

APPENDIX 4.7-1

GEOTECHNICAL REPORTS

October 22, 2024

Project No. 22023-01

To: Shea Properties
130 Vantis, Suite 200
Aliso Viejo, California, 92656

Attention: Mr. Sean McEachern, Vice President of Development

Subject: Addendum Report for Proposed Multi-Family Residential Project at Existing Anaheim Hills Festival Retail Center, Development Area 5, City of Anaheim, California

Per our recent communications, we have prepared this addendum report to the referenced NMG feasibility report (NMG, 2022) to address the Draft Environmental Impact Report (DEIR) review comments related to the potential for groundwater, liquefaction and seismic-induced landslide hazards at the subject site.

Scope of Services

The tasks below were performed for our analysis and this addendum.

- Review of our November 2022 feasibility report and other reports (listed at rear of text)
- Review and response to DEIR reviewer comments and questions
- Review and analysis of over 20 years of dewatering well data from site operator (Vestar)
- Communications with Vestar representative related to wells and well data
- Discussions and communication with you and DEIR team related to the DEIR comments

Groundwater

In our exploration described in the reference report no groundwater was encountered in our borings including a boring drilled to 15 feet deep, within 50 feet of an existing dewatering well for the former theater building. We noted in our report that significant amounts of local perched water and seepage was reported during the original rough grading of the site (Geosoils, 1992a). Hence the installation of numerous subdrains and a dewatering system for the theater consisting of 5 wells with pumps (Geosoils, 1992b). At the time of our fieldwork, the dewatering pumps appeared to be out of service. We have learned recently that four of the five dewatering wells remain functional and that the pumps are necessary to keep water from seeping into theater building stair wells that

are below ground level. We also learned that Vestar, the retail center operator, has been measuring water levels in the theater wells since their installation nearly 30 years ago.

After reviewing the data, we conclude that the groundwater being extracted by the pumps is related to local seepage along distinct and dissimilar earth layers as encountered during the original site grading. We maintain that this seepage is a local, perched groundwater condition along the southern portion of the site and not a broader groundwater table under the entire commercial site.

While the Vestar information clearly indicates the presence of groundwater impacting the theater building space, there is not sufficient detail regarding the well construction, water level collection methodology, and reliability of the water level readings to draw definitive conclusions regarding the actual groundwater levels around the wells. During some periods, the reported water levels fluctuated significantly.

Our feasibility report included recommendations for a subdrain system along the southern boundary of the site (NMG, 2022). Because the existing pumping wells have been running off and on over the last 30 years, it would be prudent to install new pumping wells in addition to the recommended subdrain system. We recommend abandoning the current wells due to their age and unknown installation methods and installing new wells to the current standards. The number of wells and their locations should be based on the actual conditions exposed during grading and placed such that they can be accessed for observation and maintenance.

Liquefaction Potential

In our feasibility report, we stated that the potential for seismic liquefaction at the site was very low due to the dense engineered fill and bedrock, and the absence of groundwater. As a clarification, we would state that the potential for liquefaction is very low, simply because the dense compacted fill and bedrock are not susceptible to liquefaction, regardless of the groundwater conditions.

Seismic Landslide Hazards

The reviewer asked that the DEIR address areas adjacent to the site that are mapped as landslide hazard zones. There is an area south/southwest of the development that is mapped by the State as having a potential for earthquake-induced landslides. That area has since been graded and developed into a residential housing tract. The overly steep slope has been removed and replaced with a stabilization keyway and engineered fill. The potential for landslide hazards is considered negligible at that site and will not have significant impact on the subject site.

If you have any questions regarding this report, please contact us. We appreciate the opportunity to provide our services.

Respectfully submitted,

NMG GEOTECHNICAL, INC.



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Principal Geologist



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Attachment: References

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REFERENCES

GeoSoils, Inc., 1992a, Final Rough Grade Soil and Geologic Report, Anaheim Hills Festival Project, Anaheim Hills, Orange County, California, Dated June 2, 1992.

GeoSoils, Inc., 1992b, Subdrainage Constructed for Perched Groundwater Mitigation, Theatre Pad and Festival Drive, Anaheim Hills Festival Project, Anaheim Hills, California, Dated August 10, 1992.

NMG Geotechnical, Inc., 2022, Geotechnical Exploration and Feasibility Report for Proposed Multi-Family Residential Development, Anaheim Hills Festival Retail Center, City of Anaheim, California, Project No. 22023-01, dated November 23, 2022



November 23, 2022

Project No. 22023-01

To: Shea Properties
130 Vantis, Suite 200
Aliso Viejo, California 92656

Attention: Mr. Sean McEachern, Vice President of Development

Subject: Geotechnical Exploration and Feasibility Report for Proposed Multi-Family Residential Development, Anaheim Hills Festival Retail Center, City of Anaheim, California

In accordance with your request, NMG Geotechnical, Inc. (NMG) has conducted a geotechnical exploration and feasibility study for the upper pad of the Anaheim Hills Festival Retail Center, in the city of Anaheim. The purpose of our study was to evaluate the geotechnical site conditions in light of the proposed improvements in order to determine project feasibility and provide preliminary geotechnical recommendations for the project design, grading, and construction.

Our scope of services was based on review of two options for conceptual site development and included review of background site information, review of prior geotechnical reports by others, drilling and logging of five hollow-stem-auger borings, advancement of seven cone penetrometer tests (CPTs), laboratory testing, geotechnical review of the preliminary design information, geotechnical analysis, and preparation of this report. This report presents our findings, conclusions and preliminary recommendations.

The site was graded in the 1990s for the retail development and is underlain by compacted artificial fill and sandstone/siltstone bedrock. The compacted artificial fill is primarily in the northern portion of the site and in a prior canyon in the east-central portion of the site. The fill is up to 38 feet thick along the former canyon alignment. Bedrock is exposed at the surface in the southwestern portion of the site and is present beneath the compacted fill. Groundwater was not encountered in borings drilled during our exploration; however, prior work at the site encountered considerable seepage in the southwest and a mitigation system was installed. Please note, environmental evaluation of onsite soils is not a part of this report and is the purview of others.

Based on our exploration, testing and analyses, we conclude that the proposed improvements are feasible from a geotechnical viewpoint. Potential constraints include differential settlement due to the varying fill depths associated with the canyon fill, previously mitigated presence of nuisance seepage, seismic shaking typical to the region and the logistics of constructing a new development while maintaining the adjacent retail development functional. The soils at the site are classified as having "low to medium" expansion potential. Geologic conditions are such that storm water infiltration is not feasible at the site.

If you have any questions regarding this report, please contact us. We appreciate the opportunity to provide our services.

Respectfully submitted,

NMG GEOTECHNICAL, INC.



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1.0 INTRODUCTION

1.1 Purpose and Scope of Services

The purpose of this study and subsurface exploration was to evaluate the existing geotechnical site conditions in light of the proposed development in order to determine project feasibility, identify geotechnical constraints and provide preliminary recommendations for design, grading, and construction.

Our scope of services was intended to collect sufficient geotechnical data to determine the site history and characterize the existing subsurface conditions, with respect to the proposed development. This report is based on the following scope of services:

- Review and compilation of available published geologic maps and geotechnical reports for the subject site and surrounding area. A list of references is included in Appendix A.
- Review of historic aerial photographs and topographic maps readily available from online sources.
- Site reconnaissance to review the conditions of the existing asphalt pavements, building exteriors and the perimeter gravity walls.
- Notification and coordination with DigAlert to identify and clear underground utilities at the boring and CPT locations. We also met with representatives of Vestar to clear the locations with their facilities/tenants.
- Drilling, sampling, and visual logging of five hollow-stem-auger borings (H-1 through H-5) to depths ranging from 15.3 to 25.8 feet below ground surface (bgs). Advancement of eight CPTs (CPT-1 through CPT-7) to depths of 6.2 to 37.3 feet were also performed. Shear wave measurements were obtained in CPT-4A to aid in the characterization of site class for seismic design. The approximate locations of these borings and CPTs are depicted on the Geotechnical Map (Plate 1) and the boring and CPT logs are included in Appendix B.
- Laboratory testing of selected samples to classify the onsite soils and evaluate in-situ moisture and density, grain-size distribution, maximum density and optimum moisture content, corrosivity, expansion index, shear strength, and consolidation. Laboratory test results are included in Appendix C.
- Review of the "Festival Hills Residential Building Site Plan" prepared by AO Architecture, received by NMG on November 10, 2022. The same plan was used as the base map to depict the geotechnical site conditions (Plate 1).
- Geotechnical evaluation and analysis of the compiled data with respect to the anticipated residential development.
- Preparation of this feasibility report with our findings, conclusions, and preliminary recommendations for the subject project.

