

Geotechnical Evaluation Landscape Maintenance Facility Irvine Great Park Irvine, California

City of Irvine

1 Civic Center Plaza | Irvine, California 92606

October 21, 2024 | Project No. 212085029



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

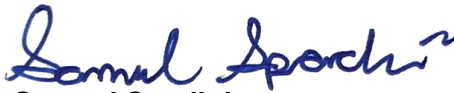
Geophysics | Engineering Geology | Laboratory Testing | Industrial Hygiene | Occupational Safety | Air Quality | GIS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

Geotechnical Evaluation
Landscape Maintenance Facility
Irvine Great Park
Irvine, California

Mr. Brian Polivka
City of Irvine
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1 INTRODUCTION

In accordance with your request and authorization, we have performed a geotechnical evaluation for the proposed Irvine Great Park (IGP) Landscape Maintenance Facility in Irvine, California. The purpose of our study was to conduct a geotechnical evaluation to develop geotechnical recommendations regarding the design and construction of the proposed improvements. This report presents our findings, conclusions, and recommendations based on our background review, site reconnaissance, subsurface evaluation, laboratory testing, and geotechnical analyses.

2 SCOPE OF SERVICES

Our scope of services included the following:

- Project coordination and planning, including scheduling the subsurface exploration and client meetings.
- Review of readily available background materials including published geologic and seismic hazards maps, stereoscopic aerial photographs, in-house information, and plans provided by the client.
- Site reconnaissance to observe the general site conditions and to locate and mark our exploratory borings for utility clearance.
- Coordination with Underground Service Alert to check underground utility conflicts.
- Subsurface exploration consisting of the drilling, logging, and sampling of three small-diameter hollow-stem auger (HSA) borings and one hand-auger boring to depths of up to approximately 41.5 feet below the ground surface. The borings were logged by a representative of our firm and bulk and relatively undisturbed soil samples were collected at selected depths for laboratory testing.
- Laboratory testing of selected soil samples including tests to evaluate in-situ moisture content and dry density, percentage of particles finer than the No. 200 sieve, expansion index, Proctor density, direct shear strength, R-Value, and soil corrosivity.
- Data compilation and engineering analysis of the information obtained from our background review, subsurface evaluation, and laboratory tests.
- Preparation of this geotechnical report presenting our findings, conclusions, and recommendations for the planned improvements at the site.

3 SITE DESCRIPTION

The approximately 10-acre project site is located centrally within the IGP in Irvine, California (Figure 1). A portion of the site is bounded by Great Park Boulevard to the east, Skyhawk to the south, Baseball Field 9 of the Sports Park to the north and Parking Lot 5 to the west. Site development activities completed prior to this study included placement of fills during grading for the IGP Sports Park, hardscape, landscaping consisting of low shrubs and trees, and utilities

consisting of storm drains and irrigation. Historical use of the property as the El Toro Marine Corps Air Base began in 1942 with various phases of expansion and upgrades to base facilities prior to its closure in 1999. Prior to 1942, the site was used for row crop agriculture. The elevation of the site ranges from approximately 299 feet above mean sea level (MSL) to the west and 311 feet above MSL to the east (DMC Engineering, 2024). An aerial image of the project area is shown on Figure 2.

4 PROJECT DESCRIPTION

Based on our understanding of the project and our review of the draft Grading Plans (DMC Engineering, 2024), development of the Landscape Maintenance Facility will include fills of up to approximately 5 feet and cuts of up to approximately 3 feet in order to create the pad elevation (Figure 3). The new structures will include an administration building, mechanic shop, maintenance facility, and storage. An exterior covered storage area is also planned. Other improvements are anticipated to include expanding portions of Parking Lot 5 to the east, asphalt drive isles and parking, gates and fences, light poles, electrical vehicle charging station, landscape materials storage, and associated utilities. Proposed grades vary from an approximate elevation of 311 feet MSL at the driveway entrance at Great Park Boulevard to an approximate elevation of 298 feet MSL adjacent to Parking Lot 5.

5 SUBSURFACE EVALUATION AND LABORATORY TESTING

Our subsurface exploration was performed on August 23, 2024 and consisted of drilling, logging, and sampling of three HSA borings using a truck-mounted drill rig to depths ranging from approximately 11.5 feet to 41.5 feet. One hand-auger boring was advanced to a depth of approximately 4 feet at the proposed east driveway entrance from Great Park Boulevard. The approximate locations of the exploratory borings are shown on Figures 2 and 3. The borings were logged by a representative from our firm. Bulk and relatively undisturbed soil samples were obtained at selected depths for laboratory testing. The logs of the exploratory borings are presented in Appendix A.

As a part of our current evaluation, we reviewed a geotechnical report for the proposed Orange County Great Park that was prepared by Lawson & Associates Geotechnical Consulting, Inc. (LGC) dated January 30, 2009. LGC performed a subsurface exploration that included a boring and a cone penetration test (CPT) sounding within the planned area of the IGP Landscape Maintenance Facility. The boring log was missing from the report. The CPT sounding (CPT-15) was performed to a depth of approximately 100 feet. The approximate location of the CPT is

shown on Figures 2 and 3, and the CPT log and seismic shear wave velocity measurements performed are included in Appendix B.

Laboratory testing was performed on representative samples from our borings to evaluate in-situ moisture content and dry density, percentage of particles finer than the No. 200 sieve, expansion index, Proctor density, direct shear strength, R-Value, and soil corrosivity. The results of the in-situ moisture content and dry density tests are presented on the boring logs in Appendix A and the laboratory testing results are presented in Appendix C.

6 GEOLOGY AND SUBSURFACE CONDITIONS

6.1 Regional Geology

The project site is located on the coastal plain of the Peninsular Ranges Geomorphic Province of southern California. This geomorphic province encompasses an area that extends approximately 125 miles from the Transverse Ranges and the Los Angeles Basin south to the Mexican border, and beyond another approximately 775 miles to the tip of Baja California. The Peninsular Ranges province varies in width from approximately 30 to 100 miles and is characterized by northwest-trending mountain range blocks separated by similarly northwest-trending faults (Norris and Webb, 1990).

Active northwest-trending fault zones in the Peninsular Ranges province include the Elsinore fault zone, San Jacinto fault zone, and the Newport-Inglewood fault zone. The active San Andreas fault zone is located northeast of the province within the adjacent 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).

6.2 Site Geology

Review of referenced geologic maps (Morton and Miller, 2006) indicates that the site is underlain by Holocene- to early Pleistocene-age alluvial fan deposits consisting of unconsolidated to moderately consolidated silt, sand, pebbly-cobbly sand and moderately to well consolidated silt, sand, gravel, and cobbles (Figure 4).

Materials encountered during our subsurface exploration consisted of fill and alluvial soils and are summarized in the following sections. More detailed descriptions of the subsurface materials are presented on the boring logs in Appendix A.

6.2.1 Fill

Fill soils were encountered in our exploratory borings to depths of up to approximately 10 feet below the ground surface and generally consisted of medium dense clayey sand and hard sandy lean clay. Varying amounts of gravel were encountered in fill soils. It is important to note that the fill encountered in our borings is anticipated to be engineered fill placed as a part of grading for the IGP Sports Park. A compaction report has not been provided for our review at the time of preparation of this report. Deeper fill materials may be present due to the previous construction activities at the site.

6.2.2 Alluvium

Alluvial soils were encountered beneath the fill in our exploratory borings to the total explored depths of up to approximately 41.5 feet. Alluvium generally consisted of loose to medium dense clayey sand, silty sand, poorly graded sand with silt, poorly graded sand with clay, and very stiff to hard lean clay and sandy lean clay.

6.3 Groundwater

Groundwater was not observed at the time of drilling in our exploratory borings that were drilled to depths of up to approximately 41.5 feet. A review of nearby boring logs submitted to the State Water Resource Control Board's GeoTracker website indicated that groundwater was recorded at approximately 145 feet below the ground surface to the west of the project site. In addition, a review of groundwater elevation contours prepared by the Orange County Water District indicate groundwater elevations between approximately 140 and 160 feet MSL (between approximately 138 and 171 feet below the ground surface) within the vicinity of the site. It should be noted that fluctuations in groundwater levels may occur due to variations in ground surface topography, subsurface stratification, precipitation, irrigation, groundwater pumping, and other factors which may not have been evident at the time of our field evaluation.

7 FAULTING AND SEISMICITY

The subject site is not located within a State of California Earthquake Fault Zone, formerly known as the Alquist-Priolo Special Studies Zone (Hart & Bryant, 2007). 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 to be significant during the design life of the proposed amphitheater improvements. Figure 5 shows the approximate site location relative to the major faults in the region. The nearest active fault is the San Joaquin fault, located approximately 1.9 miles southwest of the site (United States Geological Survey [USGS], 2008).

The principal seismic hazards evaluated at the subject site are surface fault rupture, ground motion, liquefaction, and seismically induced landslide. A brief description of these hazards and the potential for their occurrences on site are discussed in the following sections.

7.1 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 fault rupture is considered to be low. However, lurching or cracking of the ground surface as a result of nearby seismic events is possible.

7.2 Site-Specific Ground Motion

Considering the proximity of the site to active faults capable of producing a maximum moment magnitude of 6.0 or more, the project area has a high potential for experiencing strong ground motion. The 2022 California Building Code (CBC) specifies that the risk-targeted maximum considered earthquake (MCE_R) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. Based on the shear wave velocity measurements performed at CPT-15 (LGC, 2009), we calculated that the average shear wave velocity in the upper 100 feet (30 meters) of the subsurface profile (V_{s30}) is approximately 991 feet per second (ft/s) (i.e., 302 meters per second [m/s]). In accordance with Chapter 20 of the American Society of Civil Engineers (ASCE) Publication 7-16 (2016) for the Minimum Design Loads and Associated Criteria for Building and Other Structures, the site classification is Site Class D.

Per the 2022 CBC, a site-specific ground motion hazard analysis shall be performed in accordance with Section 21.2 of ASCE 7-16 for structures on Site Class D with a mapped MCE_R , 5 percent damped, spectral response acceleration parameter at a period of 1 second (S_1) greater than or equal to 0.2g. We calculated that the S_1 for the site is equal to 0.445g using the 2024 Applied Technology Council (ATC) seismic design tool (web-based); therefore, a site-specific ground motion hazard analysis was performed for the project area.

The site-specific ground motion hazard analysis consisted of the review of available seismologic information for nearby faults and performance of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA) to develop acceleration response spectrum curves corresponding to the MCE_R for 5 percent damping. Prior to the site-specific ground motion hazard analysis, we obtained the mapped seismic ground motion values and developed the mapped MCE_R response spectrum for 5 percent damping in accordance with Section 11.4 of ASCE 7-16 using the 2024 ATC seismic design tool. The depths to $V_s = 1,000$ m/s and $V_s =$

2,500 m/s are assumed to be 850 meters and 1,350 meters, respectively. These values were evaluated using the Open Seismic Hazard Analysis (OpenSHA) software developed by USGS (2021).

The 2014 new generation attenuation (NGA) West-2 relationships were used to evaluate the site-specific ground motions. The NGA relationships that we used for developing the probabilistic and deterministic response spectra are by Chiou and Youngs (2014), Campbell and Bozorgnia (2014), Boore, Stewart, Seyhan, and Atkinson (2014), and Abrahamson, Silva, and Kamai (2014). The OpenSHA software (USGS, 2021) was used for performing the PSHA. The Calculation of Weighted Average 2014 NGA Models spreadsheet by the Pacific Earthquake Engineering Research Center (PEER) was used for performing the DSHA (Seyhan, 2014).

PSHA was performed for earthquake hazards having a 2 percent chance of being exceeded in 50 years multiplied by the risk coefficients per Section 21.2.1.1 of ASCE 7-16. The maximum rotated components of ground motions were considered in PSHA with 5 percent damping. For the DSHA, we analyzed accelerations from characteristic earthquakes on active faults within the region using the hazard curves and deaggregation plots at the site obtained from the USGS Unified Hazard Tool application (USGS, 2024). A magnitude 7.2 event on the San Joaquin Hills fault with a rupture distance of 3.1 kilometers (1.9 miles) from the site was evaluated to be the controlling earthquake. Hence, the DSHA was performed for the site using this event and corrections were made to the spectral accelerations for the 84th percentile of the maximum rotated component of ground motion with 5 percent damping.

The site-specific MCE_R response spectrum was taken as the lesser of the spectral response acceleration at any period from the PSHA and DSHA, and the site-specific general response spectrum was determined by taking two-thirds of the MCE_R response spectrum with some conditions in accordance with Section 21.3 of ASCE 7-16. Figure 6 presents the site-specific MCE_R response spectrum and the site-specific design response spectrum. The mapped design response spectrum calculated in accordance with Section 11.4 of ASCE 7-16 is also presented on Figure 6 for comparison. The site-specific mapped spectral response acceleration parameters, consistent with the 2022 CBC, are provided in Section 9.2 for the evaluation of seismic loads on buildings and other structures.

7.3 Liquefaction Potential

Liquefaction is the phenomenon in which loosely deposited granular soils and non-plastic silts located below the water table undergo rapid loss of shear strength when subjected to strong earthquake-induced ground shaking. Ground shaking of sufficient duration results in the loss of

grain-to-grain contact due to a rapid rise in pore water pressure, and causes the soil to behave as a fluid for a short period of time. Liquefaction is known generally to occur in saturated or near-saturated cohesionless soils at depths shallower than 50 feet below the ground surface. Liquefaction is also known to occur in relatively fine-grained soils (i.e., sandy silt and clayey silt) with a plasticity index (PI) of less than 7. 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.

According to the State of California Seismic Hazards Zones map (California Department of Conservation, Division of Mines and Geology [CDMG], 2001) the site is not located in an area mapped as a potential liquefaction hazard zone as groundwater elevations are historically greater than 50 feet below the ground surface. Groundwater was not encountered in our exploratory borings up to a depth of 41.5 feet. A review of recent groundwater elevation maps by the Orange County Water District indicate that the groundwater elevation is greater than 50 feet below the proposed lowest surface of the Landscape Maintenance Facility. Accordingly, it is our opinion that liquefaction is not a design consideration for the project.

7.4 Landslides

Landslides may be induced by strong vibratory motion produced by earthquakes. Research and historical data indicate that seismically induced landslides tend to occur in weak soil and rock on sloping terrain. The process for zoning earthquake-induced landslides incorporates expected future earthquake shaking, existing landslide features, slope gradient and strength of earth materials on the slope.

The site does not have significant slopes and the site is not located in an area mapped as a potential seismically induced landslide hazard zone (CDMG, 2001). Based on our review of the relevant geologic maps and observation of the site conditions, landslide hazards are not a design consideration for the project provided that the new slopes are constructed in accordance with the recommendations presented in this report.

8 CONCLUSIONS

The purpose of our study was to provide an evaluation of the site with regard to the geotechnical aspects of the future development of the IGP Landscape Maintenance Facility to support the site plan design. Based on the results of our geotechnical evaluation, it is our opinion that the proposed improvements are feasible from a geotechnical perspective. Our findings and conclusions pertaining to the geotechnical aspects of the property are presented below.

- The site is underlain by fill soils and alluvial deposits. Fill soils were encountered in our exploratory borings up to a depth of approximately 10 feet below the ground surface. Fill generally consisted of medium dense clayey sand and hard sandy lean clay. Varying amounts of gravel were encountered in the fill soils. Alluvium generally consisted of loose to medium dense clayey sand, silty sand, poorly graded sand with silt, poorly graded sand with clay, and very stiff to hard lean clay and sandy lean clay.
- Near-surface soils are potentially compressible under loads from future fills or structures. Remedial grading will be needed to partly remove and recompact existing engineered fill in order to provide suitable support for the new fill, the proposed buildings, asphalt paved areas, and other structural improvements.
- Excavations in alluvium and fill materials should be feasible with standard earthmoving equipment in good working condition. Caving should be anticipated in granular soils with low cohesion.
- Our limited laboratory testing of the expansion index of near-surface soils indicates that the near-surface site soils possess a very low to low expansion potential.
- In general, the on-site soils should be suitable for reuse as compacted fill, provided that these meet the criteria provided in the Fill Material section of this report.
- Groundwater was not encountered in our exploratory borings up to a depth of approximately 41.5 feet and, based on our review of the pertinent documents, groundwater is located at depths greater than 50 feet below the ground surface. However, depth to groundwater may vary due to seasonal precipitation, subsurface conditions, irrigation, groundwater pumping, and other factors.
- There are no known active faults crossing the site, and the potential for surface ground rupture is considered to be low.
- Seismically induced liquefaction and landslides are not significant design considerations for the project site.
- Our limited laboratory corrosion testing indicates that the near-surface site soils can be classified as non-corrosive based on the California Department of Transportation (Caltrans, 2021) corrosion guidelines.

