	54.2
Soil Height Before Shearing (in.)	0.9975	0.9939	0.9901
Final Moisture Content (%)	16.5	15.9	15.3

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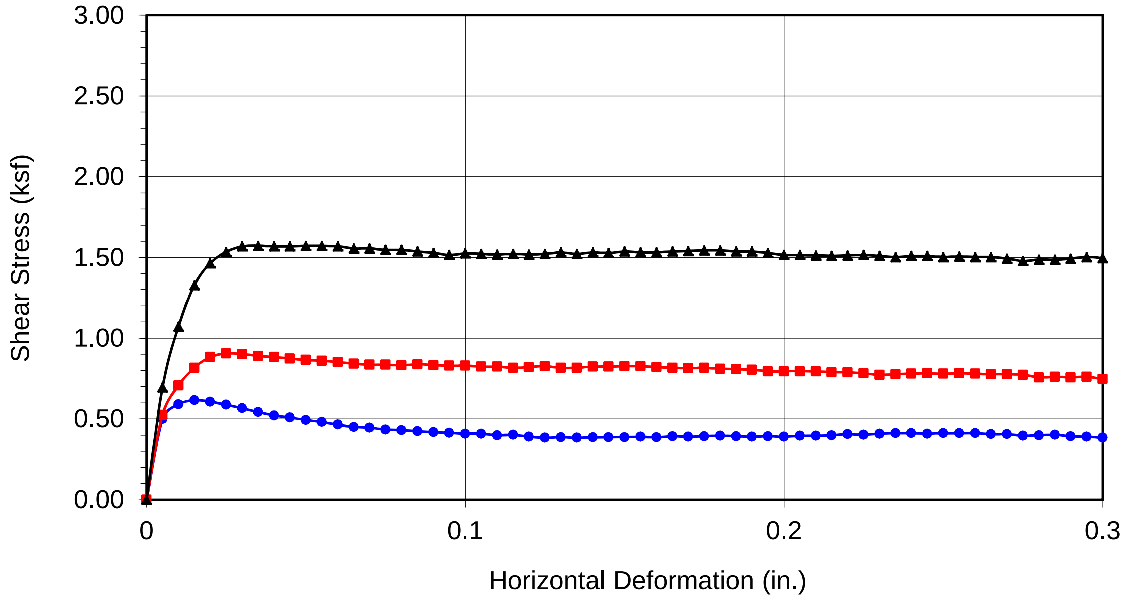
DIRECT SHEAR TEST RESULTS

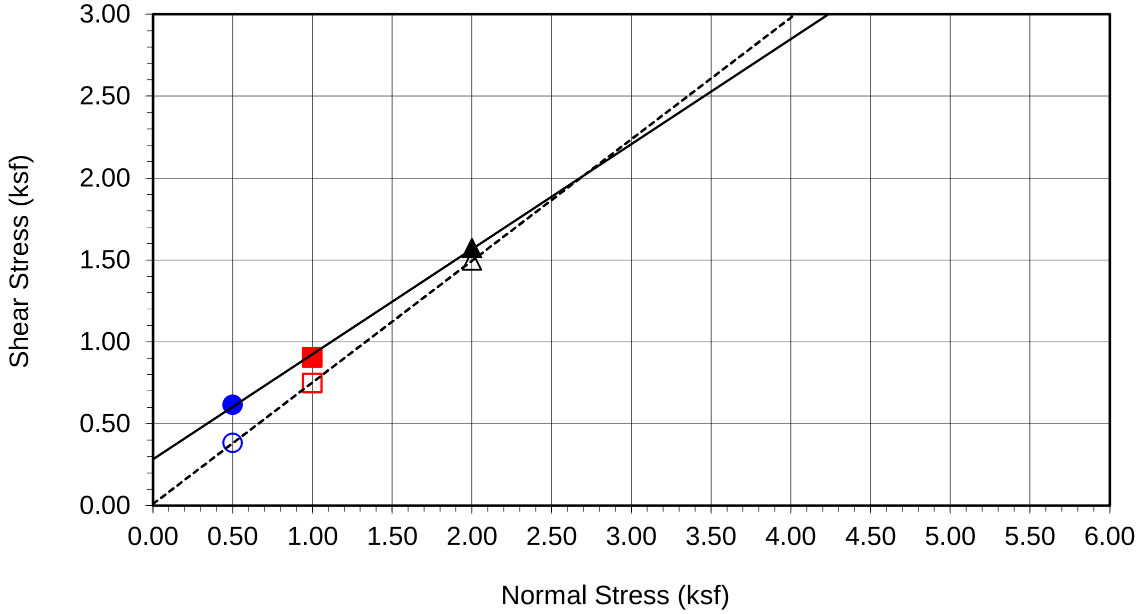
Consolidated Drained - ASTM D 3080

Project No.: 12396.001

Avalon Ballfield

07-19





Boring No.	**TP-2**
Sample No.	**B1**
Depth (ft)	**6.5-7.5**

Sample Type: 90% Remold

Soil Identification:
Yellowish brown silty, clayey gravel with sand (GC-GM)s

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	283	33
Ultimate	10	37

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.616	■ 0.905	▲ 1.572
Shear Stress @ End of Test (ksf)	○ 0.384	□ 0.748	△ 1.496
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	9.77	9.77	9.77
Dry Density (pcf)	113.3	113.4	113.4
Saturation (%)	54.1	54.2	54.2
Soil Height Before Shearing (in.)	0.9975	0.9939	0.9901
Final Moisture Content (%)	16.5	15.9	15.3

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DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 12396.001

Avalon Ballfield

07-19

Leighton

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name: Avalon Ballfield
Tested By: S. Dansby
Date: 07/09/19
Project No.: 12396.001
Input By: G. Bathala
Date: 07/16/19
Boring No.: HA-5
Depth (ft.): 0-23"
Sample No.: 1
Soil Identification: Yellowish brown silty, clayey sand with gravel (SC-SM)g

Note: Corrected dry density calculation assumes specific gravity of 2.70 and moisture content of 1.0% for oversize particles

Preparation Method:	X	Moist
		Dry
Compaction Method	X	Mechanical Ram
		Manual Ram

Scalp Fraction (%)	
#3/4	
#3/8	11.0
#4	

Rammer Weight (lb.) = 10.0
Height of Drop (in.) = 18.0
Mold Volume (ft³) 0.03320

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3818	3898	3850			
Weight of Mold (g)	1830	1830	1830			
Net Weight of Soil (g)	1988	2068	2020			
Wet Weight of Soil + Cont. (g)	437.5	428.4	425.8			
Dry Weight of Soil + Cont. (g)	407.5	391.0	380.7			
Weight of Container (g)	39.4	38.4	39.7			
Moisture Content (%)	8.15	10.61	13.23			
Wet Density (pcf)	132.0	137.3	134.1			
Dry Density (pcf)	122.1	124.2	118.5			

Maximum Dry Density (pcf) 124.3
Optimum Moisture Content (%) 10.1
Corrected Dry Density (pcf) 128.0
Corrected Moisture Content (%) 9.1

☐ **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

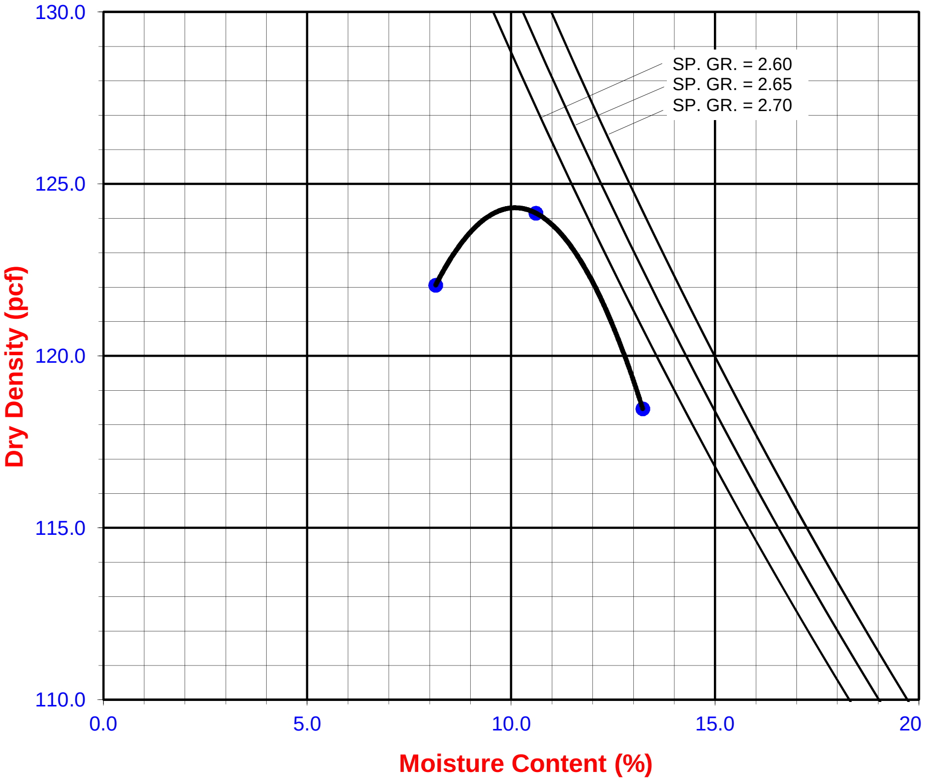
☒ **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

☐ **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:
19:55:26
GR:SA:FI

Atterberg Limits:

LL,PL,PI



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MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name: Avalon Ballfield
Tested By: S. Dansby
Date: 07/08/19
Project No.: 12396.001
Input By: J. Ward
Date: 07/09/19
Boring No.: TP-2
Depth (ft.): 6.5-7.5
Sample No.: B1
Soil Identification: Yellowish brown silty, clayey gravel with sand (GC-GM)s

Note: Corrected dry density calculation assumes specific gravity of 2.70 and moisture content of 1.0% for oversize particles; Correction per ASTM D 4718 is considered valid for soils that include up to 40% oversize particles retained on sieve #4

Preparation Method:	X	Moist
		Dry
Compaction Method	X	Mechanical Ram
		Manual Ram

Scalp Fraction (%)	
#3/4	
#3/8	
#4	47.4

Rammer Weight (lb.) = 10.0
Height of Drop (in.) = 18.0
Mold Volume (ft³) 0.03320

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3780	3905	3908			
Weight of Mold (g)	1830	1830	1830			
Net Weight of Soil (g)	1950	2075	2078			
Wet Weight of Soil + Cont. (g)	481.5	482.7	465.6			
Dry Weight of Soil + Cont. (g)	453.7	444.4	420.4			
Weight of Container (g)	39.4	39.1	40.3			
Moisture Content (%)	6.71	9.45	11.89			
Wet Density (pcf)	129.5	137.8	138.0			
Dry Density (pcf)	121.3	125.9	123.3			

Maximum Dry Density (pcf) 125.9
Optimum Moisture Content (%) 9.7
Corrected Dry Density (pcf) 143.0
Corrected Moisture Content (%) 5.6

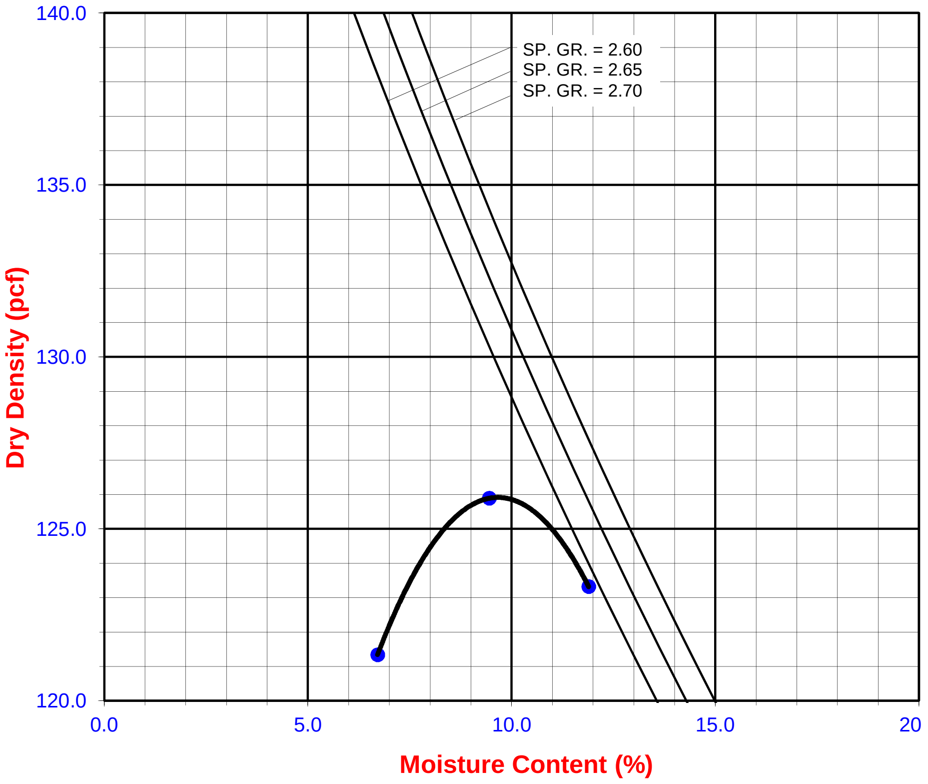
☒ **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

☐ **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

☐ **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:
GR:SA:FI

Atterberg Limits:
LL,PL,PI



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EXPANSION INDEX of SOILS

ASTM D 4829

Project Name: Avalon
Tested By: S. Felter
Date: 10/17/18
Project No.: 12181.001
Checked By: J. Ward
Date: 10/23/18
Location: North Slope
Depth (ft.): 0-1
Sample No.: B1
Soil Identification: Dark yellowish brown poorly-graded gravel with clay and sand (GP-GC)s

Dry Wt. of Soil + Cont.	(g)	1000.00
Wt. of Container No.	(g)	0.00
Dry Wt. of Soil	(g)	1000.00
Weight Soil Retained on #4 Sieve		0.00
Percent Passing # 4		100.00

MOLDED SPECIMEN		Before Test	After Test
Specimen Diameter	(in.)	4.01	4.01
Specimen Height	(in.)	1.0000	1.0095
Wt. Comp. Soil + Mold	(g)	597.00	426.94
Wt. of Mold	(g)	196.60	0.00
Specific Gravity (Assumed)		2.70	2.70
Container No.		O	O
Wet Wt. of Soil + Cont.	(g)	794.00	623.54
Dry Wt. of Soil + Cont.	(g)	720.50	559.94
Wt. of Container	(g)	0.00	196.60
Moisture Content	(%)	10.20	17.50
Wet Density	(pcf)	120.8	127.6
Dry Density	(pcf)	109.6	108.6
Void Ratio		0.538	0.553
Total Porosity		0.350	0.356
Pore Volume	(cc)	72.4	74.4
Degree of Saturation (%) [S_{meas}]		**51.2**	85.5

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
10/17/18	12:50	1.0	0	0.0320
10/17/18	13:00	1.0	10	0.0320
Add Distilled Water to the Specimen				
10/17/18	13:15	1.0	15	0.0360
10/18/18	6:30	1.0	1050	0.0415
10/18/18	7:37	1.0	1117	0.0415

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**10**

Leighton

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name: Avalon Tested By : A. Santos Date: 10/17/18
Project No. : 12181.001 Data Input By: J. Ward Date: 10/23/18

Location	North Slope			
Sample No.	B1			
Sample Depth (ft)	0-1			
Soil Identification:	Dark yellowish brown (GP-GC)s			
Wet Weight of Soil + Container (g)	128.08			
Dry Weight of Soil + Container (g)	126.70			
Weight of Container (g)	57.22			
Moisture Content (%)	1.99			
Weight of Soaked Soil (g)	100.30			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	308			
Crucible No.	11			
Furnace Temperature (°C)	860			
Time In / Time Out	11:20/12:05			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	22.1661			
Wt. of Crucible (g)	22.1475			
Wt. of Residue (g) (A)	0.0186			
PPM of Sulfate (A) x 41150	765.39			
PPM of Sulfate, Dry Weight Basis	**781**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	0.8			
PPM of Chloride (C -0.2) * 100 * 30 / B	60			
PPM of Chloride, Dry Wt. Basis	**61**			

pH TEST, DOT California Test 643

pH Value	**7.85**			
Temperature °C	20.2			

Leighton

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name: Avalon

Project No. : 12181.001

Location : North Slope

Sample No. : B1

Tested By : A. Santos Date: 10/18/18

Data Input By: J. Ward Date: 10/23/18

Depth (ft.) : 0-1

Soil Identification:* Dark yellowish brown (GP-GC)s

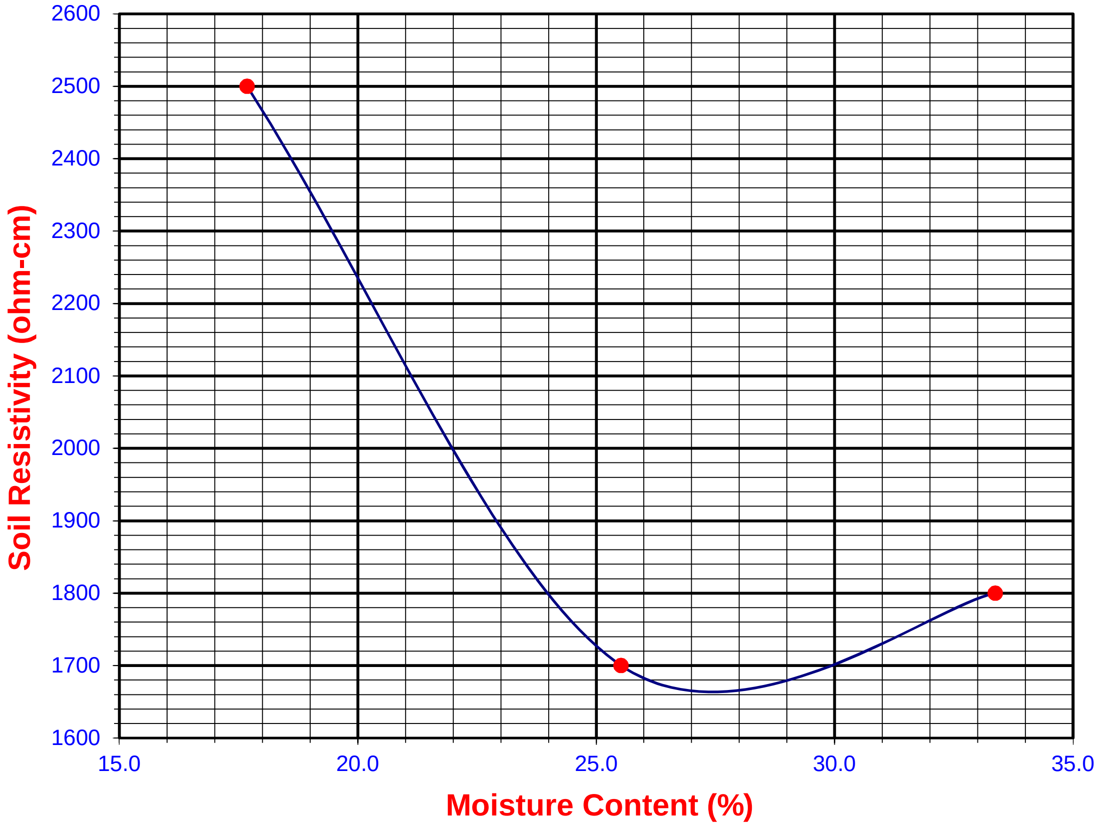
*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	17.68	2500	2500
2	30	25.52	1700	1700
3	40	33.37	1800	1800
4				
5				

Moisture Content (%) (MCi)	1.99
Wet Wt. of Soil + Cont. (g)	128.08
Dry Wt. of Soil + Cont. (g)	126.70
Wt. of Container (g)	57.22
Container No.	
Initial Soil Wt. (g) (Wt)	130.00
Box Constant	1.000

MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
1660	**27.5**	**781**	**61**	**7.85**	**20.2**



APPENDIX C

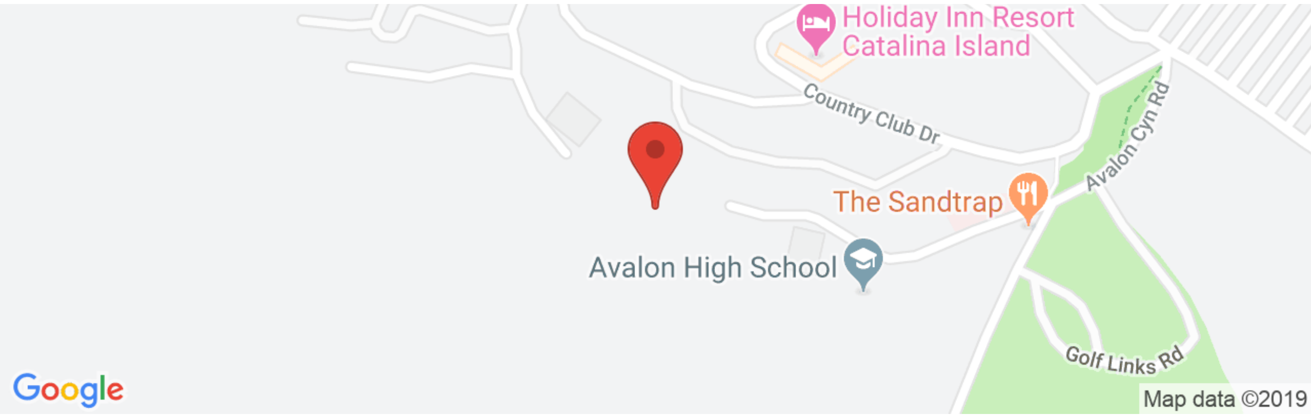
SEISMICITY DATA

Leighton

OSHPD

Avalon School Ballfield

Latitude, Longitude: 33.3392, -118.3341



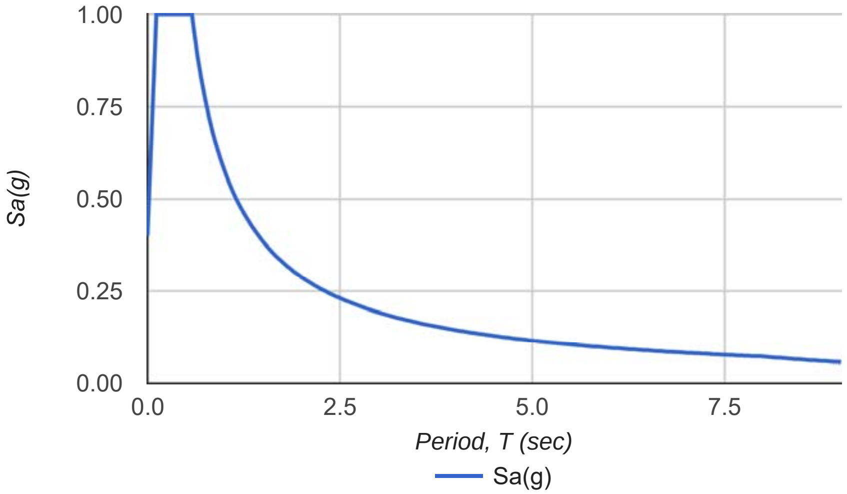
Date	9/10/2019, 7:27:51 AM
Design Code Reference Document	ASCE7-10
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	0.867	MCE_R ground motion. (for 0.2 second period)
S_1	0.332	MCE_R ground motion. (for 1.0s period)
S_{MS}	1	Site-modified spectral acceleration value
S_{M1}	0.577	Site-modified spectral acceleration value
S_{DS}	0.666	Numeric seismic design value at 0.2 second SA
S_{D1}	0.384	Numeric seismic design value at 1.0 second SA

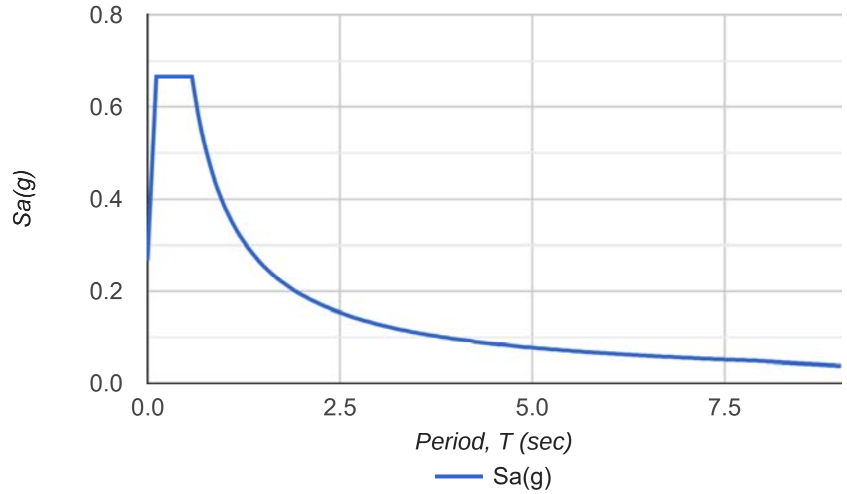
Type	Value	Description
SDC	D	Seismic design category
F_a	1.153	Site amplification factor at 0.2 second
F_v	1.735	Site amplification factor at 1.0 second
PGA	0.333	MCE_G peak ground acceleration
F_{PGA}	1.167	Site amplification factor at PGA
PGA_M	0.388	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	0.867	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	0.862	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.332	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.305	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	1.005	Mapped value of the risk coefficient at short periods

Type	Value	Description
C_{R1}	1.09	Mapped value of the risk coefficient at a period of 1 s

MCER Response Spectrum



Design Response Spectrum



DISCLAIMER

2008 National Seismic Hazard Maps - Source Parameters

New Search

Distance in Miles	Name	State	Pref Slip Rate (mm/yr)	Dip (degrees)	Dip Dir	Slip Sense	Rupture Top (km)	Rupture Bottom (km)	Length (km)
17.34	Palos Verdes	CA	3	90	V	strike slip	0	14	99
17.34	Palos Verdes Connected	CA	3	90	V	strike slip	0	10	285
24.43	Coronado Bank	CA	3	90	V	strike slip	0	9	186
29.76	Newport Inglewood Connected alt 1	CA	1.3	89		strike slip	0	11	208
29.77	Newport-Inglewood (Offshore)	CA	1.5	90	V	strike slip	0	10	66
29.77	Newport Inglewood Connected alt 2	CA	1.3	90	V	strike slip	0	11	208
29.78	Newport-Inglewood, alt 1	CA	1	88		strike slip	0	15	65
33.70	San Joaquin Hills	CA	0.5	23	SW	thrust	2	13	27
40.72	Puente Hills (Santa Fe Springs)	CA	0.7	29	N	thrust	2.8	15	11
41.66	Puente Hills (Coyote Hills)	CA	0.7	26	N	thrust	2.8	15	17
43.32	Puente Hills (LA)	CA	0.7	27	N	thrust	2.1	15	22
47.80	Elsinore;W+GI+T+J+CM	CA	n/a	84	NE	strike slip	0	16	241
47.80	Elsinore;W+GI+T	CA	n/a	84	NE	strike slip	0	14	124

47.80	Elsinore;W+GI	CA	n/a	81	NE	strike slip	0	14	83
47.80	Elsinore;W	CA	2.5	75	NE	strike slip	0	14	46
47.80	Elsinore;W+GI+T+J	CA	n/a	84	NE	strike slip	0	16	199
48.36	Santa Monica Connected alt 2	CA	2.4	44		strike slip	0.8	11	93
48.47	Anacapa-Dume, alt 2	CA	3	41	N	thrust	1.2	12	65
48.48	Santa Monica, alt 1	CA	1	75	N	strike slip	0	18	14
48.48	Santa Monica Connected alt 1	CA	2.6	51		strike slip	0	16	79
48.79	Malibu Coast, alt 1	CA	0.3	75	N	strike slip	0	8	38
48.79	Malibu Coast, alt 2	CA	0.3	74	N	strike slip	0	16	38
49.04	Anacapa-Dume, alt 1	CA	3	45	N	thrust	0	16	51
50.52	Elysian Park (Upper)	CA	1.3	50	NE	reverse	3	15	20
51.51	Hollywood	CA	1	70	N	strike slip	0	17	17
54.30	Raymond	CA	1.5	79	N	strike slip	0	16	22
54.56	Elsinore;GI+T+J	CA	n/a	86	NE	strike slip	0	17	153
54.56	Elsinore;GI+T	CA	5	90	V	strike slip	0	14	78
54.56	Elsinore;GI+T+J+CM	CA	n/a	86	NE	strike slip	0	16	195
54.56	Elsinore;GI	CA	5	90	V	strike slip	0	13	37
54.77	Rose Canyon	CA	1.5	90	V	strike slip	0	8	70
54.89	San Jose	CA	0.5	74	NW		0	15	20

						strike slip			
55.24	Chino, alt 1	CA	1	50	SW	strike slip	0	9	24
55.37	Chino, alt 2	CA	1	65	SW	strike slip	0	14	29
55.56	Verdugo	CA	0.5	55	NE	reverse	0	15	29
59.03	Elsinore;T+J	CA	n/a	86	NE	strike slip	0	17	127
59.03	Elsinore;T+J+CM	CA	n/a	85	NE	strike slip	0	16	169
59.03	Elsinore;T	CA	5	90	V	strike slip	0	14	52
59.65	Sierra Madre	CA	2	53	N	reverse	0	14	57
59.65	Sierra Madre Connected	CA	2	51		reverse	0	14	76
60.91	Clamshell-Sawpit	CA	0.5	50	NW	reverse	0	14	16

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                           *                       *
                           *    E Q S E A R C H    *
                           *                       *
                           *     Version 3.00      *
                           *                       *
                           *************************