1.2 Site Location and Existing Conditions

The approximately 10-acre site is a part of and located within the southern portion of the existing Anaheim Hills Festival retail center in the city of Anaheim, California. The site is bounded by South Festival Drive on the west and south, the existing shopping center on the north and 24-Hour Fitness building on the east (Figure 1). The subject area of this study currently consists of an existing movie theater building, asphalt parking lot with drive aisles, parking lots and overhead lighting. The northerly and southerly perimeter of the site consists of Loffelstein™ (loffel) segmented block gravity retaining walls up to approximately 15 feet in height. Based on review of GeoSoils report (1992c), it is unclear if any kind of geogrid reinforcement was installed in these perimeter walls. A fill slope up to approximately 17 feet high ascends from the northern gravity wall to the subject site.

Five "Groundwater Removal Pumps" are located between the movie theater and South Festival Drive. These pump units are housed in metal cages and the power supply to some of them appears to be disconnected.

1.3 Site History and Prior Studies

We reviewed historic aerial photographs dating from 1946 through 2021. The site was in a natural condition from 1946 through 1972. The central portion of the site had a distinct drainage running from south to north with small landslides on the west side of the drainage. Two small water reservoirs/ponds to the east of the site show in the 1946 through 1980 photos. By 1972, State Route 91 had just been widened and improved with on/off ramps to Wier Canyon. Santiago Canyon Road was complete and Wier Canyon Road was only complete to Santiago Canyon Road. By 1992, the site had been mass graded and the reservoirs/ponds to the east had been removed and there were houses/roads over the former ponds. In 1993, the movie theater and parking lot had been constructed (although the movie theater footprint was much smaller). Between 1997 and 1998, additions to both ends of the movie theater was complete and both the 24-hour Fitness and daycare facility had been constructed. From 1998 to present, the site has remained essentially unchanged.

A preliminary geologic and soils investigation prior to site development was performed by Medall, Aragon, Worswick & Associates (MAW) in 1979 for the overall shopping center development. The report includes exploratory data and provides original topography prior to development (MAW, 1979). They later prepared a supplemental report which included additional field exploration and geologic cross-sections (MAW, 1980).

The project was taken over in the late 1980s by GeoSoils, Inc. (GeoSoils), who performed additional subsurface exploration to support development of the Anaheim Hills Festival Shopping Center project (1989). GeoSoils provided observation and testing during mass grading of the site which included remedial earthwork, installation of a canyon-type subdrain, placement of compacted fill in the canyon areas, and design cuts up to 50 feet deep within the southwest portion of the site (GeoSoils, 1992a). Significant nuisance seepage was observed in the deep design cut areas upon completion of mass grading, and later in 1992. GeoSoils recommended an intricate subdrain system surrounding the movie theater which included five groundwater

pumping wells, all of which drained to a storm drain device located at the westerly entrance road off South Festival Drive (GeoSoils, 1992b). GeoSoils also provided observation and testing during subdrain installation and construction of the loffel (gravity) walls on the north and south perimeter of the site (1992c).

The pertinent prior borings, limits of compacted fill and approximate locations of the subdrain systems are depicted on the Geotechnical Map (Plate 1).

1.4 Proposed Development

Based on review of the conceptual plan, the proposed apartment development will consist of a "wrap" with residential buildings wrapped around a centrally located concrete parking garage and courtyards. We anticipate the parking structure will be on the order of four to five-levels, with possibly one-half level being subterranean. The residential structures will likely be up to five-story, wood-framed apartment buildings. A fitness center and clubhouse, leasing center, amenity building and supporting utility infrastructure is also planned.

1.5 Field Exploration

Our field exploration was performed on October 26, 2022, and included excavation of five 8-inch-diameter hollow-stem-auger borings to depths ranging from 15.3 to 25.8 feet bgs and advancement of eight CPTs to depths of 6.2 to 37.3 bgs. The borings were geotechnically logged and samples were taken at selected intervals. Relatively undisturbed soil ring samples were obtained from the borings with a 2.5-inch inside-diameter, split-barrel sampler. Samplers were driven into the soil with a 140-pound automatic safety hammer, free-falling 30 inches. The drive samples were also used to obtain a measure of resistance of the soil to penetration (recorded as blows-per-foot on our geotechnical boring logs). Representative bulk samples of onsite soils were collected from the drill cuttings and used for additional soil identification purposes. The borings were backfilled with native soils and tamped and an asphalt concrete (AC) patch was applied to the surface.

The Cone Penetration Test (CPT) uses an integrated electronic cone system that measures and records tip resistance, sleeve friction, and friction ratio parameters at 5-cm depth intervals by advancement of a 1.25-inch-diameter, pointed steel probe that is hydraulically pushed into the ground at a constant rate. The CPT provides a detailed subsurface profile and soil behavior type information. At CPT-4A, shear wave measurements were obtained at 10, 20, 30 and 35 feet deep. The shear wave is generated using an air-actuated hammer, located inside the front jack of the CPT rig. The cone tip has a triaxial geophone, which records the shear wave signal generated by the air hammer. The CPT data was used with the boring information and laboratory test data to develop a consistent interpretation of the subsurface conditions. The CPT data was utilized for the assessment of the potential liquefaction and dry sand settlement. Please note, the CPTs were all terminated due to refusal. The CPTs were backfilled with bentonite pellets and capped with AC patch.

The boring and CPT locations are depicted on Plate 1 and the geotechnical boring logs and CPT logs are included in Appendix B.

1.6 Laboratory Tests

We conducted the following tests on representative undisturbed and bulk earth samples to characterize their engineering properties:

- Moisture content and dry density;
- Particle-size sieve analysis;
- Atterberg Limits;
- Direct shear (undisturbed sample);
- Consolidation;
- Expansion Index; and
- Soil corrosivity.

Laboratory tests were conducted in general conformance with applicable ASTM International standard test methods. Laboratory test results are provided in Appendix C. In-situ moisture content and dry density data are included on the geotechnical boring logs (Appendix B).

We have also reviewed pertinent prior laboratory testing by GeoSoils (1989). Their findings are generally consistent with the NMG test results reported herein.

2.0 GEOTECHNICAL FINDINGS

2.1 Geologic Conditions and Earth Units

The subject site is located in the Peninsular Range geomorphic province, in the northwestern foothills of the Peralta Hills. During the Quaternary period, active faulting (associated with the San Andreas Fault system) subjected the region to compression and regional uplift, forming the Santa Ana Mountains and foothills. Also during the Quaternary period, active streams channels cut canyons into the emergent bedrock. However, during past grading operations, the natural terrain was modified by cutting the ridges and filling canyon areas resulting in the current site configuration.

The site is mapped as underlain by the Sycamore Canyon Member of the Puente Formation (Morton and Miller, 2006). Based on our review of the geotechnical report of rough grading prepared by GeoSoils (1992a), the subject site consists of up to approximately 38 feet of compacted fill and bedrock of the Puente Formation.