9 RECOMMENDATIONS

The recommendations presented in the following sections provide general geotechnical criteria regarding the design and construction of the proposed improvements that may be used during the current plan phase of the project. The recommendations are based on the results of our subsurface evaluation, laboratory testing, review of referenced geologic materials, experience in the general vicinity of the project area, and geotechnical analyses. The proposed work should be performed in conformance with the recommendations presented in this report, project specifications, and appropriate agency standards.

9.1 Earthwork

Based on our understanding of the project, following the demolition of the existing site improvements, the earthwork at the site is expected to consist of cut and fill grade to achieve the design grades for the building pad, parking lot, and roadways. Following rough grading, excavations for building foundations and foundations for other structures, and new utilities, are anticipated.

9.1.1 Pre-Construction Conference

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

9.1.2 Clearing and Site Preparation

Prior to performing excavations or other earthwork, the area should be cleared of existing landscape vegetation, improvements, utilities, surface obstructions, and other deleterious materials. Existing utilities within the project limits should be re-routed or protected from damage by construction activities. Materials generated from the clearing operations should be removed from the project site and disposed of at a legal dumpsite. Obstructions that extend below finished grade should be removed and replaced with compacted fill.

9.1.3 Excavation Characteristics

Based on our exploratory borings, we anticipate that excavation within the fill and alluvial materials on site may generally be accomplished with standard earthmoving equipment in good operating condition. Caving should also be anticipated in granular soils with low cohesion. Contractors should make their own independent evaluation of the excavatability of the on-site materials prior to submitting their bids.

9.1.4 Treatment of Near-Surface Soils

Remedial grading will be appropriate to prepare the ground surface for new building structure, new roadways, and other structures, such as landscape structures that will include removal of existing surficial fill to expose competent engineered fill soils. Remedial grading is not needed for structures supported on drilled pier foundations, such as light poles designed in accordance with the recommendations in this report.

In order to provide suitable support for the planned improvements, we recommend that the existing engineered fill that is dry and desiccated, and loose/soft native alluvial soils within the influence zone of the new improvements be removed to expose competent engineered

fills or native soils to the depths presented below. Based on our site observations and subsurface exploration, remedial grading up to a depth of approximately 1 foot is anticipated to remove the existing weathered fill. However, there may be deeper weathered fills in other areas that were not explored.

The limits of the remedial excavation for new structures should extend laterally so that the bottom of the excavation is approximately 5 feet beyond the outside edge of the building's footprint or structure foundation or a distance corresponding to the depth of the remedial excavation, whichever is farther. The excavation bottom should be approximately 2 feet below the deepest footing and be evaluated by our representative during the excavation work. Additional excavation of loose, soft, and/or wet areas may be appropriate, depending on our observations during construction. Prior to placement of new compacted fill, the exposed bottom should be scarified, moisture-conditioned to slightly above the laboratory optimum moisture content, and recompact to a depth of approximately 8 inches.

For the proposed landscape maintenance facility drive aisles, parking, and Parking Lot 5 expansion, we recommend that the existing weathered fill be removed to expose competent fill soils prior to placing new fill and/or aggregate base. A remedial grading depth of approximately 1 foot to expose relatively firm soil should be considered during project planning for preparing areas to receive fill. The lateral limits of the remedial grading should be extended to distance of approximately 2 feet beyond the edges of any sidewalks that will be constructed along the paved areas. Prior to placement of new compacted fill and/or aggregate base, the exposed bottom should be scarified, moisture-conditioned to slightly above the optimum moisture content, and recompact to a depth of approximately 8 inches.

In non-structural areas, some shallow remedial grading to remove organic materials and highly disturbed soils is recommended prior to placing fill to achieve future design grades. A remedial grading depth of approximately 1 to 2 feet to expose relatively firm soil should be considered during project planning for preparing areas to receive fill in non-structural areas. Prior to placement of new compacted fill, the exposed bottom should be scarified, moisture-conditioned to slightly above the optimum moisture content, and recompact to a depth of approximately 8 inches.

9.1.5 Slopes

We recommend that cut slopes and new fill slopes, such as the planned fill slope adjacent to Baseball Field 9, be constructed at slope ratios of 2:1 (horizontal to vertical) or flatter. Slopes constructed at 2:1 (horizontal to vertical) or flatter are anticipated to be stable. New fill slopes should be constructed with a keyway that is approximately 15 feet wide and should extend 2

feet or more into competent engineered fill or alluvial soils (Figure 7). Compaction of fill slopes should be performed to achieve the recommended degree of compaction to the face of the slope. This may be achieved by over-building the slope face and cutting it back to the planned finish grade. Alternatively, sheepsfoot rollers and track walking the slope may be performed for compacting slopes.

9.1.6 Temporary Excavations

We recommend that trenches and excavations be designed and constructed in accordance with the Occupational Safety and Health Administration (OSHA) regulations. These regulations provide trench sloping and shoring design parameters for excavations up to 20 feet deep based on the soil types encountered. Excavations should be designed by the contractor's engineer based on site-specific geotechnical analyses. For planning purposes, we recommend that on-site soils be considered as OSHA Type C soil.

It is our opinion that temporary slopes in site soils should be stable at inclinations of approximately 1.5:1 (horizontal to vertical) or flatter. Some surficial sloughing may occur, especially if seepage zones are encountered. Temporary slopes should be evaluated in the field in accordance with the OSHA criteria. Where temporary excavations cannot be sloped as indicated above, temporary shoring may be appropriate for the excavations.

For preliminary design purposes, the lateral earth pressure values presented on Figures 8 and 9 for temporary cantilevered and braced shoring, respectively, may be considered. The recommended design earth pressures are based on the assumption that the shoring system will be constructed without raising the ground surface elevation behind the shored sidewalls of the excavation, that there will be no surcharge loads such as soil stockpiles and construction materials, and that no loads will act above a 1:1 (horizontal to vertical) plane ascending from the base of the shoring system. For a shoring system subjected to the above-mentioned surcharge loads, the contractor should include the effect of these loads on the lateral earth pressures acting on the shored walls. The final shoring system design should be performed by the contractor's shoring engineer.

If there are existing structures or improvements that will remain in-place, care should be taken by the contractor to avoid undermining adjacent existing foundations and improvements. New excavations should not extend within the "zone of influence" of existing foundations, which is defined as a 1:1 (horizontal to vertical) plane projecting downward from the outside edge of the adjacent footing at a point 6 inches above the bottom of the footing. In the event that excavations will extend within the "zone of influence" of existing foundations, our office should

be notified and appropriate recommendations provided, such as temporary underpinning of impacted foundations and/or temporary shoring.

9.1.7 Construction Dewatering

Groundwater was not encountered in our exploratory borings at the time of drilling, and groundwater is not anticipated to impact the project. However, seepage may be encountered as a result of variations in seasonal precipitation, irrigation, leaking pipes, and variable soil conditions. The contractor should be prepared to take appropriate measures in the event that seepage is encountered during excavation operations. If seepage is encountered, disposal of groundwater should be performed in accordance with the guidelines of the Regional Water Quality Control Board.

9.1.8 Excavation Bottom Stability

In general, we anticipate that excavation bottoms in the engineered fill and alluvium will be relatively stable and should provide suitable support. Although not anticipated, excavations that encounter seepage or perched groundwater may be unstable. In general, unstable bottom conditions may be mitigated by excavation and replacement with compacted crushed aggregate or compacted fill beneath the bottom of the excavation to thicknesses of approximately 1 to 2 feet. If open-graded gravel is used for bottom stabilization, we recommend that the crushed rock be wrapped in filter fabric. Recommendations for stabilizing excavation bottoms should be based on evaluation in the field by the geotechnical consultant at the time of construction.

9.1.9 Fill Material

In general, the on-site soil should be suitable for reuse as fill and trench backfill provided those are free of trash, debris, roots, vegetation, or other deleterious materials. Fill should generally be free of rocks or lumps of material in excess of 4 inches in diameter. Rocks or hard lumps larger than approximately 4 inches in diameter should be broken into smaller pieces or should be removed from the site. Soils classified as lean and fat clay (CL and CH) are not suitable as fill materials in the structural areas. Fill used as structural backfill, should be comprised of granular, non-expansive soil that conforms to the “Greenbook” Standard Specifications for Public Works Construction (Public Works Standard, Inc., 2024) for structural backfill. “Non-expansive” is defined as soil having an expansion index (EI) of 20 or less in accordance with the ASTM International (ASTM) test method D 4829 (CBC, 2022).

Imported materials should consist of clean, non-expansive granular materials that conform to the Greenbook. The imported materials should satisfy the Caltrans (Caltrans, 2021) criteria

for non-corrosive soils (i.e., soils having a chloride concentration of less than 500 parts per million [ppm], a soluble sulfate content of less than 0.15 percent [1,500 ppm], a pH value higher than 5.5, and a minimum electrical resistivity higher than 1,500 ohm-centimeters [ohm-cm]). Materials for use as fill should be evaluated by the geotechnical consultant prior to importing. The contractor should be responsible for the uniformity of imported materials brought to the site.

9.1.10 Fill Placement and Compaction

Fill and trench backfill should be compacted in uniform horizontal lifts to a relative compaction of 90 percent or more as evaluated by ASTM D 1557 and in accordance with City of Irvine guidelines. Fill soils should be moisture-conditioned to slightly above the optimum moisture content. The optimum lift thickness of fill will depend on the type of compaction equipment used but generally should not exceed 8 inches in loose thickness. Special care should be taken to avoid pipe damage when compacting trench backfill above pipes. Placement and compaction of the fill soils should be in general accordance with the appropriate governing agencies and good construction practices.

9.2 Site-Specific Seismic Design Considerations

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 1 presents the site-specific mapped spectral response acceleration parameters in accordance with the 2022 CBC guidelines.

Table 1 – 2022 California Building Code Seismic Design Criteria	
Site-Specific Spectral Response Acceleration Parameters	Values
Site Classification	D
Mapped MCE _R Spectral Response Acceleration at Short Periods, S _s	1.244g
Mapped MCE _R Spectral Response Acceleration at 1.0-Second Period, S ₁	0.445g
MCE _R Spectral Response Acceleration at Short Periods Adjusted for Site Class, S _{MS}	1.247g
MCE _R Spectral Response Acceleration at 1.0-Second Period Adjusted for Site Class, S _{M1}	0.825g
Design Spectral Response Acceleration at Short Periods, S _{DS}	0.831g
Design Spectral Response Acceleration at 1.0-Second Period, S _{D1}	0.550g
Maximum Considered Earthquake Geometric Mean (MCE _G) Peak Ground Acceleration, PGA _M	0.570g

9.3 Building Foundations

Foundations should be designed in accordance with structural considerations and our geotechnical recommendations. In addition, requirements of the governing jurisdictions, and applicable building codes should be considered in the design of the structures. The following sections present our foundation recommendations.

9.3.1 Spread Footings

Isolated and continuous spread footings should have a width of 2 feet or more and extend to a depth of 2 feet below the adjacent finished grade. Footings should bear on compacted fill prepared per the recommendations provided in the Earthwork section of this report. Footings should be reinforced and detailed in accordance with the recommendations of the structural engineer.

Spread footings, as described above and bearing on compacted fill, may be designed using a net allowable bearing capacity of 2,500 pounds per square foot (psf). The allowable bearing capacity may be increased by 250 psf for each additional foot of depth and width, respectively, up to a value of 3,000 psf. The net allowable bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic forces. Total and differential settlements for footings designed in accordance with the above recommendations are estimated to be on the order of 1 inch and 0.5 inch over a horizontal span of 40 feet, respectively.

Footings bearing on compacted fill may be designed using a coefficient of friction of 0.35, where the total frictional resistance equals the coefficient of friction times the dead load. The footings may be designed using a passive resistance value of 350 psf per foot of depth up to a value of 3,500 psf. The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided that the passive resistance does not exceed one-half of the total allowable resistance. The passive resistance may be increased by one-third when considering loads of short duration such as wind or seismic forces.

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

9.3.2 Slabs-On-Grade

Building floor slabs should be designed by the project structural engineer based on the anticipated loading conditions. We recommend that floor slabs have a thickness of 5 inches or more and be reinforced in accordance with the recommendations of the structural engineer. The placement of reinforcements in the slab is vital for satisfactory performance. The floor slab and foundations should be tied together by extending the slab reinforcement into the footings. The slab should be underlain by a polyethylene vapor retarder, 10-mil or

thicker. The vapor retarder should further be underlain by a 4-inch-thick layer of sand or gravel with a particle size of approximately 0.4 inch or smaller. The vapor retarder is recommended in areas where moisture-sensitive floor coverings are anticipated. Soils underlying the slabs should be moisture-conditioned and compacted in accordance with the recommendations presented in this report prior to concrete placement. Joints should be constructed at intervals designed by the structural engineer to help reduce random cracking of the slab.

9.4 Hardscape

We recommend that new exterior concrete sidewalks and flatwork (hardscape) be constructed in accordance with the City of Irvine standards and have a minimum thickness of 4 inches. Concrete sidewalks and flatwork that anticipate vehicle loading should be reinforced per the recommendations of the structural engineer. The hardscape should be underlain by 4 inches of granular material such as Crushed Aggregate Base or Crushed Miscellaneous Base and installed with crack-control joints at an appropriate spacing as designed by the structural engineer to reduce the potential for shrinkage cracking. Positive drainage should be established and maintained adjacent to flatwork. To reduce the potential for differential offset, joints between the new hardscape and adjacent curbs, existing hardscape, building walls, and/or other structures, and between sections of new hardscape, should be doweled.

9.5 Retaining Walls

Retaining walls may be supported by spread footings founded on compacted fill designed in accordance with the recommendations presented in Section 9.3.1. Recommendations for lateral earth pressures to be used in design of the yielding and restrained retaining walls are provided on Figures 10 and 11, respectively. Lateral soil resistance may be obtained using a passive resistance of 350 psf per foot of depth for level ground conditions. The passive resistance may be increased by one-third when considering loads of short duration, including wind and seismic loads. Further, for sliding resistance, a friction coefficient of 0.35 may be used for the concrete and soil interface. The allowable resistance may be taken as the sum of the frictional and passive resistance provided that the passive resistance does not exceed one-half of the total allowable resistance.

Retaining walls should be backfilled with granular, low expansion potential soil. Measures should be taken to reduce the potential for build-up of moisture behind the retaining walls. Drainage design should include free-draining backfill materials and perforated drains as depicted on Figure 12.

9.6 Pole Foundations

Light poles typically impose relatively light axial loads on reinforced concrete pile foundations and the pile dimensions are generally controlled by the lateral load demand. We recommend that the pile foundations for light poles have a diameter of 18 inches or more and an embedment depth of 8 feet or more. The piles may be designed for an allowable side friction of 160 psf for downward axial loads (i.e., in compression) and 100 psf for upward axial loads (i.e., in tension). In addition, an allowable soil lateral bearing pressure of 150 psf per foot of depth can be used for evaluating the lateral pile capacity. Due to the soil arching effect, the lateral bearing pressure may be applied on an area with a width that is two times the diameter of the pile. increase to twice the value. The allowable bearing values may be increased by one-third when considering loading of short duration such as wind or seismic forces. The pile foundation design parameters provided here assume that the piles will have a center-to-center spacing of three times the pile diameter or more. The pile dimensions (i.e., diameter and design embedment) should be evaluated by the project structural engineer.

The bottoms of the drilled holes for piles should be cleaned of loose materials prior to placing steel and pouring concrete. The pile foundations should be installed within specified limits of vertical and horizontal alignment and should not exceed a batter of 2 percent over its embedment depth. Further, the top of a pile should be within 3 inches of the surveyed location.