                                 ESTIMATION OF
                            PEAK ACCELERATION FROM
                        CALIFORNIA EARTHQUAKE CATALOGS


JOB NUMBER: 12396.001
                                                     DATE: 09-10-2019

JOB NAME: Avalon School

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT


MAGNITUDE RANGE:
   MINIMUM MAGNITUDE:  4.00
   MAXIMUM MAGNITUDE:  9.00

SITE COORDINATES:
   SITE LATITUDE:  33.3392
   SITE LONGITUDE:  118.3341

SEARCH DATES:
           START DATE:   1800
           END DATE:   2019

SEARCH RADIUS:
           62.0 mi
           99.8 km

ATTENUATION RELATION:   3) Boore et al. (1997) Horiz. - NEHRP D (250)
   UNCERTAINTY (M=Median, S=Sigma): M       Number of Sigmas:  0.0
   ASSUMED SOURCE TYPE:  DS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
   SCOND:   0  Depth Source:  A
   Basement Depth:  5.00 km     Campbell SSR:        Campbell SHR:
   COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km):  0.0
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                          EARTHQUAKE SEARCH RESULTS
                          -------------------------

Page  1
-------------------------------------------------------------------------------
    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
DMG |33.3670|118.1500|04/16/1942| 72833.0|  0.0| 4.00| 0.061 | VI | 10.8( 17.4)
DMG |33.5000|118.2500|06/18/1920|10 8 0.0|  0.0| 4.50| 0.074 | VII| 12.1( 19.5)
DMG |33.5430|118.3400|09/14/1963| 35116.2|  2.2| 4.20| 0.056 | VI | 14.1( 22.6)
PAS |33.5380|118.2070|05/25/1982|134430.3| 13.7| 4.10| 0.050 | VI | 15.6( 25.0)
PAS |33.4710|118.0610|02/27/1984|101815.0|  6.0| 4.00| 0.042 | VI | 18.2( 29.3)
DMG |33.5170|118.1000|03/22/1941| 82240.0|  0.0| 4.00| 0.042 | VI | 18.2( 29.3)
PAS |33.5080|118.0710|11/20/1988| 53928.7|  6.0| 4.50| 0.052 | VI | 19.1( 30.8)
DMG |33.6330|118.4000|10/17/1934| 938 0.0|  0.0| 4.00| 0.038 |  V | 20.6( 33.2)
DMG |33.6300|118.2000|09/13/1929|132338.2|  0.0| 4.00| 0.037 |  V | 21.5( 34.6)
DMG |33.6320|118.4670|01/08/1967| 73730.4| 11.4| 4.00| 0.037 |  V | 21.6( 34.8)
DMG |33.6330|118.2000|11/01/1940|20 046.0|  0.0| 4.00| 0.037 |  V | 21.7( 34.9)
DMG |33.5610|118.0580|01/15/1937|183547.0| 10.0| 4.00| 0.036 |  V | 22.1( 35.5)
GSP |33.6583|118.3722|05/15/2013|200006.2|  1.1| 4.08| 0.038 |  V | 22.1( 35.6)
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DMG |33.6630|118.4130|01/08/1967| 738 5.3| 17.7| 4.00| 0.035 |  V | 22.8( 36.7)
DMG |33.6170|118.1170|01/20/1934|2117 0.0|  0.0| 4.50| 0.046 | VI | 22.9( 36.8)
DMG |33.5670|117.9830|07/07/1937|1112 0.0|  0.0| 4.00| 0.032 |  V | 25.6( 41.2)
DMG |33.5670|117.9830|04/17/1934|1833 0.0|  0.0| 4.00| 0.032 |  V | 25.6( 41.2)
DMG |33.6000|118.0170|12/25/1935|1715 0.0|  0.0| 4.50| 0.042 | VI | 25.6( 41.3)
DMG |33.6170|118.0330|05/21/1938| 944 0.0|  0.0| 4.00| 0.032 |  V | 25.9( 41.6)
DMG |33.5750|117.9830|03/11/1933| 518 4.0|  0.0| 5.20| 0.060 | VI | 26.0( 41.8)
DMG |33.6000|118.0000|03/11/1933| 217 0.0|  0.0| 4.50| 0.041 |  V | 26.3( 42.4)
DMG |33.6000|118.0000|03/11/1933| 231 0.0|  0.0| 4.40| 0.039 |  V | 26.3( 42.4)
DMG |33.6170|118.0170|03/14/1933|19 150.0|  0.0| 5.10| 0.056 | VI | 26.5( 42.6)
DMG |33.6170|118.0170|10/02/1933|1326 1.0|  0.0| 4.00| 0.031 |  V | 26.5( 42.6)
DMG |33.6170|118.0170|03/15/1933|111332.0|  0.0| 4.90| 0.051 | VI | 26.5( 42.6)
DMG |33.6170|117.9670|03/11/1933| 154 7.8|  0.0| 6.30| 0.100 | VII| 28.5( 45.9)
DMG |33.6830|118.0500|03/11/1933|1250 0.0|  0.0| 4.40| 0.036 |  V | 28.8( 46.4)
DMG |33.6830|118.0500|03/11/1933| 658 3.0|  0.0| 5.50| 0.065 | VI | 28.8( 46.4)
DMG |33.6540|117.9940|10/20/1961|194950.5|  4.6| 4.30| 0.034 |  V | 29.3( 47.1)
DMG |33.7000|118.0670|02/08/1940|165617.0|  0.0| 4.00| 0.029 |  V | 29.3( 47.1)
DMG |33.7000|118.0670|03/11/1933| 51022.0|  0.0| 5.10| 0.052 | VI | 29.3( 47.1)
DMG |33.7000|118.0670|07/20/1940| 4 113.0|  0.0| 4.00| 0.029 |  V | 29.3( 47.1)
DMG |33.7000|118.0670|03/11/1933| 85457.0|  0.0| 5.10| 0.052 | VI | 29.3( 47.1)
DMG |33.7590|118.2530|08/31/1938| 31814.2| 10.0| 4.50| 0.038 |  V | 29.4( 47.2)
DMG |33.6710|118.0120|10/20/1961|223534.2|  5.6| 4.10| 0.031 |  V | 29.5( 47.4)
DMG |33.7500|118.1830|08/04/1933| 41748.0|  0.0| 4.00| 0.029 |  V | 29.7( 47.7)
DMG |33.7500|118.1670|05/16/1933|205855.0|  0.0| 4.00| 0.029 |  V | 29.9( 48.2)
DMG |33.6590|117.9810|10/20/1961|20 714.5|  6.1| 4.00| 0.029 |  V | 30.0( 48.3)
DMG |33.7670|118.4500|10/11/1940| 55712.3|  0.0| 4.70| 0.041 |  V | 30.3( 48.7)
DMG |33.7330|118.1000|03/11/1933|1447 0.0|  0.0| 4.40| 0.035 |  V | 30.3( 48.8)
DMG |33.7330|118.1000|03/11/1933|15 9 0.0|  0.0| 4.40| 0.035 |  V | 30.3( 48.8)
DMG |33.7330|118.1000|03/11/1933|1350 0.0|  0.0| 4.40| 0.035 |  V | 30.3( 48.8)
DMG |33.6650|117.9790|10/20/1961|214240.7|  7.2| 4.00| 0.028 |  V | 30.4( 48.9)
DMG |33.7500|118.1330|03/11/1933|11 4 0.0|  0.0| 4.60| 0.039 |  V | 30.6( 49.3)
DMG |33.6800|117.9930|11/20/1961| 85334.7|  4.4| 4.00| 0.028 |  V | 30.6( 49.3)
PAS |33.0330|117.9440|02/22/1983| 21830.4| 10.0| 4.30| 0.033 |  V | 30.9( 49.7)
DMG |33.7700|118.4800|04/24/1931|182754.8|  0.0| 4.40| 0.035 |  V | 30.9( 49.7)
DMG |33.7830|118.4170|11/02/1940| 25826.0|  0.0| 4.00| 0.028 |  V | 31.0( 49.9)
DMG |33.7830|118.4170|10/14/1940|205111.0|  0.0| 4.00| 0.028 |  V | 31.0( 49.9)
DMG |33.7830|118.4170|11/01/1940| 725 3.0|  0.0| 4.00| 0.028 |  V | 31.0( 49.9)
DMG |33.7830|118.4170|10/12/1940| 024 0.0|  0.0| 4.00| 0.028 |  V | 31.0( 49.9)
DMG |33.7830|118.2500|11/14/1941| 84136.3|  0.0| 5.40| 0.058 | VI | 31.0( 49.9)
DMG |33.0380|118.7340|09/13/1937|221439.5| 10.0| 4.00| 0.028 |  V | 31.1( 50.0)




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                              EARTHQUAKE SEARCH RESULTS
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Page  2
-------------------------------------------------------------------------------
    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
DMG |33.7830|118.2000|12/27/1939|192849.0|  0.0| 4.70| 0.040 |  V | 31.6( 50.8)
GSP |33.6200|117.9000|04/07/1989|200730.2| 13.0| 4.50| 0.036 |  V | 31.6( 50.9)
DMG |33.7500|118.0830|03/11/1933|23 5 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 832 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 759 0.0|  0.0| 4.10| 0.029 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/31/1933|1049 0.0|  0.0| 4.10| 0.029 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 911 0.0|  0.0| 4.40| 0.034 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/30/1933|1225 0.0|  0.0| 4.40| 0.034 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933|2231 0.0|  0.0| 4.40| 0.034 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|04/02/1933|1536 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 751 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/18/1933|2052 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/12/1933|1738 0.0|  0.0| 4.50| 0.036 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/12/1933|1825 0.0|  0.0| 4.10| 0.029 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/23/1933| 840 0.0|  0.0| 4.10| 0.029 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/12/1933|2354 0.0|  0.0| 4.50| 0.036 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/23/1933|1831 0.0|  0.0| 4.10| 0.029 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/25/1933|1346 0.0|  0.0| 4.10| 0.029 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 2 5 0.0|  0.0| 4.30| 0.032 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|04/02/1933| 8 0 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933|2240 0.0|  0.0| 4.40| 0.034 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933|1025 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933|1045 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933|11 0 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/21/1933| 326 0.0|  0.0| 4.10| 0.029 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 524 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/12/1933| 546 0.0|  0.0| 4.40| 0.034 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/12/1933| 6 1 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/12/1933| 616 0.0|  0.0| 4.60| 0.037 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 2 9 0.0|  0.0| 5.00| 0.046 | VI | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 210 0.0|  0.0| 4.60| 0.037 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/12/1933|15 2 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 3 5 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 3 9 0.0|  0.0| 4.40| 0.034 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/19/1933|2123 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/20/1933|1358 0.0|  0.0| 4.10| 0.029 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 336 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 339 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 347 0.0|  0.0| 4.10| 0.029 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 436 0.0|  0.0| 4.60| 0.037 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933|1357 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 440 0.0|  0.0| 4.70| 0.040 |  V | 31.8( 51.2)
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DMG |33.7500|118.0830|04/01/1933| 642 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 513 0.0|  0.0| 4.70| 0.040 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 515 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/12/1933| 034 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933|1129 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933|1138 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933|1141 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/15/1933| 432 0.0|  0.0| 4.10| 0.029 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 439 0.0|  0.0| 4.90| 0.044 | VI | 31.8( 51.2)
DMG |33.7500|118.0830|03/12/1933| 835 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 259 0.0|  0.0| 4.60| 0.037 |  V | 31.8( 51.2)



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                          EARTHQUAKE SEARCH RESULTS
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    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
DMG |33.7500|118.0830|03/11/1933| 635 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/12/1933|1651 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/17/1933|1651 0.0|  0.0| 4.10| 0.029 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 311 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 8 8 0.0|  0.0| 4.50| 0.036 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933|1547 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 837 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 2 4 0.0|  0.0| 4.90| 0.044 | VI | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 910 0.0|  0.0| 5.10| 0.049 | VI | 31.8( 51.2)
DMG |33.7500|118.0830|03/13/1933|131828.0|  0.0| 5.30| 0.054 | VI | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 926 0.0|  0.0| 4.10| 0.029 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/13/1933|1929 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/14/1933| 036 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/14/1933|1219 0.0|  0.0| 4.50| 0.036 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 521 0.0|  0.0| 4.40| 0.034 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/12/1933| 448 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 553 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 227 0.0|  0.0| 4.60| 0.037 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 555 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 611 0.0|  0.0| 4.40| 0.034 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 618 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/16/1933|1529 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/16/1933|1530 0.0|  0.0| 4.10| 0.029 |  V | 31.8( 51.2)
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DMG |33.7500|118.0830|03/15/1933| 540 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 211 0.0|  0.0| 4.40| 0.034 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 323 0.0|  0.0| 5.00| 0.046 | VI | 31.8( 51.2)
DMG |33.7500|118.0830|03/12/1933|2128 0.0|  0.0| 4.10| 0.029 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933|1653 0.0|  0.0| 4.80| 0.042 | VI | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933|1944 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933|1956 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933|22 0 0.0|  0.0| 4.40| 0.034 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 258 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933|2232 0.0|  0.0| 4.10| 0.029 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/13/1933|1532 0.0|  0.0| 4.10| 0.029 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 230 0.0|  0.0| 5.10| 0.049 | VI | 31.8( 51.2)
DMG |33.7500|118.0830|03/12/1933| 027 0.0|  0.0| 4.40| 0.034 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/14/1933|2242 0.0|  0.0| 4.10| 0.029 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/15/1933| 2 8 0.0|  0.0| 4.10| 0.029 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 222 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/13/1933| 343 0.0|  0.0| 4.10| 0.029 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933|1147 0.0|  0.0| 4.40| 0.034 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/13/1933| 617 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/16/1933|1456 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 252 0.0|  0.0| 4.00| 0.027 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 216 0.0|  0.0| 4.80| 0.042 | VI | 31.8( 51.2)
DMG |33.7500|118.0830|03/13/1933| 432 0.0|  0.0| 4.70| 0.040 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 257 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.7500|118.0830|03/12/1933| 740 0.0|  0.0| 4.20| 0.030 |  V | 31.8( 51.2)
DMG |33.8000|118.3000|11/03/1931|16 5 0.0|  0.0| 4.00| 0.027 |  V | 31.9( 51.3)
MGI |33.8000|118.3000|12/31/1928|1045 0.0|  0.0| 4.00| 0.027 |  V | 31.9( 51.3)
DMG |33.7670|118.1170|11/04/1939|2141 0.0|  0.0| 4.00| 0.027 |  V | 32.1( 51.6)
DMG |33.7830|118.1330|11/20/1933|1032 0.0|  0.0| 4.00| 0.027 |  V | 32.7( 52.7)
DMG |33.7830|118.1330|01/13/1940| 749 7.0|  0.0| 4.00| 0.027 |  V | 32.7( 52.7)



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                             EARTHQUAKE SEARCH RESULTS
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    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
DMG |33.7830|118.1330|10/02/1933| 91017.6|  0.0| 5.40| 0.056 | VI | 32.7( 52.7)
DMG |32.8670|118.2500|02/13/1952|151337.0|  0.0| 4.70| 0.038 |  V | 33.0( 53.0)
MGI |33.8000|118.5000|06/18/1915|15 5 0.0|  0.0| 4.00| 0.026 |  V | 33.2( 53.4)
DMG |33.5450|117.8070|10/27/1969|1316 2.3|  6.5| 4.50| 0.034 |  V | 33.5( 53.9)
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DMG |33.8170|118.2170|10/22/1941| 65718.5|  0.0| 4.90| 0.042 | VI | 33.7( 54.2)
DMG |33.7500|118.0000|11/16/1934|2126 0.0|  0.0| 4.00| 0.026 |  V | 34.3( 55.1)
MGI |33.7000|117.9000|07/08/1902| 945 0.0|  0.0| 4.00| 0.025 |  V | 35.3( 56.8)
DMG |33.8500|118.2670|03/11/1933|1425 0.0|  0.0| 5.00| 0.043 | VI | 35.5( 57.1)
DMG |33.8500|118.2670|03/11/1933| 629 0.0|  0.0| 4.40| 0.031 |  V | 35.5( 57.1)
GSP |32.9000|118.0070|06/20/2009|010030.6| 14.0| 4.10| 0.026 |  V | 35.7( 57.5)
DMG |32.8170|118.3500|12/26/1951| 04654.0|  0.0| 5.90| 0.068 | VI | 36.1( 58.0)
GSP |32.9750|118.7910|03/04/1992|190627.0|  6.0| 4.20| 0.027 |  V | 36.5( 58.7)
PAS |32.9710|117.8700|07/13/1986|1347 8.2|  6.0| 5.30| 0.048 | VI | 37.0( 59.5)
PAS |32.9900|117.8490|07/13/1986|14 133.0| 12.0| 4.60| 0.033 |  V | 37.0( 59.5)
DMG |33.8670|118.2170|06/19/1944| 3 6 7.0|  0.0| 4.40| 0.030 |  V | 37.1( 59.6)
DMG |33.8670|118.2170|06/19/1944| 0 333.0|  0.0| 4.50| 0.032 |  V | 37.1( 59.6)
DMG |33.8000|118.0000|10/21/1913| 938 0.0|  0.0| 4.00| 0.024 |  V | 37.2( 59.8)
DMG |33.8670|118.2000|11/13/1933|2128 0.0|  0.0| 4.00| 0.024 |  V | 37.2( 59.9)
USG |33.0170|117.8170|07/16/1986|1247 3.7| 10.0| 4.11| 0.026 |  V | 37.2( 59.9)
USG |33.0170|117.8170|07/14/1986| 11112.6| 10.0| 4.12| 0.026 |  V | 37.2( 59.9)
PAS |32.9860|117.8440|10/01/1986|201218.6|  6.0| 4.00| 0.024 |  V | 37.4( 60.1)
GSP |32.8667|118.6535|11/10/2014|084242.9|  5.1| 4.11| 0.026 |  V | 37.5( 60.3)
DMG |33.8830|118.3170|03/11/1933|1457 0.0|  0.0| 4.90| 0.039 |  V | 37.6( 60.4)
GSP |32.9850|117.8180|06/21/1995|211736.2|  6.0| 4.30| 0.028 |  V | 38.6( 62.1)
USG |32.7700|118.3340|06/16/1985|1027 0.7|  5.0| 4.14| 0.025 |  V | 39.3( 63.2)
DMG |33.9030|118.4310|11/29/1938|192115.8| 10.0| 4.00| 0.023 | IV | 39.3( 63.3)
MGI |33.9000|118.2000|10/08/1927|1914 0.0|  0.0| 4.60| 0.032 |  V | 39.5( 63.5)
GSP |32.9700|117.8100|04/04/1990|085439.3|  6.0| 4.00| 0.023 | IV | 39.6( 63.7)
PAS |32.9450|117.8310|07/29/1986| 81741.8| 10.0| 4.10| 0.024 |  V | 39.8( 64.1)
PAS |32.9700|117.8030|07/14/1986| 03246.2| 10.0| 4.00| 0.023 | IV | 39.9( 64.2)
PAS |32.9330|117.8410|07/29/1986| 81741.6| 10.0| 4.30| 0.027 |  V | 40.0( 64.4)
GSP |32.7600|118.2880|08/16/2001|180433.8|  6.0| 4.40| 0.028 |  V | 40.1( 64.5)
GSP |33.9220|118.2700|10/28/2001|162745.6| 21.0| 4.00| 0.023 | IV | 40.4( 65.0)
MGI |33.8000|117.9000|05/22/1902| 740 0.0|  0.0| 4.30| 0.027 |  V | 40.4( 65.1)
PAS |32.9450|117.8060|09/07/1984|11 313.4|  6.0| 4.30| 0.026 |  V | 40.9( 65.8)
DMG |33.9000|118.1000|07/08/1929|1646 6.7| 13.0| 4.70| 0.033 |  V | 41.0( 66.0)
GSP |33.9380|118.3360|05/18/2009|033936.3| 13.0| 4.70| 0.032 |  V | 41.3( 66.5)
DMG |32.7500|118.2000|06/25/1939| 149 0.0|  0.0| 4.50| 0.029 |  V | 41.4( 66.6)
GSP |32.7340|118.3340|08/16/2001|220628.1| 25.0| 4.20| 0.025 |  V | 41.8( 67.2)
DMG |33.7670|117.8170|08/22/1936| 521 0.0|  0.0| 4.00| 0.022 | IV | 41.9( 67.5)
GSP |33.5150|119.0330|08/24/2010|054216.9| 16.0| 4.00| 0.022 | IV | 42.1( 67.7)
DMG |33.9390|118.2050|01/11/1950|214135.0|  0.4| 4.10| 0.023 | IV | 42.1( 67.7)
GSP |32.7280|118.2230|01/29/2009|084159.0|  0.0| 4.20| 0.024 |  V | 42.7( 68.7)
PAS |33.9190|118.6270|01/19/1989| 65328.8| 11.9| 5.00| 0.036 |  V | 43.4( 69.9)
DMG |33.9500|118.1330|10/25/1933| 7 046.0|  0.0| 4.30| 0.025 |  V | 43.7( 70.4)
DMG |32.7180|118.1720|04/28/1938| 6 728.0| 10.0| 4.50| 0.028 |  V | 43.9( 70.6)
PAS |32.9470|117.7360|01/15/1989|153955.2|  6.0| 4.20| 0.024 | IV | 43.9( 70.7)
MGI |33.8000|117.8000|11/07/1926|1948 0.0|  0.0| 4.60| 0.029 |  V | 44.2( 71.2)
MGI |33.8000|117.8000|11/10/1926|1723 0.0|  0.0| 4.60| 0.029 |  V | 44.2( 71.2)
MGI |33.8000|117.8000|05/20/1917| 945 0.0|  0.0| 4.00| 0.021 | IV | 44.2( 71.2)
MGI |33.8000|117.8000|11/04/1926|2238 0.0|  0.0| 4.60| 0.029 |  V | 44.2( 71.2)
MGI |33.8000|117.8000|11/09/1926|1535 0.0|  0.0| 4.60| 0.029 |  V | 44.2( 71.2)
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MGI |33.8000|117.8000|05/19/1917| 635 0.0|  0.0| 4.00| 0.021 | IV | 44.2( 71.2)




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                         EARTHQUAKE SEARCH RESULTS
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    |       |        |          |  TIME   |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC)  |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec | (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
MGI |33.8000|117.8000|05/19/1917| 719 0.0|  0.0| 4.00| 0.021 | IV | 44.2( 71.2)
PAS |33.6300|119.0200|10/23/1981|172816.9| 12.0| 4.60| 0.029 |  V | 44.3( 71.3)
DMG |33.4300|119.0960|10/31/1969|103929.0|  7.3| 4.80| 0.032 |  V | 44.4( 71.4)
DMG |33.3390|119.1040|10/24/1969|202642.5| -1.8| 4.70| 0.031 |  V | 44.4( 71.5)
DMG |33.9830|118.3000|02/11/1940|192410.0|  0.0| 4.00| 0.021 | IV | 44.5( 71.6)
PAS |32.7560|117.9880|01/12/1975|212214.8| 15.3| 4.80| 0.032 |  V | 45.0( 72.4)
GSP |32.7260|118.0680|12/27/2000|002714.1|  6.0| 4.10| 0.022 | IV | 45.0( 72.5)
PAS |33.9330|118.6690|10/17/1979|205237.3|  5.5| 4.20| 0.023 | IV | 45.3( 72.9)
DMG |33.9500|118.6320|08/31/1930| 04036.0|  0.0| 5.20| 0.039 |  V | 45.5( 73.2)
MGI |34.0000|118.3000|06/22/1920|2035 0.0|  0.0| 4.00| 0.021 | IV | 45.7( 73.5)
MGI |34.0000|118.3000|06/30/1920| 350 0.0|  0.0| 4.00| 0.021 | IV | 45.7( 73.5)
MGI |34.0000|118.3000|09/03/1905| 540 0.0|  0.0| 5.30| 0.041 |  V | 45.7( 73.5)
MGI |34.0000|118.4000|01/29/1927|2324 0.0|  0.0| 4.00| 0.021 | IV | 45.8( 73.7)
MGI |34.0000|118.4000|02/22/1920|1610 0.0|  0.0| 4.60| 0.028 |  V | 45.8( 73.7)
MGI |34.0000|118.4000|10/01/1930| 040 0.0|  0.0| 4.60| 0.028 |  V | 45.8( 73.7)
MGI |34.0000|118.4000|02/07/1927| 429 0.0|  0.0| 4.60| 0.028 |  V | 45.8( 73.7)
DMG |34.0000|118.4170|12/07/1938| 338 0.0|  0.0| 4.00| 0.021 | IV | 45.9( 73.8)
T-A |34.0000|118.2500|03/21/1880|1425 0.0|  0.0| 4.30| 0.024 |  V | 45.9( 73.8)
T-A |34.0000|118.2500|09/23/1827| 0 0 0.0|  0.0| 5.00| 0.035 |  V | 45.9( 73.8)
T-A |34.0000|118.2500|01/10/1856| 0 0 0.0|  0.0| 5.00| 0.035 |  V | 45.9( 73.8)
T-A |34.0000|118.2500|03/26/1860| 0 0 0.0|  0.0| 5.00| 0.035 |  V | 45.9( 73.8)
T-A |34.0000|118.2500|05/04/1857| 6 0 0.0|  0.0| 4.30| 0.024 |  V | 45.9( 73.8)
T-A |34.0000|118.2500|01/17/1857| 1 0 0.0|  0.0| 4.30| 0.024 |  V | 45.9( 73.8)
T-A |34.0000|118.2500|05/02/1856| 810 0.0|  0.0| 4.30| 0.024 |  V | 45.9( 73.8)
MGI |34.0000|118.2000|06/26/1917|2120 0.0|  0.0| 4.60| 0.028 |  V | 46.3( 74.5)
MGI |34.0000|118.2000|06/26/1917|2130 0.0|  0.0| 4.60| 0.028 |  V | 46.3( 74.5)
MGI |34.0000|118.2000|02/13/1917|13 5 0.0|  0.0| 4.60| 0.028 |  V | 46.3( 74.5)
MGI |34.0000|118.2000|06/26/1917| 424 0.0|  0.0| 4.00| 0.020 | IV | 46.3( 74.5)
MGI |34.0000|118.2000|06/26/1917|2115 0.0|  0.0| 4.60| 0.028 |  V | 46.3( 74.5)
PAS |33.9440|118.6810|01/01/1979|231438.9| 11.3| 5.00| 0.035 |  V | 46.3( 74.5)
DMG |33.9670|118.0500|01/30/1941| 13446.9|  0.0| 4.10| 0.022 | IV | 46.3( 74.5)
PAS |33.6370|119.0560|10/23/1981|191552.5|  6.3| 4.60| 0.028 |  V | 46.4( 74.6)
GSP |32.6850|118.1380|06/20/1997|053855.0|  6.0| 4.20| 0.023 | IV | 46.6( 74.9)
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DMG |34.0000|118.5000|06/22/1920| 248 0.0|  0.0| 4.90| 0.033 |  V | 46.6( 75.0)
MGI |34.0000|118.5000|03/08/1918|1230 0.0|  0.0| 4.00| 0.020 | IV | 46.6( 75.0)
DMG |34.0000|118.5000|08/04/1927|1224 0.0|  0.0| 5.00| 0.034 |  V | 46.6( 75.0)
DMG |34.0000|118.5000|11/08/1914|1140 0.0|  0.0| 4.50| 0.027 |  V | 46.6( 75.0)
DMG |34.0000|118.5000|03/06/1918|1820 0.0|  0.0| 4.00| 0.020 | IV | 46.6( 75.0)
MGI |34.0000|118.5000|06/23/1920|1220 0.0|  0.0| 4.00| 0.020 | IV | 46.6( 75.0)
MGI |34.0000|118.5000|11/19/1918|2018 0.0|  0.0| 5.00| 0.034 |  V | 46.6( 75.0)
PAS |32.7590|117.9060|10/18/1976|172753.1| 13.8| 4.20| 0.022 | IV | 47.1( 75.8)
DMG |32.8000|117.8330|01/24/1942|214148.0|  0.0| 4.00| 0.020 | IV | 47.2( 75.9)
GSP |32.6810|118.1090|06/20/1997|043540.5|  6.0| 4.70| 0.029 |  V | 47.3( 76.1)
GSP |33.9920|118.0820|03/16/2010|110400.2| 18.0| 4.40| 0.025 |  V | 47.3( 76.2)
GSP |33.9325|117.9158|03/29/2014|040942.2|  5.1| 5.10| 0.036 |  V | 47.5( 76.4)
GSP |34.0200|118.1800|06/12/1989|172225.5| 16.0| 4.10| 0.021 | IV | 47.8( 77.0)
DMG |32.6800|118.0770|10/28/1973|22 0 2.7|  8.0| 4.50| 0.026 |  V | 47.9( 77.1)
DMG |33.6040|119.1050|03/25/1956| 332 2.3|  8.2| 4.20| 0.022 | IV | 48.0( 77.3)
GSP |33.8060|117.7150|03/07/2000|002028.2| 11.0| 4.00| 0.020 | IV | 48.0( 77.3)
GSP |33.6920|119.0580|05/30/2012|051400.8| 16.0| 4.00| 0.020 | IV | 48.3( 77.7)
GSP |34.0300|118.1800|06/12/1989|165718.4| 16.0| 4.40| 0.024 |  V | 48.5( 78.1)
DMG |33.8540|117.7520|10/04/1961| 22131.6|  4.3| 4.10| 0.021 | IV | 48.8( 78.6)
MGI |34.0000|118.0000|05/05/1929| 735 0.0|  0.0| 4.00| 0.019 | IV | 49.5( 79.7)



                            -------------------------
                            EARTHQUAKE SEARCH RESULTS
                            -------------------------