Artificial compacted fill materials (**Map Symbol: Af**) were encountered in Borings H-3, H-4 and H-5. The compacted fill materials encountered generally consist of mottled olive brown to dark gray brown clayey sand and sandy clay that was damp to moist and very dense to hard, plastic, and locally iron-stained. Although the grading report by GeoSoils (1992a) did not include removal bottom elevations, it did include some isolated fill thickness values. We modeled the limits and depths of compacted fill from the fill thickness values by GeoSoils and by comparing the original topography with their recommended remedial measures (GeoSoils, 1989). The areas of fill are illustrated on the Geotechnical Map (Plate 1) and Cross-Sections A-A' and B-B' (Plate 2).

The southern and southwestern portion of the site consists of sedimentary bedrock of the Puente Formation, Sycamore Canyon Member (**Map Symbol: Tpsc**). This late Miocene-aged marine formation was encountered in all our borings and typically consists of interlayered sandstone and siltstone/claystone. The sandstone was found to be light brown, damp, very dense, micaceous and locally friable. The clayey siltstone/silty claystone was generally dark gray to black, damp, hard, micaceous, slightly plastic with some visible layers dipping 25 to 35 degrees.

Some geologic mapping was performed during the prior grading operations (GeoSoils, 1992a), including limits of the earth units described above, as well as local lenses of terrace deposits (**Map Symbol: Qt**) consisting of red-brown clay, silt, sand and gravel. They also mapped locations of faults, clay beds, groundwater seepage (and subdrains), and bedding generally dipping 15 to 48 degrees to the northwest.

2.2 Groundwater

Groundwater was not encountered during our exploration to a maximum depth of 25.8 feet bgs and groundwater was not encountered during the initial site exploration to depths of 100 feet below original ground surface (MAW, 1979 and GeoSoils, 1989). During rough grading of the site in the early 1990s, considerable nuisance seepage was observed and mapped in the design

cut area in the southwest portion of the site – specifically south of the movie theater, within South Festival Drive and within the residential development to the south. Based on review of GeoSoils (1992a), the rough grading included over 50 feet of design cut in this area and several clay beds, as well as a fault were mapped in the keyway immediately south of Festival Drive. Upon completion of rough grading and prior to development of the movie theater, GeoSoils documented installation of an elaborate subdrain system, including five groundwater pumping wells that outlet into an assumed storm drain system in the northwest portion of the site (GeoSoils, 1992b). This subdrain system is depicted on Plate 1.

The historic high groundwater mapping by California Division of Mines and Geology (CDMG, 1997 and 2000) indicates high groundwater at the site is greater than 40 feet deep.

2.3 Regional Faulting, Seismicity and Seismic Hazards

The subject site is not located within a fault-rupture hazard zone as defined by the Alquist-Priolo Special Studies Zone Act (CGS, 2018). No active faults are mapped at the site (Jennings and Bryant, 2010) and no evidence of active faulting was observed during prior studies (Appendix A) or this exploration. The potential for primary ground rupture at the site is considered low. The primary seismic hazard at the subject site is ground shaking due to a future earthquake on a regional active fault.

Using the USGS deaggregation computer program (USGS, 2022) and the site coordinates of 33.8652 degrees north latitude and 117.7512 degrees west longitude, the closest major active faults to the site are the Peralta Hills Fault located 1.7 miles southwest of the site and the Whittier Fault located 1.9 miles to the northeast; with the Whittier Fault the controlling fault for seismicity. The maximum moment magnitude (M_w) for the controlling fault is 7.5, which would be generated from the Whittier Fault.

Properties in southern California are subject to seismic hazards of varying degrees depending upon the proximity, degree of activity, and capability of nearby faults. These hazards can be primary (i.e., directly related to the energy release of an earthquake, such as surface rupture and ground shaking) or secondary (i.e., related to the effect of earthquake energy on the physical world which can cause phenomena, such as liquefaction and ground lurching).

The subject site is not located within a potential liquefaction hazard zone (CDMG, 1998 and 2001). Based on site specific geotechnical exploration and considering the site is underlain by very dense to hard artificial fill and sedimentary bedrock, as well as the absence of groundwater, the potential for seismic liquefaction at the subject site is considered very low. Tsunami and seiche are not considered hazards at the site due to the elevation and distance from the ocean or any substantial confined body of water.

2.4 Engineering Characteristics of Onsite Soils

The following includes a summary of the subsurface geotechnical conditions based on the boring logs and laboratory test results performed on in-situ samples (Appendix C).

Soil Properties: Grain-size distribution tests were conducted on two near-surface samples. The fines content (passing No. 200 sieve) varied from 35 to 40 percent. The plasticity indices (PI) for these samples were 12 and 19, respectively. In general, the near-surface artificial fill soils encountered during our exploration were classified as clayey sand (SC), silty sand (SM) and low plasticity sandy silty clays (CL).

Shear Strength: Direct shear testing conducted on two relatively undisturbed ring samples of the artificial fill resulted in both peak and ultimate internal friction angles of 30 degrees with cohesions of 1,150 and 650 pounds per square foot (psf). A sandstone bedrock sample test resulted in a peak strength of 42 degrees with 300 psf cohesion and an ultimate friction angle of 36 degrees

Consolidation: Consolidation tests were performed on four relatively undisturbed ring samples of the artificial fill from depths of 5 and 10 feet. The samples have very low to low consolidation potential with no hydro-collapse potential.

Expansion Potential: Two expansion potential tests on bulk samples collected from 1 to 5 feet below ground resulted in indices of 26 and 51, which corresponds to "low" to "medium" potential.

Soluble Sulfate Content and Soil Corrosivity: The soluble sulfate exposure of two bulk samples from the upper 5 feet was classified as "S0 to S1" per Table 19.3.1.1 of ACI-318-14. Saturated resistivities ranged from 800 to 880 ohm-cm ("severely corrosive" to ferrous metals). pH levels were near normal and chloride contents were relatively low (less than 140 ppm).

2.5 Settlement Potential

Significant increases in grades above existing grades are not anticipated for the proposed project. Therefore, post-construction settlements of structures will be based on the foundation loads of those structures and the consolidation properties of the earth materials at the site. Large areas of the planned structure footprints have bedrock at the surface or relatively thin fills over bedrock. The heaviest proposed structure, the parking structure, is currently planned over an area with bedrock at the surface. Therefore, the settlement of that structure is expected to be relatively minor.

Before site development, a relatively broad drainage canyon traversed the site in a generally north-south direction. The canyon subdrain installed during site rough grading is shown on the Geotechnical Map and represents the bottom trace of this canyon. The deepest fill within the proposed building footprint is on the order of 35 feet deep and located within this filled-in canyon. The apartment structure to the east of the planned parking structure will straddle this canyon fill as illustrated on Cross-Section B-B'. Fortunately, the artificial fill at the site is medium dense/dense and stiff/hard with relatively low consolidation potential based on our drilling and laboratory testing. The underlying bedrock is also dense to hard and may be considered essentially incompressible under the anticipated structural loads.

We performed feasibility level settlement analyses of the canyon fill assuming 6 to 6.5-foot square column footings with loads ranging from 100 to 150 kips. Using software-based Westergaard and Boussinesq stress transfer methods, the resulting total settlements ranged from $\frac{1}{2}$ to $\frac{3}{4}$ inch. Foundations founded on fills of 5 feet or less or on the bedrock will undergo negligible post-construction settlement. We estimate that differential settlements between columns in the vicinity of the deeper canyon fills would be on the same order of magnitude as the total settlements ($\frac{1}{2}$ to $\frac{3}{4}$ inch).

Potential settlement induced by earthquakes (seismic settlement) should be negligible due to the density of the engineered fill and bedrock underlying the site.

2.6 Infiltration Feasibility

Based on the data we have compiled to date, infiltration or percolation of storm water into the subsurface is not considered feasible. Although the regional groundwater table is very deep, nuisance seepage needed to be mitigated at the site and the pad consists entirely of very dense bedrock and compacted fill. Furthermore, any infiltration system would need to be deep enough to not adversely impact the adjacent retail site to the south (which, due to the depth, would be founded in bedrock).