9.7 Underground Utilities

We anticipate that utility pipelines will be supported on alluvial deposits or compacted fill. The depths of the pipelines are not known; however, we anticipate that the pipe invert depths will not exceed 10 feet. Trenches should not be excavated parallel to building footings. If needed, trenches can be excavated adjacent to a continuous footing, provided that the bottom of the trench is located above a 1:1 (horizontal to vertical) plane projected downward from the outside edge of the adjacent footing at a point 6 inches above the bottom of the footing. Utility lines that cross beneath footings should be encased in concrete below the footing.

9.7.1 Pipe Bedding

We recommend that pipelines be supported on 4 inches or more of granular bedding material. Bedding material should be placed around pipe zones to 1 foot or more above the top of the pipe. The bedding material should be classified as sand, be free of organic material, and have a sand equivalent (SE) of 30 or more. We do not recommend that gravel be used for bedding material because of the nature of the subsurface material. It has been our experience

that the voids within the gravel are sufficiently large to allow fines to migrate into the voids, thereby creating the potential for sinkholes and depressions to develop at the ground surface.

Special care should be taken not to allow voids beneath and around the pipe. Compaction of the bedding material and backfill should proceed along both sides of the pipe concurrently. Trench backfill, including bedding material, should be placed and compacted with mechanical equipment in accordance with the recommendations presented in the following section. Jetting should not be allowed.

9.7.2 Trench Backfill

Based on our subsurface evaluation, the on-site soils should generally be suitable for re-use as trench backfill provided that those are free of trash, debris, roots, vegetation, contaminated material, deleterious materials and rock/lumps of material larger than approximately 4 inches in diameter. We recommend that trench backfilling be in general conformance with the Greenbook. Fill should be moisture-conditioned to slightly above the optimum moisture content. Wet soils should be allowed to dry to slightly above the optimum moisture content prior to their placement as trench backfill. Trench backfill should be compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557. Lift thickness for backfill will depend on the type of compaction equipment utilized, but fill should generally be placed in horizontal lifts not exceeding 8 inches in loose thickness. Special care should be exercised to avoid damaging the pipe during compaction of the backfill.

9.7.3 Modulus of Soil Reaction

The modulus of soil reaction is used to characterize the stiffness of soil backfill placed along the sides of buried flexible pipelines for the purpose of evaluating deflection caused by the weight of the backfill above the pipe. We recommend that a modulus of soil reaction of 1000 pounds per square inch (psi) be used for design provided that granular bedding material be placed adjacent to the pipe, as recommended above.

9.8 Preliminary Pavement Design

We anticipate that future new parking lots and access roads will include new flexible pavement. Traffic loading information was not available for our design at the time of preparation of this report. For planning purposes, we have assumed traffic index (TI) values of 5.0 and 6.0 for the preliminary design of pavement sections. The TI of 5.0 is generally used for designing parking stall and/or driveway pavements subjected to relatively light passenger vehicles. The TI of 6.0 is generally used for designing pavements in driveway areas that are subjected to relatively light passenger

vehicles and periodic heavy equipment/truck traffic. Laboratory testing was performed on a representative subgrade soil sample and indicated an R-value of 48.

Based on the TIs described above, a design R-value of 40 (in anticipation of the pavements underlain by soil subgrade having more fines content than that was tested), and guidelines of the Caltrans Highway Design Manual (Caltrans, 2020b), we have developed the following preliminary pavement sections for the project (Table 2). We recommend that these pavement sections be re-evaluated once project-specific TIs are developed and the as-graded near-surface earth materials are further tested.

Table 2 – Preliminary Flexible Pavement Structural Sections			
Traffic Index	AC/CAB or CMB (inches)	Full Depth AC (inches)	PCC (inches)
5.0	3.0/5.0	5.5	6.0
6.0	3.0/6.0	6.5	7.0
Notes: AC – Asphalt Concrete CAB – Crushed Aggregate Base CMB – Crushed Miscellaneous Base PCC – Portland Cement Concrete with a 28-day compressive strength of 2,500 psi			

Subgrade soils in areas to be paved should be prepared as recommended in the Earthwork section of this report. Expansive clayey soils, if encountered during grading, should not be placed within the upper 12 inches of the subgrade to reduce the potential for pavement damage. CAB or CMB materials should conform to the Greenbook, Section 200. The CAB/CMB should be compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557. CAB/CMB should be placed at slightly above the optimum moisture content as evaluated by ASTM D 1557. AC should conform to Section 203 of the Greenbook and should be compacted to a relative compaction of 95 percent. Final pavement sections should be selected based on the actual anticipated traffic loading conditions and evaluation of the subgrade materials at the time of construction.

9.9 Corrosivity

The corrosion potential of the site soils was evaluated using the results of a selected representative sample obtained from the exploratory boring. Laboratory testing was performed to evaluate pH, minimum electrical resistivity, soluble sulfate, and chloride content. Soluble sulfate content is addressed in the following section of this report. The soil pH and minimum resistivity tests were performed in accordance with California Test (CT) method 643. The test for chloride content of the soils was performed using CT 422. Sulfate testing was performed in general accordance with CT 417. The laboratory test results are presented in Appendix C.

The pH of the tested sample was 6.9. The electrical resistivity was 1,789 ohm-cm. The chloride content was 65 ppm, and the sulfate content was 20 ppm (i.e., 0.002 percent). Based on the laboratory test results and Caltrans criteria (2021), the project site can be classified as non-corrosive site, which is defined as having earth materials with a pH of 5.5 or more, chloride concentrations of 500 ppm or less, sulfate concentration of 0.15 percent (i.e., 1,500 ppm) or less, or an electrical resistivity of 1,500 ohm-centimeters or more. We recommend that a corrosion engineer be consulted for further evaluation and recommendations.

9.10 Concrete

Concrete in contact with soil or water that contains high concentrations of soluble sulfates can be subject to chemical deterioration. Based on the American Concrete Institute (ACI, 2022) criteria, the potential for sulfate attack is negligible for water-soluble sulfate contents in soil ranging from 0.00 to 0.10 percent by weight and moderate for water-soluble sulfate contents ranging from 0.10 to 0.20 percent by weight. The potential for sulfate attack is severe for water-soluble sulfate contents ranging from 0.20 to 2.00 percent by weight and very severe for water-soluble sulfate contents over 2.00 percent by weight. Laboratory testing indicated the sulfate content of the sample was 0.002 percent, representing a negligible potential for sulfate attack (ACI, 2022). However due to the potential for variability of site soils, Type II/V cement should be considered for concrete construction. The concrete should have a water-cement ratio no higher than 0.50 by weight for normal weight aggregate concrete.

In order to reduce the potential for shrinkage cracks in the concrete during curing, we recommend that the concrete for the proposed structures be placed with a slump of 4 inches based on ASTM C 143. The slump should be checked periodically at the site prior to concrete placement. We also recommend that crack control joints be provided in slabs in accordance with the recommendations of the structural engineer to reduce the potential for distress due to minor soil movement and concrete shrinkage. We further recommend that concrete cover over reinforcing steel for slabs-on-grade and foundations be provided in accordance with CBC (2022). The structural engineer should be consulted for additional concrete specifications

9.11 Drainage

Positive surface drainage is imperative for satisfactory site performance. Positive drainage should be provided and maintained to transport surface water away from the foundations and other site improvements. Positive drainage is defined as a slope of 2 percent or more over a distance of 5 feet or more away from the foundations. Surface water should not be allowed to pond adjacent to foundation elements.

10 LIMITATIONS

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

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

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

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified, and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

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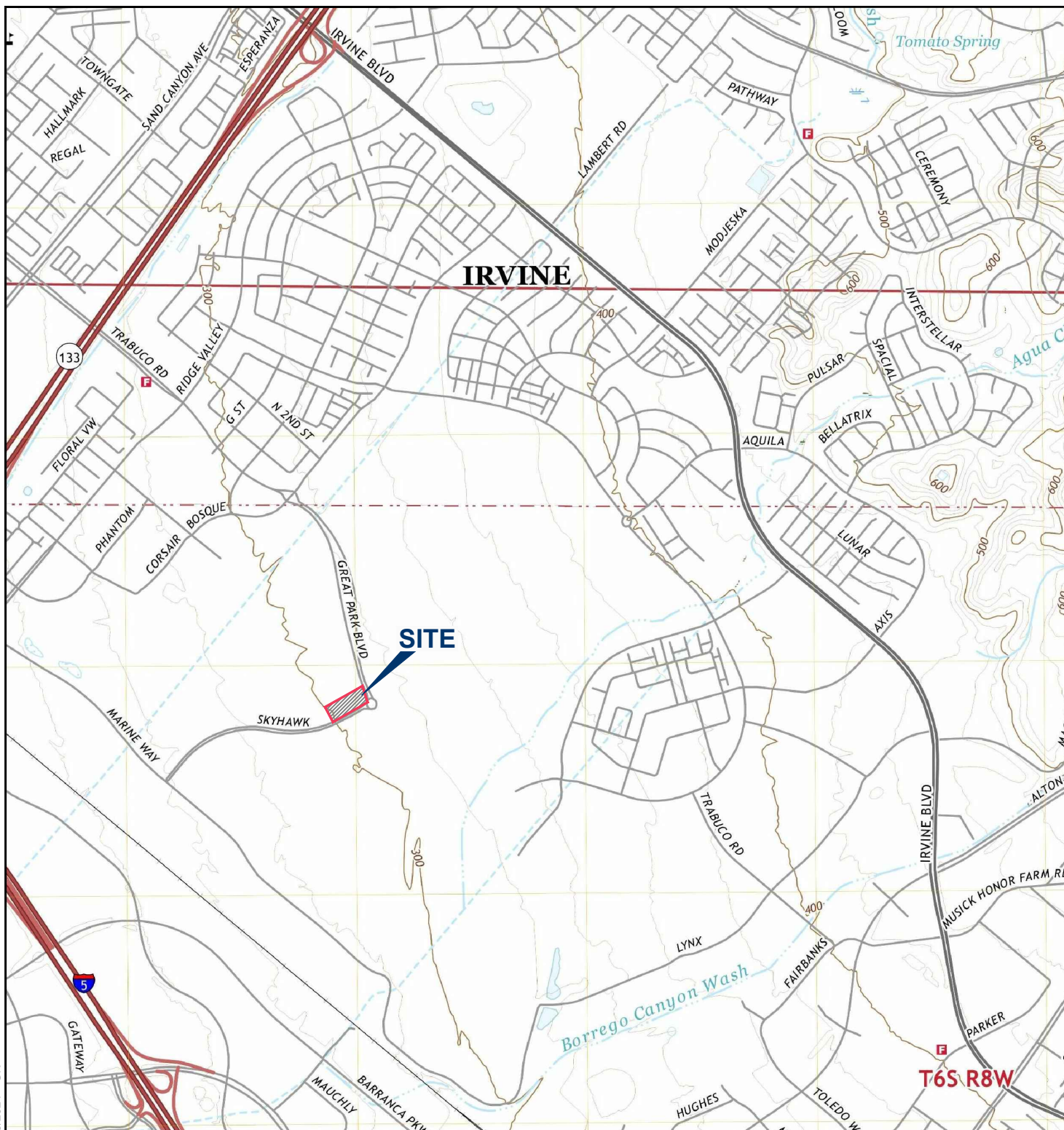
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FIGURES

212085029.dwg_SL 10/15/2024 GK



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

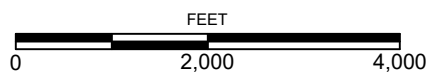
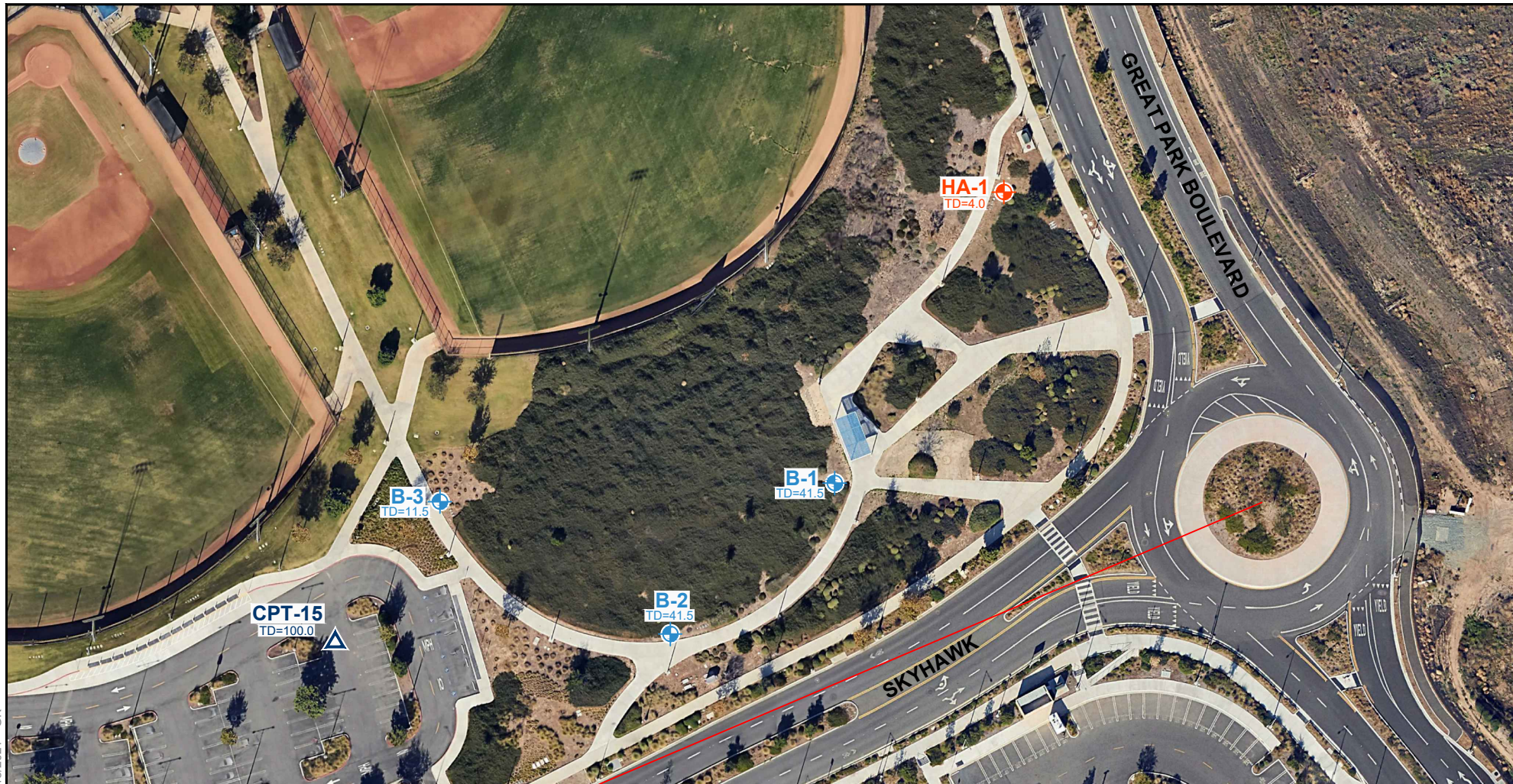
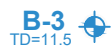


FIGURE 1

212085029.dwg_SAEI 10/18/2024 GK



LEGEND



B-3
TD=11.5

HSA BORING;
TD=TOTAL DEPTH IN FEET



CPT-15
TD=100.0

CONE PENETRATION TEST, LGC (2009);
TD=TOTAL DEPTH IN FEET



HA-1
TD=4.0

HAND-AUGER BORING;
TD=TOTAL DEPTH IN FEET

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. | REFERENCE: GOOGLE EARTH, 2024.