Page   6
-------------------------------------------------------------------------------
    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
MGI |34.0000|118.0000|05/05/1929| 1 7 0.0|  0.0| 4.60| 0.027 |  V | 49.5( 79.7)
MGI |34.0000|118.0000|12/25/1903|1745 0.0|  0.0| 5.00| 0.033 |  V | 49.5( 79.7)
PAS |32.7140|117.9100|10/18/1976|172652.6| 15.1| 4.20| 0.022 | IV | 49.7( 79.9)
DMG |33.2910|119.1930|10/24/1969| 82912.1| 10.0| 5.10| 0.035 |  V | 49.7( 79.9)
GSP |34.0590|118.3870|09/09/2001|235918.0|  4.0| 4.20| 0.021 | IV | 49.8( 80.1)
DMG |33.9960|117.9750|06/15/1967| 458 5.5| 10.0| 4.10| 0.020 | IV | 49.8( 80.2)
GSP |33.9613|117.8923|03/29/2014|213245.9|  9.3| 4.14| 0.021 | IV | 49.9( 80.3)
GSP |33.9040|117.7910|08/08/2012|163322.1| 10.0| 4.50| 0.025 |  V | 50.0( 80.4)
GSP |33.9050|117.7920|08/08/2012|062334.1| 10.0| 4.50| 0.025 |  V | 50.0( 80.4)
GSP |33.9090|117.7920|06/14/2012|031715.7|  9.0| 4.00| 0.019 | IV | 50.2( 80.8)
GSP |33.9070|117.7880|08/29/2012|203100.3|  9.0| 4.10| 0.020 | IV | 50.2( 80.8)
PAS |33.6710|119.1110|09/04/1981|155050.3|  5.0| 5.30| 0.038 |  V | 50.2( 80.9)
PAS |33.9650|117.8860|01/01/1976|172012.9|  6.2| 4.20| 0.021 | IV | 50.3( 80.9)
GSP |32.6260|118.1510|06/20/1997|080413.6|  6.0| 4.60| 0.026 |  V | 50.4( 81.1)
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PAS |34.0490|118.1010|10/01/1987|144541.5| 13.6| 4.70| 0.028 |  V  | 50.8( 81.8)
DMG |33.6820|117.5530|07/05/1938|18 655.7| 10.0| 4.50| 0.025 |  V  | 50.8( 81.8)
PAS |34.0500|118.0870|10/01/1987|155953.5| 10.4| 4.00| 0.019 | IV  | 51.1( 82.2)
PAS |34.0520|118.0900|10/01/1987|151231.8| 10.8| 4.70| 0.027 |  V  | 51.2( 82.3)
GSP |33.9170|117.7760|09/03/2002|070851.9| 12.0| 4.80| 0.029 |  V  | 51.2( 82.4)
MGI |34.0800|118.2600|07/16/1920|18 8 0.0|  0.0| 5.00| 0.032 |  V  | 51.3( 82.6)
PAS |34.0600|118.1000|10/01/1987|1449 5.9| 11.7| 4.70| 0.027 |  V  | 51.5( 83.0)
DMG |33.5830|119.1830|02/10/1952|135055.0|  0.0| 4.00| 0.019 | IV  | 51.7( 83.2)
DMG |32.7170|117.8330|11/06/1950|205546.0|  0.0| 4.40| 0.023 | IV  | 51.8( 83.4)
PAS |34.0610|118.0790|10/01/1987|144220.0|  9.5| 5.90| 0.051 | VI  | 51.9( 83.6)
PAS |34.0730|118.0980|10/04/1987|105938.2|  8.2| 5.30| 0.037 |  V  | 52.4( 84.4)
MGI |34.1000|118.3000|07/16/1920|2022 0.0|  0.0| 4.60| 0.025 |  V  | 52.6( 84.6)
MGI |34.1000|118.3000|07/16/1920|2127 0.0|  0.0| 4.60| 0.025 |  V  | 52.6( 84.6)
MGI |34.1000|118.3000|07/26/1920|1215 0.0|  0.0| 4.00| 0.019 | IV  | 52.6( 84.6)
MGI |34.1000|118.3000|07/16/1920|2130 0.0|  0.0| 4.60| 0.025 |  V  | 52.6( 84.6)
PAS |34.0760|118.0900|10/01/1987|1448 3.1| 11.7| 4.10| 0.019 | IV  | 52.8( 84.9)
PAS |32.6250|118.0090|07/11/1981|215029.4|  5.0| 4.30| 0.022 | IV  | 52.8( 84.9)
DMG |33.8000|117.6000|09/16/1903|1210 0.0|  0.0| 4.00| 0.018 | IV  | 52.9( 85.1)
MGI |33.8000|117.6000|04/22/1918|2115 0.0|  0.0| 5.00| 0.031 |  V  | 52.9( 85.1)
GSP |34.0958|118.4912|06/02/2014|023643.9|  4.3| 4.16| 0.020 | IV  | 53.0( 85.3)
MGI |34.1000|118.2000|01/27/1860| 830 0.0|  0.0| 4.30| 0.022 | IV  | 53.1( 85.4)
MGI |34.1000|118.2000|05/02/1916|1432 0.0|  0.0| 4.00| 0.018 | IV  | 53.1( 85.4)
MGI |34.1000|118.2000|04/21/1921|1538 0.0|  0.0| 4.00| 0.018 | IV  | 53.1( 85.4)
DMG |33.6990|117.5110|05/31/1938| 83455.4| 10.0| 5.50| 0.040 |  V  | 53.5( 86.1)
PAS |34.0770|118.0470|02/11/1988|152555.7| 12.5| 4.70| 0.026 |  V  | 53.5( 86.2)
GSG |33.9530|117.7610|07/29/2008|184215.7| 14.0| 5.30| 0.036 |  V  | 53.7( 86.4)
DMG |33.7170|117.5170|06/19/1935|1117 0.0|  0.0| 4.00| 0.018 | IV  | 53.8( 86.5)
MGI |34.1000|118.1000|07/11/1855| 415 0.0|  0.0| 6.30| 0.061 | VI  | 54.2( 87.3)
DMG |33.7170|117.5070|08/06/1938|22 056.0| 10.0| 4.00| 0.018 | IV  | 54.3( 87.3)
GSP |33.9550|117.7460|12/14/2001|120135.5| 13.0| 4.00| 0.018 | IV  | 54.3( 87.4)
DMG |33.7250|117.4980|01/03/1956| 02548.9| 13.7| 4.70| 0.026 |  V  | 55.0( 88.5)
GSP |33.9510|117.7090|01/05/1998|181406.5| 11.0| 4.30| 0.021 | IV  | 55.5( 89.2)
GSP |34.1340|118.4862|03/17/2014|132536.9|  9.2| 4.39| 0.022 | IV  | 55.6( 89.4)
MGI |34.1000|118.0000|01/27/1930|2026 0.0|  0.0| 4.60| 0.024 |  V  | 55.9( 90.0)
DMG |33.7480|117.4790|06/22/1971|104119.0|  8.0| 4.20| 0.019 | IV  | 56.7( 91.3)
DMG |33.7330|117.4670|10/26/1954|162226.0|  0.0| 4.10| 0.018 | IV  | 56.8( 91.4)
GSP |33.7330|117.4660|09/02/2007|172914.0|  2.0| 4.70| 0.025 |  V  | 56.9( 91.5)
PAS |34.1490|118.1350|12/03/1988|113826.4| 13.3| 4.90| 0.028 |  V  | 57.1( 91.8)
GSP |33.1660|119.3020|11/15/2009|224527.1|  6.0| 4.30| 0.020 | IV  | 57.1( 92.0)




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                         EARTHQUAKE SEARCH RESULTS
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Page  7
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--------------------------------------------------------------------------------
    |       |        |          |  TIME   |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
PAS |34.0060|117.7390|02/18/1989| 717 4.8|  3.3| 4.30| 0.020 | IV | 57.3( 92.3)
DMG |32.5290|118.0820|05/26/1973|234633.3|  8.0| 4.30| 0.020 | IV | 57.8( 93.0)
T-A |34.1700|118.1700|03/07/1888|1554 0.0|  0.0| 4.30| 0.020 | IV | 58.1( 93.5)
MGI |34.0000|117.7000|12/03/1929| 9 5 0.0|  0.0| 4.00| 0.017 | IV | 58.4( 94.0)
DMG |34.1000|118.8000|05/10/1911|1340 0.0|  0.0| 4.00| 0.017 | IV | 58.9( 94.9)
GSP |32.4970|118.1450|10/19/2005|085126.1| 12.0| 4.20| 0.019 | IV | 59.2( 95.2)
DMG |33.7000|117.4000|05/15/1910|1547 0.0|  0.0| 6.00| 0.049 | VI | 59.3( 95.4)
DMG |33.7000|117.4000|05/13/1910| 620 0.0|  0.0| 5.00| 0.029 |  V | 59.3( 95.4)
DMG |33.7000|117.4000|04/11/1910| 757 0.0|  0.0| 5.00| 0.029 |  V | 59.3( 95.4)
DMG |34.0170|118.9670|04/16/1948|222624.0|  0.0| 4.70| 0.024 |  V | 59.3( 95.4)
DMG |32.5000|118.5500|02/24/1948| 81510.0|  0.0| 5.30| 0.034 |  V | 59.3( 95.4)
GSP |34.0490|118.9150|02/19/1995|212418.1| 15.0| 4.30| 0.020 | IV | 59.3( 95.4)
GSP |34.0690|118.8820|05/02/2009|011113.7| 14.0| 4.40| 0.021 | IV | 59.4( 95.6)
MGI |34.0000|119.0000|12/14/1912| 0 0 0.0|  0.0| 5.70| 0.041 |  V | 59.5( 95.8)
DMG |34.0000|119.0000|09/24/1827| 4 0 0.0|  0.0| 7.00| 0.082 | VII| 59.5( 95.8)
DMG |32.8500|117.4830|02/23/1943| 92112.0|  0.0| 4.00| 0.017 | IV | 59.7( 96.1)
PAS |34.0160|118.9880|10/26/1984|172043.5| 13.3| 4.60| 0.023 | IV | 60.0( 96.5)
DMG |33.9500|117.5830|04/11/1941| 12024.0|  0.0| 4.00| 0.017 | IV | 60.3( 97.1)
DMG |32.5830|117.8000|04/19/1939| 741 0.0|  0.0| 4.50| 0.022 | IV | 60.7( 97.7)
DMG |34.1000|117.8000|03/31/1931|2033 0.0|  0.0| 4.00| 0.017 | IV | 60.8( 97.9)
PAS |34.0540|118.9640|04/13/1982|11 212.2| 16.6| 4.00| 0.016 | IV | 61.2( 98.5)
DMG |33.9900|119.0580|05/29/1955|164335.4| 17.4| 4.10| 0.017 | IV | 61.2( 98.5)
GSP |34.2150|118.5100|01/19/1994|140914.8| 17.0| 4.50| 0.021 | IV | 61.3( 98.7)
GSP |34.2130|118.5370|01/17/1994|123055.4| 18.0| 6.70| 0.068 | VI | 61.4( 98.9)
DMG |33.4000|119.4000|07/24/1947|1654 2.0|  0.0| 4.30| 0.019 | IV | 61.6( 99.1)
GSP |33.6660|119.3300|03/16/2002|213323.8|  7.0| 4.60| 0.023 | IV | 61.6( 99.2)
PAS |33.9060|119.1660|05/23/1978| 91650.8|  6.0| 4.00| 0.016 | IV | 61.8( 99.4)
GSP |32.4550|118.1630|10/16/2005|211135.0| 10.0| 4.90| 0.026 |  V | 61.8( 99.5)

********************************************************************************
-END OF SEARCH-   346 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH:   1800  TO  2019

LENGTH OF SEARCH TIME:   220  years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 10.8 MILES (17.4 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.100 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
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  a-value=  4.024
  b-value=  0.955
  beta-value=  2.198


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TABLE OF MAGNITUDES AND EXCEEDANCES:
------------------------------------