2.7 Existing Improvements

There are several existing improvements within and along the perimeter of the site. The perimeter gravity walls along both the northerly and southerly boundary are up to approximately 15 feet high. These walls were designed and constructed based on the current site configuration. The existing asphalt pavement parking lot consists of 2.5 to 5 inches of asphalt concrete (AC) over 3 to 5 inches of aggregate base (AB). Utility line manholes and catch basins, including storm drain, sewer, water, and electric vaults, were observed along the drive aisles, near the toe-of-slope and along the perimeter of the site. Also, five groundwater pumping wells are located along the rear of the existing movie theater.

2.8 Earthwork Shrinkage, Bulking and Subsidence

The loss or gain of volume (shrinkage or bulking, respectively) of excavated natural materials and re-compaction as fill varies according to earth material type and location. This volume change is represented as a percentage shrinkage (volume loss) and as a percentage bulking (volume gain) after re-compaction of a unit volume of cut in this same material in its natural state. We anticipate the compacted fill materials may shrink up to 1 percent or bulk up to 1 percent (essentially they should have no volume change) and the bedrock materials may bulk on the order of 2 to 5 percent. Also, due to prior site use, ground subsidence at the site is anticipated to be negligible.

3.0 CONCLUSIONS AND PRELIMINARY RECOMMENDATIONS

3.1 General Conclusions and Recommendation

Based on the results of our subsurface exploration and feasibility study, construction of the proposed improvements, as described herein, is considered geotechnically feasible. Limited additional exploration, soil testing, and analyses may be required once the site plan with structure type and structure locations are finalized in order to provide more detailed design level recommendations.

The preliminary recommendations in this report are provided for broad feasibility and planning purposes. More detailed recommendations for design and construction will be provided following additional exploration (as needed), testing, and analysis, based on the final grading structure design information. These preliminary recommendations are considered minimum and may be superseded by more restrictive requirements of others. In addition to the following recommendations, General Earthwork and Grading Specifications are provided in Appendix E.

3.2 Protection of Existing Improvements and Utilities

Existing buildings, improvements and utilities adjacent to the site that are to be protected in-place should be located and visually marked prior to demolition and grading operations. Excavations adjacent to improvements to be protected in-place or any utility easement should be performed with care so as not to destabilize the adjacent ground. Utility lines that are to be abandoned (if any) should be removed and the excavation should be backfilled and compacted in accordance with the recommendations provided herein.

Stockpiling of soils (more than 5 feet in height) at or near existing structures and over utility lines that are to remain in-place (if any) should not be allowed without review by the geotechnical consultant and the structure/utility line owner(s).

We recommend that the conditions (fascia block, drainage devices, etc.) of the existing gravity walls along the perimeter of the site be visually assessed prior to and during demolition, grading and construction.

3.3 Demolition and Site Preparation

We anticipate the existing theater building has shallow spread footing foundations; hence, no removals of deep foundations are anticipated for that building. Significant amounts of concrete and asphalt concrete will be generated from site demolition. Portland Cement Concrete (PCC) and AC are likely to be suitable to be re-used as a crushed miscellaneous base product should crushing operations be feasible at the site. We anticipate the five pumping units south of the movie theater will also be abandoned. These should be completely removed along with the associated piping (electrical, drain lines, etc.) and disposed of offsite. Recommendations for additional subdrainage are provided in Section 3.5.

Prior to grading, the site should be cleared of deleterious materials (including vegetation, concrete, AC and existing utility pipelines to be demolished) and disposed of offsite.

3.4 Grading and Earthwork

General: Grading and excavations should be performed in accordance with the requirements of the City of Anaheim and NMG's General Earthwork and Grading Specifications provided in Appendix E.

Remedial Removals/Overexcavation: The recommended geotechnical remedial removals across the site include removal of the near-surface disturbed soils following completion of demolition operations and overexcavation of the bedrock. The anticipated depth of removals in existing compacted fill areas is generally on the order of 1 to 2 feet below existing grades. In areas near prior trees and bushes, the removals may need to be deeper in order to remove the roots/root balls. Deeper excavations, if any, during the remedial removal operations should be laid-back at a minimum of 1.5H:1V inclination to reduce the potential for failures near the property lines.

The majority of the site is anticipated to expose sandstone and siltstone bedrock at finish grade. Due to the potential for variations in the bedrock lithology, we recommend a minimum 3-foot overexcavation and recompaction of the bedrock materials below the future building pads to provide more uniform subgrades for slabs-on-grade and foundations. Within courtyards, drive aisles and surface parking, overexcavation is not deemed necessary; however, the upper 6 inches of subgrade should be processed as outlined below. Although very dense and locally cemented, we anticipate the bedrock materials to be rippable with conventional earthmoving equipment.

As mentioned earlier, we understand that the majority of the utility pipelines will be abandoned. Following clearance of the existing utility lines, the remedial removals should include removal of remnants of pipe lines/conduits and backfill soils, along with disturbed soils to expose competent native soils and replacement with compacted fill.

Fill Placement: The soils exposed at the bottom of the remedial removals and excavations should be observed and accepted by the geotechnical consultant prior to placement of fill materials. After the remedial removals and overexcavation, the bottoms should be scarified and moisture-conditioned prior to placement of fill. Onsite soil materials that are relatively free of organic materials and debris are considered suitable to be used as fill materials below the building slabs and footings. The soils should be mixed to provide a uniform blend of material. Fill should be placed in nearly horizontal loose lifts of approximately 8 inches in thickness, moisture-conditioned to 2 to 3 percent above optimum moisture content, and compacted to a minimum relative compaction of 90 percent (per ASTM D1557). We recommend that fills in the upper 18 inches be placed at 1.2 times optimum moisture content in order to mitigate the moderately expansive soils at the site and to expedite construction. Fills placed against ground sloping more than 5H:1V should be keyed and benched into competent material as the new fill is placed.

3.5 Groundwater and Subdrainage

The outlet for the subdrain system located in the western portion of the site should be located in the field and the functionality should be assessed. Future remedial earthwork in the area will include overexcavation of bedrock, which will likely include removal of portions of the subdrain system. We also anticipate the groundwater pumping units will be demolished/abandoned. If encountered and/or broken, the subdrain system should be repaired and/or removed.

We also recommend an additional subdrain be installed along the southern perimeter of the site due to the site geological conditions, including the mapped faults and clay beds to the south. The subdrain should consist of a 4-foot-deep trench excavation with 4-inch-diameter Schedule 40 PVC pipe surrounded by 3 cubic feet of $\frac{1}{2}$ - to $\frac{3}{4}$ -inch crushed rock and wrapped in filter fabric (Mirafi 140N or approved equivalent). The subdrain should be installed such that it flows a minimum of 1.5 percent to a suitable outlet device (catch basin or storm drain pipeline).

3.6 Settlement

From the settlement analysis described in Section 2.5, and assuming the proposed site development as shown on Plate 1, total post-construction settlement of structures is anticipated to be on the order of 1 to 1.5 inches. Maximum differential static settlements should be on the order of $\frac{1}{2}$ -inch and $\frac{3}{4}$ -inch over a span of 30 feet. Note that these maximum settlements should be isolated to the portions of buildings that straddle the area of deeper canyon fill (eastern half of the site). Seismic settlement is not expected to be significant.

Additional settlement analyses may be necessary once grading and structural designs are more complete and if additional exploration and laboratory testing is performed.

3.7 Building Foundations and Slabs

The slab and foundations should be designed by the project structural engineer based on the proposed structure type and the anticipated loading conditions. As mentioned in the prior settlement section, shallow spread footings should be feasible for the parking structure provided the structure is not located over the deeper canyon fill. If it is, shallow foundations may still be feasible for the parking structure but with some remedial measures.