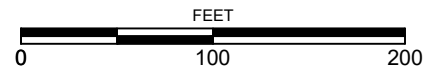
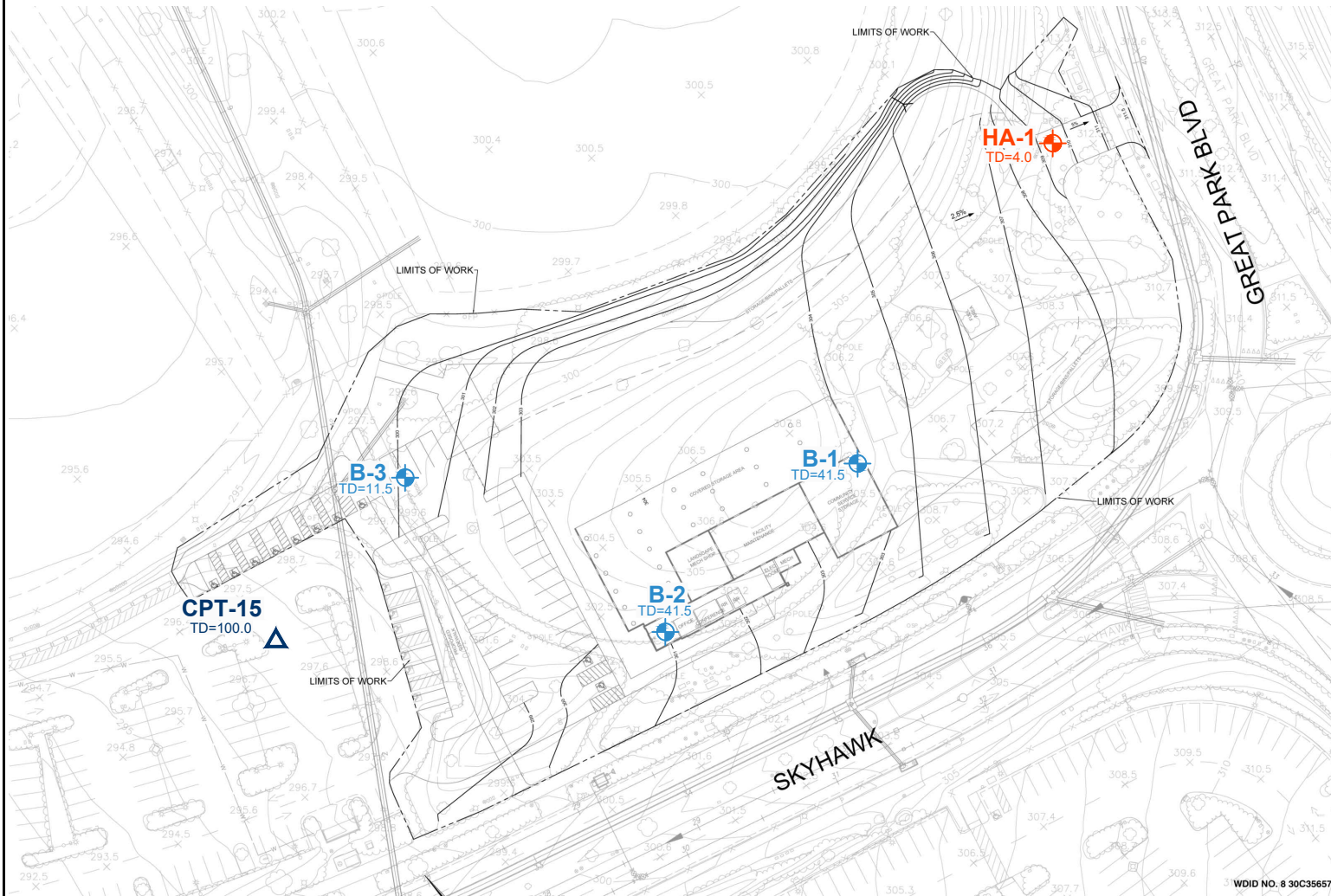


FIGURE 2



LEGEND	
B-3 TD=11.5	HSA BORING; TD=TOTAL DEPTH IN FEET
HA-1 TD=4.0	HAND-AUGER BORING; TD=TOTAL DEPTH IN FEET
CPT-15 TD=100.0	CONE PENETRATION TEST, LGC (2009); TD=TOTAL DEPTH IN FEET

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. | REFERENCE: DMC, 2024.

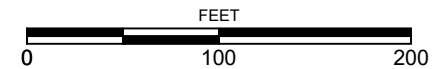
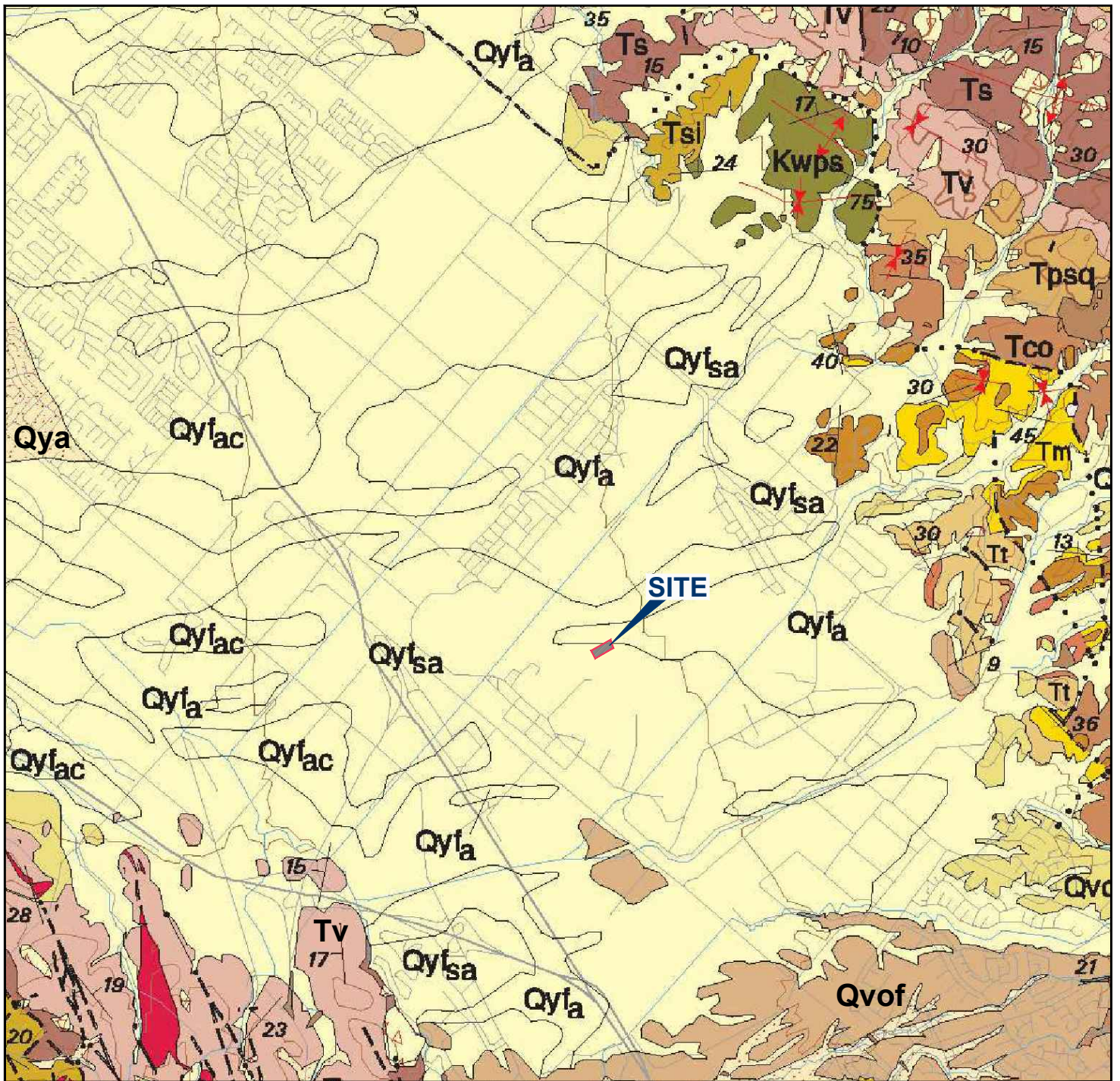


FIGURE 3

212085029.dwg_RG 10/15/2024 GK

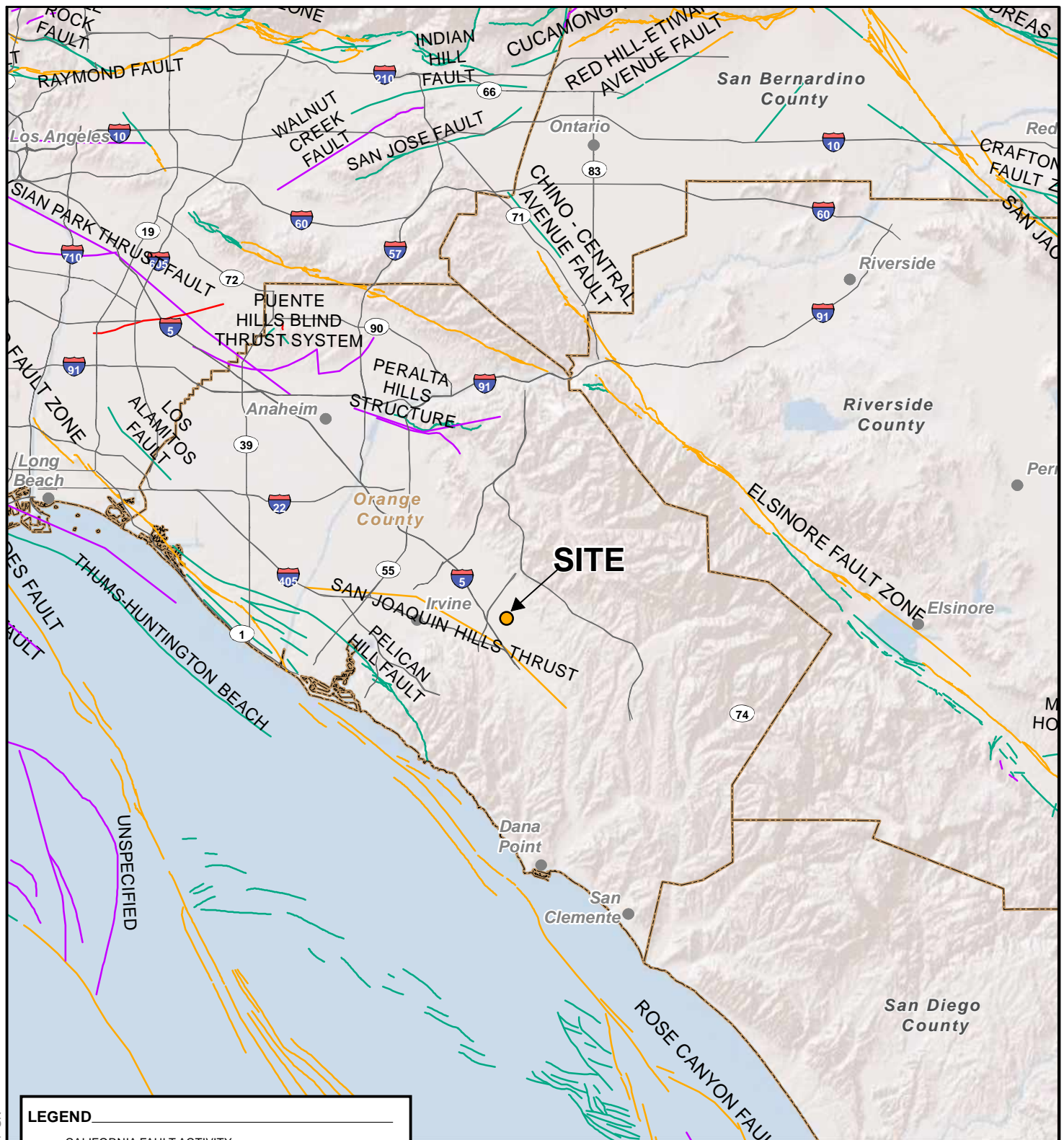


LEGEND

Qyf	YOUNG ALLUVIAL FAN DEPOSITS	Tco	OSO MEMBER		GEOLOGIC CONTACT
Qya	YOUNG AXIAL-CHANNEL DEPOSITS	Tv	VAQUEROS FORMATION		FAULT
Qvoa	VERY OLD AXIAL-CHANNEL DEPOSITS	Ts	SESPE FORMATION		
Qvof	VERY OLD ALLUVIAL-FAN DEPOSITS	Tt	TOPANGA GROUP		

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

FIGURE 4



LEGEND

CALIFORNIA FAULT ACTIVITY

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

SOURCES: QUATERNARY FAULTS DATABASE - U.S. GEOLOGICAL SURVEY AND CALIFORNIA GEOLOGICAL SURVEY, QUATERNARY FAULT AND FOLD DATABASE FOR THE UNITED STATES, ACCESSED SEPTEMBER 09, 2024, AT: <https://www.usgs.gov/programs/earthquake-hazards/faults>, ESRI, 2023.



NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

FIGURE 5

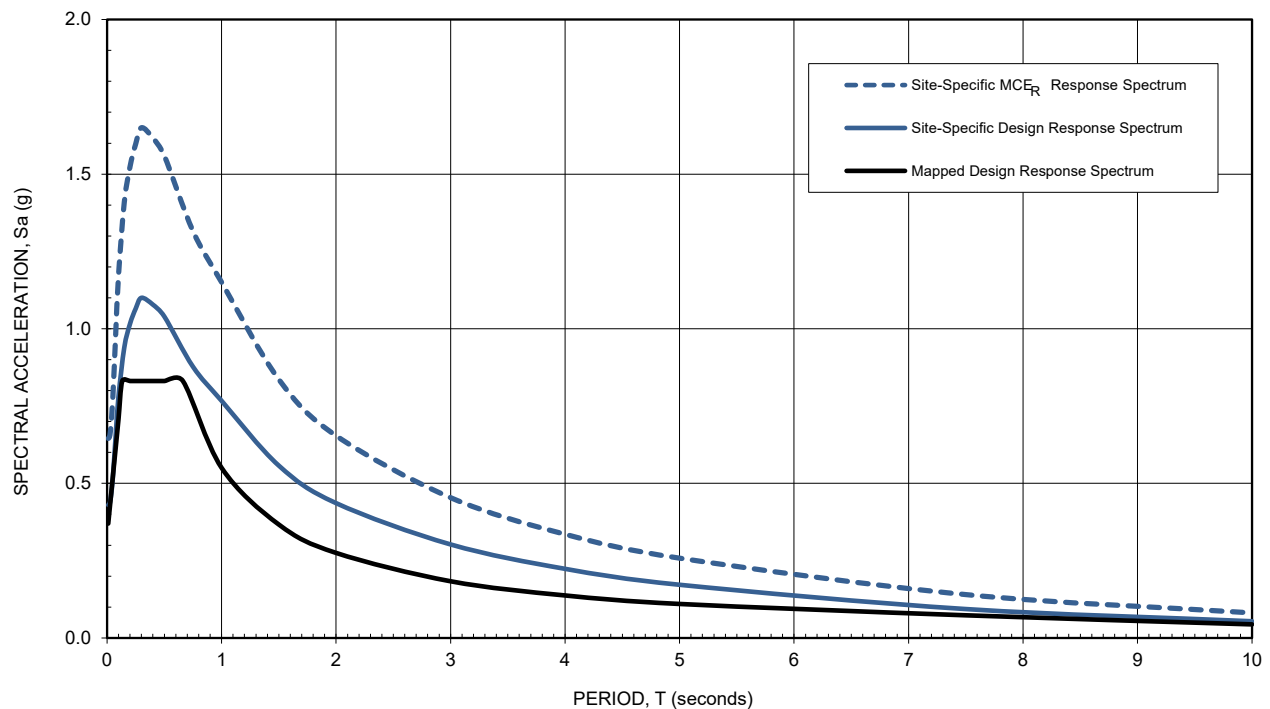
FAULT LOCATIONS

IRVINE GREAT PARK LANDSCAPE MAINTENANCE FACILITY
IRVINE, CALIFORNIA

PERIOD (seconds)	SITE-SPECIFIC MCE _R RESPONSE SPECTRUM Sa (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa (g)
0.010	0.645	0.430
0.020	0.650	0.433
0.030	0.671	0.448
0.050	0.776	0.517
0.075	0.981	0.654
0.100	1.173	0.782
0.150	1.408	0.939
0.200	1.524	1.016
0.250	1.598	1.065
0.300	1.650	1.100
0.400	1.617	1.078