  Earthquake | Number of Times | Cumulative
   Magnitude |    Exceeded     | No. / Year
  -----------+-----------------+------------
     4.0     |       346       |   1.57991
     4.5     |       124       |   0.56621
     5.0     |        43       |   0.19635
     5.5     |        10       |   0.04566
     6.0     |         5       |   0.02283
     6.5     |         2       |   0.00913
     7.0     |         1       |   0.00457
```

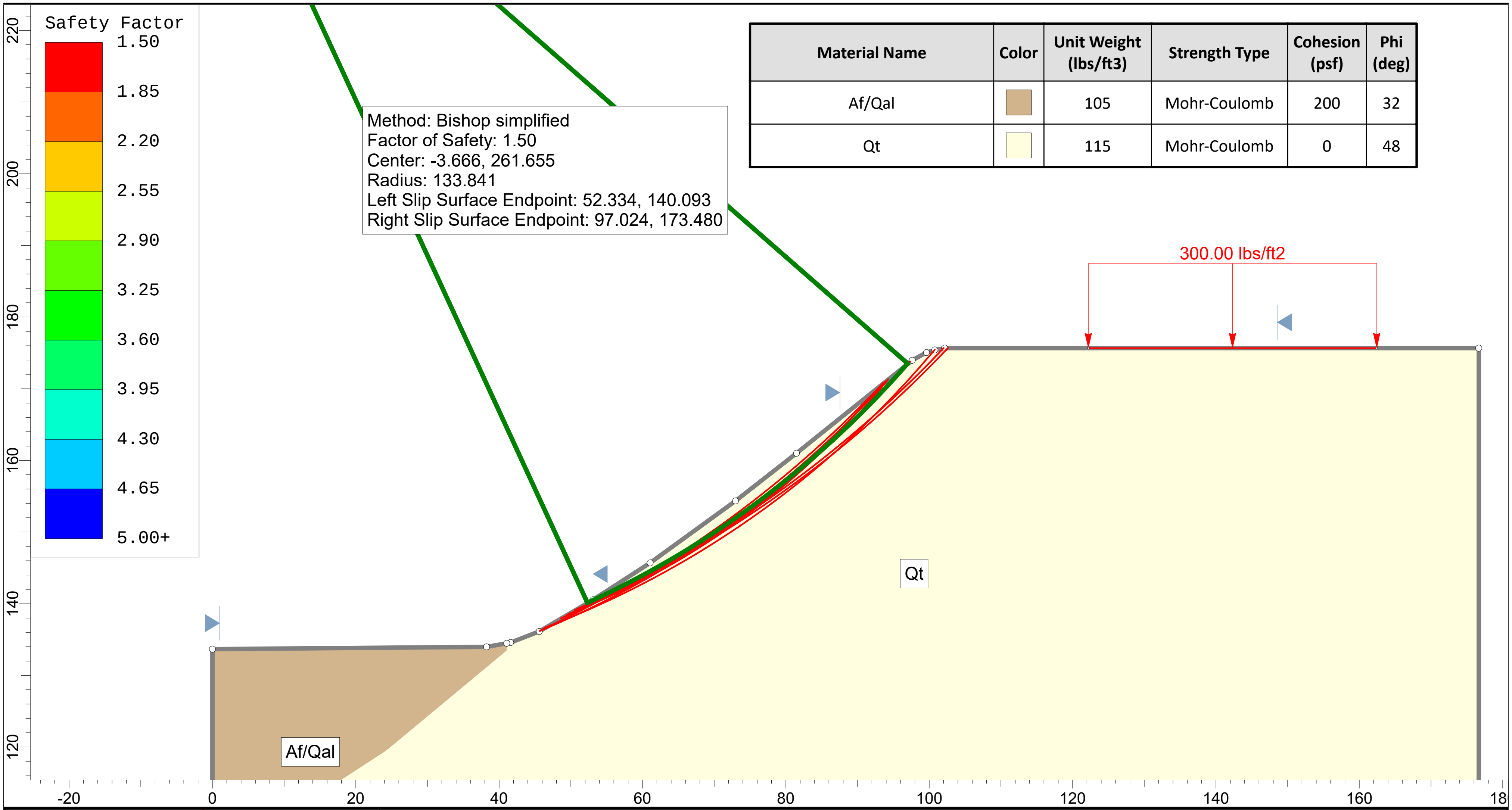
APPENDIX D

SLOPE STABILITY ANALYSIS

Section A - A'

Global Stability

\\Ds-irv\project\INFOCUS PROJECTS\12001-12500\12396 LBUSD Avalon Geo\001 Geo Investigation\Analyses\Slope Stability\Sec A_Global Static.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Af/Qal	🟫	105	Mohr-Coulomb	200	32
Qt	🟨	115	Mohr-Coulomb	0	48

Method: Bishop simplified
Factor of Safety: 1.50
Center: -3.666, 261.655
Radius: 133.841
Left Slip Surface Endpoint: 52.334, 140.093
Right Slip Surface Endpoint: 97.024, 173.480

SLIDEINTERPRET 8.023

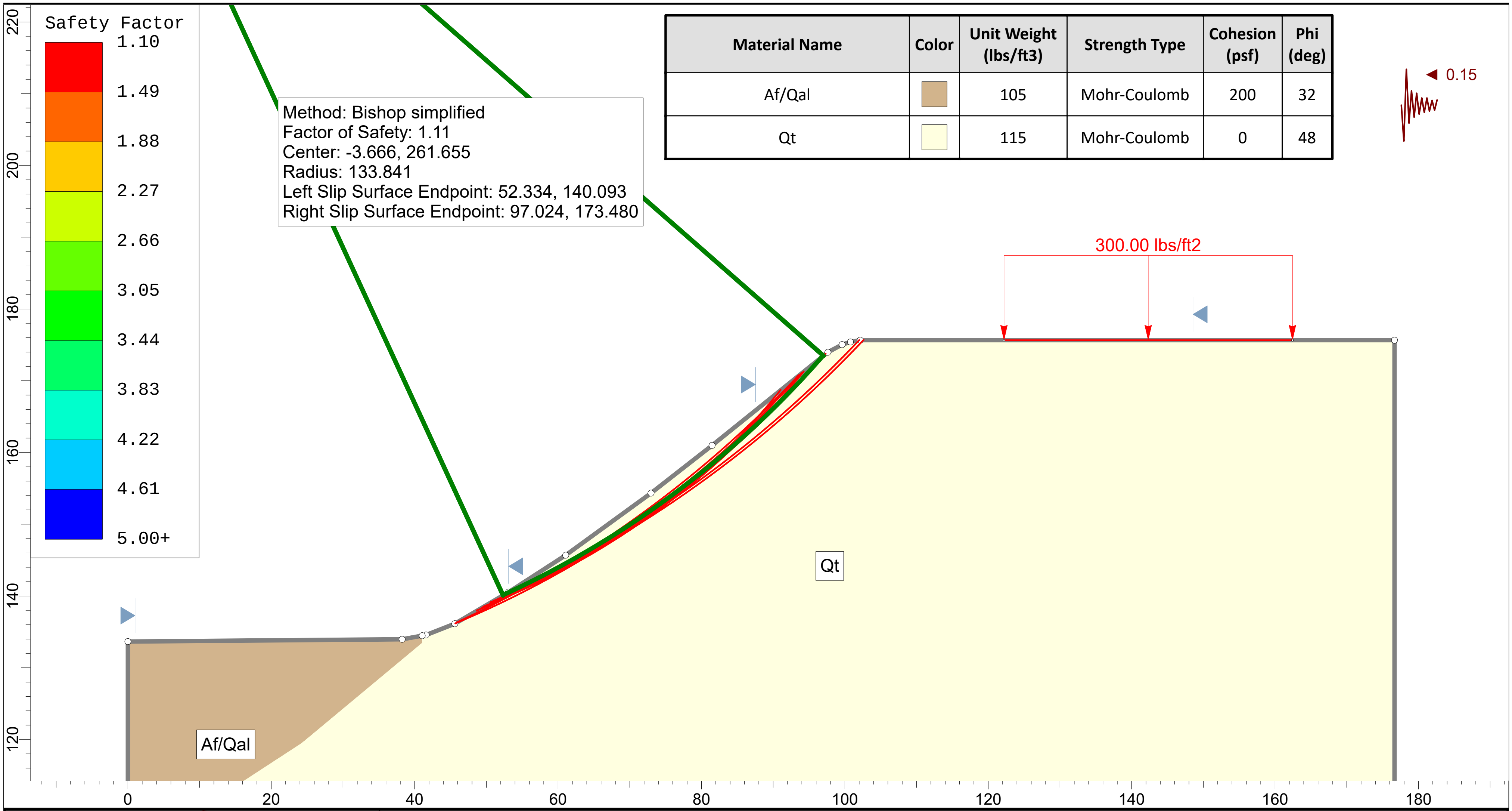
Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

Project	**Avalon K-12 Sports Field and Site Improvements, Avalon, California**
Analyzed By	CD
Units	feet
Scale	1:240
File Name:	\\Ds-irv\project\INFOCUS PROJECTS\12001-12500\12396 LBUSD Avalon Geo\001 Geo Investigation\Analyses\Slope Stability\Sec A_Global Static.slim
Project No.:	**12396.001**
Date	September 2019
Condition	**Static**

Section A - A'

Global Stability

\\Ds-irv\project\INFOCUS PROJECTS\12001-12500\12396 LBUSD Avalon Geo\001 Geo Investigation\Analyses\Slope Stability\Sec A_Global Pstat.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Af/Qal	🟫	105	Mohr-Coulomb	200	32
Qt	🟨	115	Mohr-Coulomb	0	48

Method: Bishop simplified
Factor of Safety: 1.11
Center: -3.666, 261.655
Radius: 133.841
Left Slip Surface Endpoint: 52.334, 140.093
Right Slip Surface Endpoint: 97.024, 173.480

SLIDEINTERPRET 8.023

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

Project
Avalon K-12 Sports Field and Site Improvements, Avalon, California

Analyzed By CD

Units feet

Scale 1:240

File Name: \\Ds-irv\project\INFOCUS PROJECTS\12001-12500\12396 LBUSD Avalon Geo\001 Geo Investigation\Analyses\Slope Stability\Sec A_Global Pstat.slim

Project No.: **12396.001**

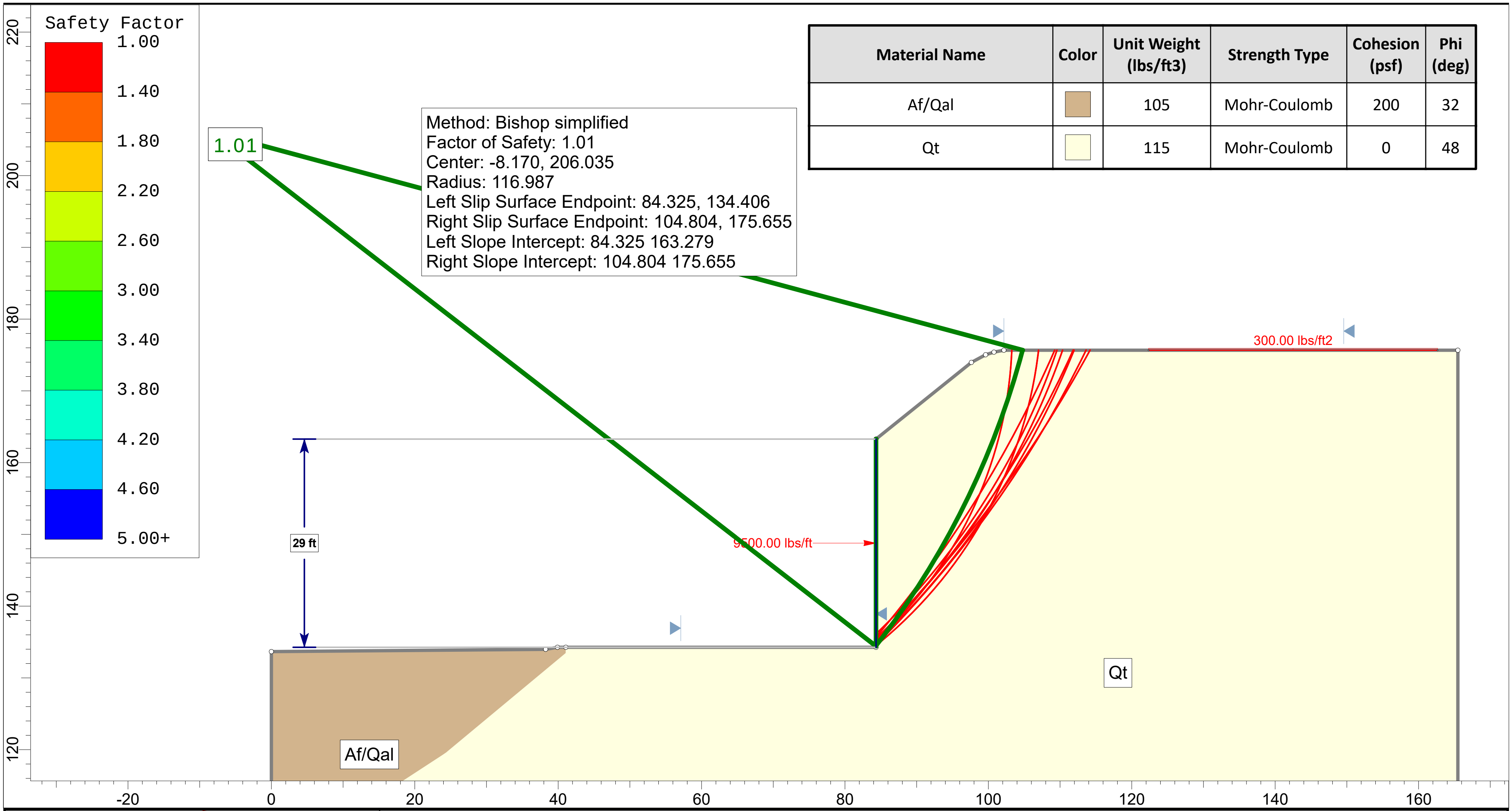
Date September 2019

Condition **Pseudostatic**

Section A - A' Proposed Retaining Wall

Global Stability

\\Ds-irv\project\INFOCUS PROJECTS\12001-12500\12396 LBUSD Avalon Geo\001 Geo Investigation\Analyses\Slope Stability\Sec A_RW Stat FS=1.slim



Method: Bishop simplified
Factor of Safety: 1.01
Center: -8.170, 206.035
Radius: 116.987
Left Slip Surface Endpoint: 84.325, 134.406
Right Slip Surface Endpoint: 104.804, 175.655
Left Slope Intercept: 84.325 163.279
Right Slope Intercept: 104.804 175.655

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Af/Qal	🟫	105	Mohr-Coulomb	200	32
Qt	🟨	115	Mohr-Coulomb	0	48

SLIDEINTERPRET 8.023

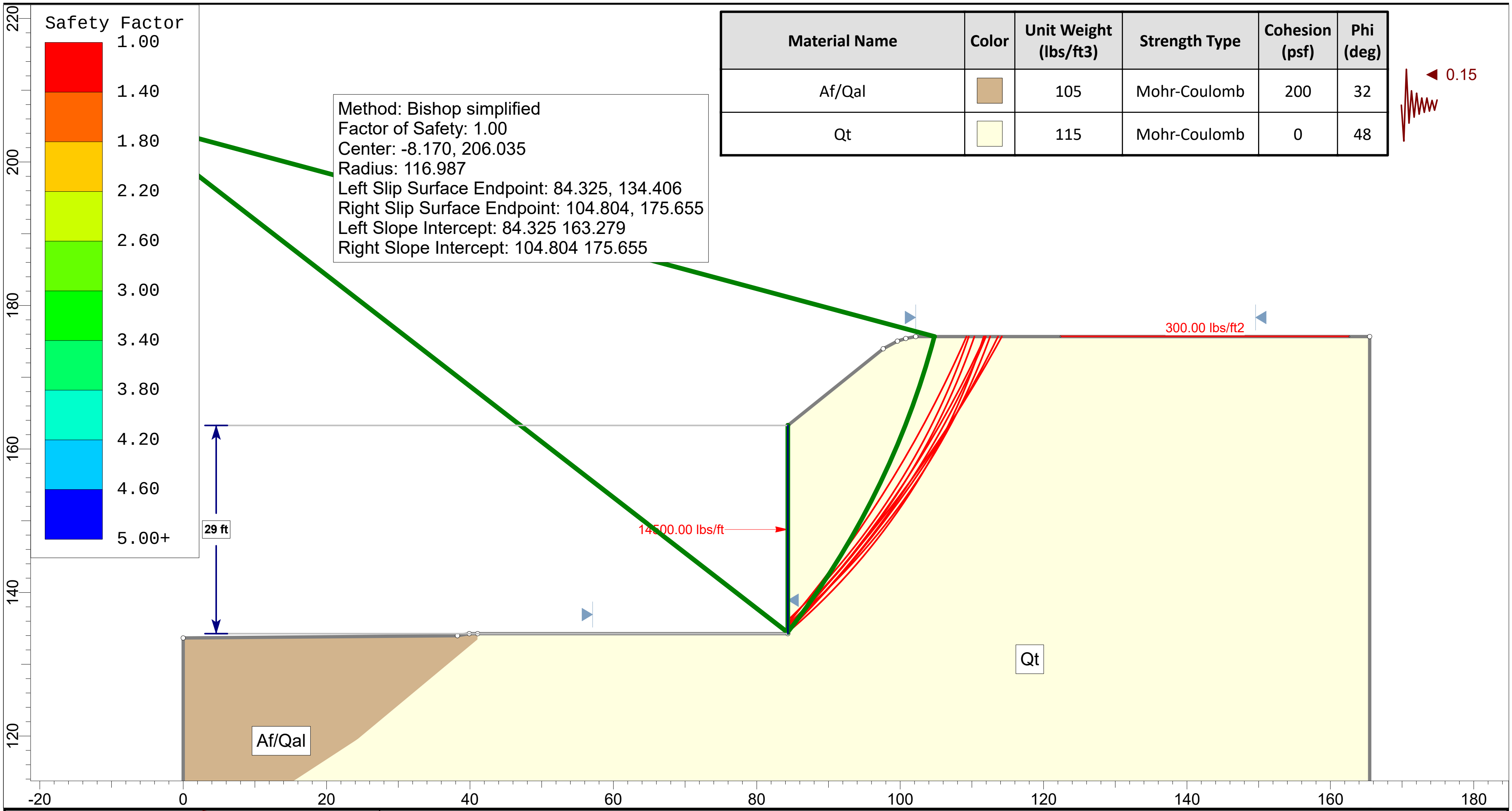
Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

Project	**Avalon K-12 Sports Field and Site Improvements, Avalon, California**		
Analyzed By CD	*Units* feet	*Scale* 1:240	*File Name:* \\Ds-irv\project\INFOCUS PROJECTS\12001-12500\12396 LBUSD Avalon Geo\001 Geo Investigation\Analyses\Slope Stability\Sec A_RW Stat FS=1.slim
Date September 2019	*Condition* **Static**		*Project No.:* **12396.001**

Section A - A' Proposed Retaining Wall

Global Stability

\\Ds-irv\project\INFOCUS PROJECTS\12001-12500\12396 LBUSD Avalon Geo\001 Geo Investigation\Analyses\Slope Stability\Sec A_RW P-Stat FS=1.slim



Method: Bishop simplified
Factor of Safety: 1.00
Center: -8.170, 206.035
Radius: 116.987
Left Slip Surface Endpoint: 84.325, 134.406
Right Slip Surface Endpoint: 104.804, 175.655
Left Slope Intercept: 84.325 163.279
Right Slope Intercept: 104.804 175.655

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Af/Qal	🟫	105	Mohr-Coulomb	200	32
Qt	⬜	115	Mohr-Coulomb	0	48

SLIDEINTERPRET 8.023

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

Project
Avalon K-12 Sports Field and Site Improvements, Avalon, California

Analyzed By CD

Units feet

Scale 1:240

File Name: \\Ds-irv\project\INFOCUS PROJECTS\12001-12500\12396 LBUSD Avalon Geo\001 Geo Investigation\Analyses\Slope Stability\Sec A_RW P-Stat FS=1.slim

Project No.: **12396.001**

Date September 2019

Condition **Pseudostatic**

Section B - B'

Global Stability

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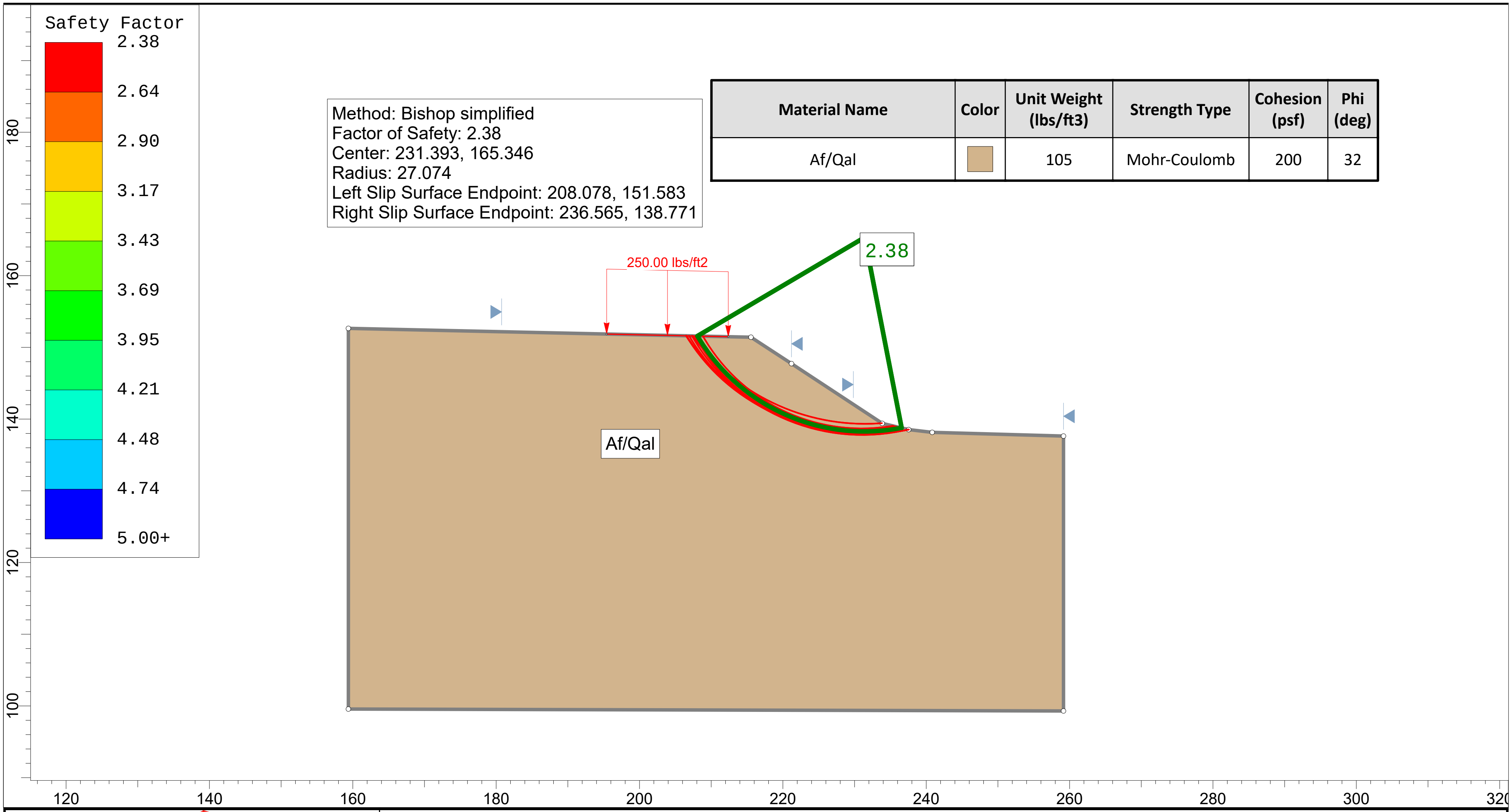

Method: Bishop simplified
Factor of Safety: 2.38
Center: 231.393, 165.346
Radius: 27.074
Left Slip Surface Endpoint: 208.078, 151.583
Right Slip Surface Endpoint: 236.565, 138.771

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Af/Qal	🟫	105	Mohr-Coulomb	200	32

SLIDEINTERPRET 8.023

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

Project	**Avalon K-12 Sports Field and Site Improvements, Avalon, California**			
Analyzed By CD	*Units* feet	*Scale* 1:240	*File Name:* \\Ds-irv\project\INFOCUS PROJECTS\12001-12500\12396 LBUSD Avalon Geo\001 Geo Investigation\Analyses\Slope Stability\Sec B_Global Static.slim	*Project No.:* **12396.001**
Date September 2019	*Condition* **Static**			

Section B - B'

Global Stability

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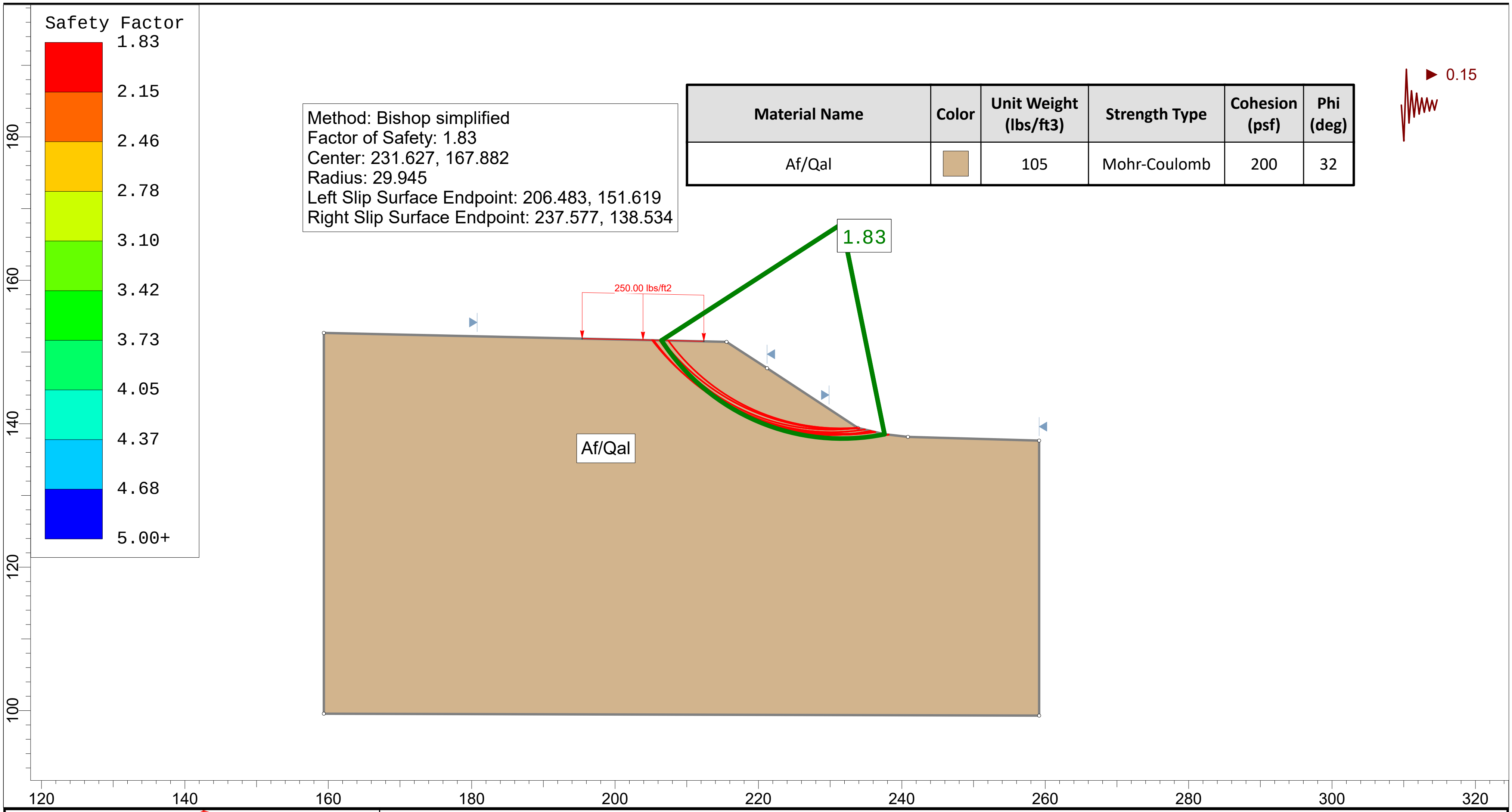


Method: Bishop simplified
Factor of Safety: 1.83
Center: 231.627, 167.882
Radius: 29.945
Left Slip Surface Endpoint: 206.483, 151.619
Right Slip Surface Endpoint: 237.577, 138.534

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Af/Qal	🟫	105	Mohr-Coulomb	200	32

SLIDEINTERPRET 8.023

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

Project
Avalon K-12 Sports Field and Site Improvements, Avalon, California

Analyzed By CD

Units feet

Scale 1:240

File Name: \\Ds-irv\project\INFOCUS PROJECTS\12001-12500\12396 LBUSD Avalon Geo\001 Geo Investigation\Analyses\Slope Stability\Sec B_Global P-stat.slim

Project No.: **12396.001**

Date September 2019

Condition **Pseudostatic**

APPENDIX E

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROGH GRADING

APPENDIX E

LEIGHTON CONSULTING, INC.
EARTHWORK AND GRADING GUIDE SPECIFICATIONS

TABLE OF CONTENTS

E-1.0 GENERAL

E-1.1 Intent

These Earthwork and Grading Guide Specifications are for grading and earthwork shown on the current, approved grading plan(s) and/or indicated in the Leighton Consulting, Inc. geotechnical report(s). These Guide Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the project-specific recommendations in the geotechnical report shall supersede these Guide Specifications. Leighton Consulting, Inc. shall provide geotechnical observation and testing during earthwork and grading. Based on these observations and tests, Leighton Consulting, Inc. may provide new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

E-1.2 Role of Leighton Consulting, Inc.

Prior to commencement of earthwork and grading, Leighton Consulting, Inc. shall meet with the earthwork contractor to review the earthwork contractor's work plan, to schedule sufficient personnel to perform the appropriate level of observation, mapping and compaction testing. During earthwork and grading, Leighton Consulting, Inc. shall observe, map, and document subsurface exposures to verify geotechnical design assumptions. If observed conditions are found to be significantly different than the interpreted assumptions during the design phase, Leighton Consulting, Inc. shall inform the owner, recommend appropriate changes in design to accommodate these observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include (1) natural ground after clearing to receiving fill but before fill is placed, (2) bottoms of all "remedial removal" areas, (3) all key bottoms, and (4) benches made on sloping ground to receive fill.

Leighton Consulting, Inc. shall observe moisture-conditioning and processing of the subgrade and fill materials, and perform relative compaction testing of fill to determine the attained relative compaction. Leighton Consulting, Inc. shall provide *Daily Field Reports* to the owner and the Contractor on a routine and frequent basis.

E-1.3 The Earthwork Contractor

The earthwork contractor (Contractor) shall be qualified, experienced and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Guide

Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing grading and backfilling in accordance with the current, approved plans and specifications.

The Contractor shall inform the owner and Leighton Consulting, Inc. of changes in work schedules at least one working day in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that Leighton Consulting, Inc. is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish earthwork and grading in accordance with the applicable grading codes and agency ordinances, these Guide Specifications, and recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of Leighton Consulting, Inc., unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, adverse weather, etc., are resulting in a quality of work less than required in these specifications, Leighton Consulting, Inc. shall reject the work and may recommend to the owner that earthwork and grading be stopped until unsatisfactory condition(s) are rectified.

E-2.0 PREPARATION OF AREAS TO BE FILLED

E-2.1 Clearing and Grubbing

Vegetation, such as brush, grass, roots and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies and Leighton Consulting, Inc.. Care should be taken not to encroach upon or otherwise damage native and/or historic trees designated by the Owner or appropriate agencies to remain. Pavements, flatwork or other construction should not extend under the “drip line” of designated trees to remain.

Leighton Consulting, Inc. shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 3 percent of organic materials (by dry weight: ASTM D 2974). Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area. As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

E-2.2 Processing

Existing ground that has been declared satisfactory for support of fill, by Leighton Consulting, Inc., shall be scarified to a minimum depth of 6 inches (15 cm). Existing ground that is not satisfactory shall be over-excavated as specified in the following Section E-2.3. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

E-2.3 Overexcavation

In addition to removals and over-excavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be over-excavated to competent ground as evaluated by Leighton Consulting, Inc. during grading. All undocumented fill soils under proposed structure footprints should be excavated

E-2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), (>20 percent grade) the ground shall be stepped or benched. The lowest bench or key shall be a minimum of 15 feet (4.5 m) wide and at least 2 feet (0.6 m) deep, into competent material as evaluated by Leighton Consulting, Inc.. Other benches shall be excavated a minimum height of 4 feet (1.2 m) into competent material or as otherwise recommended by Leighton Consulting, Inc.. Fill placed on ground sloping flatter than 5:1 (horizontal to vertical units), (<20 percent grade) shall also be benched or otherwise over-excavated to provide a flat subgrade for the fill.

E-2.5 Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by Leighton Consulting, Inc. as suitable to receive fill. The Contractor shall obtain a written acceptance (*Daily Field Report*) from Leighton Consulting, Inc. prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys and benches.

E-3.0 FILL MATERIAL

E-3.1 Fill Quality

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by Leighton Consulting, Inc. prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to Leighton Consulting, Inc. or mixed with other soils to achieve satisfactory fill material.

E-3.2 Oversize

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 6 inches (15 cm), shall not be buried or placed in fill unless location, materials and placement methods are specifically accepted by Leighton Consulting, Inc.. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 feet (3 m) measured vertically from finish grade, or within 2 feet (0.61 m) of future utilities or underground construction.

E-3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of Section E-3.1, and be free of hazardous materials (“contaminants”) and rock larger than 3-inches (8 cm) in largest dimension. All import soils shall have an Expansion Index (EI) of 20 or less and a sulfate content no greater than (≤) 500 parts-per-million (ppm). A representative sample of a potential import source shall be given to Leighton Consulting, Inc. at least four full working days before importing begins, so that suitability of this import material can be determined and appropriate tests performed.

E-4.0 FILL PLACEMENT AND COMPACTION

E-4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill, as described in Section E-2.0, above, in near-horizontal layers not exceeding 8 inches (20 cm) in loose thickness. Leighton Consulting, Inc. may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers, and only if the building officials with the appropriate jurisdiction approve. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

E-4.2 Fill Moisture Conditioning

Fill soils shall be watered, dried back, blended and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM) Test Method D 1557.

E-4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, each layer shall be uniformly compacted to not-less-than (≥) 90 percent of the maximum dry density as determined by ASTM Test Method D 1557. In some cases, structural fill may be specified (see project-specific geotechnical report) to be uniformly compacted to at-least (≥) 95 percent of the ASTM D 1557 modified Proctor laboratory maximum dry density. For fills thicker than (>) 15 feet (4.5 m), the portion of fill deeper than 15 feet below proposed finish grade shall be compacted to 95 percent of the ASTM D 1557 laboratory maximum density. Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

E-4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by back rolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet (1 to 1.2 m) in fill elevation, or by other methods producing satisfactory results acceptable to Leighton Consulting, Inc.. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of the ASTM D 1557 laboratory maximum density.

E-4.5 Compaction Testing

Field-tests for moisture content and relative compaction of the fill soils shall be performed by Leighton Consulting, Inc.. Location and frequency of tests shall be at our field representative(s) discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

E-4.6 Compaction Test Locations

Leighton Consulting, Inc. shall document the approximate elevation and horizontal coordinates of each density test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that Leighton

Consulting, Inc. can determine the test locations with sufficient accuracy. Adequate grade stakes shall be provided.

E-5.0 EXCAVATION

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by Leighton Consulting, Inc. during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by Leighton Consulting, Inc. based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, then observed and reviewed by Leighton Consulting, Inc. prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by Leighton Consulting, Inc..

E-6.0 TRENCH BACKFILLS

E-6.1 Safety

The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations. Work should be performed in accordance with Article 6 of the *California Construction Safety Orders*, 2009 Edition or more current (see also: http://www.dir.ca.gov/title8/sb4a6.html).

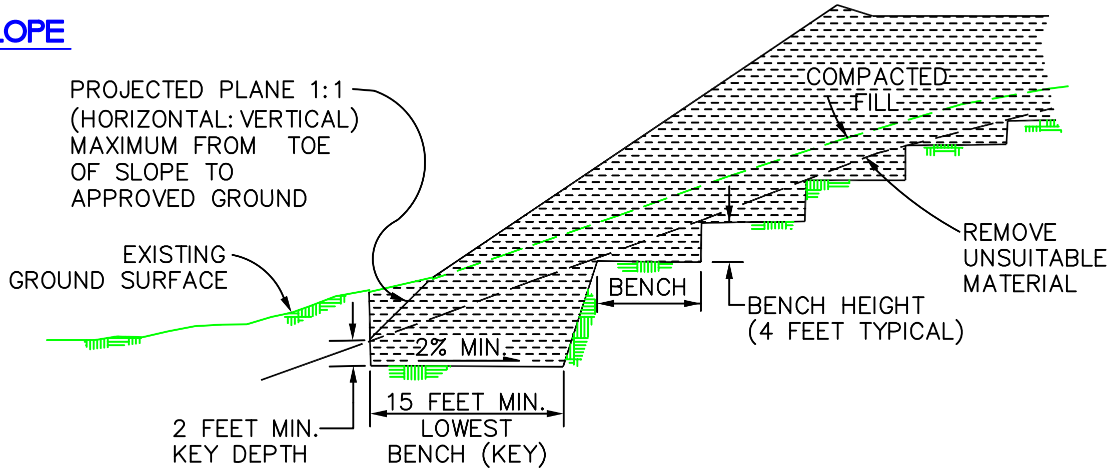
E-6.2 Bedding and Backfill

All utility trench bedding and backfill shall be performed in accordance with applicable provisions of the 2018 Edition of the *Standard Specifications for Public Works Construction* (Green Book). Bedding material shall have a Sand Equivalent greater than 30 (SE>30). Bedding shall be placed to 1-foot (0.3 m) over the top of the conduit, and densified by jetting in areas of granular soils, if allowed by the permitting agency. Otherwise, the pipe-bedding zone should be backfilled with Controlled Low Strength Material (CLSM) consisting of at least one sack of Portland cement per cubic-yard of sand, and conforming to Section 201-6 of the 2018 Edition of the *Standard Specifications for Public Works Construction* (Green Book). Backfill over the bedding zone shall be placed and densified mechanically to a minimum of 90 percent of relative compaction (ASTM D 1557) from 1 foot (0.3 m) above the top of the conduit to the surface. Backfill above the pipe zone shall **not** be jetted. Jetting of the bedding around the conduits shall be observed by Leighton Consulting, Inc. and backfill above the pipe zone (bedding) shall be observed and tested by Leighton Consulting, Inc..

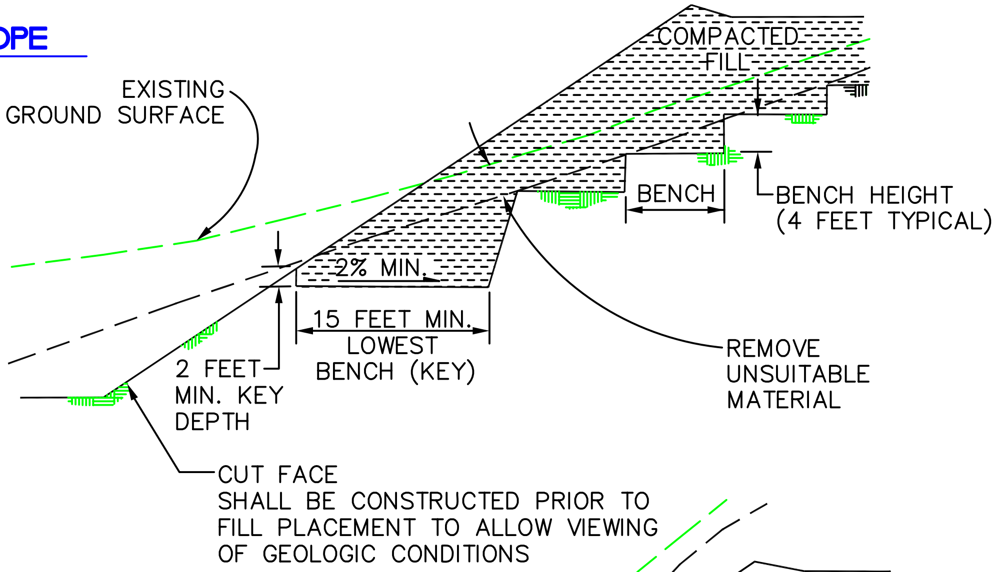
E-6.3 Lift Thickness

Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to Leighton Consulting, Inc. that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method, and only if the building officials with the appropriate jurisdiction approve.

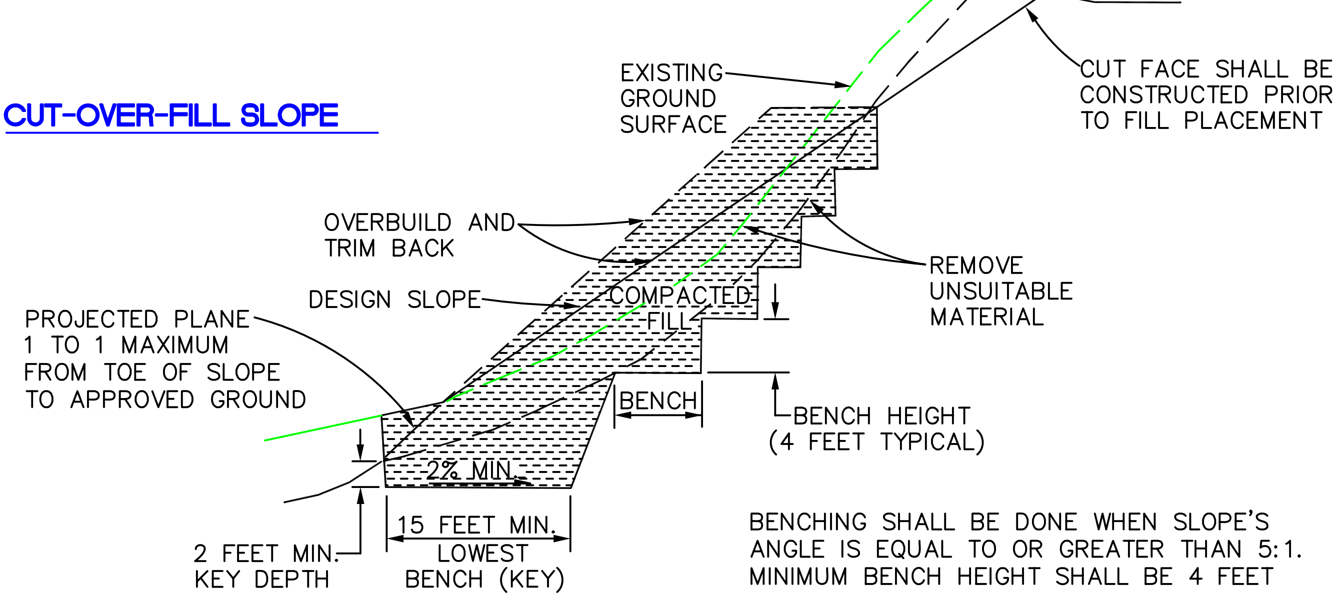
FILL SLOPE



FILL-OVER-CUT SLOPE



CUT-OVER-FILL SLOPE

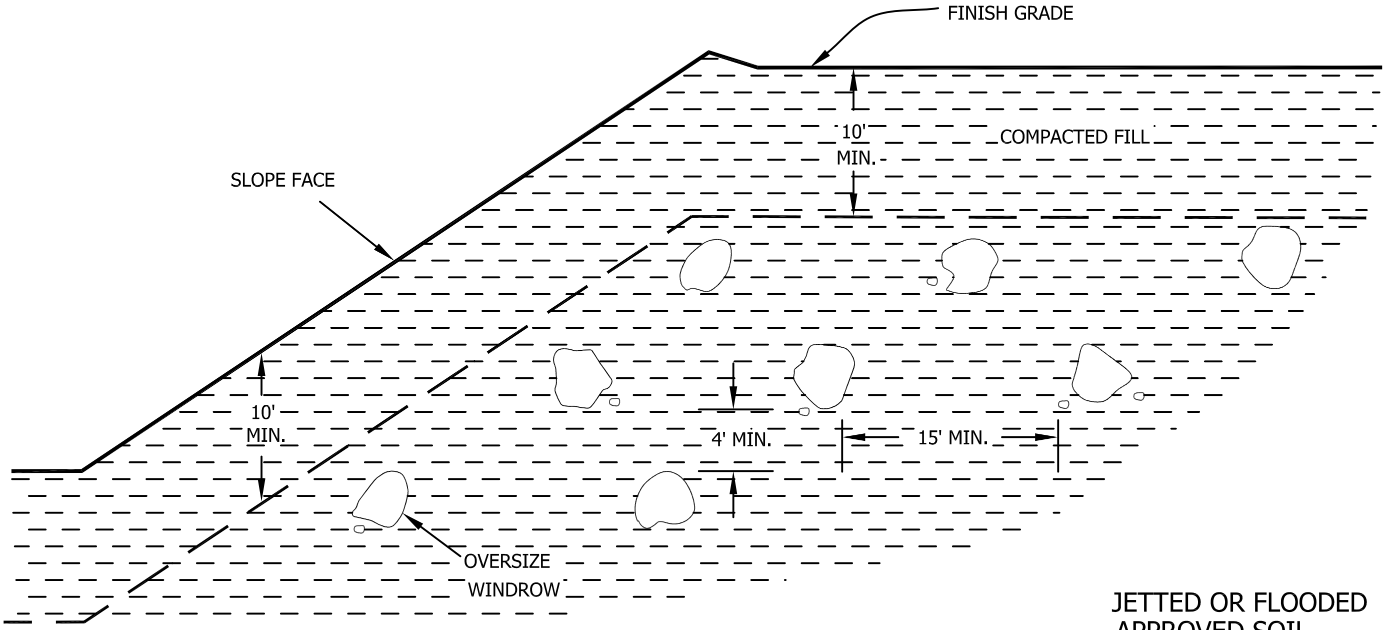


BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

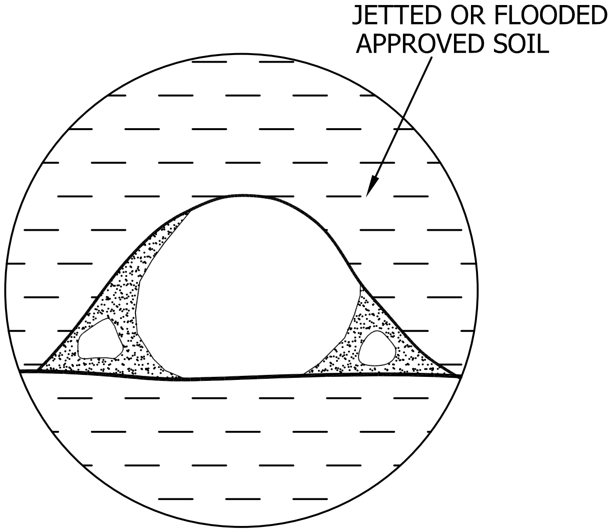
KEYING AND BENCHING

GENERAL EARTHWORK AND GRADING
SPECIFICATIONS
STANDARD DETAILS A

Leighton

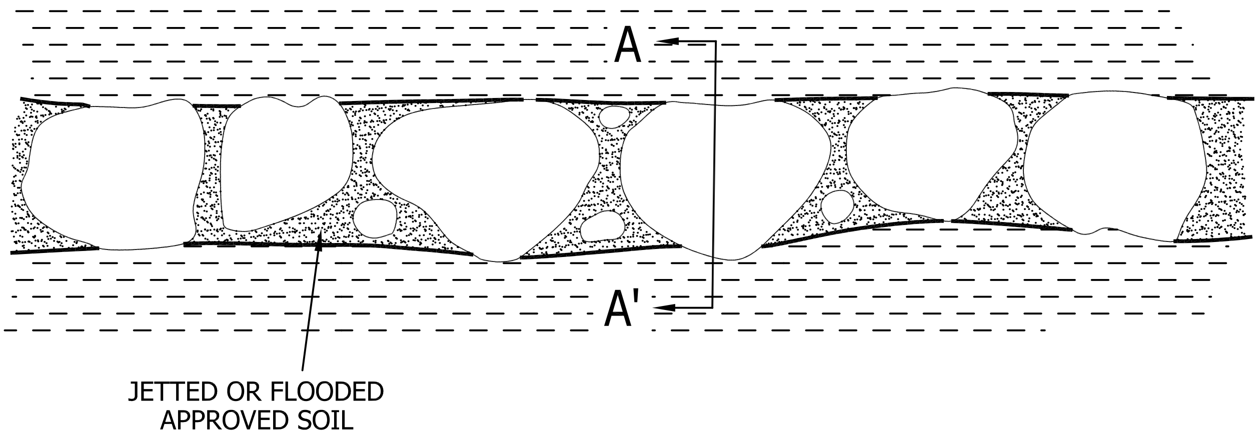


- Oversize rock is larger than 8 inches in largest dimension.
- Backfill with approved soil jetted or flooded in place to fill all the voids.
- Do not bury rock within 10 feet of finish grade.
- Windrow of buried rock shall be parallel to the finished slope face.



SECTION A-A'

PROFILE ALONG WINDROW



OVERSIZE ROCK DISPOSAL

GENERAL EARTHWORK AND GRADING
SPECIFICATIONS
STANDARD DETAILS B

Leighton

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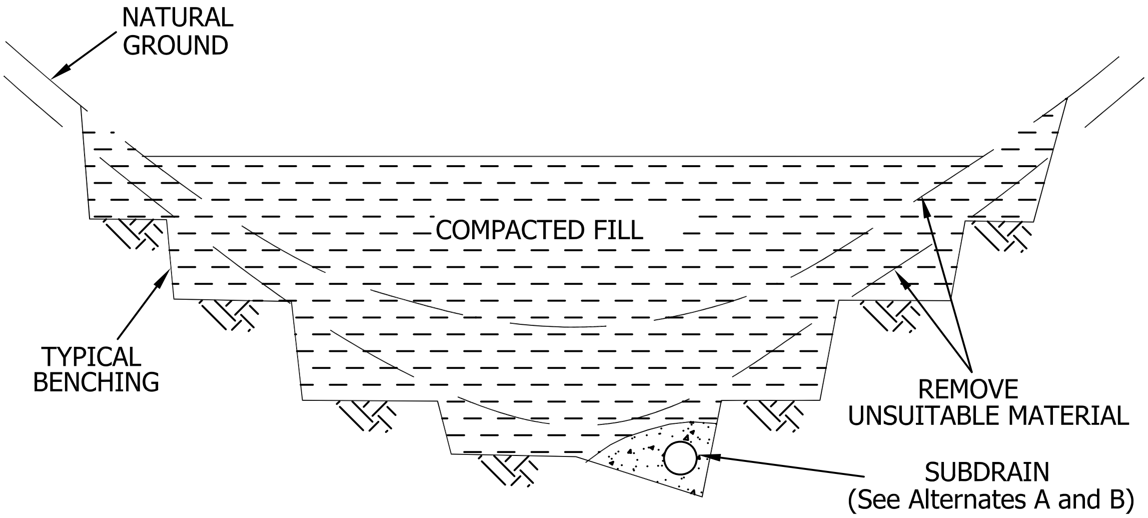
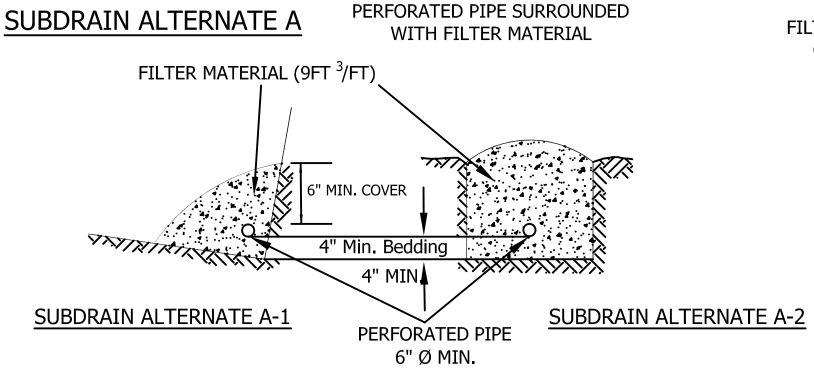


SUBDRAIN ALTERNATE A

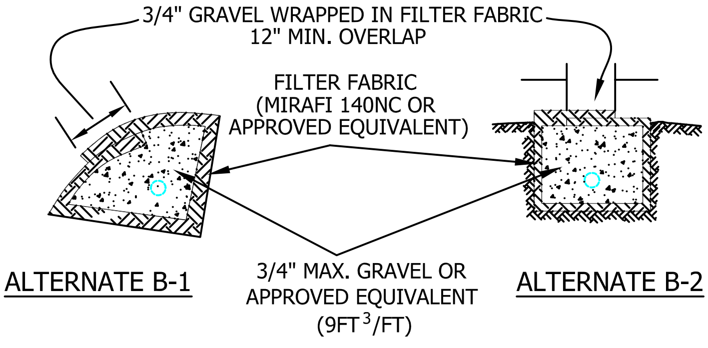


FILTER MATERIAL

FILTER MATERIAL SHALL BE CLASS 2 PERMEABLE MATERIAL PER STATE OF CALIFORNIA STANDARD SPECIFICATION, OR APPROVED ALTERNATE. CLASS 2 GRADING AS FOLLOWS:

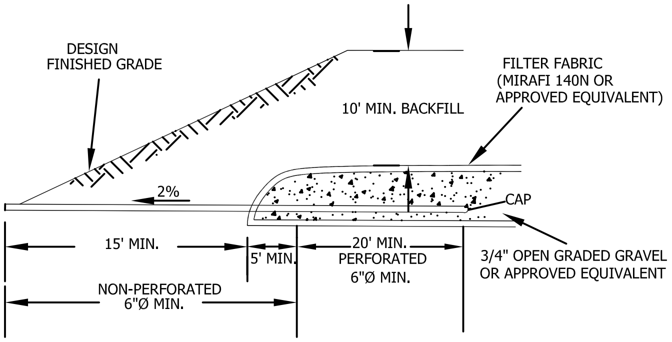
Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

SUBDRAIN ALTERNATE B



○ PERFORATED PIPE IS OPTIONAL PER GOVERNING AGENCY'S REQUIREMENTS

DETAIL OF CANYON SUBDRAIN TERMINAL



CANYON
SUBDRAIN

GENERAL EARTHWORK AND GRADING
SPECIFICATIONS
STANDARD DETAILS C

Leighton

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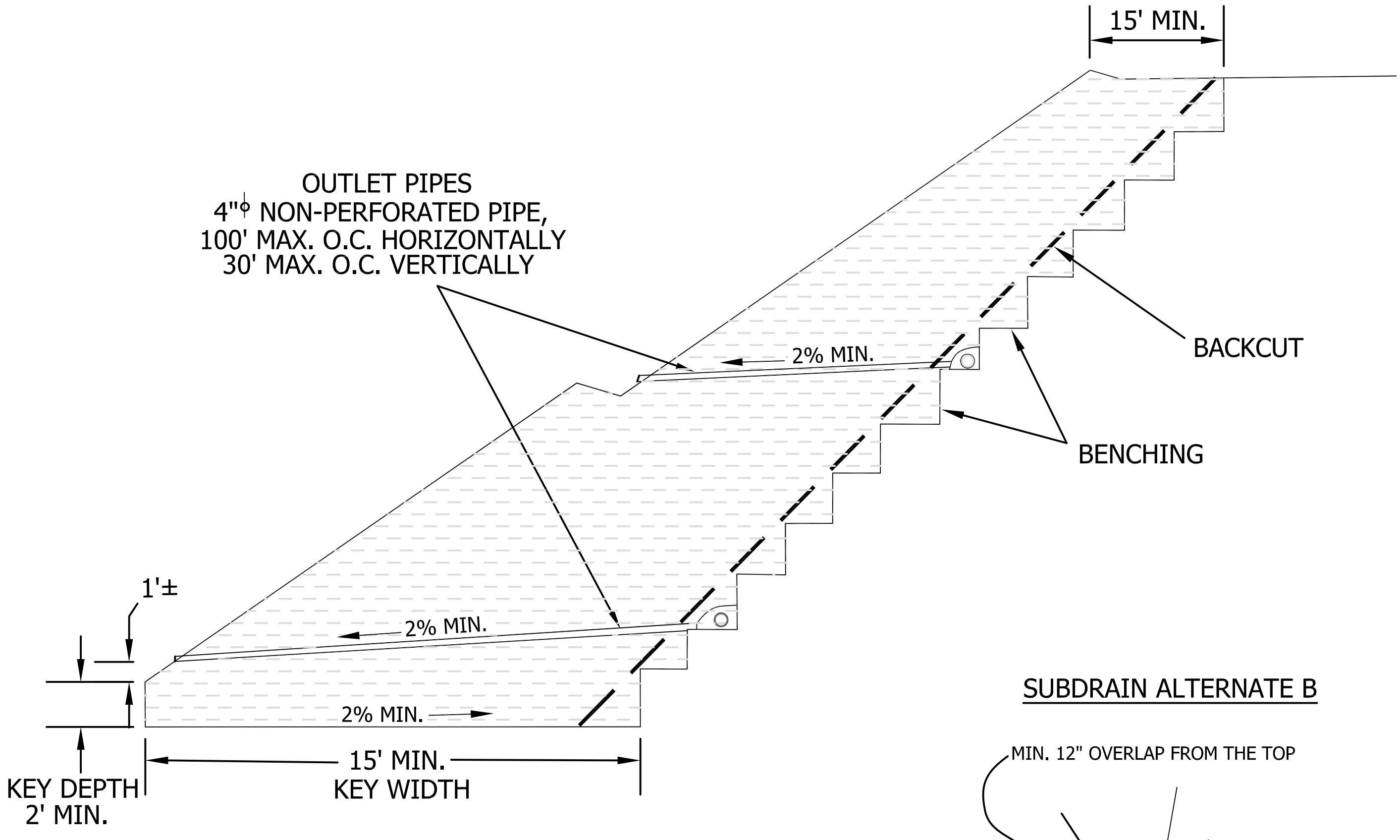
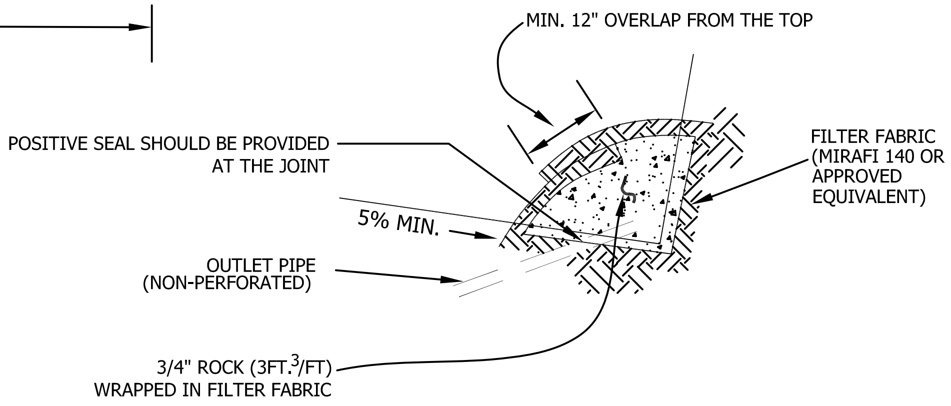
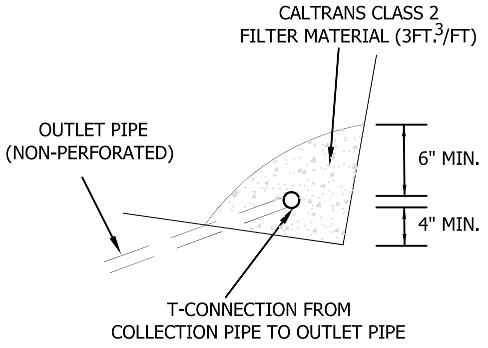


SUBDRAIN ALTERNATE B



SUBDRAIN ALTERNATE A



- SUBDRAIN INSTALLATION - Subdrain collector pipe shall be installed with perforations down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" to 1/2" if drilled holes are used. All subdrain pipes shall have a gradient at least 2% towards the outlet.
- SUBDRAIN PIPE - Subdrain pipe shall be ASTM D2751, ASTM D1527 (Schedule 40) or SDR 23.5 ABS pipe or ASTM D3034 (Schedule 40) or SDR 23.5 PVC pipe.
- All outlet pipe shall be placed in a trench and, after fill is placed above it, rodded to verify integrity.

BUTTRESS OR REPLACEMENT FILL SUBDRAINS

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAILS D

Leighton

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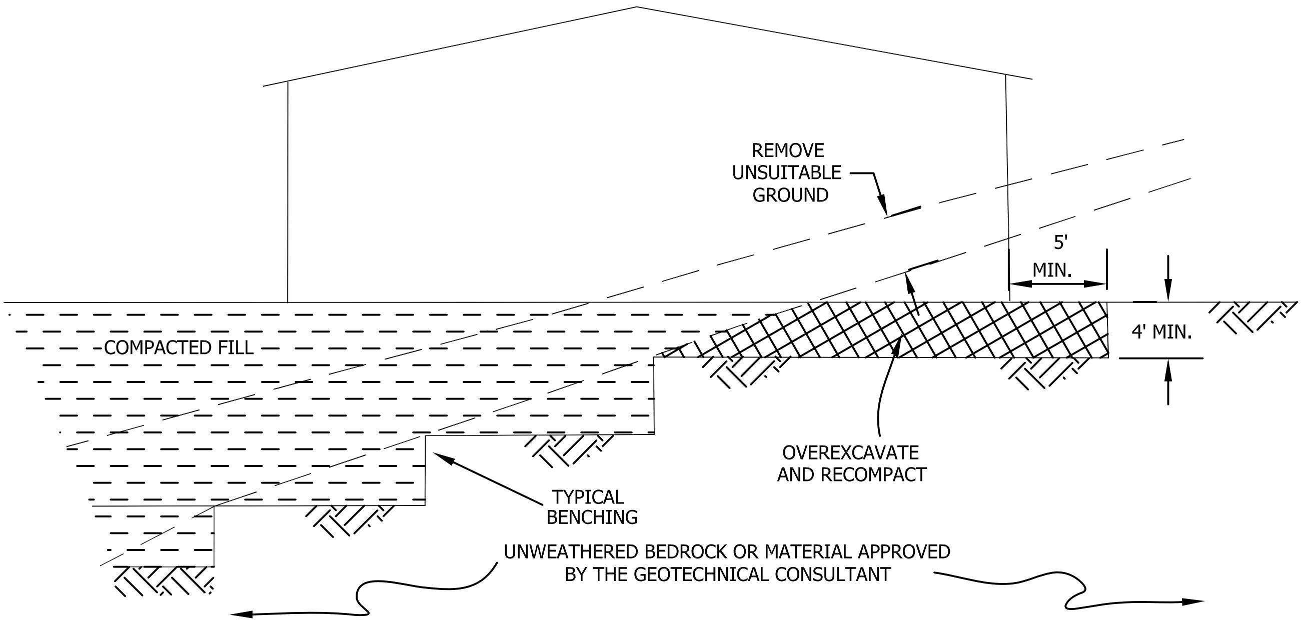
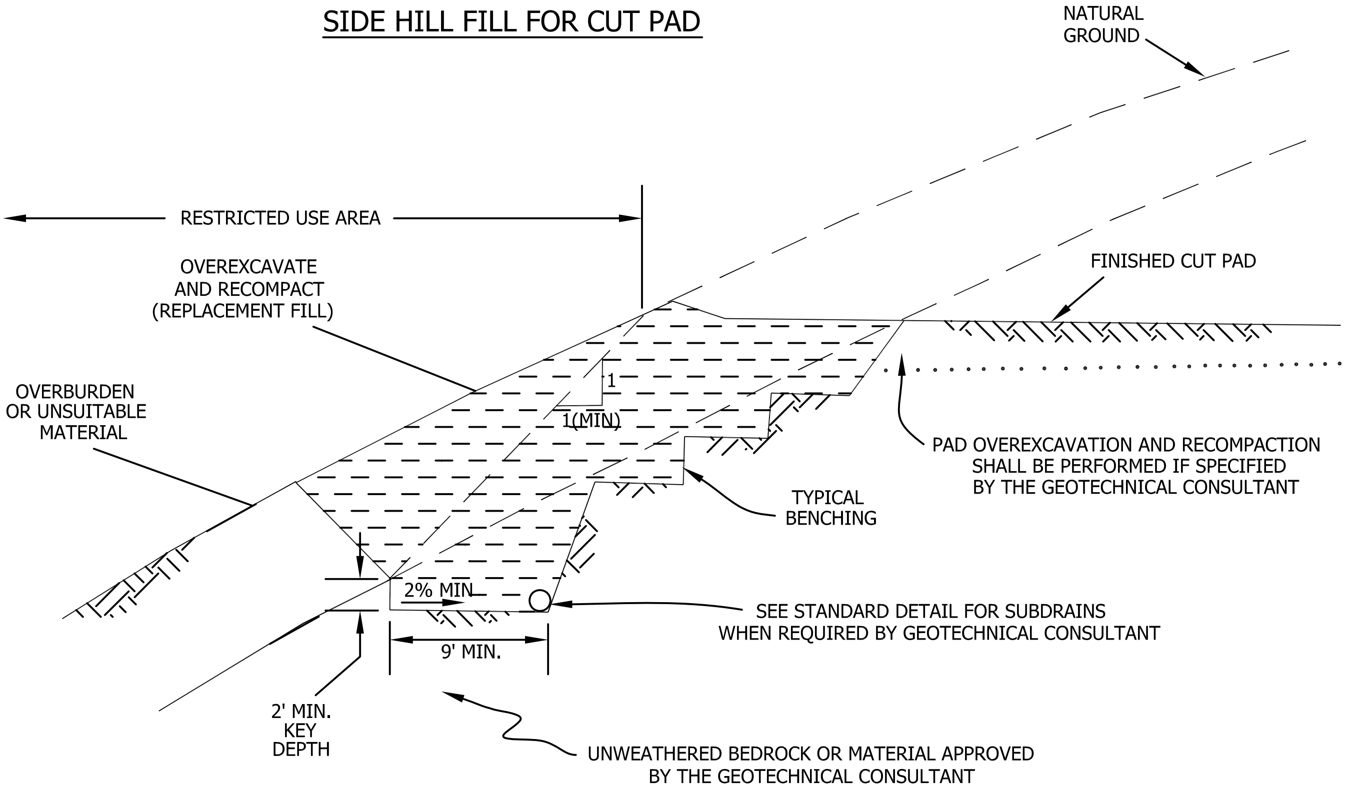
CUT-FILL TRANSITION LOT OVEREXCAVATION



SIDE HILL FILL FOR CUT PAD



TRANSITION LOT FILLS AND SIDE HILL FILLS

GENERAL EARTHWORK AND GRADING
SPECIFICATIONS
STANDARD DETAILS E

Leighton

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APPENDIX F

GEOPROFESSIONAL BUSINESS ASSOCIATION (GBA)
IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL REPORT

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

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GEOTECHNICAL AND GEOLOGICAL ENGINEERING INVESTIGATION REPORT