The near-surface soils at the site have "low" to "medium" expansion potentials; therefore, the apartment buildings will require either a post-tension slab-on-grade system or a slab design per the Wire Reinforcement Institute method.

The recommended net allowable bearing capacities for foundations on compacted fill is expected to start at 2,000 psf with increases up to 4,000 psf. Bearing capacities for foundation on the existing bedrock should range from 4,000 to 5,000 psf.

We understand that partial subterranean parking is being considered. The local groundwater table is deep within the vicinity of the parking structure and no special subdrainage or slab design related to groundwater will be necessary.

The current building footprints along north side of the site are sufficiently far away from the existing gravity retaining wall and therefore no foundation setbacks related to that wall or other significant slopes is anticipated.

3.8 Preliminary Seismic Design Parameters

The following table summarizes the seismic design criteria for the subject site. These seismic design parameters are developed in accordance with ASCE 7-16 and 2019 CBC. The seismic response coefficient, C_s , should be determined per the parameters provided below and using equation 12.8-2 of ASCE 7-16.

Please note, as of January 1, 2023 the 2022 CBC will be adopted by local governing agencies. The updated code will include ASCE7-16 Supplements 1 through 3, which includes either site specific ground motion hazard analysis or a 50 percent increase for the value of S_{M1} and the subsequent applications of S_{M1} (which specifically include S_{D1} and T_s) for all sites considered to be Site Class D. The following values do not include the increase to S_{M1} ; however, updated seismic design parameters should be provided during the design phase of the project.

<i>Selected Seismic Design Parameters from 2019 CBC/ASCE 7-16</i>	<i>Seismic Design Values</i>	<i>Reference</i>
Latitude	33.8652 North	
Longitude	117.7512 West	
Controlling Seismic Source	Whittier Fault	USGS, 2022
Distance to Controlling Seismic Source	1.9 mi	USGS, 2022
Site Class per Table 20.3-1 of ASCE 7-16	D	SEA/OSHPD, 2022
Spectral Acceleration for Short Periods (S_s)	1.84 g	SEA/OSHPD, 2022
Spectral Accelerations for 1-Second Periods (S_1)	0.65 g	SEA/OSHPD, 2022
Site Coefficient F_a , Table 11.4-1 of ASCE 7-16	1.0	SEA/OSHPD, 2022
Site Coefficient F_v , Table 11.4-2 of ASCE 7-16	1.7	
Design Spectral Response Acceleration at Short Periods (S_{DS}) from Equation 11.4-3 of ASCE 7-16	1.23 g	SEA/OSHPD, 2022
Design Spectral Response Acceleration at 1-Second Period (S_{D1}) from Equation 11.4-4 of ASCE 7-16	0.74 g	
T_s , S_{D1} / S_{DS} , Section 11.4.6 of ASCE 7-16	0.60 sec	
T_L , Long-Period Transition Period	8 sec	SEA/OSHPD, 2022
Peak Ground Acceleration Corrected for Site Class Effects (PGA_M) from Equation 11.8-1 of ASCE 7-16	0.86 g	SEA/OSHPD, 2022
Seismic Design Category, Section 11.6 of ASCE 7-16	D	

3.9 Soil Corrosivity and Soluble Sulfate Contents

Existing soil is classified as "severely corrosive" to ferrous metals in contact with soil. Soluble sulfates were found to be in the S0 to S1 classification. Additional soil testing is recommended to determine whether the S0 classification may be more prevalent at the site.

3.10 Pavements

The following pavement sections should be considered preliminary. Actual pavement sections should be designed based on laboratory R-value testing after re-grading of the site.

3.10.1 Non-vehicular Pavements

As the onsite soils are, at worst, moderately expansive, non-vehicular and non-structural PCC slabs may be 4 inches thick, unreinforced, and with no subbase required. Moderate pre-saturation of such flatwork subgrade soil will be recommended.

3.10.2 Vehicular Pavements

Onsite soils are a mixture of sands, silts and clays and are therefore considered "fair" in terms of vehicular pavement subgrades. The existing pavement sections encountered in the borings for this study were consistently 4 to 5 inches of AC over 4 to 5 inches of AB. Future AC pavements sections are expected to be of this same general magnitude for the planned residential development.

PCC pavements should range from 4 to 5 inches thick with nominal reinforcements (No. 3 rebar, 24 inches on-center). Trash enclosure pavements should be thicker with heavier reinforcement.

3.11 Temporary Excavations and Shoring

The majority of the proposed parking structure is anticipated to expose bedrock. If one-half to one level of subterranean parking is planned, temporary excavations should be laid back at 1H:1.5V slope ratios in order to account for potential adverse bedding conditions. For vertical temporary excavations, cantilever soldier pile or sheet pile walls should be feasible and parameters should be provided during the future design phase.

3.12 Utility Installation and Trench Backfill

Excavations should conform to the latest edition of OSHA requirements (shoring or layback of trench or excavation walls). The near-surface soils across most of the site are anticipated to be classified as Type C for CalOSHA trenching and shoring excavation requirements, due to the potential for adverse bedding conditions.

The published OSHA shoring design systems may be used for conventional shoring excavations less than 20 feet in depth. Soil loadings are provided below that do not include the effects of the additional loads from other surcharges. The geotechnical consultant should review the conditions during the deeper excavation and installation of shoring. Care should be taken at all times by personnel and/or equipment operators working adjacent to the excavations.

Native soils should generally be suitable for use as trench backfill. Backfill materials should be compacted to a minimum relative compaction of 90 percent (per ASTM D1557). We recommend that moisture content of native backfill be over optimum moisture content. Native soils that are

found to be wet will require reprocessing (e.g., mixing or drying) to achieve the uniform moisture content. Select backfill may be used in lieu of native soils.

Special bedding material may be required for trenches that encounter wet and/or soft soils at pipe depth. This will include overexcavating the trench 12 inches and providing the trench bottom with an additional layer of compacted crushed rock to create a firm bottom. Specific bedding and shading requirements for the pipelines should be provided by the governing agency.

If high-density polyethylene (HDPE) pipe is proposed for the development, then excavation, installation, bedding, shading and backfilling should be in strict accordance with the project and manufacturer's requirements. HDPE pipe has specific requirements for the width of the trench excavation. Also, HDPE pipe requires appropriate bedding and compaction of select granular backfill material in the pipe zone in order to provide uniform and adequate support. The initial backfill in the pipe zone (haunch) needs to be properly placed around the HDPE pipe, distributed by shovel to provide uniform support, "knifed-in" to remove voids and tamped for compaction.

3.13 Additional Geotechnical Exploration

The preliminary recommendations herein are based on the proposed residential design as depicted on Plate 1. Future modifications to the conceptual design and the rough grading plan should be reviewed by the geotechnical consultant. Additional geotechnical exploration may be needed at the site to better assess the subsurface soil conditions, once the actual development plans are prepared. Additional laboratory testing and engineering analysis may also need to be performed for the final design.