PERIOD (seconds)	SITE-SPECIFIC MCE _R RESPONSE SPECTRUM Sa (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa (g)
0.500	1.559	1.040
0.750	1.316	0.877
1.000	1.150	0.767
1.500	0.836	0.557
2.000	0.654	0.436
3.000	0.454	0.303
4.000	0.335	0.224
5.000	0.258	0.172
7.500	0.140	0.094
10.000	0.081	0.054

$S_{MS} = 1.485 \text{ g}$ $S_{M1} = 1.361 \text{ g}$ $S_{DS} = 0.990 \text{ g}$ $S_{D1} = 0.908 \text{ g}$ $PGA_M = 0.624 \text{ g}$



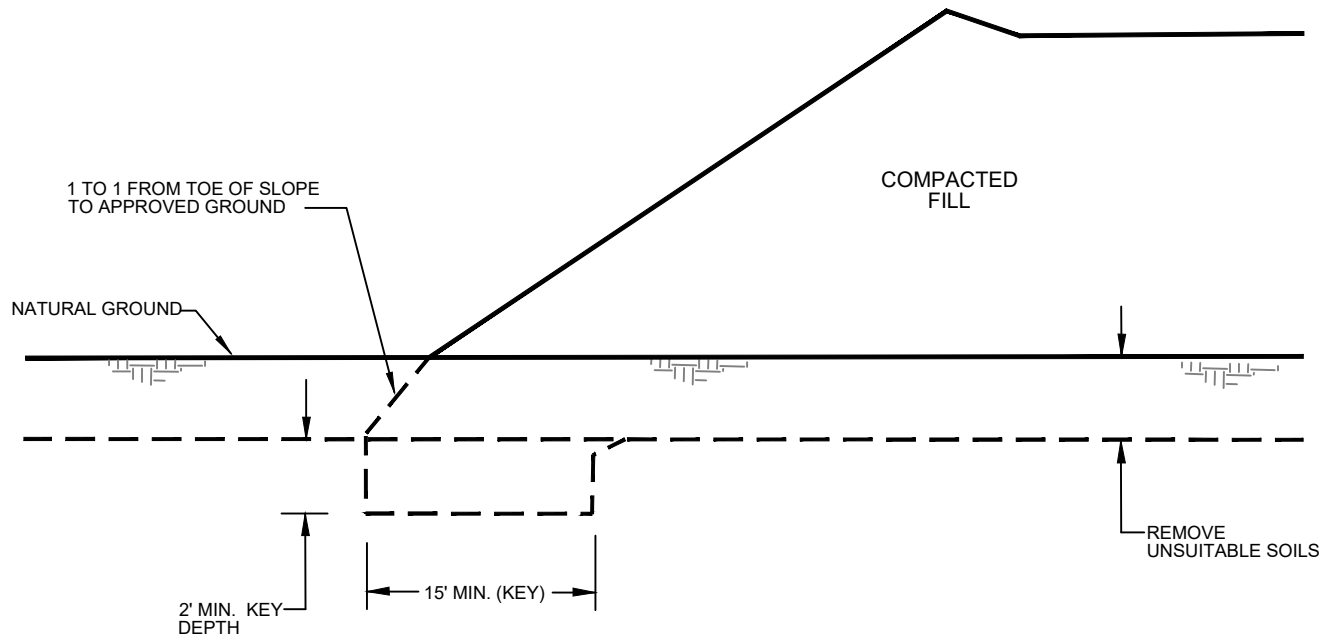
NOTES:

- 1 The probabilistic ground motion spectral response accelerations are based on the risk-targeted Maximum Considered Earthquake (MCE_R) having a 2% probability of exceedance in 50 years in the maximum direction using the Chiou & Youngs (2014), Campbell & Bozorgnia (2014), Boore et al. (2014), and Abrahamson et al. (2014) attenuation relationships and the risk coefficients per ASCE 7-16 Section 21.2.1.1.
- 2 The deterministic ground motion spectral response accelerations are the 84th percentile geometric mean values in the maximum direction using the Chiou & Youngs (2014), Campbell & Bozorgnia (2014), Boore et al. (2014), and Abrahamson et al. (2014) attenuation relationships for deep soil sites considering a Mw 7.2 event on the San Joaquin Hills fault zone located 3.1 kilometers from the site. It conforms with the lower bound limit per ASCE 7-16 Section 21.2.2.
- 3 The Site-Specific MCE_R Response Spectrum is the lesser of the spectral ordinates of the deterministic and probabilistic accelerations at each period per ASCE 7-16 Section 21.2.3. The Site-Specific Design Response Spectrum conforms with the lower bound limit per ASCE 7-16 Section 21.3.
- 4 The Mapped Design Response Spectrum is computed from the mapped spectral ordinates modified for Site Class D (stiff soil profile) per ASCE 7-16 Section 11.4. It is presented for the sake of comparison.

FIGURE 6

ACCELERATION RESPONSE SPECTRA

IRVINE GREAT PARK LANDSCAPE MAINTENANCE FACILITY
IRVINE, CALIFORNIA

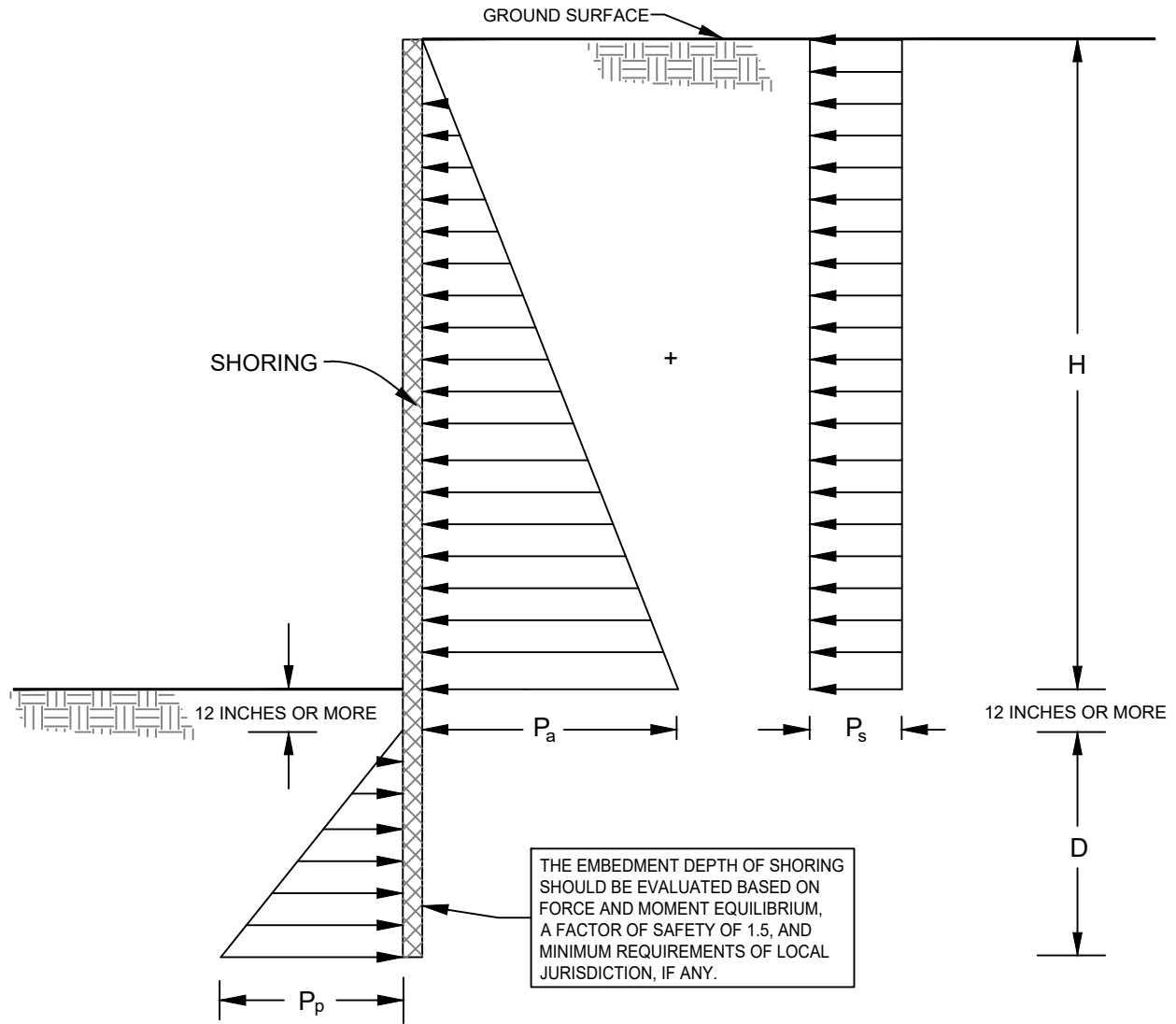


NOT TO SCALE

FIGURE 7

FILL KEY DETAIL

IRVINE GREAT PARK LANDSCAPE MAINTENANCE FACILITY
IRVINE, CALIFORNIA



NOTES:

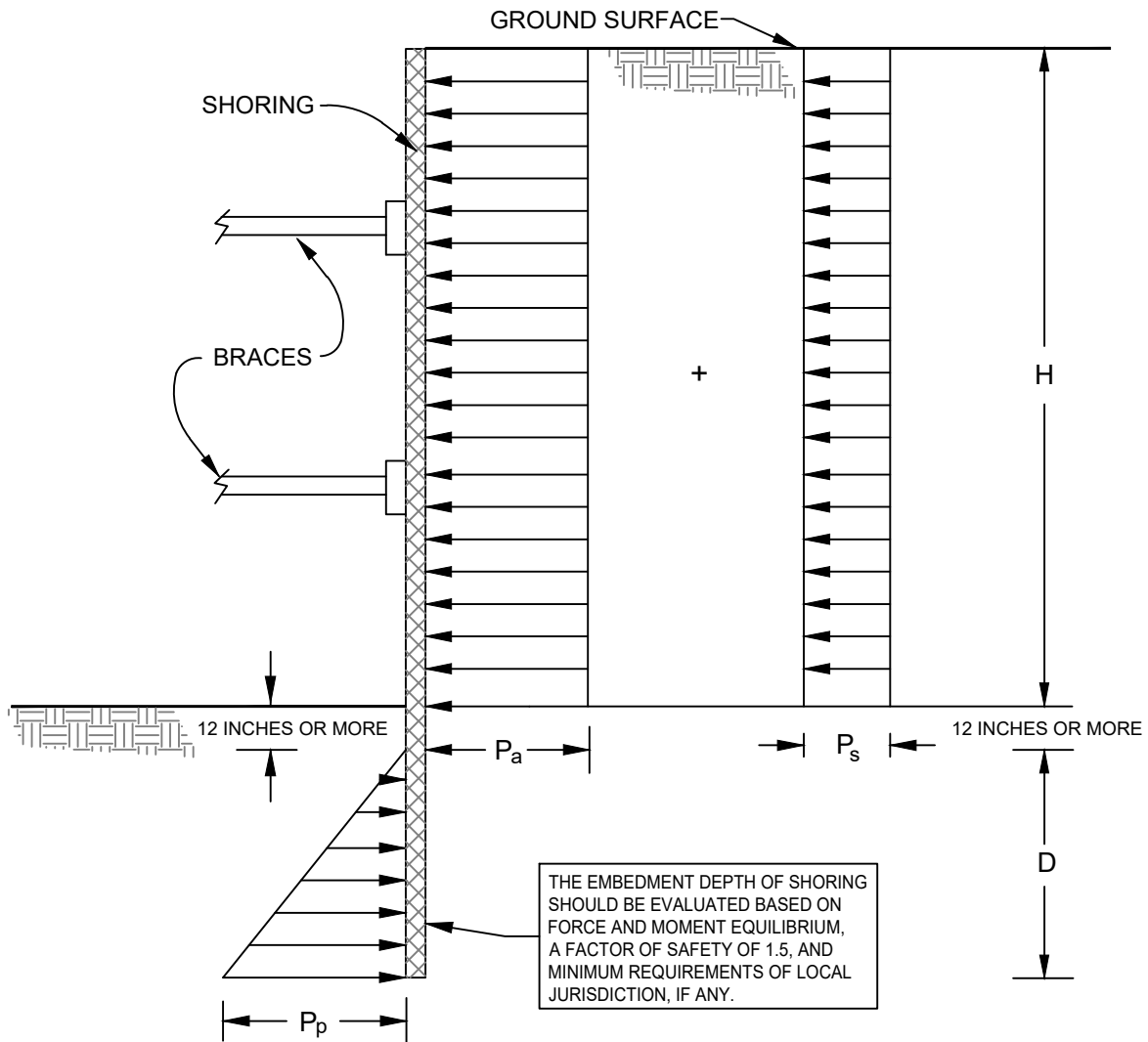
1. ACTIVE LATERAL EARTH PRESSURE, P_a
 $P_a = 43H$ psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
 $P_s = 86$ psf
3. PASSIVE LATERAL EARTH PRESSURE, P_p
 $P_p = 300D$ psf
4. ASSUMES GROUNDWATER IS NOT PRESENT
5. H AND D ARE IN FEET

NOT TO SCALE

FIGURE 8

**LATERAL EARTH PRESSURES FOR
TEMPORARY CANTILEVERED SHORING**

IRVINE GREAT PARK LANDSCAPE MAINTENANCE FACILITY
IRVINE, CALIFORNIA



NOTES:

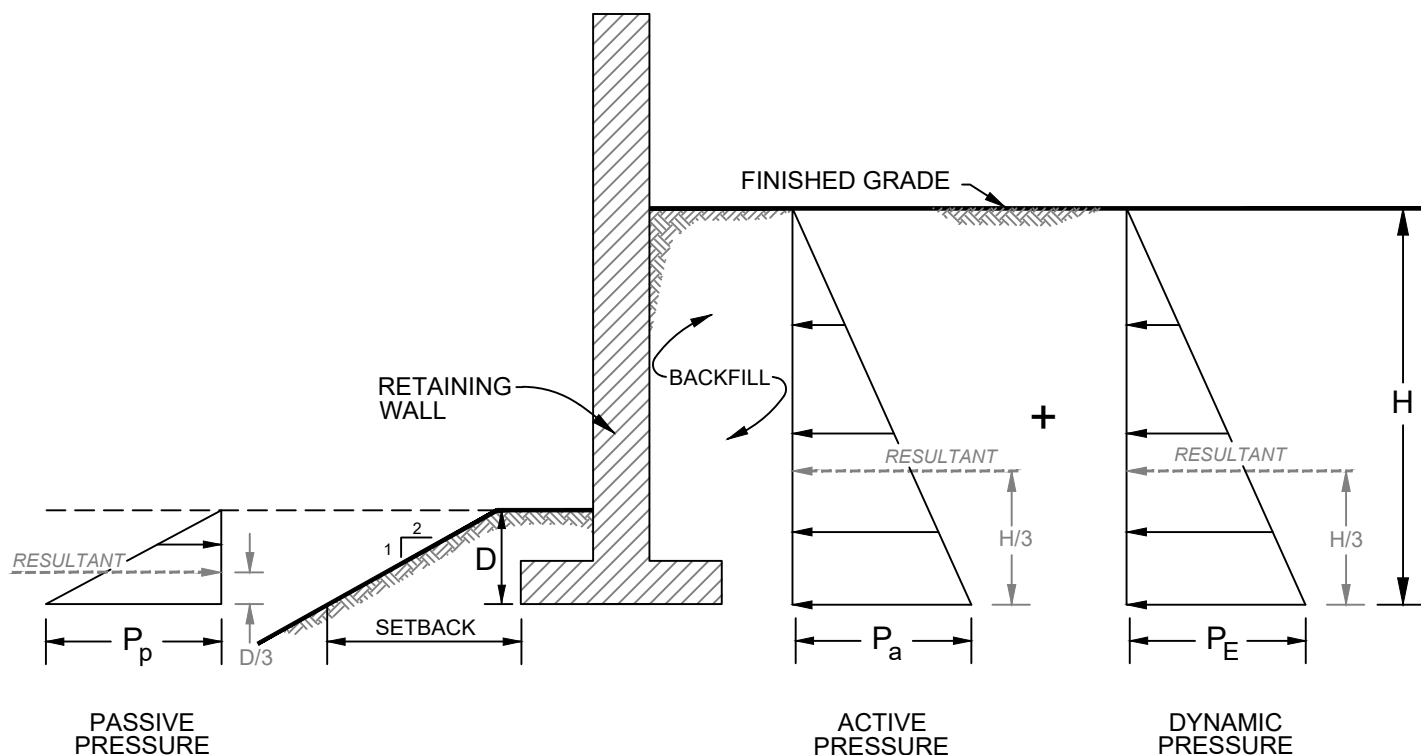
1. APPARENT LATERAL EARTH PRESSURE, P_a
 $P_a = 28H$ psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
 $P_s = 127$ psf
3. PASSIVE LATERAL EARTH PRESSURE, P_p
 $P_p = 300D$ psf
4. ASSUMES GROUNDWATER IS NOT PRESENT
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H AND D ARE IN FEET