VARIOUS CAMPUS UPGRADES
AVALON K-12 SCHOOL
200 FALLS CANYON ROAD
AVALON, SANTA CATALINA ISLAND
CALIFORNIA 90704

PREPARED FOR:
LONG BEACH UNIFIED SCHOOL DISTRICT
2425 WEBSTER AVENUE
LONG BEACH, CA 90810

PREPARED BY:
KOURY ENGINEERING & TESTING, INC.
14280 EUCLID AVENUE
CHINO, CALIFORNIA 91710

PROJECT NO. 19-1126

FEBRUARY 6, REVISED MARCH 20, 2020

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February 6, Revised March 20, 2020
Project No. 19-1126

Long Beach Unified School District
2425 Webster Avenue
Long Beach, CA 90810

Attention: Misters Alex Rosas & Elston Soares
Facilities Development & Planning
2425 Webster Avenue
Long Beach, CA 90810

SUBJECT: Geotechnical and Geological Engineering Investigation
Avalon K-12 School – Campus Upgrades
200 Falls Canyon Road
Avalon, Santa Catalina Island, CA 90704

1. INTRODUCTION

This report presents the results of a preliminary Geotechnical and Geological Investigation (Geohazard Study) performed by Koury Engineering & Testing, Inc. (Koury) for the design and construction of the proposed upgrades at the Avalon K-12 School located at 200 Falls Canyon Road, Avalon, California. The study was performed to evaluate the subsurface soil conditions in the area of the proposed upgrades in order to provide geotechnical recommendations for design and construction. This report includes our findings and recommendations for the design and construction of the proposed site improvements.

The recommendations provided within this submittal are based on the results of our field exploration, laboratory testing and engineering analyses. Our services were performed in general accordance with our Proposal No. 19-1126, dated October 31, 2019.

Our professional services have been performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this report. This report has been prepared exclusively for the Long Beach Unified School District and their consultants for the subject project. The report has not been prepared for use by other parties and may not contain sufficient information for the purposes of other parties or other uses.

2. SITE CONDITIONS

The Avalon K-12 School is bounded by Falls Canyon Road on the north, Avalon Canyon Road to the east, undisturbed vegetated hillside to the south and commercial properties and open land on the west. The main access to the site is from Falls Canyon Road. Building 2000 for which a moment frame will be added is located within the eastern portion of the campus and the proposed new ramps and retaining walls are located south and west of Building 6000, and west of Building 5000 located in the western portion of the campus.

The portion of the campus for the proposed new ramps and retaining walls are presently occupied by fencing and a preexisting unpaved pedestrian path leading to the eastern edge of the grass playfield on the west side of Building 5000. The slope is inclined up toward the west-northwest for a total slope height of approximately 19 feet above Building 5000 pad grade. There is an existing ramp and sidewalk on the east side of Building 5000 leading to the lower level hardscape playground along the south side of Building 6000. The concrete ramp has a curb and metal hand rail. The sidewalk is truncated by asphalt at the upper and lower hardscape playground. There are a few planters with grass and trees paralleling some sections of the walkway.

The portions of the site where improvements are proposed generally slope to the east, and the ground surface lies at elevations between about 125 and 85 feet (NAVD88). Drainage of the site is generally by sheet flow toward the east.

3. PROPOSED IMPROVEMENTS

Koury understands that the Long Beach Unified School District is planning to perform some modernization at Avalon K-12 School, which includes construction of new retaining walls and ramps near Buildings 5000 and 6000. It is also understood that a moment frame will be constructed near the center portion of the kitchen area inside Building 2000. The associated improvements may also include new poles, utility lines, sidewalks, landscape and hardscape areas and accessibility upgrades required for path of travel, parking area and restroom. The proposed ramps will be located on the east side of Building 5000 and will be directed to an asphalt apron leading to a new concrete pavement ramp along the south side of Building 6000. A new site wall will be constructed near the southwest corner of Building 5000 and a new concrete retaining wall will be constructed northeast of Building 5000 along the pre-existing pedestrian path leading to the grass playfield.

Architectural and structural design details for the proposed moment frame and retaining walls were not provided. Koury assumed maximum column loads of about 20 kips and maximum wall loads not exceeding 3 kips per lineal foot.

4. FIELD EXPLORATION

The field exploration program consisted of drilling four shallow test borings (B-1 through B-4) on January 6th and 7th, 2020, using hand-held auger drilling equipment. The borings were drilled to depths ranging between about 3½ and 10 feet below the existing ground surface.

The locations of the borings are shown on the Boring Location Map, Figure A-2, presented in Appendix A. California ring samples and bulk samples were obtained from the borings for laboratory testing. The depths, blow counts, and description of the samples are shown on the attached boring logs presented in Appendix B of this report.

5. LABORATORY TESTING

Laboratory tests, including moisture content, dry unit weight and #200 sieve analysis were performed to aid in the classification of the materials encountered and to evaluate their engineering properties. The results of pertinent laboratory tests are presented on the boring logs in Appendix B, and/or in Appendix C.

6. SOIL CONDITIONS

The subsurface soil profile consists of fill underlain by alluvial deposits. The fill depth was found to range between about 2½ and 5 feet at the boring locations. Deeper fill may be encountered between and/or beyond the borings.

The fill encountered in the borings consists of various mixtures of silty sand with gravel, sand with silt and gravel, and gravel with silt and sand. The fill was found to be generally moist and loose to medium dense.

The underlying alluvium at shallow depth appears to consist predominantly of interbedded gravel with silt and sand, silty sand, and silty gravel with cobbles and boulders. The alluvium was found to be generally medium dense.

The soils encountered at shallow depths are generally slightly moist to moist except for localized areas that were very moist. The moisture contents of the soils were found to range from about 4 to 14 percent with an average of about 7 percent. Our #200 sieve wash tests indicated that the fines contents generally range from about 5 to 23 percent with an average of about 11½ percent. Based on the test data, the dry unit weights range from about 96 to 122 pcf with an average of about 114 pcf. The gravel contents of the sample tested range from about 7 to 60 percent with an average of approximately 40 percent.

Variations in the soil conditions as well as detailed descriptions are indicated on the attached boring logs in Appendix B. The soil conditions described in this report are based on the soils observed in the test borings drilled for this investigation and the laboratory test results. Variations between and beyond the borings should be anticipated.

7. GROUNDWATER

The areas of the proposed improvements lie at approximately elevations 88 to 125 feet (NAVD88). Groundwater was not encountered in the borings drilled for this study. According to the City of Avalon Canyon Watershed presentation dated 12-17-2015, there are two water wells in the golf course area with water at roughly elevations 10 to 15 feet (NAVD88). If that data is extrapolated to Building 2000, the groundwater would be about 75 feet below ground surface at Building 2000. These water wells are located about 400 feet east of the campus, and the west end of the campus is at approximately elevation 65 feet, which would indicate that the groundwater depth could be at least 50 feet below ground surface on the campus.

8. SITE GEOLOGY

The site is located on Santa Catalina Island, one of the island groups known as the Channel Islands, off the coast of southern California. Santa Catalina Island lies about 20 miles south of San Pedro Hill, the nearest point on the mainland. The general trend of the island is northwest to west. Its length is approximately 21 miles, with an average width of about 3 miles. The island topography is very bold and rugged and is dominated by steep ridges and V-shaped canyons.

The southern half of the island is underlain by Tertiary intrusive rocks including rhyolite, andesite, and basalt. Most of the City of Avalon, including the school campus, is underlain by young alluvium and alluvium fan deposits from Quaternary time. The borings drilled during our investigation on January 6th & 7th, 2020, encountered fill and alluvial materials consisting predominantly of gravel and sand; thus, consistent with regional mapping.

9. OIL WELL

The State of California Department of Conservation, Division of Oil, Gas and Geothermal Resources, indicates that Avalon K-12 School is located 26 miles southwest of the Huntington Beach Oil/Gas Field and 28 miles southwest of the Wilmington Oil/Gas Field. The nearest plugged oil well is located about 21 miles northeast of the site, the nearest idle hole is located about 25 miles northeast of the site and the nearest active well is located about 27 miles northeast of the site (See Figure A-7, in Appendix A).

During our subsurface exploration, we did not observe oil-field derived hazardous or toxic materials within the borings drilled to the maximum depth of 10 feet. No hazardous materials associated with oil fields are anticipated within the proposed improvement areas.

10. SEISMIC CONSIDERATIONS

10.1. General

The Avalon K-12 School, like the rest of Southern California and its coastal area, is located within a seismically active region as a result of being located near the active margin between the North

American and Pacific tectonic plates. The principal source of seismic activity is movement along the northwest-trending regional faults such as the San Andreas, San Jacinto, Newport-Inglewood and Whittier-Elsinore fault zones.

By definition of the California Geological Survey (CGS), an active fault is one which has had surface displacement within the Holocene Epoch (roughly the last 11,000 years). The CGS has defined a potentially active fault as any fault which has been active during the Quaternary Period (approximately the last 2,000,000 years). These definitions are used in delineating Earthquake Fault Zones as mandated by the Alquist-Priolo Geologic Hazard Zones Act of 1972 and as subsequently revised in 1997 as the Alquist-Priolo Earthquake Fault Zones. The intent of the act is to require fault investigations on sites located within Special Studies Zone to preclude new construction of certain inhabited structures across the trace of active faults.

The subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Based on the California Geological Survey, there are four active faults located within about 30 miles of Avalon. The Palos Verdes/Coronado Bank fault is located approximately 18 miles northeast of the site. The Cabrillo Fault is located about 23 miles northeast of the site. The San Diego Trough fault is located about 27 miles southeast of the site and the Newport Inglewood Fault is located approximately 29 miles east of the site (see Figure A-4, Appendix A).

Based on the information available at this time, it is our opinion that a Mw7.2 earthquake may occur on the nearest segment of the Palos Verdes Fault, a Mw6.5 earthquake may occur on the Cabrillo Fault, a Mw7.3 earthquake may occur on the San Diego Trough Fault, and a Mw7.2 earthquake may occur on the Newport-Inglewood Fault. Large earthquakes could occur on other faults in the general area, but because of their greater distance and/or lower probability of occurrence, they may be less important to the site from a seismic shaking standpoint.

Due to the proximity of the site to the Newport-Inglewood Fault, near field effects from strong ground motion associated with a large earthquake along this fault may occur at the site. These near field effects, including “fling” and directivity of strong ground motion, may result in significantly higher accelerations at the site.

According to the EQSEARCH program, within a search radius of 60 miles, about 40 earthquakes of magnitude 5 or greater have been recorded up to the year 2000. Within that same period, there are records of 4 earthquakes of magnitude 6 or greater, 1 earthquake of magnitude 6.5 or greater and 1 earthquakes of magnitude 7 or greater within the same search area. During that time period, the most significant earthquake for the site had a magnitude of 6.3 and was reported to have occurred in 1933 at a location about 28 miles from the site. Using the attenuation relationship of Campbell and Bozorgnia for alluvium (1997), the past highest acceleration at the site was calculated to be on the order of 0.06g. A summary of the earthquakes with magnitudes 5 and greater is attached in Appendix D.

10.2. Landsliding

There are two known landslide areas in Avalon; one is located along the road to Pebbly Beach and the other is located in the vicinity of Vieudelou Avenue, Hill Street, Olive Street and Maiden Lane. These areas show evidence of rockfall and creep, respectively. Both areas are located about 1 mile and ½ miles away from the site, respectively. No evidence for landsliding was observed on or in the immediate vicinity of the site at the time of our field exploration. Based on topographic conditions, landsliding is considered a low potential hazard at the site.

10.3. Liquefaction

Liquefaction may occur when saturated, loose to medium dense, cohesionless soils are densified by ground shaking or vibrations. The densification results in increased pore water pressures if the soils are not sufficiently permeable to dissipate these pressures during and immediately following an earthquake. When the pore water pressure is equal to or exceeds the overburden pressure, liquefaction of the affected soil layers occurs. For liquefaction to occur, three conditions are required:

- Ground shaking of sufficient magnitude and duration;
- Groundwater level at or above the level of the susceptible soils during the ground shaking; and
- Soils that are susceptible to liquefaction.

The General Plan/Safety Elements of the City of Avalon indicates a low potential for liquefaction to occur at the site. The County of Los Angeles GIS does not show the site as being located in a liquefaction zone. The groundwater level appears to be at least 50 feet below ground surface and the potential site acceleration is mild (PGA_M = 0.529, deaggragated earthquake magnitude = 6.1). Therefore, the potential for liquefaction and significant seismic settlement is considered low.

10.4. Tsunamis and Seiches

The proposed improvement areas are located at elevations exceeding 85 feet and ½ mile away from the coastline. There is no mapped major reservoir in the immediate vicinity and upslope of the site. Therefore, tsunamis and seiches are considered low potential hazards.

11. FLOODING

The project site lies within an area of minimal flood hazard (Zone X) as shown on the FEMA Flood Map #06037C2204F, effective date September 26, 2008 (Figure A-5, Appendix A). Based on the County of Los Angeles GIS and the General Plan/Safety Elements of the City of Avalon, the site is not located in a 100-year return flood zone. The site is also not located within a dam inundation zone; however, the site is located within a 500-year flood zone according to the County of Los Angeles GIS. Flooding is not considered a high potential hazard to the site.

12. COLLAPSIBLE SOILS

Soils prone to collapse are generally young and deposited by flash floods and wind. The onsite soils have been mapped as young alluvium; however, the soil moisture contents were generally found to be within a few percent of optimum, no open skeleton soil structure was observed, and the percentage of silt that in often associated with collapsible soil was in the low range. Therefore, the potential for collapse is considered low. Overerexcavation and recompaction, and appropriate drainage are recommended to mitigate the potential for hydrocollapse.

13. CONCLUSIONS AND RECOMMENDATIONS

13.1. General

In our opinion, the planned improvements are feasible from a geotechnical engineering point of view provided the geotechnical recommendations presented in this report are followed. The main concerns from a geotechnical standpoint are the presence of undocumented fill and loose soil near the ground surface.

The following sections contain preliminary geotechnical recommendations for the design and construction of the proposed improvements and include our recommendations and discussions about bearing capacity, settlement, flatwork, slabs-on-grade, temporary excavations, and utility trenches.

13.2. Grading

13.2.1. Building Footings

The thickness of undocumented fill encountered at the boring locations range from about 2½ to 5 feet. We recommend removing all undocumented fill from the areas of proposed footings. The exact thickness of undocumented fill should be verified at the time of grading.

For the proposed moment frame footings, we recommend complete overexcavation of the existing fill and the subgrade to at least 3 feet below the proposed footings unless testing at the time of construction indicates at least 90 percent relative compaction. Where feasible, the overexcavation should extend laterally at least 3 feet beyond the footing edge. The recommended overexcavation for site work, site walls and concrete mechanical pads is 2½ feet below the foundations with a lateral horizontal extension of at least 3 feet beyond the edge of footings unless indicated otherwise.

Following subgrade approval by the Geotechnical Engineer, the bottom of the removal excavation should be scarified to a depth of 8 inches, moisture conditioned to above optimum and recompacted to 92% relative compaction as determined by ASTM D1557.

All fill placed below footings or within the building pad should be compacted to at least 95% relative compaction at a moisture content above optimum unless approved otherwise by the Geotechnical

Consultant at the time of construction. All fill should be deemed as “failing” and unsuitable if the moisture content is less than the recommended value unless determined otherwise by the Geotechnical Engineer at the time of construction.

13.2.2. Exterior Flatwork, Sidewalk and Pavement Areas

Similarly to the building footprint areas, all abandoned utilities should be removed, and the excavations should be backfilled with engineered fill. We recommend overexcavating 15 inches of subgrade material and placing at least 15 inches of new engineered fill for the subgrade of all new non-structural flatwork and pavement. Prior to backfill placement, the subgrade should be scarified to a depth of 10 inches, moisture conditioned and recompacted to 92% relative compaction.

Except for pavement areas, all fill outside the structure areas should be compacted to at least 92% relative compaction at moisture content above optimum except as indicated otherwise by the Geotechnical Engineer.

13.2.3. General Grading Requirements

1. All fills, unless otherwise specifically stated in the report, should be compacted to at least 92 percent of the maximum dry density as determined by ASTM D1557 Method of Soil Compaction and to 95 percent relative compaction for building support.

2. No fill should be placed until the area to receive the fill has been adequately prepared and approved by the Geotechnical Consultant or his representative.

3. Fill soils should be kept free of debris and organic material.