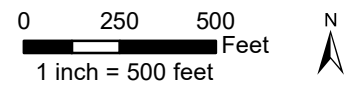
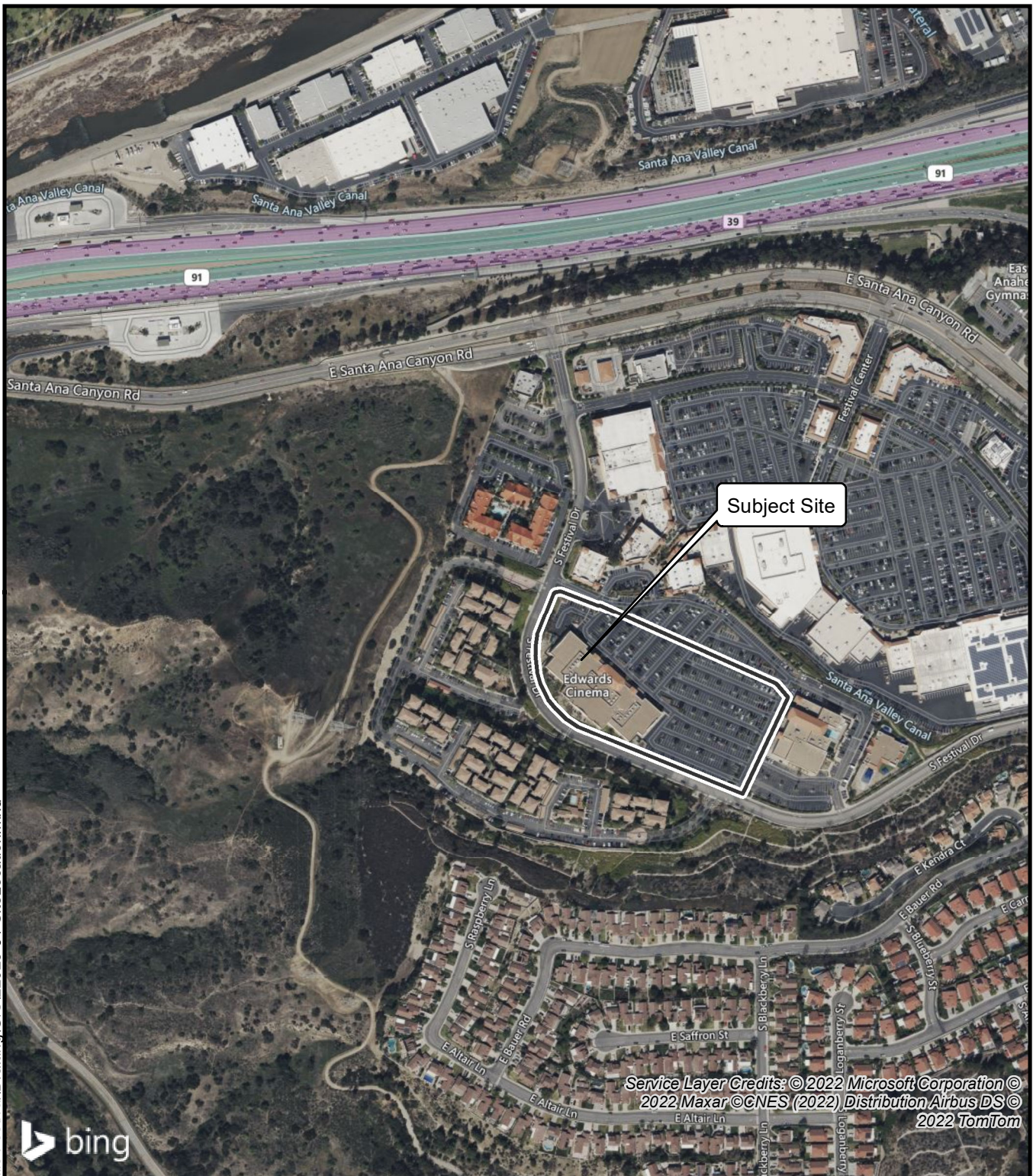
4.0 LIMITATIONS

This report has been prepared for the exclusive use of our client, Shea Properties for the specific residential project proposed at the Anaheim Hills retail center. This report or its contents should not be used or relied upon for other projects or purposes or by other parties without the written consent of NMG and the involvement of a geotechnical professional. The means and methods used by NMG for this study are based on local geotechnical standards of practice, care, and requirements of governing agencies. No warranty or guarantee, express or implied is given.

The findings, conclusions, and recommendations herein are professional opinions based on interpretations and inferences made from geologic and engineering data from specific locations and depths, observed or collected at a given time. By nature, geologic conditions can vary from point to point, can be very different in between points, and can also change over time. Our conclusions and recommendations are subject to verification and/or modification during excavation and construction when more subsurface conditions are exposed.

NMG's expertise and scope of services did not include assessment of potential subsurface environmental contaminants or environmental health hazards.

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SITE LOCATION MAP

FESTIVAL HILLS
CITY OF ANAHEIM, CALIFORNIA

Project Number: 22023-01

By: TM/LY

Project Name: Shea Prop/Anaheim Hills Festival

Date: 11/23/2022

Figure 1



P:\2022\22023-01 Shea Prop Anaheim Hills Festival\Drafting\GIS\22023-01 SeismicHazards.mxd



Legend

- Earthquake-Induced Landslide Zones
- Liquefaction Zones

0 250 500
Feet
1 inch = 1,000 feet



SEISMIC HAZARDS AND FAULT ZONES MAP

Base: California Geological Survey, Earthquake Zones of Required Investigation, Black Star Canyon and Orange Quadrangles
Dated: January 17, 2001 and April 15, 1998

FESTIVAL HILLS
CITY OF ANAHEIM, CALIFORNIA

Project Number: 22023-01

By: TM/LY

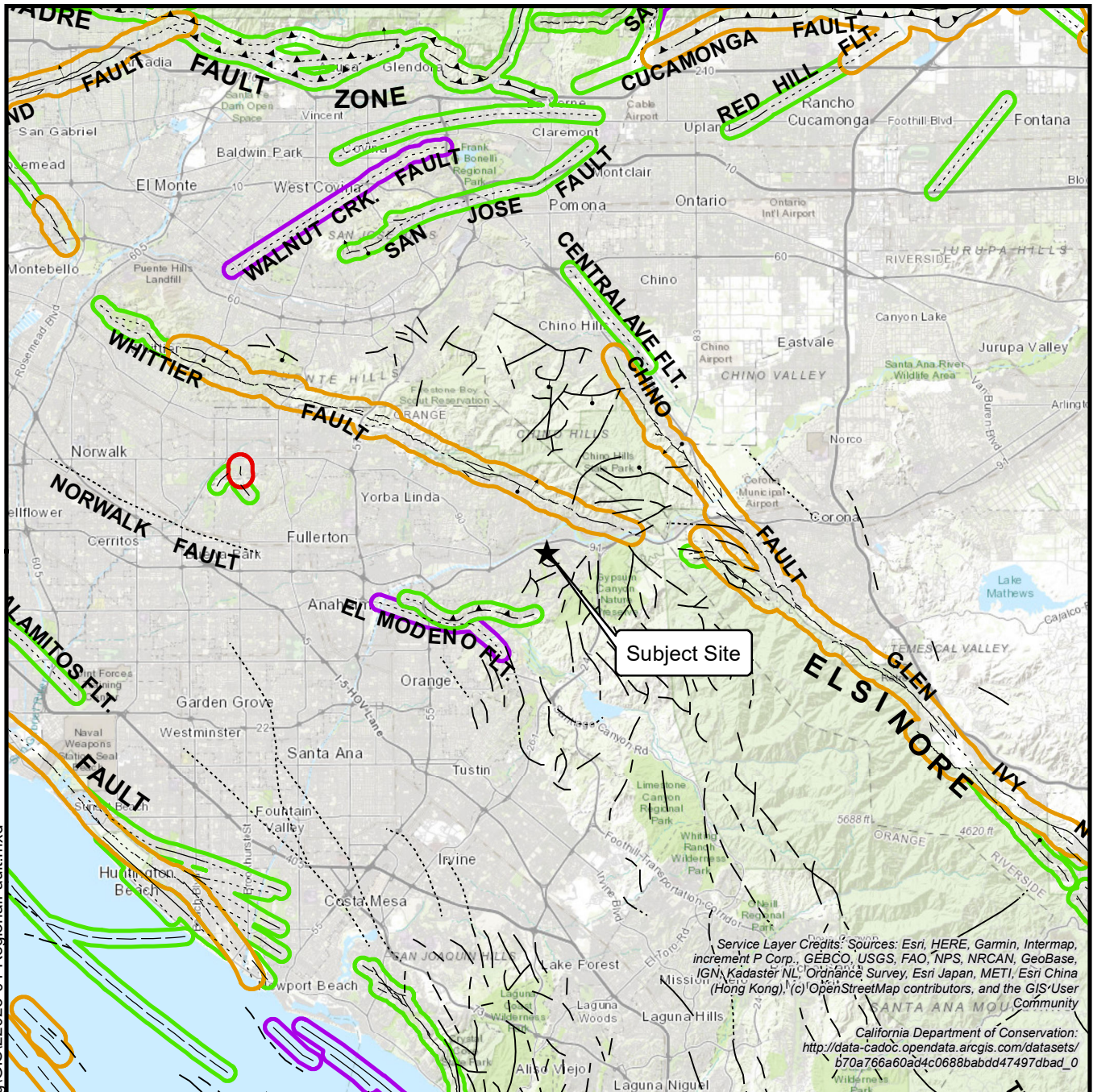
Project Name: Shea Prop/Anaheim Hills Festival