NOT TO SCALE

FIGURE 9

LATERAL EARTH PRESSURES FOR BRACED EXCAVATION

IRVINE GREAT PARK LANDSCAPE MAINTENANCE FACILITY
IRVINE, CALIFORNIA



NOTES:

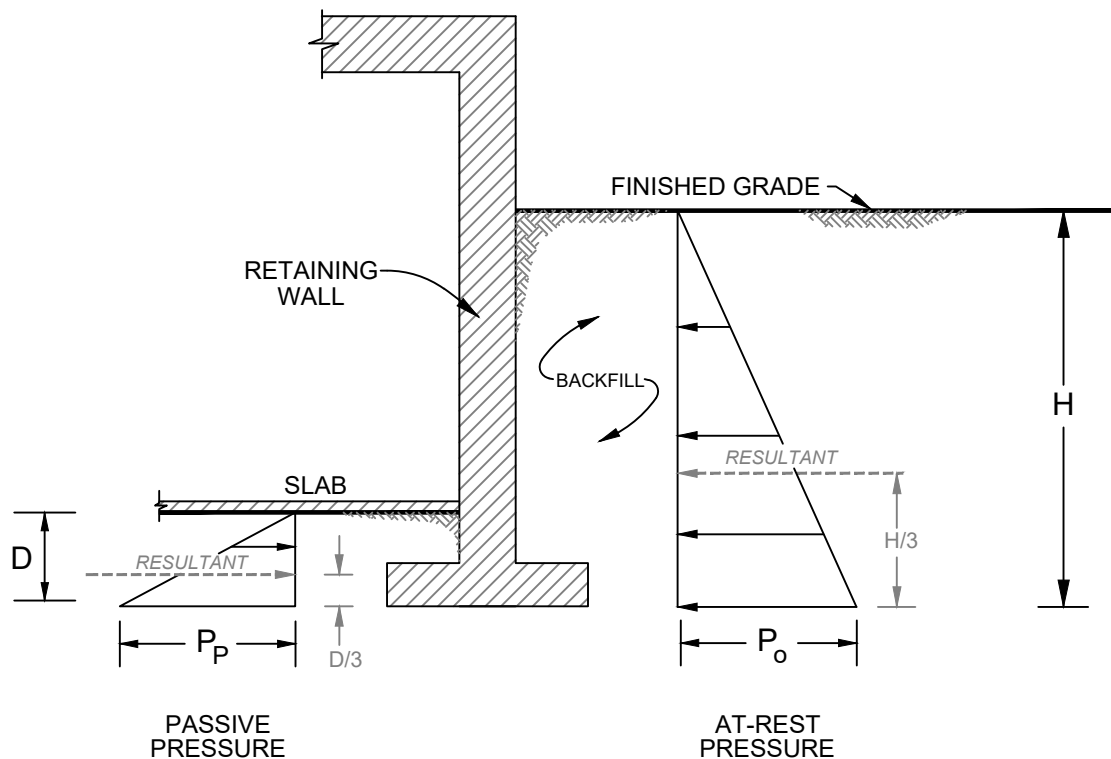
1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. STRUCTURAL GRANULAR BACKFILL MATERIALS AS SPECIFIED IN GREENBOOK SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. P_E IS CALCULATED IN ACCORDANCE WITH THE RECOMMENDATIONS OF MONONOBÉ AND MATSUO (1929), AND ATIK AND SITAR (2010)
5. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
6. H AND D ARE IN FEET
7. SETBACK SHOULD BE IN ACCORDANCE WITH THE CURRENT VERSION OF THE APPLICABLE BUILDING CODE

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure ($\text{lb/ft}^2/\text{ft}^{(1)}$)	
P_a	Level Backfill with Granular Soils ⁽²⁾	2H:1V Sloping Backfill with Granular Soils ⁽²⁾
	37H	57H
P_E	19H	25H
P_p	Level Ground	2H:1V Descending Ground
	350D	140D

NOT TO SCALE

FIGURE 10



NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. STRUCTURAL, GRANULAR BACKFILL MATERIALS AS SPECIFIED IN GREENBOOK SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. DYNAMIC LATERAL EARTH PRESSURE IS IGNORED AND DEEMED INAPPLICABLE DUE TO THE AT-REST CONDITION OF THE RETAINING WALL
5. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
6. H AND D ARE IN FEET

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

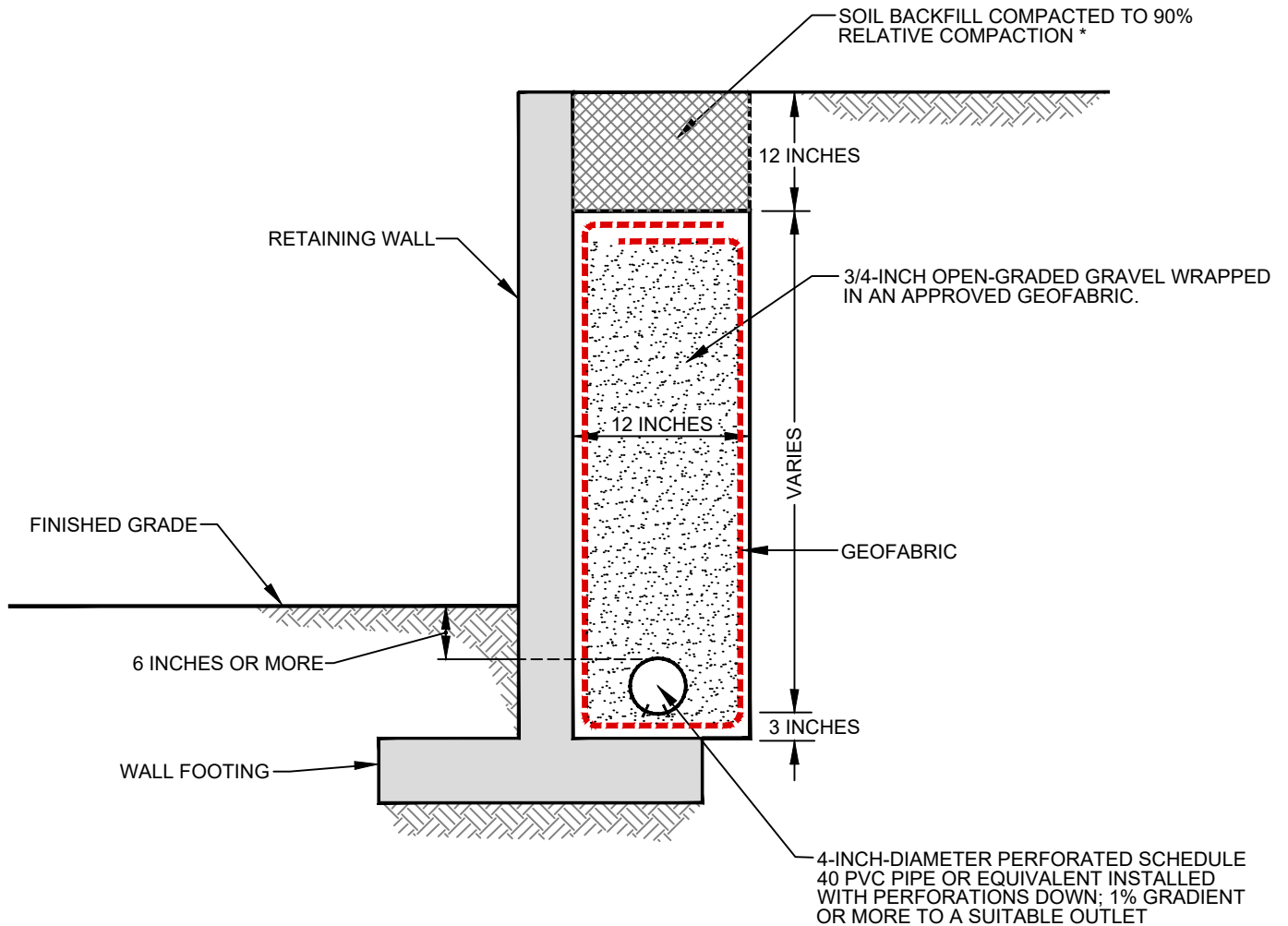
Lateral Earth Pressure	Equivalent Fluid Pressure (lb/ft ² /ft) ⁽¹⁾	
P _o	Level Backfill with Granular Soils ⁽²⁾	2H:1V Sloping Backfill with Granular Soils ⁽²⁾
	56H	82H
P _p	Level Ground	2H:1V Descending Ground
	350D	140D

NOT TO SCALE

FIGURE 11

LATERAL EARTH PRESSURES FOR RESTRAINED RETAINING WALLS

IRVINE GREAT PARK LANDSCAPE MAINTENANCE FACILITY
IRVINE, CALIFORNIA



*BASED ON ASTM D1557

NOT TO SCALE

FIGURE 12



APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

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

Bulk Samples

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

The Standard Penetration Test (SPT) Sampler

Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of approximately 1.4 inches. The sampler was driven into the ground 12 to 18 inches with a 140-pound hammer falling freely from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.





























Field Procedure for the Collection of Relatively Undisturbed Samples

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

The Modified Split-Barrel Drive Sampler

The sampler, with an external diameter of 3.0 inches, was lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sampler barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sampler barrel in the brass rings, sealed, and transported to the laboratory for testing.

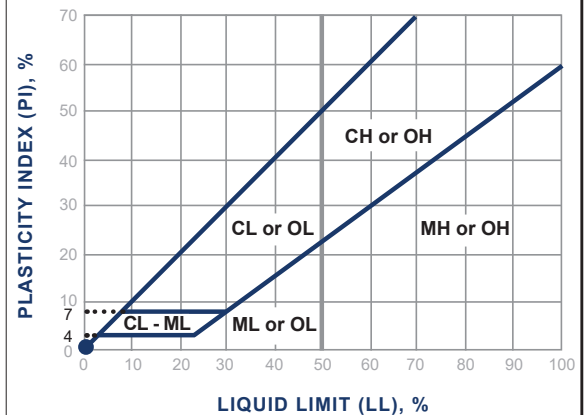
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Secondary Divisions	
			Group Symbol	Group Name
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	GRAVEL more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	 GW	well-graded GRAVEL
			 GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	 GW-GM	well-graded GRAVEL with silt
			 GP-GM	poorly graded GRAVEL with silt
			 GW-GC	well-graded GRAVEL with clay
			 GP-GC	poorly graded GRAVEL with clay
		GRAVEL with FINES more than 12% fines	 GM	silty GRAVEL
			 GC	clayey GRAVEL
			 GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	 SW	well-graded SAND
			 SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	 SW-SM	well-graded SAND with silt
			 SP-SM	poorly graded SAND with silt
			 SW-SC	well-graded SAND with clay
			 SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	 SM	silty SAND
			 SC	clayey SAND
			 SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	SILT and CLAY liquid limit less than 50%	INORGANIC	 CL	lean CLAY
			 ML	SILT
			 CL-ML	silty CLAY
		ORGANIC	 OL (PI > 4)	organic CLAY
			 OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	 CH	fat CLAY
			 MH	elastic SILT
		ORGANIC	 OH (plots on or above "A"-line)	organic CLAY
			 OH (plots below "A"-line)	organic SILT
		Highly Organic Soils	 PT	Peat

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.075 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.075"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

Plasticity Chart



Apparent Density - Coarse-Grained Soil

Apparent Density	Spooling Cable or Cathead		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	Modified Split Barrel (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

Consistency - Fine-Grained Soil

Consistency	Spooling Cable or Cathead		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	Modified Split Barrel (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

BORING LOG EXPLANATION SHEET

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

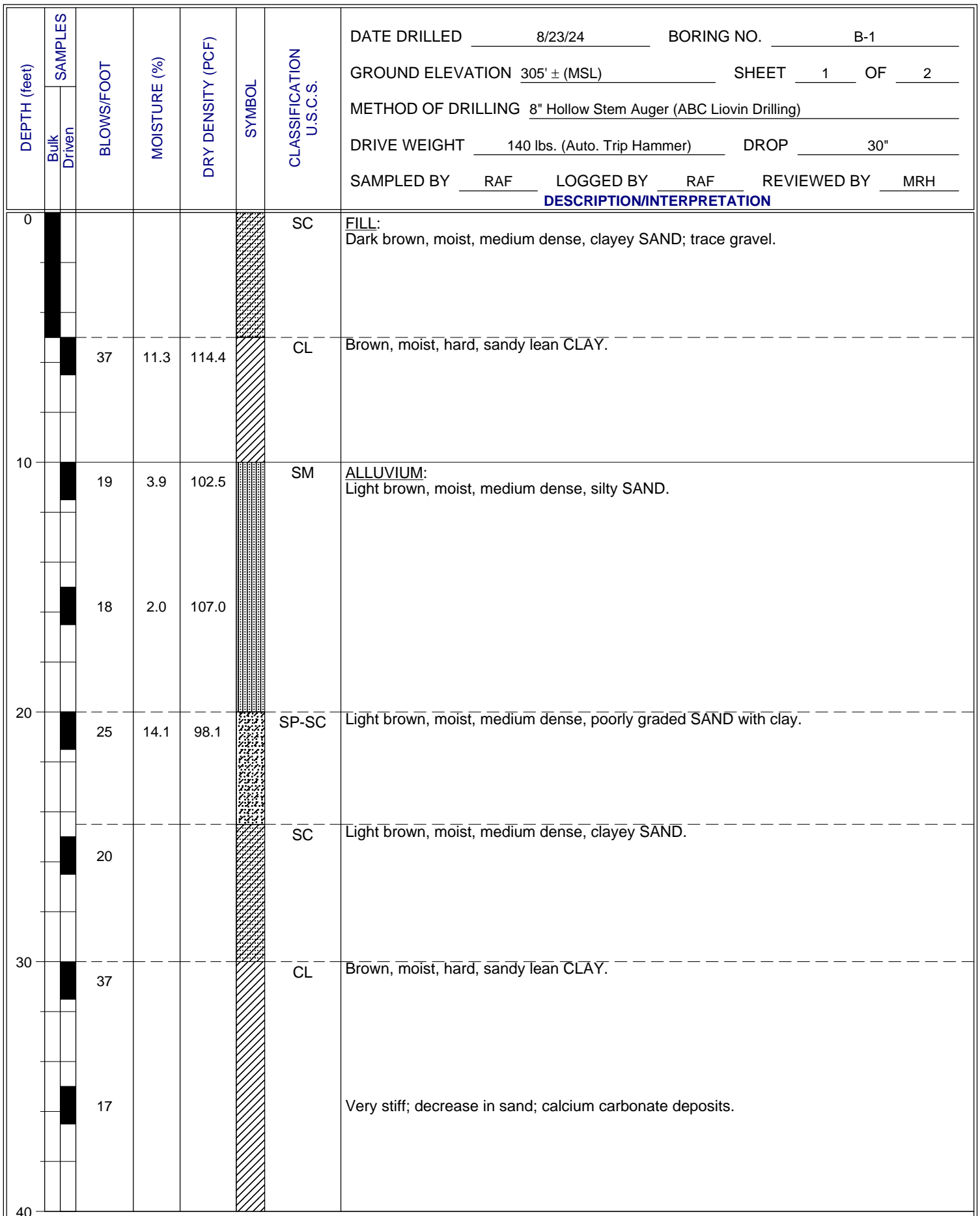


FIGURE A- 1


DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>8/23/24</u> BORING NO. <u>B-1</u> GROUND ELEVATION <u>305' ± (MSL)</u> SHEET <u>2</u> OF <u>2</u> METHOD OF DRILLING <u>8" Hollow Stem Auger (ABC Liovin Drilling)</u> DRIVE WEIGHT <u>140 lbs. (Auto. Trip Hammer)</u> DROP <u>30"</u> SAMPLED BY <u>RAF</u> LOGGED BY <u>RAF</u> REVIEWED BY <u>MRH</u> DESCRIPTION/INTERPRETATION		
	Bulk	Driven								
40			20				SC	ALLUVIUM: (Continued) Brown, moist, medium dense, clayey SAND. Total Depth = 41.5 feet. Groundwater not encountered during drilling. Backfilled with soil generated from drilling on 8/23/24. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.		
50										
60										
70										
80										