4. Rocks or hard fragments larger than 2 inches may not be placed in the fill within two feet of footings or slabs without approval of the Geotechnical Consultant or his representative, and in a manner specified for each occurrence. Cobble size exceeding 6 inches should not be placed in fill. There should not be any concentrations of particles sizes of 2 inches or greater; proper mixing should be performed. If encountered, oversize materials should be disposed outside the structural fill and flatwork areas at the locations designated by the District representative.

5. The fill material should be placed in lifts which, when loose, should not exceed 8 inches per lift. Each lift should be spread evenly and should be thoroughly mixed during the spreading operation to obtain uniformity of material and moisture.

6. When the moisture content of the fill material is lower than the specified value or is too low to obtain adequate compaction, water should be added and thoroughly dispersed until

the soil has a moisture content as recommended above unless indicated otherwise in this report and/or by the Geotechnical Engineer at the time of construction.

7. When the moisture content of the fill material is too high to obtain adequate compaction, the fill material should be aerated by blading or other satisfactory methods until the soil has a moisture content as specified herein.

8. Permanent fill and cut slopes should not be constructed at gradients steeper than 2:1(H: V).

We recommend that all excavated soils be pre-mixed and moisture conditioned outside the fill area prior to reuse as fill.

13.3. Fill Materials

13.3.1. Onsite Materials

Borings B-1 and B-3 encountered silty sand with gravel near the ground surface and Borings B-2 and B-4 encountered poorly graded sand with silt and gravel to gravel with silt and sand, respectively. The onsite granular materials encountered in the borings are deemed suitable for re-used as engineered fill provided all oversize particles are removed and the soils are properly moisture conditioned and mixed prior to placement. Some import materials may be needed for backfilling purpose. The imported materials being used for backfilling should have a low expansion potential (EI less than 20), and should comply with the specifications of this report.

Overexcavation and re-compaction will induce fill shrinkage. Many factors such as mixing, relative compaction of the fill, and topographic approximations will affect shrinkage. We cannot estimate the exact amount of shrinkage; however, in our opinion, the shrinkage may be on the order of 10 to 15 percent for existing soils excavated and recompacted to 92 percent relative compaction. This estimate does not include the material that will be required to fill in the excavations after the removal of any subsurface structures from the prior use of the site and removal of oversize particles and topsoil where present.

13.3.2. Import

Import materials should contain sufficient fines (binder material) to be relatively impermeable and result in a stable subgrade when compacted. The imported materials should have an expansion index (EI) less than 20 and should be free of organic materials, debris, and cobbles larger than 2½ inches with no more than 35% passing the # 200 sieve. A bulk sample of potential import material, weighing at least 35 pounds, should be submitted to the Geotechnical Consultant at least 48 hours before fill operations. Other than aggregate base and bedding sand, all proposed import materials should be tested for corrosivity, should be environmentally cleared from contamination and should be approved by the Geotechnical Consultant prior to being imported onsite.

13.4. Temporary Excavations

Temporary excavations adjacent to un-surcharged areas are anticipated to be stable vertically to a depth up to 3½ feet in fill and alluvium. For deeper excavations up to a depth of 6 feet, we recommend a gradient no steeper than 1:1 (H:V) for unsurcharged excavations unless shoring is used.

The tops of slopes should be barricaded to prevent vehicles and storage loads within 6 feet of the tops of the slopes, or within a distance equal to at least the height of the slope, whichever is greater. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes; we should be advised of such heavy vehicle loadings so that specific setback requirements can be established. When excavating adjacent to existing footings or building supports, proper means should be employed to prevent any possible damage to the existing structures. Un-shored excavations should not extend below a 1½:1 (H:V) plane extending downward from the lower edge of adjacent footings and should start at least 2 feet away from the footing edge. Where there is insufficient space to slope back an excavation, shoring may be required. All regulations of State and Federal OSHA should be followed.

Temporary excavations are assumed to be those that will remain un-shored for a period of time not exceeding one week. In dry weather, the excavation slopes should be kept moist, but not soaked. If excavations are made during the rainy season (normally from November through April), particular care should be taken to protect slopes against erosion. Mitigative measures, such as installation of berms, plastic sheeting, or other devices, may be warranted to prevent surface water from flowing over or ponding at the top of excavations.

13.5. Floor Slabs

13.5.1. General

Any new building floor slabs should be underlain by at least 3 feet of engineered fill. The building floor slabs-on-grade, as a minimum, should have a thickness of 5 inches and should contain as a minimum No. 4 bars spaced a maximum of 16 inches on centers, in both directions or as recommended otherwise by the Structural Engineer. The Structural Engineer should ultimately determine the size and spacing of the reinforcement to be used. We recommend a concrete strength of at least 3500 psi unless determined otherwise by the Structural Engineer.

For the proposed new mechanical room floor, we recommend overexcavating the subgrade at least 2½ feet below the slab bottom and to process the exposed bottom as indicated above. The slab thickness should be based on the requirements of the proposed equipment to be installed; however, it should not be less than 5 inches. The slab should be underlain by at least 4 inches of crushed aggregate base.

13.6.2 Moisture Sensitive Floor Covering

Water vapor transmitted through floor slabs is a common cause of floor covering problems. In areas where moisture-sensitive floor coverings (such as tile, hardwood floors, linoleum or carpeting) are planned, a vapor retarder should be installed below the concrete slab to reduce excess vapor transmission through the slab.

The function of the recommended impermeable membrane (vapor retarder) is to reduce the amount of soil moisture or water vapor that is transmitted through the floor slab. The membrane should be at least 15-mil thick, Class A, and care should be taken to preserve the continuity and integrity of the membrane beneath the floor slab. The vapor retarder should conform to ASTM E1745.

A capillary break below the slab may be used at the discretion of the project Architect. At least 4 inches of free draining gravel or coarse sand, with no more than 2 percent passing the ASTM No. 200 sieve, should be placed below the vapor retarder to serve as a capillary break. The gravel or sand layer should be vibrated in place to achieve a minimum of 92% relative compaction per ASTM D1557. The gradation for the free draining material should conform to the requirements for No. 4 Concrete Aggregates as specified in Section 200-1.4 of the Standard Specifications for Public Works Construction (Greenbook).

Another factor affecting vapor transmission through floor slabs is the water to cement ratio in the concrete used for the floor slab. A high water to cement ratio increases the porosity of the concrete, thereby facilitating the transmission of water vapor through the slab. The project Structural

Engineer should provide recommendations for design of the building slab in accordance with the latest version of the applicable codes. The placement of sand above the vapor retarder is the purview of the Structural Engineer.

13.6. Seismic Coefficients

Under the Earthquake Design Regulations of Chapter 16A, Section 1613A of the CBC 2016, and based on the mapped values, the coefficients and factors presented in Table 1 were calculated using the USGS web site (Figure A-6, Response Spectrum).

The site class is determined in accordance with ASCE 7 Chapter 20 using either shear wave velocity, SPT blow count or undrained shear strength. For a site to be classified as Site Class D the weighted average SPT blow count should be between 15 and 50 and the average weighted undrained shear strength should be between 1,000 and 2,000 psf within the upper 100 feet of soil. The SPT blow count test results presented on the boring logs indicate that the requirements for Class D are met.

Table 1 – Seismic Coefficients and Factors

Site Class (CBC 2019 – 1613A.3.2)	D
Seismic Design Category based on Occupancy Category III (CBC 2019-1604A.5 &1613A.3.5)	D
Mapped Acceleration Parameter for Short Period (0.2 Second), S_S	1.090
Mapped Acceleration Parameter for 1.0 Second, S_1	0.374
Adjusted Maximum Spectral Response Parameter for Short Period (0.2 Second), S_{MS}	1.160
Adjusted Maximum Spectral Response Parameter for 1.0 Second Period, S_{M1}	*0.577
Design Spectral Response Acceleration Parameter, S_{DS}	0.773
Design Spectral Response Acceleration Parameter, S_{D1}	*0.385
Peak Ground Acceleration (PGA_M)	0.529

Project Site Coordinates: Longitude: W-118.33193º Latitude: N33.338403º (WGS84)
*Based on F_v of 1.81. See Section 11.4.8 of ASCE 7-16 for calculation requirements

13.7. Shallow Foundations

General: For the purpose of preparing this report, we assumed that the proposed new columns will impose a maximum column load of about 50 kips and new wall loads less than 5 kips per lineal foot. The recommendations for preparation of the subgrade underlying the footings are provided in the “Earthwork” Section of this report. The Structural Engineer should design the foundations in accordance with the requirements of the applicable building code.

Footings should have a minimum width of 2 feet for isolated footings and 18 inches for continuous footings. The bottom of building footings should be located at least 24 inches below the lowest adjacent finish grade, and reinforcement should consist of a minimum of two No. 5 bars, top and bottom or equivalent as determined by the Structural Engineer.

The proposed moment frame may be supported on isolated and/or strip footings designed using a net allowable bearing pressure of 1,800 pounds per square foot (psf) for footings supported on at least 3 feet of engineered fill as indicated in the grading section of this report. This bearing pressure may be increased by 300 psf for each additional foot of depth or width to a maximum of 2200 psf. A one-third increase in the bearing value may be used when considering wind or seismic loads. In the event of new footings located within one footing width of an existing footing, we recommend reducing the bearing pressure of the new footing by 30 percent.

Minor footings may be required for low height exterior landscape walls (5 feet or less in height), ramp wall, retaining walls, masonry fence wall, ball wall (8 feet high at Building 3000) or other small ancillary structures. These footings should be supported on at least 2½ feet of new engineered fill and should be embedded at least 18 inches below the lowest adjacent grade. A vertical bearing pressure of 1,800 psf may be used for these footings. This bearing pressure may be increased by 1/3 for wind or seismic loads.

Lateral Resistance of Footings: Lateral load resistance may be derived from passive resistance along the vertical sides of the foundations, friction acting at the base of the foundations, or a combination of the two. A coefficient of friction of 0.35 may be used between the footings and the supporting soils comprised of engineered fill. The passive resistance of level properly compacted fill soils in direct contact with the footings may be assumed to be equal to the pressure developed by a fluid with a density of 250 pcf, to a maximum pressure of 2,500 psf. To generate this passive pressure resistance, the ground surface in front of the wall should be level for a distance of at least 10 feet or 3½ times the height providing the passive pressure, whichever is greater. A one-third increase in the passive value may be used for wind or seismic loads. The frictional resistance and the passive resistance of the soils may be combined provided that the passive resistance is reduced by one third. We recommend that the first foot of soil cover be neglected in the passive resistance calculations if the ground surface is not protected from erosion or disturbance by a slab, pavement or in a similar manner.

Estimated Settlement of Footings: Based on the results of our analyses and provided that our recommendations in preceding sections of this report are followed, we estimate that the total static settlement of isolated and/or strip footings under sustained loads will be on the order of ¾ inch for the anticipated maximum structural load. The maximum static differential settlement, over a horizontal distance of 40 feet, is anticipated to be on the order of ½ the total settlement for similarly loaded footings.

Pole Footings: As a typical foundation, cast-in-place drilled piers (caisson shafts) are usually used to support the axial and lateral loads of this kind of structure. Lateral loads on the foundation shaft for pole may be resisted by the passive resistance utilized by the surrounding soils. The passive resistance when the ground surface is level, may be assumed to be equal to the pressure developed by a fluid with a density of 150 pcf, with zero point 1.5 feet deep below ground surface and to a maximum value 1,500 psf. These values apply to the design of the poles when they are adversely affected by 0.5 inch of lateral movement at ground level. These lateral resistance values should not be multiplied by 2 as addressed in section 1806A.3.4 of CBC 2010. If the embedment depth is obtained in accordance to the Pole Formula provided in CBC 2019 (Equation 18A-1), the allowable lateral soil-bearing pressure based on a depth of one-third of the embedment depth (S_1 in Section 1807A.3.2.1 of CBC 2019) may be calculated according to the aforementioned lateral resistance values. If movement of 0.5 inch or greater at the ground surface is allowed, the equivalent fluid pressure may be multiplied by 2. No additional resistance increase is recommended for wind or seismic loading.

For vertical support, we recommend a side skin friction resistance of 300 psf. The upper 3 feet of soils should be neglected for the skin friction resistance, and the weight of the foundation may be assumed to be taken by bearing if the loose soils are removed from the bottom of the hole at the time of the excavation. Pole footing can be installed in the existing soil conditions and no overexcavation is recommended for the lateral bearing capacity. However, the upper 1.5 feet of soil should be neglected for both restrained and unrestrained design.

13.8. Retaining Walls

We have assumed that retaining walls, if needed, will have heights in the range of 1½ to 5 feet. Design earth pressures for retaining walls depend primarily on the allowable wall movement, wall inclination, type of backfill materials, backfill slopes, surcharges, and drainage. The earth pressures provided assume that non-expansive soil backfill will be used and a drainage system will be installed behind the walls so that external water pressure will not develop. A drainage system should be provided behind the walls to reduce the potential for development of hydrostatic pressure. If a drainage system is not installed, the cantilever level-backfilled walls, under static conditions, should be designed to resist a hydrostatic pressure equal to that developed by a fluid with a density of 95 pcf for the full height of the wall.

Determination of whether the active or at-rest condition is appropriate for design will depend on the flexibility of the wall. Walls that are free to rotate at least 0.002 radians (deflection at the top of the wall of at least 0.002 x H, where H is the unbalanced wall height) may be designed for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for at-rest conditions. The recommended static active and at-rest earth pressures are provided in the following table.

Table 2 - Earth Pressures for Retaining Walls

Wall Movement	Backfill Condition	Equivalent Fluid Pressure
Free to Deflect	Level	40
Free to Deflect	3:1 Slope	45
Restrained	Level	65

The above lateral earth pressures do not include the effects of surcharge (e.g., traffic, footings, sloping ground) or compaction-induced wall pressures. Any surcharge (live, including traffic, dead load, or slope) located within a 1:1 plane drawn upward from the base of the excavation should be added to the lateral earth pressures. The lateral contribution of a uniform surcharge load located immediately behind walls may be calculated by multiplying the surcharge by 0.33 for cantilevered walls and 0.5 for restrained walls. For vehicular surcharge adjacent to driveways or parking areas a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot traffic surcharge, should be used.

Walls should be waterproofed using appropriate membranes, and properly drained or designed to resist hydrostatic pressures. The waterproofing membrane should be covered with a protection board or equivalent to prevent perforation during backfilling.

Except for the upper 18 inches feet, the backfill immediately behind retaining walls (minimum horizontal distance of 12 inches measured perpendicular to the wall) should consist of free-draining ¾-inch crushed rock wrapped with filter fabric. The upper 1½ feet of cover backfill should consist of relatively impervious onsite material. A 4-inch diameter perforated PVC pipe, placed perforations down at the bottom of the crushed rock layer, leading to a suitable gravity outlet, should be installed at the base of the walls. As an alternative to extending the crushed rock to within 1½ feet of the ground surface for the wall drain, geocomposite panel drains may be used. With wall drain panels, the 4-inch diameter perforated pipe located at the heel of the wall/footing should be surrounded with one cubic foot of ¾-inch crushed rock wrapped with filter fabric; the pipe invert should be supported on about 1½ inches of crushed rock. All drainage should be directed offsite in non-erosive devices.

In the event of a large earthquake, the lateral earth pressure on walls may be significant. When combining both static and seismic lateral earth pressures, a decreased factor of safety may be used in the design of retaining walls when checking for sliding and overturning stability. For cantilever walls, we have calculated the seismic increment of lateral pressure using the Mononobe-Okabe equation assuming the seismic coefficient to be 0.42 of the peak acceleration (PGA_M). We suggest using a dynamic earth pressure increment of 18 psf/ft for cantilever yielding walls with level backfill assuming the walls will not exceed 8 feet in height. The pressure should be taken as an inverted triangular distribution with the zero-pressure point at the toe of the wall and 18 H (psf where H in feet) at the top of the wall, where H is the wall height in feet. For sloping backfill at 3:1 (H:V) behind the wall, a dynamic earth pressure increment of 36 psf/ft may be used for cantilever yielding walls. The point of application of the dynamic thrust may be taken at 0.6H above the toe of the wall. The Structural Engineer should determine if a seismic increment of lateral earth pressure is applicable based on wall heights and allowable wall movements.

13.9. Utility Trench Backfill