Date: 11/23/2022

Figure 2



P:\2022\2023-01 Shea Prop Anaheim Hills Festival\Drafting\GIS\2023-01 RegionalFault.mxd



Legend

Faults

- Certain
- - Approximately Located
- Concealed

Reactivity of Movement

- Historic
- Holocene
- Late Quaternary
- Quaternary

0 3 6 Miles
1 inch = 6 miles



REGIONAL FAULT MAP

Base: California Geological Survey, Fault Activity Map of California, 2010

FESTIVAL HILLS
CITY OF ANAHEIM, CALIFORNIA

Project Number: 22023-01

By: TM/LY

Project Name: Shea Prop/Anaheim Hills Festival

Date: 11/23/2022

Figure 3





LEGEND

EARTH UNITS - CIRCLED WHERE BURIED

Af ARTIFICIAL FILL, PLACED UNDER OBSERVATION AND TESTING OF GEOSOLS, INC. (1992a) APPROXIMATE LIMITS HIGHLIGHTED IN GREEN
Qt TERRACE DEPOSIT
Tpssc PUENTE FORMATION, SYCAMORE CANYON MEMBER

SYMBOLS - LOCATIONS ARE APPROXIMATE, QUERIED WHERE UNCERTAIN

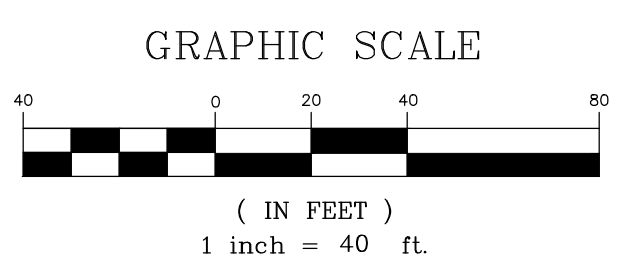
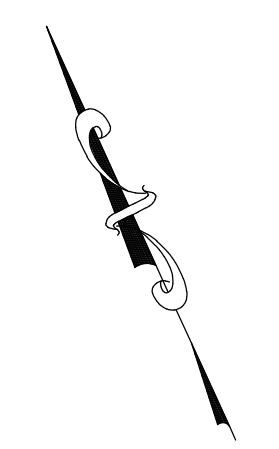
--- GEOLOGIC CONTACT, DOTTED WHERE BURIED
--- FAULT, DOTTED WHERE BURIED
--- LIMITS OF EXISTING FILL
--- CLAY BED (BURIED)
--- GEOLOGIC ATTITUDES- LOCATIONS ARE APPROXIMATE, DASHED WHERE BURIED
--- BEDDING
--- FAULT/SHEAR SURFACE ATTITUDE

OTHER SYMBOLS - LOCATIONS ARE APPROXIMATE

● **H-5** HOLLOW STEM AUGER BORING, THIS INVESTIGATION SHOWING TOTAL DEPTH, DEPTH TO EARTH UNITS AND DEPTH TO GROUNDWATER (5 TOTAL)
▲ **CPT-7** CONE PENETROMETER TEST, SHOWING TOTAL DEPTH
● **GB-21** BUCKET AUGER BORING BY GEO SOILS, INC. (1989), SHOWING TOP OF BORING ELEVATION, TOTAL DEPTH, DEPTH TO EARTH UNITS AND DEPTH TO GROUNDWATER (6 TOTAL)
● **B-10** BUCKET AUGER BORING BY MEDALL, ARAGON, WORSWICK & ASSOCIATES (1979), SHOWING TOP OF BORING ELEVATION, TOTAL DEPTH, DEPTH TO EARTH UNITS AND DEPTH TO GROUNDWATER (2 TOTAL)

OTHER SYMBOLS - CONTINUED

--- EXISTING SUBDRAIN WITH PIPE DIAMETER LABELED, ARROW INDICATES FLOW DIRECTION (GEOSOLS, 1992a, b, c)
--- GEOLOGIC CROSS-SECTION (2 TOTAL)
--- THICKNESS OF COMPACTED FILL (GEOSOLS, INC. 1992a)



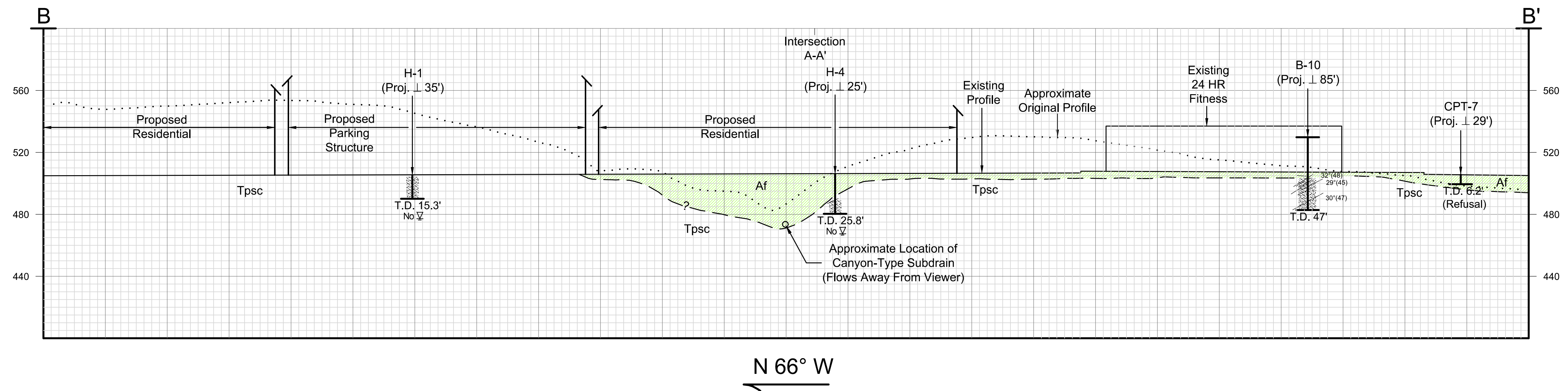
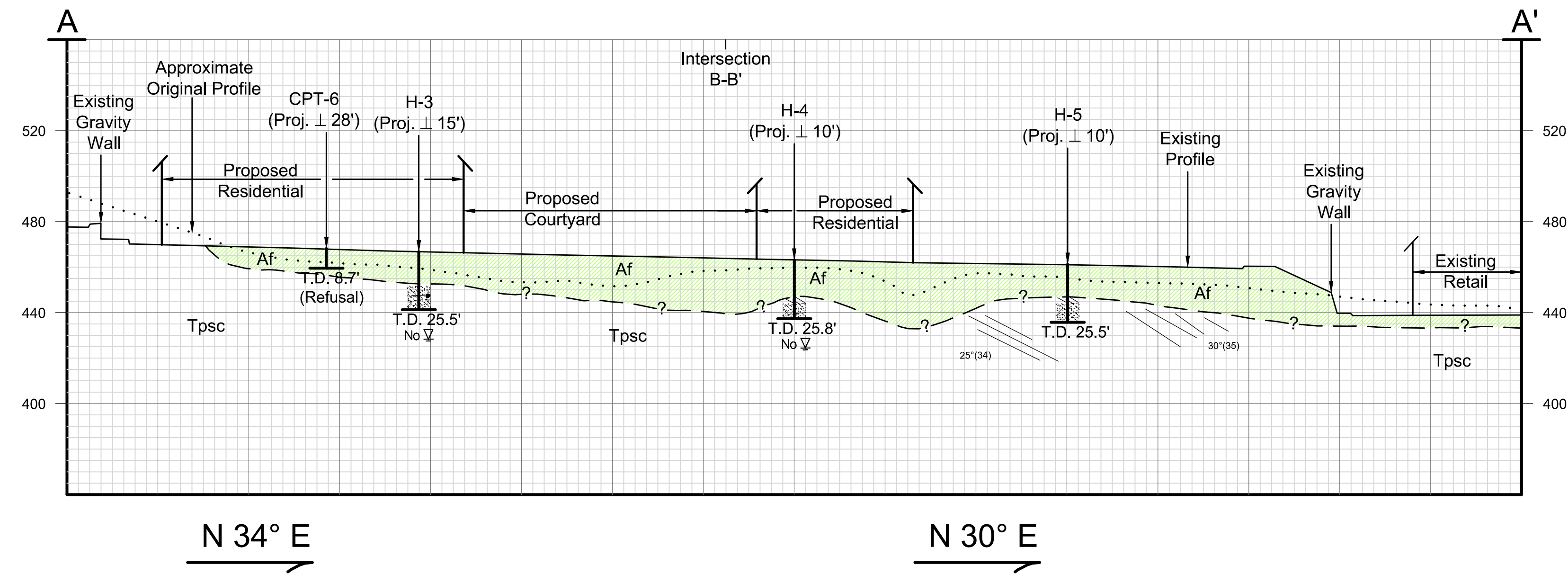
GEOTECHNICAL MAP
FESTIVAL HILLS
CITY OF ANAHEIM, CALIFORNIA