FIGURE A- 2

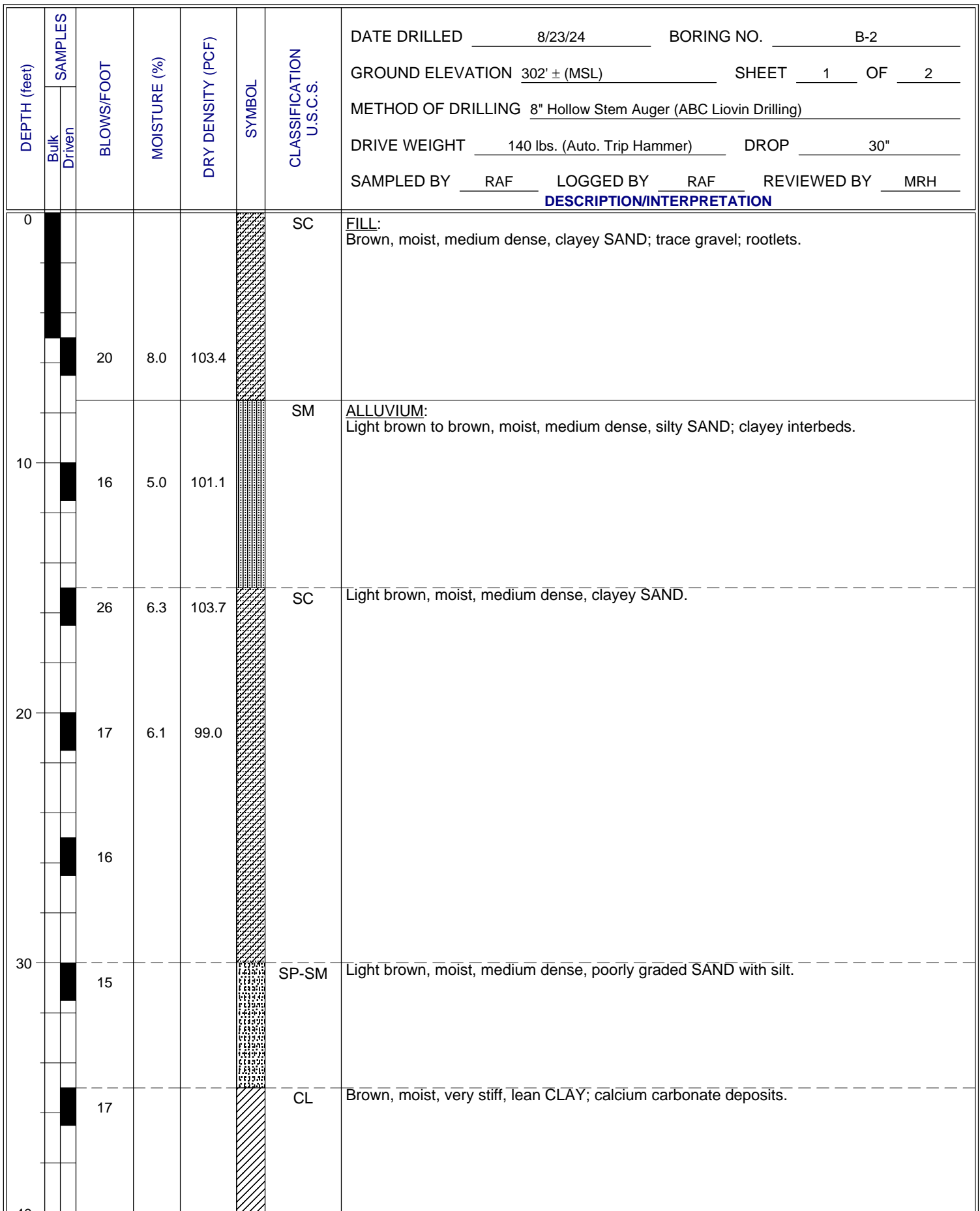


FIGURE A- 3


DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>8/23/24</u> BORING NO. <u>B-2</u> GROUND ELEVATION <u>302' ± (MSL)</u> SHEET <u>2</u> OF <u>2</u> METHOD OF DRILLING <u>8" Hollow Stem Auger (ABC Liovin Drilling)</u> DRIVE WEIGHT <u>140 lbs. (Auto. Trip Hammer)</u> DROP <u>30"</u> SAMPLED BY <u>RAF</u> LOGGED BY <u>RAF</u> REVIEWED BY <u>MRH</u>		
	Bulk	Driven						DESCRIPTION/INTERPRETATION		
40			36				CL	ALLUVIUM: (Continued) Brown to red, moist, hard, sandy lean CLAY; calcium carbonate stringers; manganese oxidation. Total Depth = 41.5 feet. Groundwater not encountered during drilling. Backfilled with soil generated from drilling on 8/23/24. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.		
50										
60										
70										
80										

FIGURE A- 4

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>8/23/24</u> BORING NO. <u>B-3</u> GROUND ELEVATION <u>298' ± (MSL)</u> SHEET <u>1</u> OF <u>1</u> METHOD OF DRILLING <u>8" Hollow Stem Auger (ABC Liovin Drilling)</u> DRIVE WEIGHT <u>140 lbs. (Auto. Trip Hammer)</u> DROP <u>30"</u> SAMPLED BY <u>RAF</u> LOGGED BY <u>RAF</u> REVIEWED BY <u>MRH</u> DESCRIPTION/INTERPRETATION	
	Bulk	Driven							
0							SC	FILL: Brown, moist, medium dense, clayey SAND; trace gravel; rootlets.	
			14	4.2	104.8		SP-SM	ALLUVIUM: Light brown, moist, loose, poorly graded SAND with silt.	
10			6					Brown.	
								Total Depth = 11.5 feet. Groundwater not encountered during drilling. Backfilled with soil generated from drilling on 8/23/24. <u>Notes:</u> Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.	
20									
30									
40									

FIGURE A- 5

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>8/23/24</u> BORING NO. <u>HA-1</u> GROUND ELEVATION <u>311' ± (MSL)</u> SHEET <u>1</u> OF <u>1</u> METHOD OF DRILLING <u>4" Hand Auger (ABC Liovin Drilling)</u> DRIVE WEIGHT <u>140 lbs. (Auto. Trip Hammer)</u> DROP <u>30"</u> SAMPLED BY <u>RAF</u> LOGGED BY <u>RAF</u> REVIEWED BY <u>MRH</u>	
	Bulk	Driven						DESCRIPTION/INTERPRETATION	
0							SC	FILL: Brown, moist, medium dense, clayey SAND; trace gravel.	
10								Total Depth = 4 feet. Groundwater not encountered during drilling. Backfilled with soil generated from drilling on 8/23/24. <u>Notes:</u> Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.	
20									
30									
40									

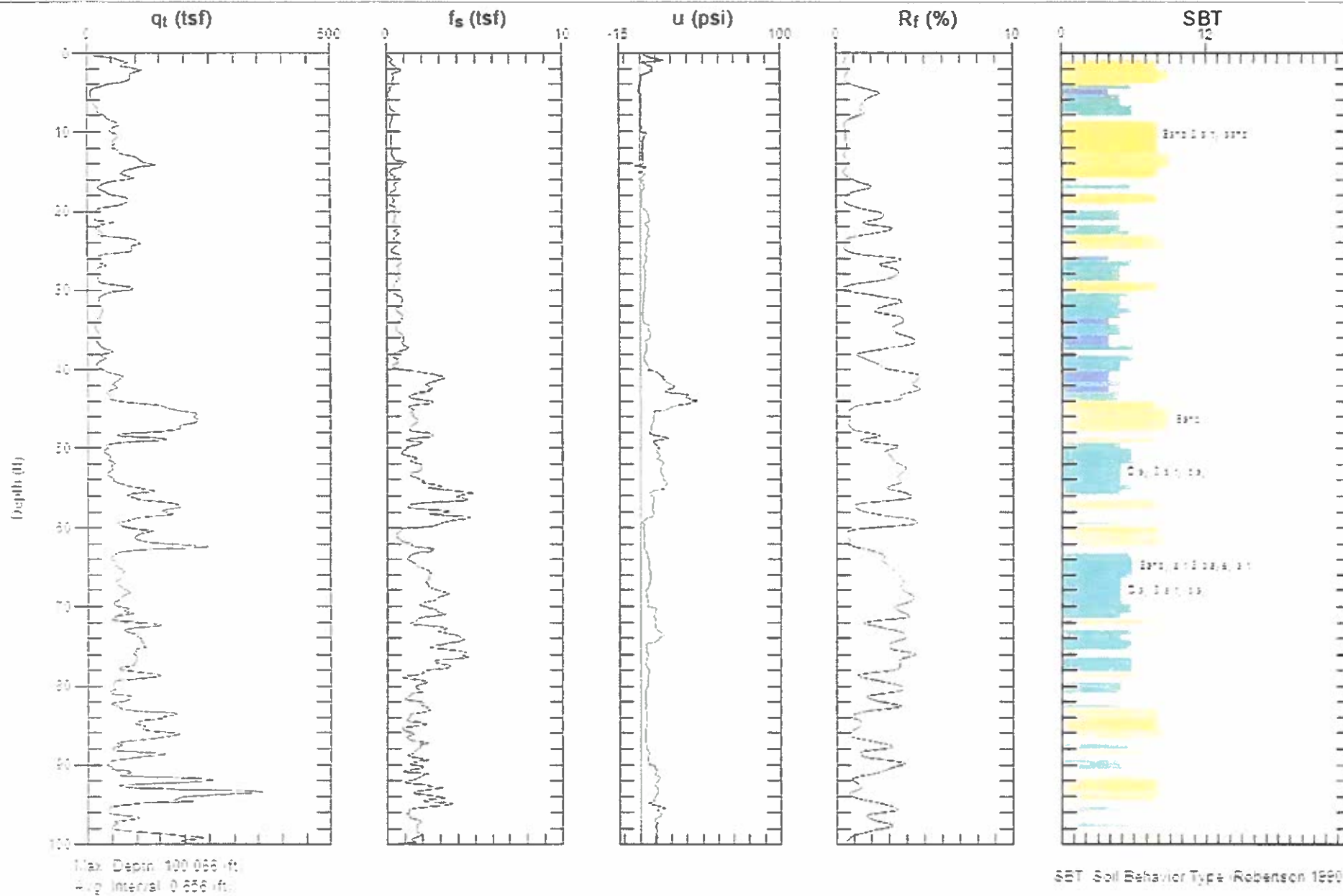


APPENDIX B

Previous CPT Log (LGC 2009)



Date: 1/28/2008 08:50





Shear Wave Velocity Calculations

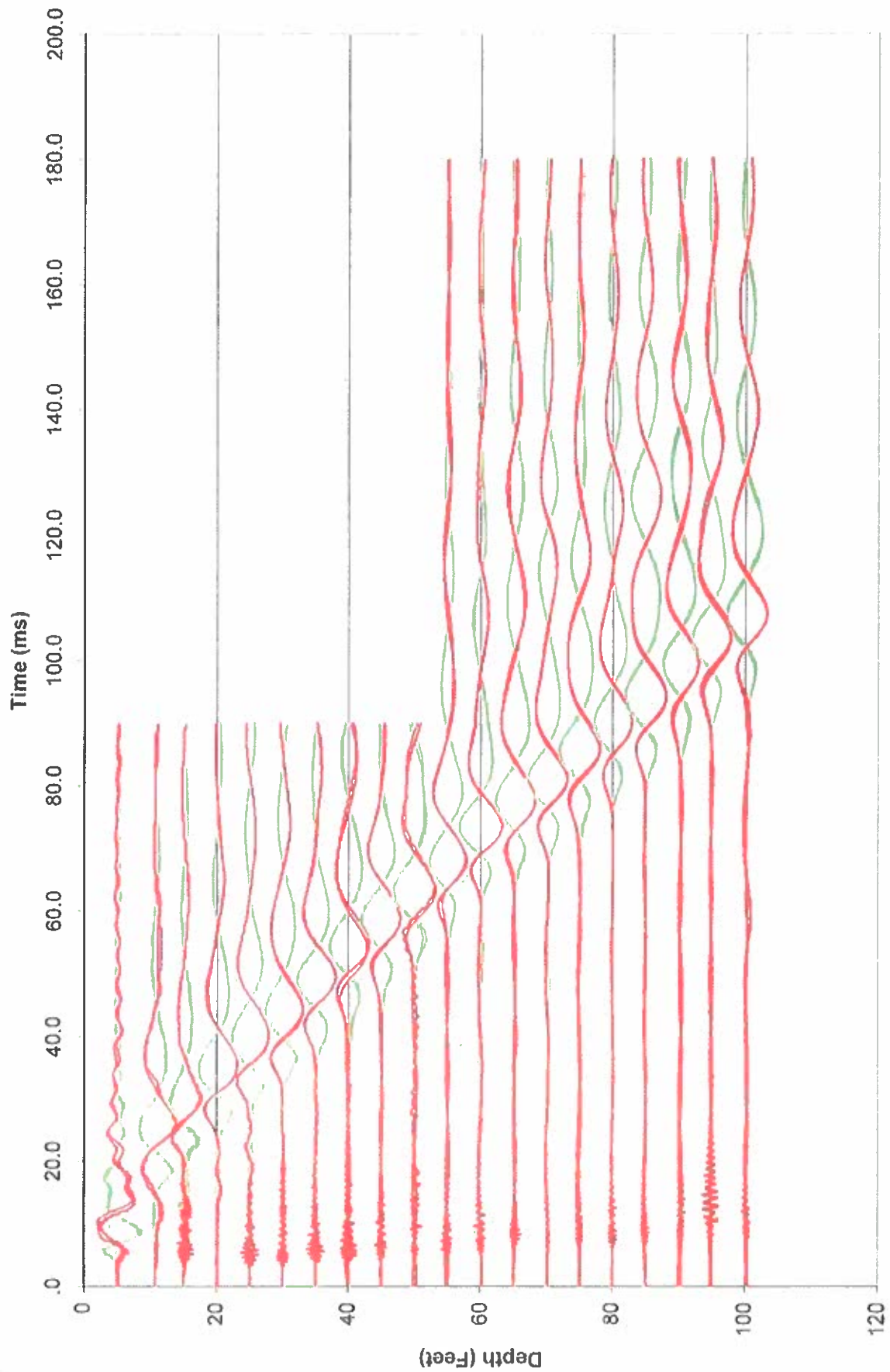
EL TORO MCAS
CPT-15

Geophone Offset: 0.66 Feet
Source Offset: 1.67 Feet

1/28/2008

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
5.09	4.43	4.73	4.73	9.6000			
10.01	9.35	9.49	4.76	16.8500	7.2500	657.2	6.89
15.09	14.43	14.53	5.03	23.8000	6.9500	724.3	11.89
20.01	19.35	19.42	4.90	30.7500	6.9500	704.6	16.89
25.10	24.44	24.50	5.07	38.1500	7.4000	685.2	21.90
30.02	29.36	29.41	4.91	45.5500	7.4000	663.7	26.90
35.10	34.44	34.49	5.08	53.0000	7.4500	681.6	31.90
40.03	39.37	39.40	4.92	59.0000	6.0000	819.4	36.91
45.11	44.45	44.48	5.08	64.2500	5.2500	967.9	41.91
50.03	49.37	49.40	4.92	68.7500	4.5000	1092.9	46.91
55.12	54.46	54.48	5.08	73.8500	5.1000	996.6	51.92
60.20	59.54	59.57	5.08	77.9000	4.0500	1255.1	57.00
65.12	64.46	64.49	4.92	82.4500	4.5500	1081.2	62.00
70.05	69.39	69.41	4.92	86.1500	3.7000	1329.7	66.93
75.13	74.47	74.49	5.08	90.2500	4.1000	1240.0	71.93
80.05	79.39	79.41	4.92	94.1000	3.8500	1277.9	76.93
85.14	84.48	84.49	5.08	99.1000	5.0000	1016.8	81.93
90.06	89.40	89.41	4.92	103.2000	4.1000	1200.1	86.94
95.47	94.81	94.83	5.41	107.6500	4.4500	1216.3	92.11
100.07	99.41	99.42	4.59	111.4000	3.7500	1224.7	97.11

Waveforms for Sounding CPT-16





APPENDIX C

Laboratory Testing

APPENDIX C

LABORATORY TESTING

Classification

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

In-Place Moisture and Density Tests

The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

200 Wash

An evaluation of the percentage of particles finer than the No. 200 sieve in selected soil samples was performed in general accordance with ASTM D 1140. The results of the tests are presented on Figure C-1.