Bedding material surrounding utility lines and extending to a point 12 inches above the lines should consist of either sand, fine-grained gravel, or sand-cement slurry to support and/or to protect the lines. A minimum of 4-inch thick bedding material should be placed below the bottom of the utility lines, on a firm and unyielding subgrade. The bedding material should meet the specifications provided in the latest edition of the “Standard Specifications for Public Works Construction” (Greenbook). Sand or gravel should be compacted in accordance with the Greenbook specifications.

Above the bedding, up to finished subgrade in areas other than landscape and up to one foot below flatworks and pavements, utility trenches should be backfilled with onsite materials or imported granular materials and mechanically compacted to at least 92% of the maximum dry density of the soils.

For utility trenches within the building areas, the backfill should be compacted to the minimum required relative compaction indicated under the “Grading” section of this report. The backfill material should be observed, tested and approved by the Geotechnical Consultant. The trench bedding materials should be placed in accordance with Section 306-6 of the “Standard Specifications for Public Works Construction” (Greenbook).

When adjacent to any footings, utility trenches and pipes should be laid above an imaginary line measured at a gradient of 1½ (H:V) projected down from the bottom edges of any footings. Otherwise, the pipe should be designed to accept the lateral effect from the footing load, or the footing bottom should be deepened as needed to comply with this requirement. Backfill consisting of 2-sack sand cement slurry may also be used.

13.10. Drainage

Foundation, slab, flatwork, and pavement performance depend greatly on proper drainage within and along the boundary of the improvements. Perimeter grades around the buildings should be sloped in a manner allowing water to drain away from the structures and not pond next to the foundations. Roof downdrains should be connected to underground pipes carrying water away from the structure areas or have extenders so water does not drain and pond next to the structures. Per the 2019 CBC, landscape areas within 10 feet of structures should slope away at gradients of at least 5 percent. Paved areas within 10 feet of structures should slope away at gradients of at least 2 percent. Proper drainage is recommended for all surfaces to reduce the risk of settlement due to hydroconsolidation. We recommend minimizing the size and number of planters adjacent to buildings and using drought resistant planting.

13.11. Asphalt Concrete (AC) Pavement

The required pavement structural sections depend on the expected wheel loads, volume of traffic, and subgrade soils. The characteristics of subgrade soils are determined by R-value testing. Based on soil classification and our experience with R-value testing correlations with fines contents and plasticity index, we anticipate an R-value of at least 35 for the sand. The R-value should be confirmed with additional tests, if necessary, at the time of construction. The following pavement sections were calculated based assumed traffic indices of 4, 5, 5.5 and 6. We recommend a traffic index of at least 5.5 for driveways where trucks, including trash trucks and fire trucks will have access. The project Civil Engineer should determine the traffic index to be used for different areas of the site. For pedestrian traffic and playground, we recommend at least 2½ inches of asphalt concrete underlain by 3 inches of aggregate base or 4 inches of asphalt concrete over subgrade soils compacted to 95 percent relative compaction.

Table 3 – Alternative Pavement Sections for Vehicular Traffic

Traffic Index	Asphalt Thickness (Inches)	Base Course (CAB) Thickness (Inches)
4	3.0	4.0
5	3.0	5.0

5.5	3.5	5.5
6	4.0	6.0

Base course material should consist of Crushed Aggregate Base (CAB) as defined by Section 200-2.2 of the Standard Specifications for Public Works Construction ("Greenbook"). Base course and asphalt concrete should be compacted to at least 95 percent of the maximum dry density of that material.

The subgrade underlying the pavement areas should be overexcavated 15 inches below the proposed base course layer. Prior to fill placement, the exposed surface should be scarified to a minimum depth of 8 inches, moisture conditioned above optimum moisture content and compacted to at least 92% of the maximum dry density obtained per ASTM D1557. The upper 12 inches of subgrade soils should be compacted to at least 95% relative compaction. The subgrade should be in a "non-pumping" condition at the time of compaction.

Any onsite surficial organic soils within landscaped/turf areas should not be used as subgrade materials. Where feasible, the overexcavation should be laterally extended a minimum of 2 feet beyond the perimeters and edges of parking areas, roadways and curbs. Any abandoned footing and/or underground concrete structure within the work limit should be removed entirely and the excavation should be backfilled to grade.

In order to increase pavement performance and to extend the pavement life, concrete curbs and gutters could be deepened to extend below the base course material and be seated in the compacted subgrade. Priority should be given to areas where heavier traffic is anticipated and where irrigation may be greater. The intent of deepening the curbs and gutters is to form a "cut-off" wall to reduce the amount of water flow through the base course material from adjacent landscaped areas. Subgrade soils, which become soaked as a result of water flowing through base course material, can reduce the life of the pavement and cause heaving of the pavement. Where feasible, the curbs should be deepened to an elevation at least 6 inches below the bottom level of the proposed base course section.

13.12. Portland Cement Concrete (PCC) Vehicular Pavement

The grading recommendations for vehicular PCC pavement are provided in Section 13.2.2 of this report. Base course material used in the pavement sections should consist of Crushed Aggregate Base (CAB) as defined by Section 200-2.2 of the Standard Specifications for Public Works Construction (Greenbook 2012). The aggregate base course should be compacted to at least 95% of the maximum dry density of that material.

The recommendations presented herein should be used for design and construction of the slabs and pertaining grading work underlying vehicular pavement areas. A minimum modulus of rupture of 550 psi for concrete has been assumed in designing of the PCC pavement sections; this corresponds to a concrete compressive strength of approximately 4,000 psi at 28 days. A qualified design professional should specify where heavy duty and standard duty slabs are used based on the anticipated type and frequency of traffic. The recommended PCC pavement sections are provided in the following table.

Table 4 - PCC Pavement Sections

Pavement Type	Portland Cement Concrete Thickness (inches)	Base Course (CAB) Thickness (inches)
Light Duty	6.0	4.0
Heavy Duty	7.0	6.0

These concrete pavement sections should be increased for bus traffic where applicable. The following recommendations should also be incorporated into the design and construction of PCC pavement sections:

- The pavement sections should be reinforced with No. 3 rebars spaced at 18 inches on centers each way to reduce the potential for shrinkage cracking.
- Joint spacing in feet should not exceed twice the slab thickness in inches, e.g., 12 feet for a 6-inch thick slab. Regardless of slab thickness, joint spacing should not exceed 15 feet.
- Layout joints should form square panels. When this is not practical, rectangular panels can be used if the long dimension is no more than 1.5 times the short one.
- Control joints should have a depth of at least 1/4 the slab thickness, e.g., 1 inch for a 4-inch thick slab.

- Where the pavement does not abut against a curb or gutter, an 8-inch thickened edge should be constructed.
- Pavement section design assumes that proper maintenance such as sealing, and repair of localized distress will be performed on a periodic basis.

Exterior concrete slabs for pedestrian traffic or landscape should be at least four inches thick. Weakened plane joints should be located at intervals of no more than about 6 feet unless slabs thicker than 4 inches are used. The pavement sections should be reinforced with No. 3 rebars spaced no further than 18 inches on centers each way to reduce the potential for shrinkage cracking.

If pedestrian pavers are used, they should be supported on one inch of sand underlain by 4 inches of crushed aggregate base (CAB). For light vehicle traffic, the pavers should be underlain by one inch of sand and at least 9 inches of aggregate base (CAB). For heavy duty traffic area, we recommend increasing the aggregate base thickness to 14 inches. A separation/reinforcing fabric should be placed on the prepared subgrade prior to placement of the aggregate base.

14. SOIL EXPANSIVITY

The subsurface soils encountered at shallow depths range from silty sand to gravel with sand. These types of material generally have a low susceptibility to expansion when facing seasonal cycles of saturation/desiccation. The recommendations provided in this report regarding drainage, moisture content during compaction and other pertinent recommendations for site improvements should be incorporated into the design and construction.

15. SOIL CORROSIVITY

The corrosion potential of the onsite materials to steel and buried concrete was preliminarily evaluated. Laboratory testing was performed on selected soil samples to evaluate pH, minimum resistivity, chloride and soluble sulfate content. The test results are presented in the following table.

Table 5 - Corrosion Test Results

Boring	Depth (ft)	Minimum Resistivity (ohm-cm)	pH	Soluble Sulfate Content (ppm)	Soluble Chloride Content (ppm)
B-1	0-1	6580	7.8	22	20

These tests are only an indicator of soil corrosivity for the samples tested. Other soils found on site may be more, less, or of a similar corrosive nature. Imported fill materials should be tested to confirm that their corrosion potential is not significantly more severe than those noted. The concentrations of soluble sulfates indicate that the potential of sulfate attack on concrete in contact with the onsite soils is “negligible” based on ACI 318 Table 4.3.1. Cement Type II may be used in the concrete. Maximum water-cement ratios are not specified for the sulfate concentrations; however, the Structural Engineer should select a concrete with appropriate strength.

Further interpretation of the corrosivity test results, including the resistivity value, and providing corrosion design and construction recommendations are the purview of a corrosion specialists/consultants.

16. OBSERVATION AND TESTING

This report has been prepared assuming that Koury Engineering & Testing, Inc. will perform all geotechnical-related field observations and testing. If the recommendations presented in this report are utilized, and observation of the geotechnical work is performed by others, the party performing the observations must review this report and assume responsibility for the recommendations contained herein. That party would then assume the title of “Geotechnical Consultant of Record”. A representative of the Geotechnical Consultant should be present to observe all grading operations as well as all footing excavations.

17. CLOSURE

The findings and recommendations presented in this report were based on the results of our field and laboratory investigations, combined with professional engineering experience and judgment. The report was prepared in accordance with generally accepted engineering principles and practice. We make no other warranty, either expressed or implied. Subsurface variations between borings should be anticipated. Koury should be notified if subsurface conditions are encountered, which differ from those described in this report since updated recommendations may be required. Samples obtained during this investigation will be retained in our laboratory for a period of 45 days from the date of this report and will be disposed after this period.

Should you have any questions concerning this submittal or the recommendations contained herewith, please do not hesitate to call our office.

Respectfully submitted,

KOURY ENGINEERING & TESTING, INC.

Jacques Roy

JACQUES B. ROY
NO. 2077
Ex. Date 9/30/21

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EIRIK F. HAENSCHKE
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Engineering Geologist

Distribution: 1. Addressee (a pdf copy via e-mail)
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APPENDICES

Appendix A: Maps and Plans

Vicinity Map – Figure A-1
Boring Location Map – Figure A-2
Geology Map – Figure A-3
Fault Map – Figure A-4
Flood Map – Figure A-5
Response Spectrum – Figure A-6
Oil and Gas Map – Figure A-7

Appendix B: Field Exploratory Boring Logs

Borings B-1 through B-4

Appendix C: Laboratory Test Results and Calculations

Appendix D: Historical Earthquake Data

EQSEARCH Results

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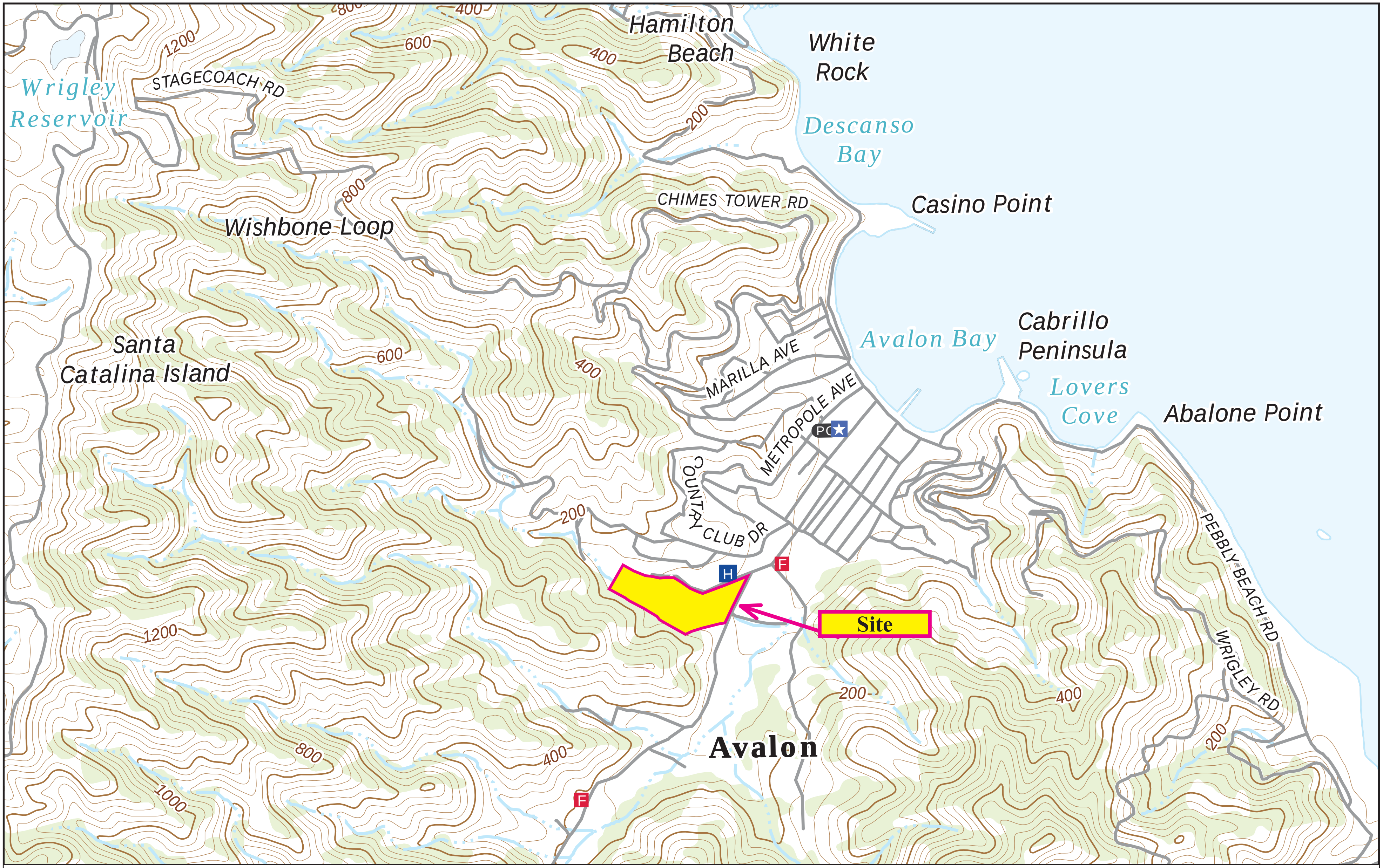
APPENDIX A

Maps and Plans



Reference: USGS Topographic Map, Santa Catalina East Quadrangle, California-Los Angeles County, 7.5 Minutes Series, 2018 - Contour Interval 40 ft.

0 1 2 Mile

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Project Name:	Project No.: 19-1126	Drawing Title:	Figure:
Avalon K-12 School Campus Upgrades	Date: February 2020	Vicinity Map	A-1

LEGEND

● B-4 Boring Location and Number

KOURY PASSION INTEGRITY QUALITY

Project Name:	Project No.: 19-1126	Drawing Title:	Figure:
Avalon K-12 School Campus Upgrades	Date: February 2020	Boring Location Map	A-2

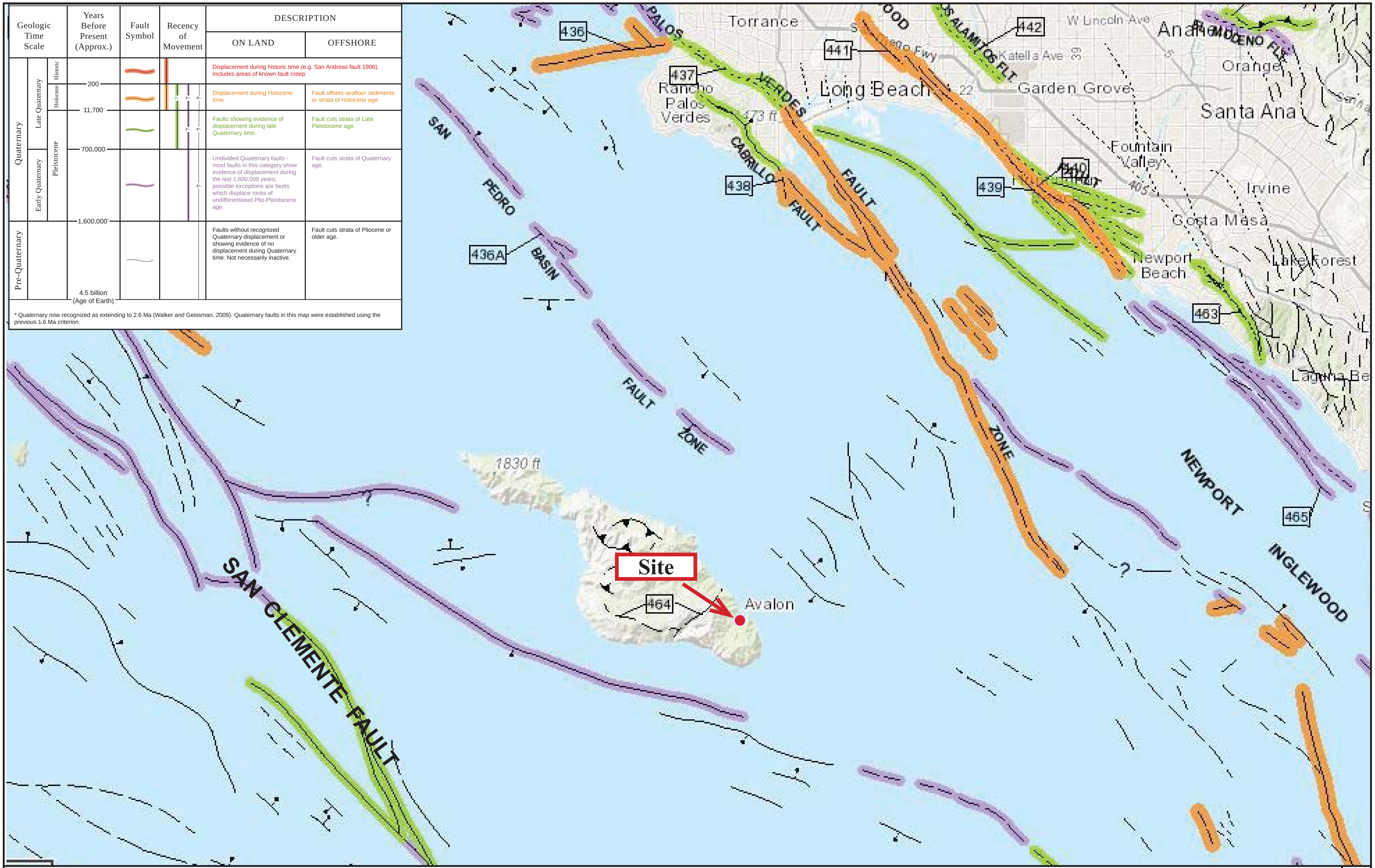
LEGEND

Qal	Alluvium	Ql	Quarternary Lake Deposits

0 3 Mile

Reference: Geologic Map of California, Long Beach Sheet, Compilation by Charles W. Jennings (1962)

KOURY PASSION INTEGRITY QUALITY	Project Name: **Avalon K-12 School Site Upgrades**	Project No.: **19-1126** Date: **February 2020**	Drawing Title: **Geology Map**	Figure: **A-3**



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
						ON LAND	OFFSHORE
Quaternary	Late Quaternary	Historic	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	11,700			Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	700,000			Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		1,600,000*			Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			4.5 billion (Age of Earth)			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.

Reference: Fault Activity Map of California (2010) - California Geological Survey
Department of Conservation Web Site @ http://maps.conservation.ca.gov/cgs/fam/ - See Figure A-4a, for explanation

0 10 Mile

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Project Name:	Project No.: 19-1126	Drawing Title:	Figure:
Avalon K-12 School Campus Upgrades	Date: February 2020	**Fault Map**	**A-4**

EXPLANATION

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. Concealed faults in the Great Valley are based on maps of selected subsurface horizons, so locations shown are approximate and may indicate structural trend only. All offshore faults based on seismic reflection profile records are shown as solid lines where well defined, dashed where inferred, queried where uncertain.

FAULT CLASSIFICATION COLOR CODE (Indicating Recency of Movement)

——— — — — - - - - - - ? -	Fault along which historic (last 200 years) displacement has occurred and is associated with one or more of the following: (a) a recorded earthquake with surface rupture. (Also included are some well-defined surface breaks caused by ground shaking during earthquakes, e.g. extensive ground breakage, not on the White Wolf fault, caused by the Arvin-Tehachapi earthquake of 1952). The date of the associated earthquake is indicated. Where repeated surface ruptures on the same fault have occurred, only the date of the latest movement may be indicated, especially if earlier reports are not well documented as to location of ground breaks. (b) fault creep slippage - slow ground displacement usually without accompanying earthquakes. (c) displaced survey lines.
1906 ▶ ◀ 1906 1838 ▷ ◁ 1838	A triangle to the right or left of the date indicates termination point of observed surface displacement. Solid red triangle indicates known location of rupture termination point. Open black triangle indicates uncertain or estimated location of rupture termination point.
▶ 1951 ◀	Date bracketed by triangles indicates local fault break.
1992	No triangle by date indicates an intermediate point along fault break.
CREEP	Fault that exhibits fault creep slippage. Hachures indicate linear extent of fault creep. Annotation (creep with leader) indicates representative locations where fault creep has been observed and recorded.
1969 1968 ■ ■ 1968	Square on fault indicates where fault creep slippage has occured that has been triggered by an earthquake on some other fault. Date of causative earthquake indicated. Squares to right and left of date indicate termi- nal points between which triggered creep slippage has occurred (creep either continuous or intermittent between these end points).
——— — — — - - - - - - ? -	Holocene fault displacement (during past 11,700 years) without historic record. Geomorphic evidence for Holocene faulting includes sag ponds, scarps showing little erosion, or the following features in Holocene age deposits: offset stream courses, linear scarps, shutter ridges, and triangular faceted spurs. Recency of faulting offshore is based on the interpreted age of the youngest strata displaced by faulting.
——— — — — - - - - - - ? -	Late Quaternary fault displacement (during past 700,000 years). Geomorphic evidence similar to that described for Holocene faults except features are less distinct. Faulting may be younger, but lack of younger overlying deposits precludes more accurate age classification.
——— — — — - - - - - - ? -	Quaternary fault (age undifferentiated). Most faults of this category show evidence of displacement some- time during the past 1.6 million years; possible exceptions are faults which displace rocks of undifferenti- ated Plio-Pleistocene age. Unnumbered Quaternary faults were based on Fault Map of California, 1975. See Bulletin 201, Appendix D for source data.
——— — — — - - - - - - ? -	Pre-Quaternary fault (older that 1.6 million years) or fault without recognized Quaternary displacement. Some faults are shown in this category because the source of mapping used was of reconnaissnce nature, or was not done with the object of dating fault displacements.

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.
——▼—— — ▼ — - - - ▼ - ? -	Low angle fault (barbs on upper plate). Fault surface generally dips less than 45° but locally may have been subsequently steepened. On offshore faults, barbs simply indicate a reverse fault regardless of steepness of dip

OTHER SYMBOLS

491	Numbers refer to annotations listed in the appendices of the accompanying report. Annotations include fault name, age of fault displacement, and pertinent references including Earthquake Fault Zone maps where a fault has been zoned by the Alquist-Priolo Earthquake Fault Zoning Act. This Act requires the State Geolo- gist to delineate zones to encompass faults with Holocene displacement.
—·—·—·—·—·—	Structural discontinuity (offshore) separating differing Neogene structural domains. May indicate disconti- nuities between basement rocks.
//////////////	Brawley Seismic Zone, a linear zone of seismicity locally up to 10 km wide associated with the releasing step between the Imperial and San Andreas faults

KOURY PASSION INTEGRITY QUALITY	**Project Name:** **Avalon K-12 School Campus Upgrades**	**Project No.:** **19-1126** **Date:** **February 2020**	**Drawing Title:** **Fault Map Legend**	**Figure:** **A-4a**

National Flood Hazard Layer FIRMette

SPECIAL FLOOD HAZARD AREAS	🟦	Without Base Flood Elevation (BFE) *Zone A, V, A99* With BFE or Depth *Zone AE, AO, AH, VE, AR*
	▨	Regulatory Floodway
OTHER AREAS OF FLOOD HAZARD	🟧	0.2% Annual Chance Flood Hazard, Ar of 1% annual chance flood with avera depth less than one foot or with draina areas of less than one square mile *Zo*
	▨	Future Conditions 1% Annual Chance Flood Hazard *Zone X*
	▨	Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
	▨	Area with Flood Risk due to Levee *Zone*
OTHER AREAS	NO SCREEN	Area of Minimal Flood Hazard *Zone X*
	☐	Effective LOMRs
	🟧	Area of Undetermined Flood Hazard *Z*
GENERAL STRUCTURES	– — – –	Channel, Culvert, or Storm Sewer
	ı ı ı ı ı ı ı	Levee, Dike, or Floodwall
OTHER FEATURES	Ⓑ 20.2 17.5	Cross Sections with 1% Annual Chanc Water Surface Elevation
	⑧ – –	Coastal Transect
	~~ 513 ~~~	Base Flood Elevation Line (BFE)
	═	Limit of Study
	—	Jurisdiction Boundary
	--- — --- —	Coastal Transect Baseline
	- —	Profile Baseline
	—	Hydrographic Feature
MAP PANELS	☐	Digital Data Available
	☐	No Digital Data Available
	☒	Unmapped

N

The pin displayed on the map is an approxim point selected by the user and does not repre an authoritative property location.

This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The basemap shown complies with FEMA's basemap accuracy standards

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 6/21/2019 at 12:21:48 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

This map image is void if the one or more of the following map elements do not appear: basemap imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date. Map images for unmapped and unmodernized areas cannot be used for regulatory purposes.

Project Name:	Project No.: 19-1126	Drawing Title:	Figure:
Avalon K-12 School Campus Upgrades	Date: February 2020	Flood Map	A-5

Avalon K-12 School

200 Falls Canyon Rd, Avalon, CA 90704, USA

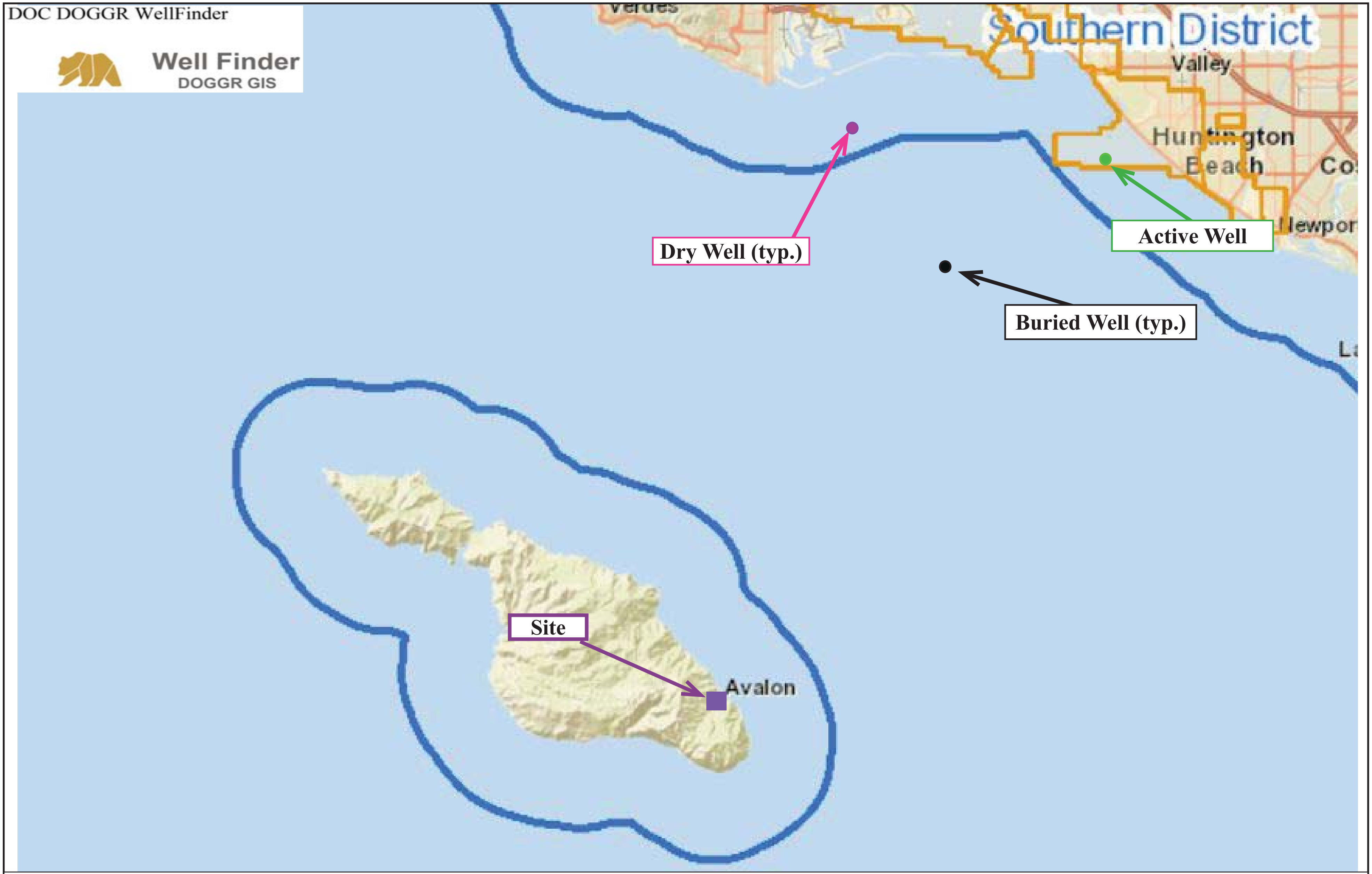
Latitude, Longitude: 33.3384035, -118.3319303

Date	1/21/2020, 3:12:30 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.09	MCE_R ground motion. (for 0.2 second period)
S_1	0.374	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.16	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	0.773	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1.064	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.467	MCE_G peak ground acceleration
F_{PGA}	1.133	Site amplification factor at PGA
PGA_M	0.529	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.09	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.2	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.374	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.413	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.562	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.908	Mapped value of the risk coefficient at short periods

Project Name:	Project No.:	Drawing Title:	Figure:
Avalon K-12 School Campus Upgrades	19-1126 Date: February 2020	Response Spectrum	A-6



Reference: California Department of Conservation, Division of Oil, Gas & Thermal Resources Well Finder (DOGGR)

0 5 mile

N

KOURY
PASSION INTEGRITY QUALITY

Project Name: Avalon K-12 School Campus Upgrades	Project No.: 19-1126 Date: February 2020	Drawing Title: Oil & Gas Map	Figure: A-7

APPENDIX B

Field Exploratory Boring Logs

KEY TO LOGS

SOILS CLASSIFICATION					
MAJOR DIVISIONS			GRAPHIC LOG	USCS SYMBOL	TYPICAL NAMES
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	**GRAVELS** MORE THAN 50% OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	**CLEAN GRAVELS** LESS THAN 5% FINES	░	**GW**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			░	**GP**	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES MORE THAN 12% FINES	▥	**GM**	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
			▤	**GC**	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS 50% OR MORE OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	**CLEAN SANDS** LESS THAN 5% FINES	▣	**SW**	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			░	**SP**	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES MORE THAN 12% FINES	▦	**SM**	SILTY SANDS, SAND-SILT MIXTURES
			▩	**SC**	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS 50% OR MORE OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	**SILTS AND CLAYS** LIQUID LIMIT IS LESS THAN 50		▥	**ML**	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			▨	**CL**	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			▩	**OL**	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT IS 50 OR MORE		▧	**MH**	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR GRAVELLY ELASTIC SILTS
			▨	**CH**	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			▣	**OH**	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			▤	**PT**	PEAT AND OTHER HIGHLY ORGANIC SOILS

GRAIN SIZES							
SILT AND CLAY	**SAND**			**GRAVEL**		**COBBLES**	**BOULDERS**
	FINE	**MEDIUM**	**COARSE**	**FINE**	**COARSE**		
	#200	#40	#10	#4	3/4"	3"	12"
SIEVE SIZES							

KEY TO LOGS (continued)

SPT/CD BLOW COUNTS VS. CONSISTENCY/DENSITY

FINE-GRAINED SOILS (SILTS, CLAYS, etc.)			GRANULAR SOILS (SANDS, GRAVELS, etc.)		
CONSISTENCY	***BLOWS/FOOT**		**RELATIVE DENSITY**	***BLOWS/FOOT**	
	SPT	**CD**		**SPT**	**CD**
SOFT	0-4	0-4	VERY LOOSE	0-4	0-8
FIRM	5-8	5-9	LOOSE	5-10	9-18
STIFF	9-15	10-18	MEDIUM DENSE	11-30	19-54
VERY STIFF	16-30	19-39	DENSE	31-50	55-90
HARD	over 30	over 39	VERY DENSE	over 50	over 90

* CONVERSION BETWEEN CALIFORNIA DRIVE SAMPLERS (CD) AND STANDARD PENETRATION TEST (SPT) BLOW COUNT HAS BEEN CALCULATED USING "FOUNDATION ENGINEERING HAND BOOK" BY H.Y. FANG. (**VALUES ARE FOR 140 Lbs HAMMER WEIGHT ONLY**)

DESCRIPTIVE ADJECTIVE VS. PERCENTAGE

DESCRIPTIVE ADJECTIVE	PERCENTAGE REQUIREMENT
TRACE	1 - 10%
LITTLE	10 - 20%
SOME	20 - 35%
AND	35 - 50%

*THE FOLLOWING "DESCRIPTIVE TERMINOLOGY/ RANGES OF MOISTURE CONTENTS" HAVE BEEN USED FOR MOISTURE CLASSIFICATION IN THE LOGS.

APPROXIMATE MOISTURE CONTENT DEFINITION

DEFINITION	DESCRIPTION
DRY	Dry to the touch; no observable moisture
SLIGHTLY MOIST	Some moisture but still a dry appearance
MOIST	Damp, but no visible water
VERY MOIST	Enough moisture to wet the hands
WET	Almost saturated; visible free water

Boring Log

Project No. 19-1126
Project Name : Avalon K12

Boring No. B-1
Sheet : 1 **Of :** 1

Drilling Method : 4" Hand Auger
Sampling Method : Bulk - CD
Hammer Weight: 50 lbs. **Drop Height:** 18"
Location : See Figure A-2

Ground Elevation:
Drilling Co.: Koury
Date Drilled : 1/06/2020

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Equivalent SPT	Depth (ft)	Sample Location	Graphic Log	Soil Type (USCS)	Description	Additional Tests
				0	☒		SM	Grass over topsoil	Gradation Fines = 17% Gravel = 30%
1	9.5			1	☒			**FILL:** **Silty SAND with GRAVEL;** fine to coarse sand, loose to medium dense, moist, very dark brown	
2			12	2	■			no recovery	
3	9.3	114	14	3	■				Fines = 19% Gravel = 38%
4	13.4			4	☒				Fines = 12% Gravel = 38%
5	4.9		30	5	■		GP-GM	**ALLUVIUM:** **GRAVEL with SILT and SAND;** fine to coarse sand, moist, brown	Fines = 6% Gravel = 60%
6	6.0			6	☒				Fines = 5% Gravel = 47%
7	9.0		24	7	■		SM	**Silty SAND;** fine to medium, trace to little gravel, moist, dark brown	Fines = 13% Gravel = 14%
8	6.0		15	9	■		GM	**Silty GRAVEL**; fine to coarse sand, angular gravel, moist, dark brown	
				10				End of Boring @ 10' due to refusal on cobbles No groundwater encountered	
				11					
				12					
				13					
				14					
				15					
				16					
				17					
				18					
				19					
				20					

▽ ☒ ■ ☒

Boring Log

Project No. 19-1126
Project Name : Avalon K12

Boring No. B-2
Sheet : 1 **Of :** 1

Drilling Method : 4" Hand Auger
Sampling Method : Bulk - CD
Hammer Weight: 50 lbs. **Drop Height:** 18"
Location : See Figure A-2

Ground Elevation:
Drilling Co.: Koury
Date Drilled : 1/06/2020

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Equivalent SPT	Depth (ft)	Sample Location	Graphic Log	Soil Type (USCS)	Description	Additional Tests
				0		■	**PCC**	4" of portland cement concrete, no aggregate base	
1	4.1			1	☒	▨	**SP-SM**	**FILL:** **SAND with SILT and GRAVEL;** fine to coarse sand, loose to medium dense, subrounded gravel, brown	Gradation Fines = 9% Gravel = 39%
2	4.6		25	2	■			no recovery	Fines = 7% Gravel = 36%
3	4.2			3	☒	▨	**GP-GM**	**ALLUVIUM:** **GRAVEL with SILT and SAND;** fine to coarse sand, moist, brown	Fines = 6% Gravel = 48%
				4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20				End of Boring @ 3' 6" due to refusal on cobble No groundwater encountered	

▽ ☒ ■ ☒

Boring Log

KOURY
PASSION INTEGRITY QUALITY

Project No. 19-1126
Project Name : Avalon K12

Boring No. B-3
Sheet : 1 **Of :** 1

Drilling Method : 4" Hand Auger
Sampling Method : Bulk - CD
Hammer Weight: 50 lbs. **Drop Height:** 18"
Location : See Figure A-2

Ground Elevation:
Drilling Co.: Koury
Date Drilled : 1/06/2020

Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Equivalent SPT	Depth (ft)	Soil Type (USCS)	Description	Additional Tests
				0		Exposed subgrade/gravel mix	
1	5.6			1	SM	**FILL:** **Silty SAND with GRAVEL;** fine to coarse sand, loose to medium dense, moist, brown	Fines = 17% Gravel = 39%
2	5.7	122	30	2			Fines = 23% Gravel = 22%
3	4.4			3	GP	**ALLUVIUM:** **GRAVEL with SILT and SAND;** fine to coarse sand, surrounded gravel, moist, dark brown	Fines = 10% Gravel = 52%
4	4.1			4			Fines = 20% Gravel = 55%
				5		End of Boring @ 4' 6" No groundwater encountered	
				6			
				7			
				8			
				9			
				10			
				11			
				12			
				13			
				14			
				15			
				16			
				17			
				18			
				19			
				20			

▽ ⊠ ■ ☒

Boring Log

KOURY
PASSION INTEGRITY QUALITY

Project No. 19-1126
Project Name : Avalon K12

Boring No. B-4
Sheet : 1 **Of :** 1

Drilling Method : 4" Hand Auger
Sampling Method : Bulk - CD
Hammer Weight: 50 lbs. **Drop Height:** 18"
Location : See Figure A-2

Ground Elevation:
Drilling Co.: Koury
Date Drilled : 1/06/2020

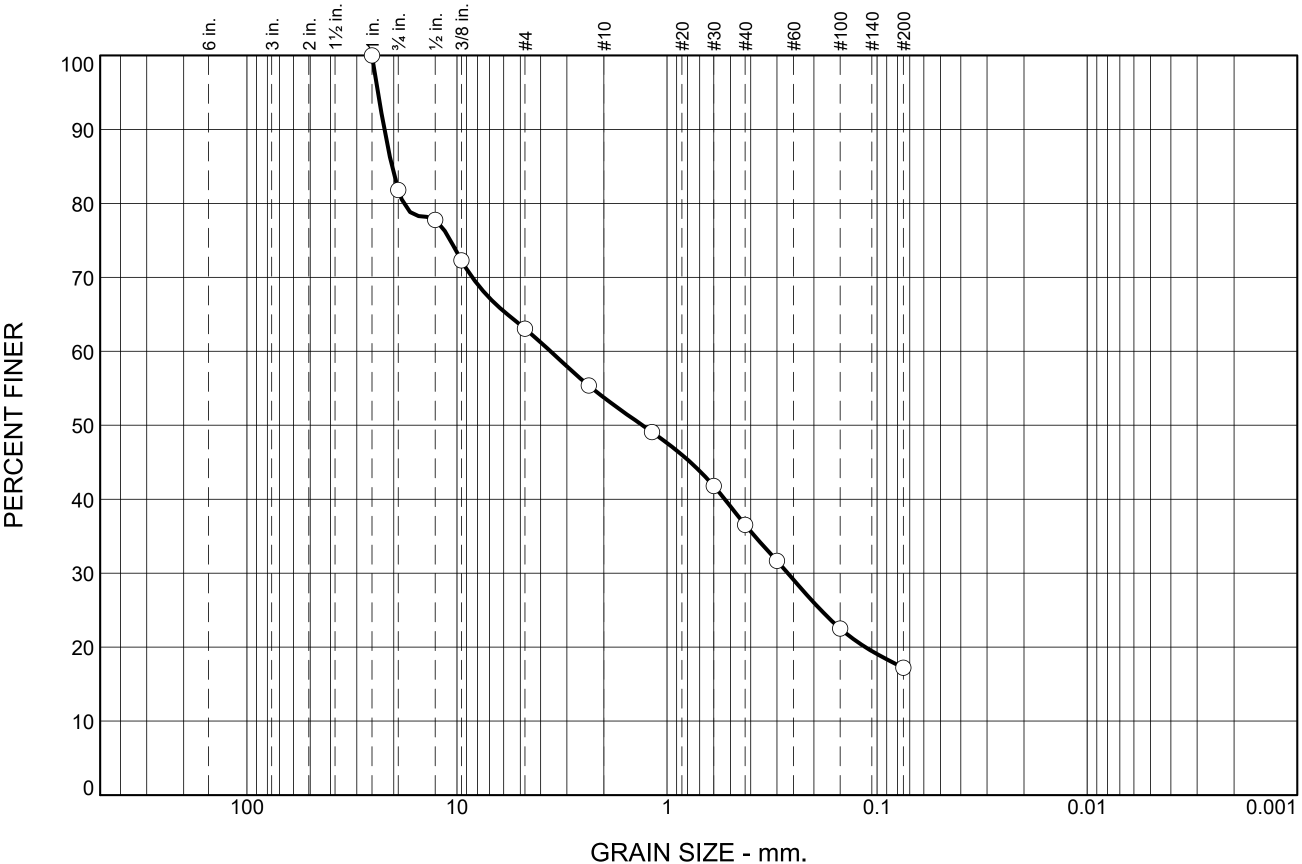
Sample No.	Moisture Content (%)	Dry Unit Weight (pcf)	Equivalent SPT	Depth (ft)	Sample Location	Graphic Log	Soil Type (USCS)	Description	Additional Tests
				0	⊠			Grass over topsoil	
								FILL: **GRAVEL with SILT and SAND;** fine to coarse sand, loose to medium dense, moist, dark brown	Gradation Fines = 7% Gravel = 49%
1	6.6			1	⊠				
				2			GP-GM		
2	6.5	122	24/6"		■				Fines = 6% Gravel = 45%
3	6.1			3	⊠			**ALLUVIUM:** **GRAVEL with SILT and SAND;** fine to coarse sand, subrounded gravel, moist, dark brown	Fines = 5% Gravel = 53%
				4					
4	14.3	96		5	■		SM	**Silty SAND;** fine to medium, trace to little gravel, moist, dark brown	Fines = 16% Gravel = 7%
				6				End of Boring @ 5' 6" Subgrade very moist @ 3', no groundwater encountered	
				7					
				8					
				9					
				10					
				11					
				12					
				13					
				14					
				15					
				16					
				17					
				18					
				19					
				20					

▽ ⊠ ■ ☒

APPENDIX C

Laboratory Test Results & Calculations

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	18.2	18.7	9.4	17.2	19.3	17.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1	100.0		
3/4	81.8		
1/2	77.8		
3/8	72.3		
#4	63.1		
#8	55.4		
#16	49.1		
#30	41.8		
#40	36.5		
#50	31.7		
#100	22.5		
#200	17.2		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 22.1524 D_{85}= 20.4113 D_{60}= 3.5866
D_{50}= 1.3101 D_{30}= 0.2665 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Lab #6341.

Location: B1 @ 0' - 1'
Sample Number: 2010-010 Series

Date: 1/13/20

Koury Engineering & Testing, Inc.

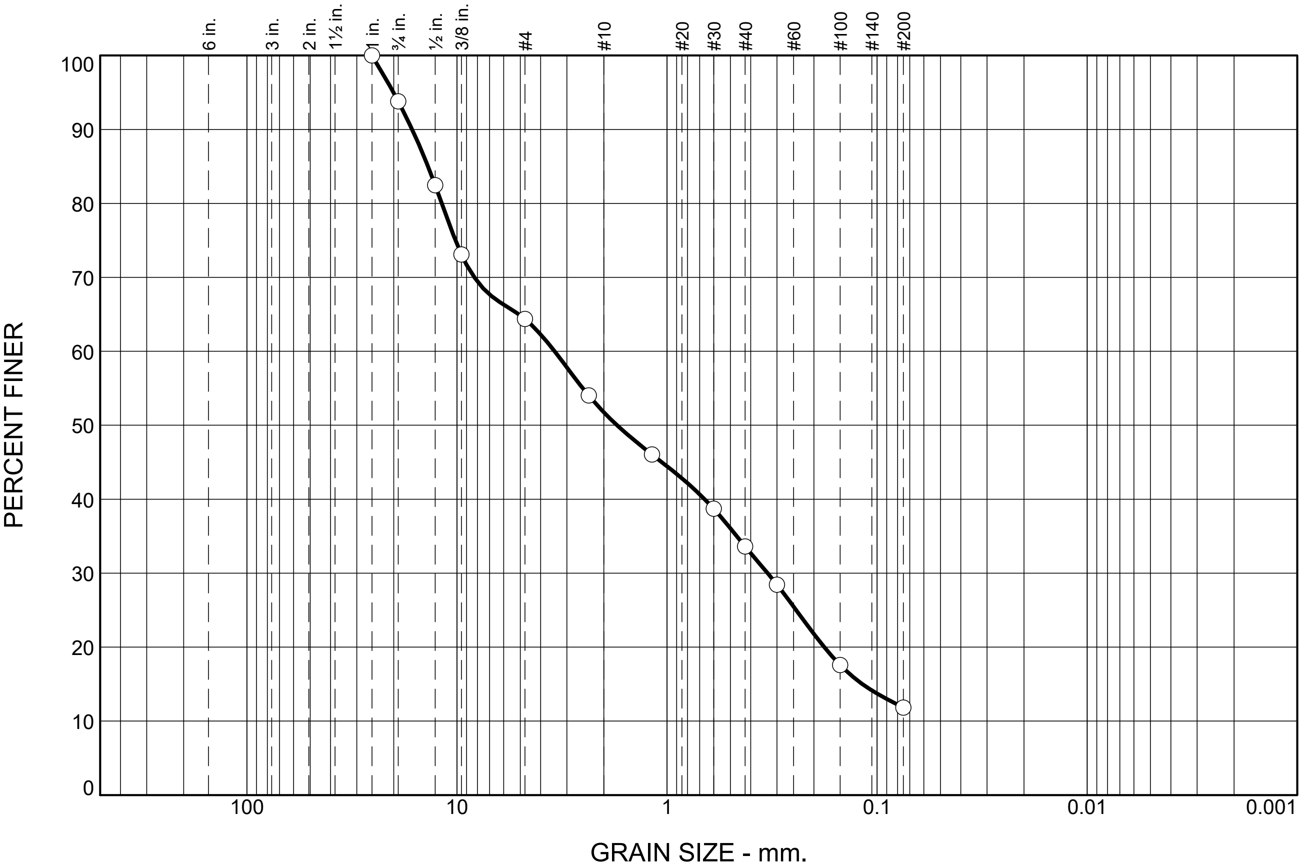
Chino, CA

Client:
Project: Avalon ES - HVAC
Project No: 19-1126 **Figure**

Tested By: Mathew F. Perry **Checked By:**

These results are for the exclusive use of the client for whom they were obtained. They apply only to the samples tested and are not indicitive of apparently identical samples.

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	6.2	29.4	12.6	18.2	21.8	11.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1	100.0		
3/4	93.8		
1/2	82.5		
3/8	73.1		
#4	64.4		
#8	54.0		
#16	46.1		
#30	38.7		
#40	33.6		
#50	28.4		
#100	17.6		
#200	11.8		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 16.3573 D_{85}= 13.7433 D_{60}= 3.4314
D_{50}= 1.7295 D_{30}= 0.3322 D_{15}= 0.1175
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Lab #6341.

Location: B2 @ 0" - 18"
Sample Number: 2020-010 Series

Date: 1/13/20

Koury Engineering & Testing, Inc.

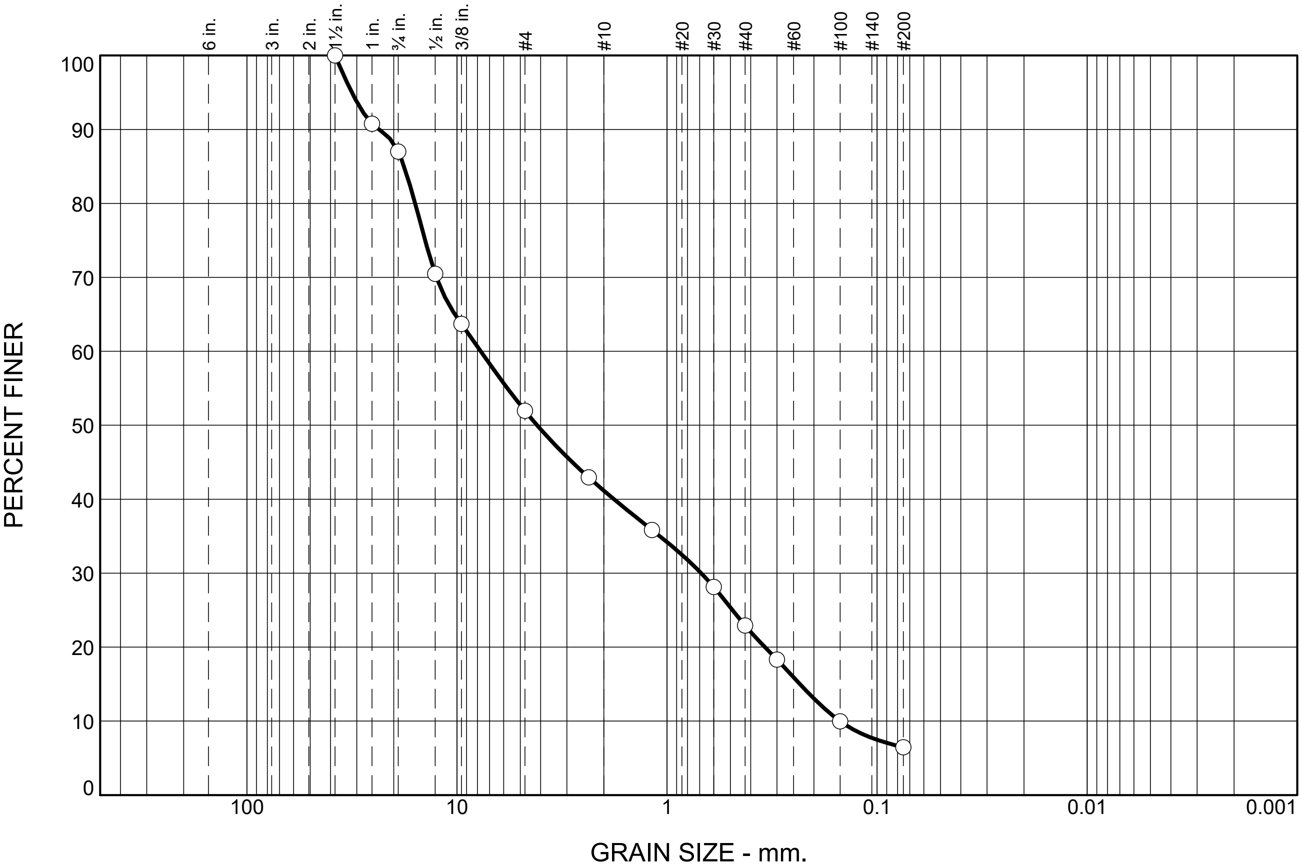
Chino, CA

Client:
Project: Avalon ES - HVAC
Project No: 19-1126 **Figure**

Tested By: Mathew F. Perry **Checked By:** ____________________

These results are for the exclusive use of the client for whom they were obtained. They apply only to the samples tested and are not indicitive of apparently identical samples.

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	13.0	35.0	10.8	18.3	16.4	6.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.5	100.0		
1	90.8		
3/4	87.0		
1/2	70.5		
3/8	63.7		
#4	52.0		
#8	42.9		
#16	35.8		
#30	28.1		
#40	22.9		
#50	18.3		
#100	10.0		
#200	6.5		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 23.5519 D_{85}= 17.8558 D_{60}= 7.7114
D_{50}= 4.1552 D_{30}= 0.6880 D_{15}= 0.2331
D_{10}= 0.1504 C_u= 51.26 C_c= 0.41

Classification

USCS= AASHTO=

Remarks

Lab #6341.

Location: B4 @ 0.5' - 2'
Sample Number: 2010-010 Series

Date: 1/13/20

Koury Engineering & Testing, Inc.

Chino, CA

Client:
Project: Avalon ES - HVAC
Project No: 19-1126 **Figure**

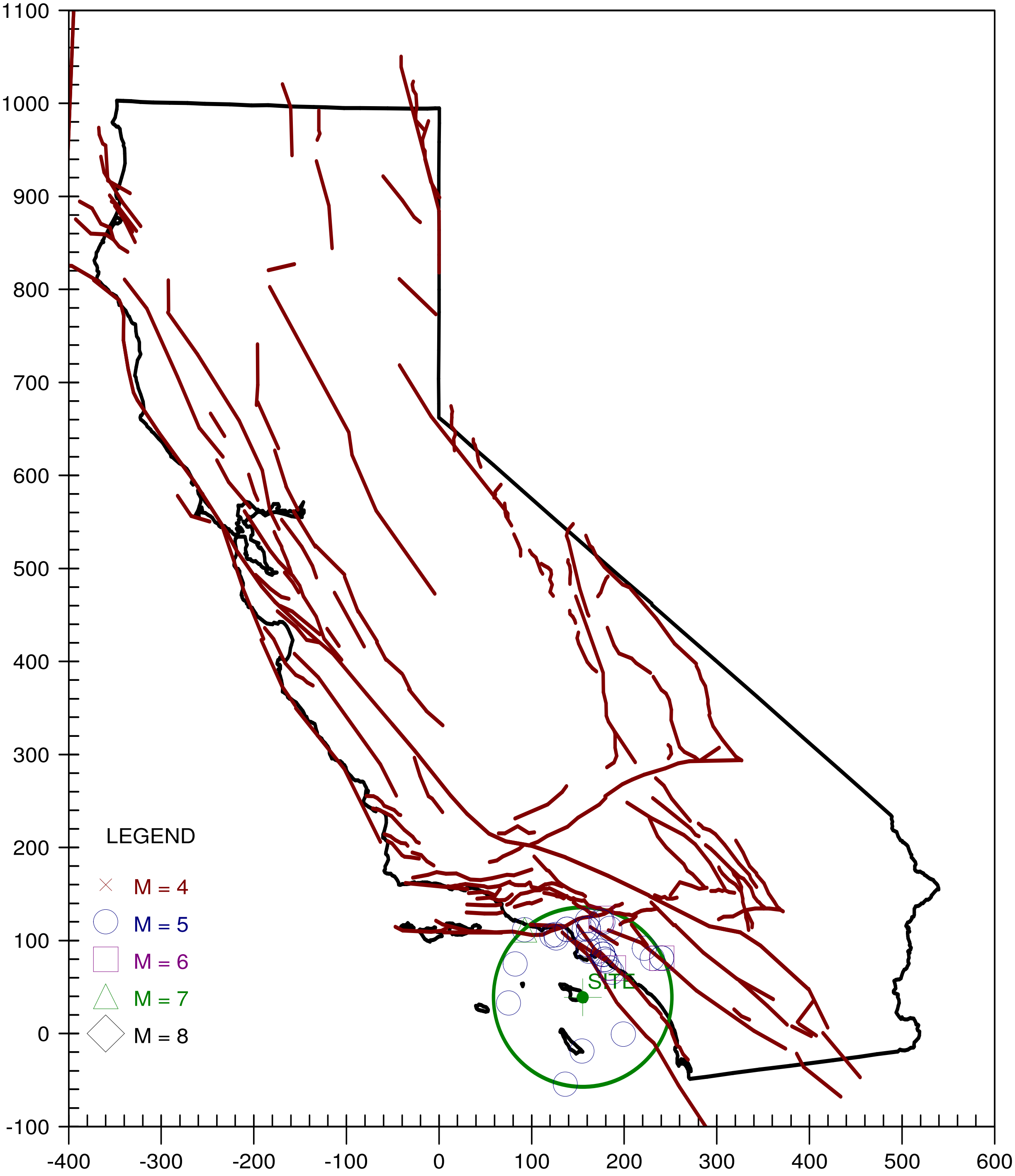
Tested By: Mathew F. Perry **Checked By:** ______________

APPENDIX D

Historical Earthquake Data

EARTHQUAKE EPICENTER MAP

Avalon K-12 School



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                           *************************
                           *                       *
                           *    E Q S E A R C H    *
                           *                       *
                           *     Version 3.00      *
                           *                       *
                           *************************

                                 ESTIMATION OF
                            PEAK ACCELERATION FROM
                        CALIFORNIA EARTHQUAKE CATALOGS


JOB NUMBER: 19-1126
                                                     DATE: 02-05-2020

JOB NAME: Avalon K-12 School

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT


MAGNITUDE RANGE:
   MINIMUM MAGNITUDE:  5.00
   MAXIMUM MAGNITUDE:  9.00

SITE COORDINATES:
   SITE LATITUDE:  33.3384
   SITE LONGITUDE:  118.3319

SEARCH DATES:
           START DATE:   1800
           END DATE:   2000

SEARCH RADIUS:
           60.0 mi
           96.6 km

ATTENUATION RELATION:  14) Campbell & Bozorgnia (1997 Rev.) - Alluvium
   UNCERTAINTY (M=Median, S=Sigma): M       Number of Sigmas:  0.0
   ASSUMED SOURCE TYPE:  DS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
   SCOND:   0  Depth Source:  A
   Basement Depth:  5.00 km     Campbell SSR:  0     Campbell SHR:  0
   COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km):  3.0
















                            -------------------------
                            EARTHQUAKE SEARCH RESULTS
                            -------------------------

Page  1
-------------------------------------------------------------------------------
```

```
    |       |        |          |   TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC)  |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
DMG |33.5750|117.9830|03/11/1933| 518 4.0|  0.0| 5.20| 0.028 |  V | 25.9( 41.7)
DMG |33.6170|118.0170|03/14/1933|19 150.0|  0.0| 5.10| 0.025 |  V | 26.4( 42.5)
DMG |33.6170|117.9670|03/11/1933| 154 7.8|  0.0| 6.30| 0.059 | VI | 28.5( 45.8)
DMG |33.6830|118.0500|03/11/1933| 658 3.0|  0.0| 5.50| 0.031 |  V | 28.8( 46.3)
DMG |33.7000|118.0670|03/11/1933| 51022.0|  0.0| 5.10| 0.022 | IV | 29.3( 47.1)
DMG |33.7000|118.0670|03/11/1933| 85457.0|  0.0| 5.10| 0.022 | IV | 29.3( 47.1)
DMG |33.7830|118.2500|11/14/1941| 84136.3|  0.0| 5.40| 0.025 |  V | 31.1( 50.0)
DMG |33.7500|118.0830|03/11/1933| 910 0.0|  0.0| 5.10| 0.019 | IV | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 230 0.0|  0.0| 5.10| 0.019 | IV | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 323 0.0|  0.0| 5.00| 0.018 | IV | 31.8( 51.2)
DMG |33.7500|118.0830|03/11/1933| 2 9 0.0|  0.0| 5.00| 0.018 | IV | 31.8( 51.2)
DMG |33.7500|118.0830|03/13/1933|131828.0|  0.0| 5.30| 0.023 | IV | 31.8( 51.2)
DMG |33.7830|118.1330|10/02/1933| 91017.6|  0.0| 5.40| 0.024 | IV | 32.8( 52.7)
DMG |33.8500|118.2670|03/11/1933|1425 0.0|  0.0| 5.00| 0.015 | IV | 35.5( 57.2)
DMG |32.8170|118.3500|12/26/1951| 04654.0|  0.0| 5.90| 0.031 |  V | 36.0( 58.0)
PAS |32.9710|117.8700|07/13/1986|1347 8.2|  6.0| 5.30| 0.018 | IV | 36.8( 59.3)
PAS |33.9190|118.6270|01/19/1989| 65328.8| 11.9| 5.00| 0.011 | III| 43.5( 70.0)
DMG |33.9500|118.6320|08/31/1930| 04036.0|  0.0| 5.20| 0.013 | III| 45.6( 73.4)
MGI |34.0000|118.3000|09/03/1905| 540 0.0|  0.0| 5.30| 0.014 | III| 45.7( 73.6)
T-A |34.0000|118.2500|01/10/1856| 0 0 0.0|  0.0| 5.00| 0.011 | III| 45.9( 73.9)
T-A |34.0000|118.2500|09/23/1827| 0 0 0.0|  0.0| 5.00| 0.011 | III| 45.9( 73.9)
T-A |34.0000|118.2500|03/26/1860| 0 0 0.0|  0.0| 5.00| 0.011 | III| 45.9( 73.9)
PAS |33.9440|118.6810|01/01/1979|231438.9| 11.3| 5.00| 0.010 | III| 46.4( 74.6)
DMG |34.0000|118.5000|08/04/1927|1224 0.0|  0.0| 5.00| 0.010 | III| 46.7( 75.1)
MGI |34.0000|118.5000|11/19/1918|2018 0.0|  0.0| 5.00| 0.010 | III| 46.7( 75.1)
MGI |34.0000|118.0000|12/25/1903|1745 0.0|  0.0| 5.00| 0.009 | III| 49.5( 79.7)
DMG |33.2910|119.1930|10/24/1969| 82912.1| 10.0| 5.10| 0.010 | III| 49.8( 80.1)
PAS |33.6710|119.1110|09/04/1981|155050.3|  5.0| 5.30| 0.012 | III| 50.4( 81.1)
MGI |34.0800|118.2600|07/16/1920|18 8 0.0|  0.0| 5.00| 0.009 | III| 51.4( 82.7)
PAS |34.0610|118.0790|10/01/1987|144220.0|  9.5| 5.90| 0.018 | IV | 52.0( 83.6)
PAS |34.0730|118.0980|10/04/1987|105938.2|  8.2| 5.30| 0.011 | III| 52.5( 84.4)
MGI |33.8000|117.6000|04/22/1918|2115 0.0|  0.0| 5.00| 0.009 | III| 52.8( 85.0)
DMG |33.6990|117.5110|05/31/1938| 83455.4| 10.0| 5.50| 0.013 | III| 53.4( 85.9)
MGI |34.1000|118.1000|07/11/1855| 415 0.0|  0.0| 6.30| 0.024 | IV | 54.2( 87.3)
DMG |33.7000|117.4000|04/11/1910| 757 0.0|  0.0| 5.00| 0.007 | II | 59.2( 95.2)
DMG |33.7000|117.4000|05/15/1910|1547 0.0|  0.0| 6.00| 0.017 | IV | 59.2( 95.2)
DMG |33.7000|117.4000|05/13/1910| 620 0.0|  0.0| 5.00| 0.007 | II | 59.2( 95.2)
DMG |32.5000|118.5500|02/24/1948| 81510.0|  0.0| 5.30| 0.009 | III| 59.2( 95.3)
DMG |34.0000|119.0000|09/24/1827| 4 0 0.0|  0.0| 7.00| 0.036 |  V | 59.7( 96.0)
MGI |34.0000|119.0000|12/14/1912| 0 0 0.0|  0.0| 5.70| 0.013 | III| 59.7( 96.0)

*******************************************************************************
-END OF SEARCH-   40 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH:   1800  TO  2000

LENGTH OF SEARCH TIME:   201  years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 25.9 MILES (41.7 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.059 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
  a-value=  1.617
  b-value=  0.530
  beta-value=  1.219


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TABLE OF MAGNITUDES AND EXCEEDANCES:
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  Earthquake | Number of Times | Cumulative
   Magnitude |    Exceeded     | No. / Year
  -----------+-----------------+------------
     4.0     |       40        |   0.19900
     4.5     |       40        |   0.19900
     5.0     |       40        |   0.19900
     5.5     |        9        |   0.04478
     6.0     |        4        |   0.01990
     6.5     |        1        |   0.00498
     7.0     |        1        |   0.00498
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KOURY
PASSION INTEGRITY QUALITY

THE KOURY DIFFERENCE

We are a key member of the construction team while safeguarding the public. We improve operational logistics and provide superior quality control through the continuing development of our engineering staff and technical expertise, utilization of classroom training and field supervisors, thus defining the industry standard.

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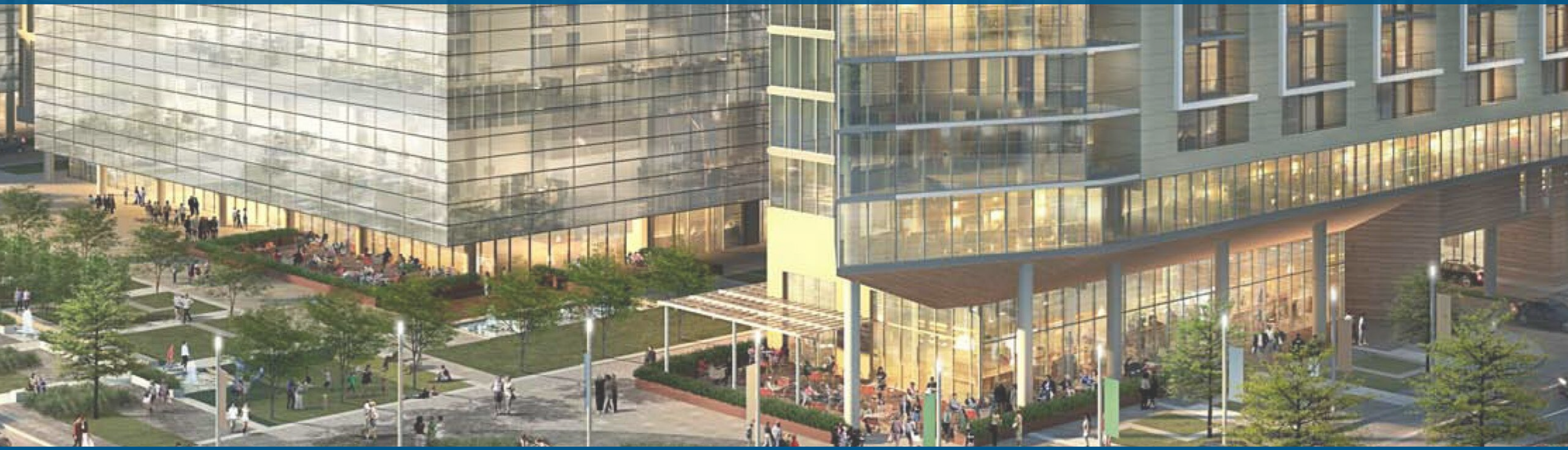

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