Expansion Index Tests

The expansion index of selected materials was evaluated in general accordance with ASTM D 4829. Specimens were molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and were inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of these tests are presented on Figure C-2.

Proctor Density Test

The maximum dry density and optimum moisture content of a selected representative soil sample was evaluated using the Modified Proctor method in general accordance with ASTM D 1557. The results of the test are summarized on Figure C-3.

Direct Shear Tests

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

R-Value

The resistance value, or R-value, for site soils was evaluated in general accordance with California Test (CT) 301. A sample was prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test result is shown on Figure C-6.

Soil Corrosivity Tests

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

SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-1	0.0-5.0	CLAYEY SAND	97	33	SC
B-1	5.0-6.5	SANDY LEAN CLAY	100	52	CL
B-1	10.0-11.5	SILTY SAND	98	15	SM
B-2	5.0-6.5	CLAYEY SAND	100	38	SC
B-2	10.0-11.5	SILTY SAND	100	15	SM
B-3	0.0-4.0	CLAYEY SAND	98	33	SC
B-3	5.0-6.5	POORLY GRADED SAND WITH SILT	100	11	SP-SM
HA-1	0.0-4.0	CLAYEY SAND	98	27	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

FIGURE C-1

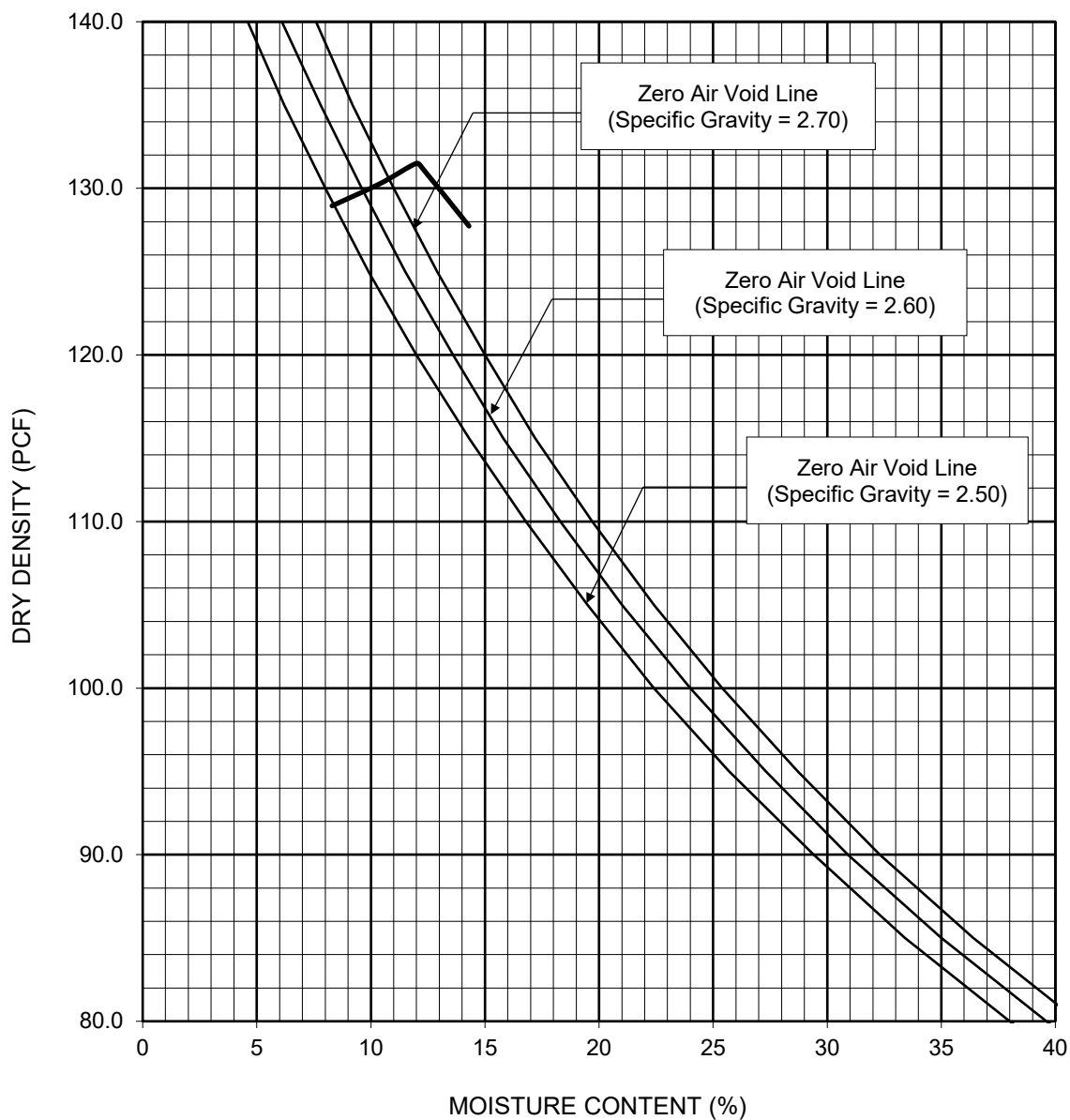
SAMPLE LOCATION	SAMPLE DEPTH (ft)	INITIAL MOISTURE (percent)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (percent)	VOLUMETRIC SWELL (in)	EXPANSION INDEX	POTENTIAL EXPANSION
B-1	0.0-5.0	8.5	117.1	16.0	0.022	22	Low
B-3	0.0-4.0	8.3	116.1	14.7	0.003	3	Very Low

PERFORMED IN GENERAL ACCORDANCE WITH

☐ UBC STANDARD 18-2

☒ ASTM D 4829

FIGURE C-2



Sample Location	Depth (ft)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (percent)
B-1	0.0-5.0	Dark Brown Clayey SAND	131.5	12.0
Dry Density and Moisture Content Values Corrected for Oversize (ASTM D 4718)			N/A	N/A

PERFORMED IN GENERAL ACCORDANCE WITH

☒ ASTM D 1557

☐ ASTM D 698

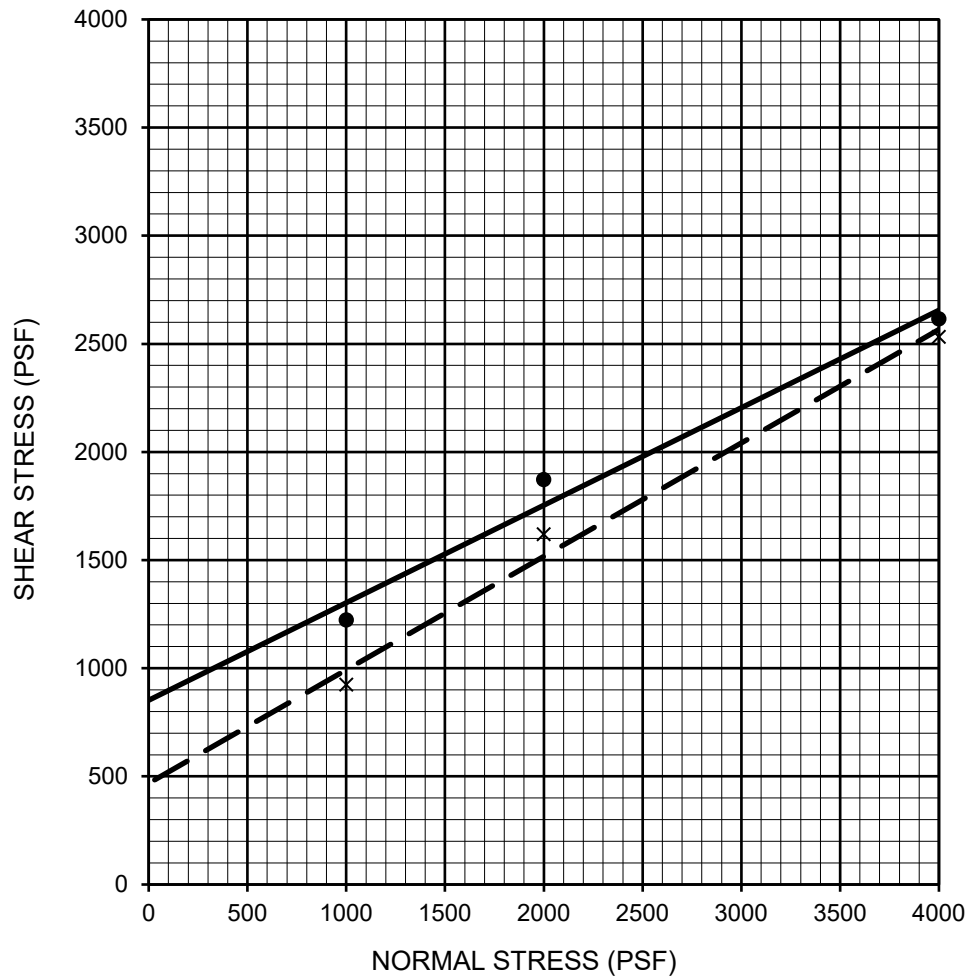
METHOD

☐ A

☐ B

☒ C

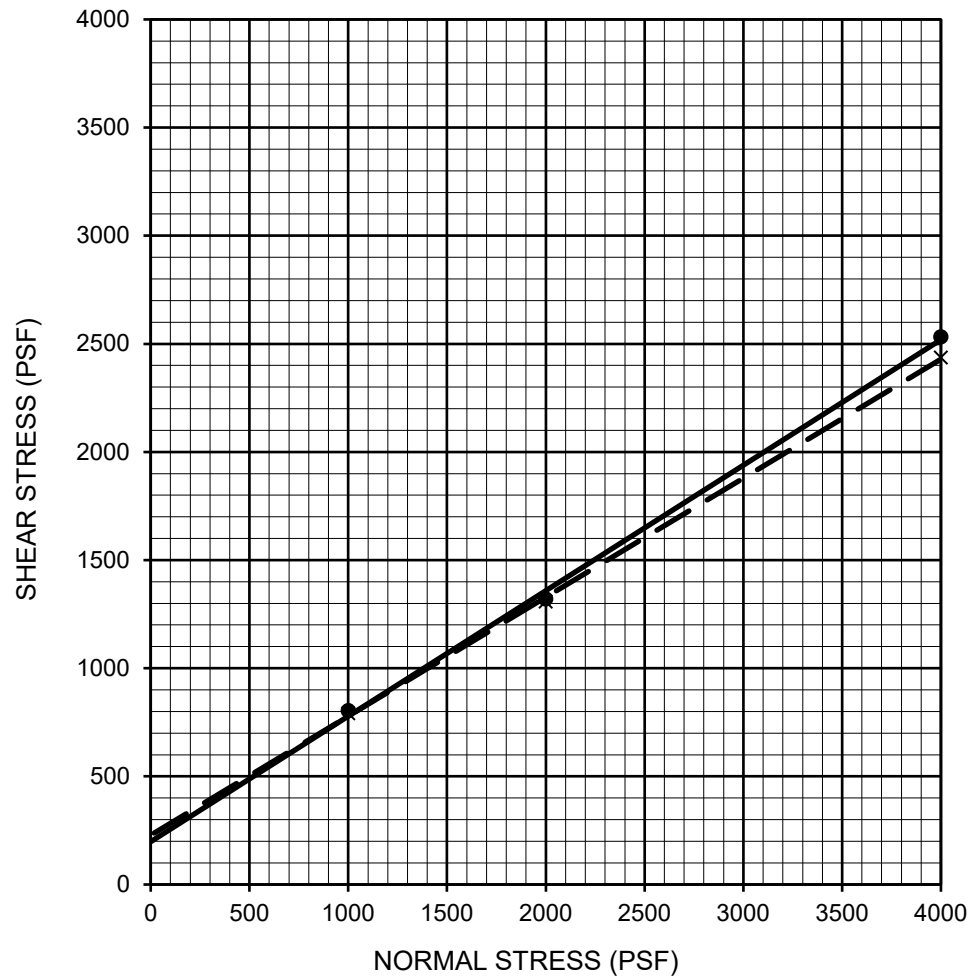
FIGURE C-3



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
CLAYEY SAND	—●—	B-1	0.0-5.0	Peak	852	24	SC
CLAYEY SAND	- - X - -	B-1	0.0-5.0	Ultimate	468	28	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080 ON A SAMPLE REMOLDED TO 90% RELATIVE COMPACTION

FIGURE C-4



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
CLAYEY SAND	—●—	B-2	5.0-6.5	Peak	198	30	SC
CLAYEY SAND	- - X - -	B-2	5.0-6.5	Ultimate	228	29	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE C-5

SAMPLE LOCATION	SAMPLE DEPTH (ft)	SOIL TYPE	R-VALUE
B-3	0.0-4.0	SC	48

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

FIGURE C-6

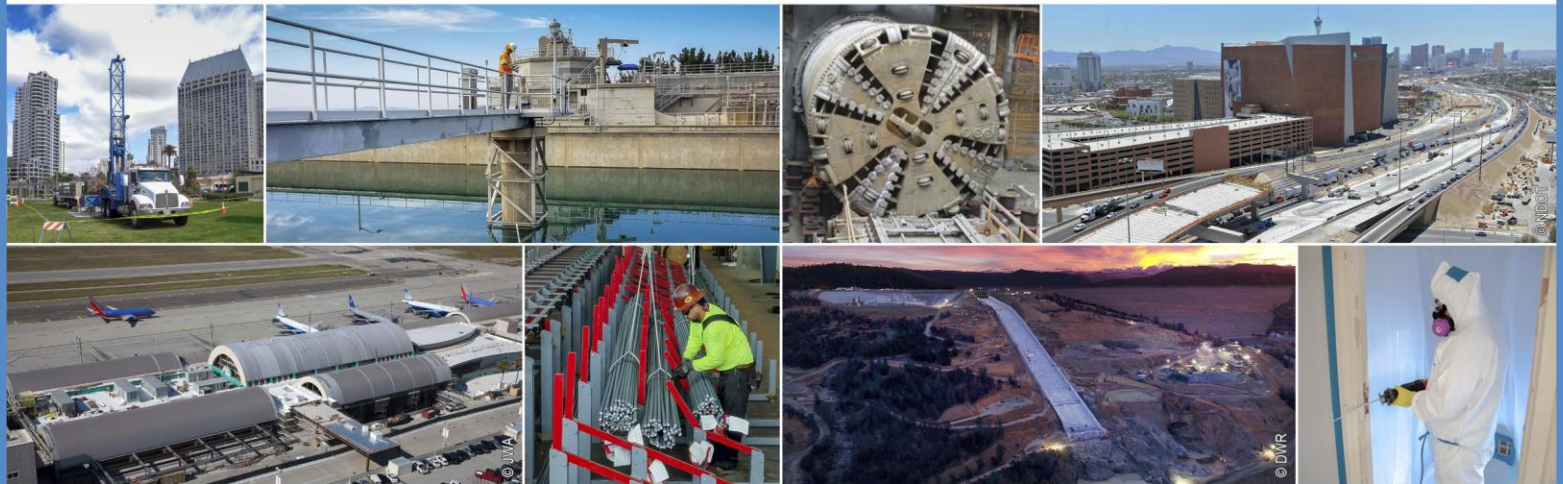
SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH ¹	RESISTIVITY ¹ (ohm-cm)	SULFATE CONTENT ²		CHLORIDE CONTENT ³ (ppm)
				(ppm)	(%)	
B-2	0.0-5.0	6.9	1,789	20	0.002	65

¹ PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643

² PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417

³ PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

FIGURE C-7



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