Project No.: 22023-01
Project Name: Shea Prop/Anaheim Hills Festival
Date: 11/23/2022

By: TM/LY
SCALE: 1" = 40'

NMG
Geotechnical, Inc.

Plate 1
Drawing: P:\2022\2023-01 Shea Prop Anaheim Hills Festival\Drafting\WP\2023-01 Plate.dwg
Last Saved: Wed Nov 23, 2022 - 10:50am
Last Plotted: Tue Nov 29, 2022 - 1:50pm
By: jschneider



APPENDIX A

APPENDIX A

REFERENCES

- California Division of Mines and Geology (CDMG), 1997, Seismic Hazard Zone Report for the Orange 7.5-Minute Quadrangle, California, Seismic Hazard Zone Report 011.
- California Division of Mines and Geology (CDMG), 1998, Seismic Hazard Zones, Orange Quadrangle, Official Map dated April 15, 1998.
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- GeoSoils, Inc., 1992d, Final Soils Compaction Report (Trench Backfills and Site Improvements), Anaheim Hills Festival Project, Anaheim Hills, California, Project No. 1993-C1, D1-OC, dated October 14, 1992.
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APPENDIX A

REFERENCES (Continued)

- Medall, Aragon, Worswick and Associates, Inc., 1980, Supplemental Geologic and Soils Studies, Phase I, Grading and Culvert Construction, Santa Ana Canyon Road Relocation, Bauer Ranch, Anaheim, California for Kaufman and Broad Homes, Inc., Project No. S1628-I, dated July 8, 1980.
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- Structural Engineers Association/Office of Statewide Health Planning and Development, 2022, U.S. Seismic Design Maps, web site address: <https://seismicmaps.org/>; Date Accessed: November 20, 2022.
- U.S. Geological Survey, 2022, Unified Hazard Tool, NSHM 2014 Dynamic Deaggregation Program; web site address: <https://earthquake.usgs.gov/hazards/interactive/>; Date Accessed: November 20, 2022.

APPENDIX B

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GP	POORLY GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT 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 GREATER THAN 50			MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
			HIGHLY ORGANIC SOILS		

NOTE: Dual symbols are used to indicate gravels or sand with 5-12% fines and soils with fines classifying as CL-ML. Symbols separated by a slash indicate borderline soil classifications.

Sampler and Symbol Descriptions

- Modified California sample (D-#)
 - Standard Penetration Test (S-#)
 - Shelby tube sample (T-#)
 - Large bulk sample (B-#)
 - Small bulk sample (SB-#)
 - Approximate depth of groundwater during drilling
 - Approximate depth of static groundwater
- Note: Number of blows required to advance driven sample 12 inches (or length noted).

Laboratory and Field Test Abbreviations

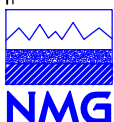
- AL** Atterberg limits (plasticity)
- CC** Chemical Testing incl. Soluble Sulfate
- CN** Consolidation
- DS** Direct Shear
- EI** Expansion Index
- GS** Grain Size Analysis (Sieve, Hydro. and/or -No. 200)
- MD** Maximum Density and Optimum Moisture
- RV** Resistance Value (R-Value)
- SE** Sand Equivalent
- UU** Unconsolidated Undrained Shear Strength

GENERAL NOTES

- Soil classifications are based on the Unified Soil Classification System and include color, moisture, and relative density or consistency. Field descriptions have been modified to reflect results of laboratory tests where deemed appropriate. Bedrock descriptions are based on visual classification and include rock type, moisture, color, grain size, strength, and weathering.
- Descriptions on these boring logs apply only at the specific boring locations and at the time the borings were drilled. They are not warranted to be representative of subsurface conditions at other locations or times.

KEY TO LOG OF BORING

Shea/ Anaheim Hills Festival
Anaheim Hills, CA
PROJECT NO. 22023-01

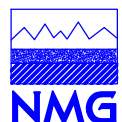


Geotechnical, Inc.

Date(s) Drilled	10/26/22	Logged By	BF	H-1 Sheet 1 of 1
Drilling Company	2R Drilling, Inc.	Drill Bit Size/Type	8"	
Drill Rig Type	CME 75 Hollow Stem	Hammer Data	140 lbs. @ 30 Inch Drop	
Sampling Method(s)	Modified California, Bulk			
Approximate Groundwater Depth: No Groundwater Encountered.				Total Depth Drilled (ft)15.3
Comments				Approximate Ground Surface Elevation (ft)464.0 msl

Elevation (ft)	SAMPLES			Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	Type	Number	Blows per foot						
0						Surface: Parking lot, 2.5" of asphaltic concrete over 3" of aggregate base.			
					SANDSTONE	Puente Formation, Sycamore Canyon Member (Tpsc) @ 0.5': Light yellowish brown (2.5Y 6/3) fine SANDSTONE, damp, very dense, micaceous, trace silt, slightly friable.			B-1 @ 1-5'
-460		B-1							
5		D-1	50/6"			@ 5': Light yellowish brown (2.5Y 6/4) fine SANDSTONE, damp, very dense, micaceous, slightly friable.	7.5	117.8	DS
10		D-2	50/5"			@ 10': Light yellowish brown (2.5Y 6/4) fine SANDSTONE, damp, very dense, micaceous, slightly friable.	6.2	105.5	
-450									
15		D-3	50/4"			@ 15': Light yellowish brown (2.5Y 6/4) fine SANDSTONE, damp, very dense, micaceous, slightly friable.	11.6		Disturbed
20						Notes: Total Depth: 15.3 Feet. No Groundwater Encountered. Backfilled with Cuttings and Tamped. Patched with Quickset Concrete and Black Dye.			
-440									
25									

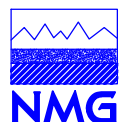
LOG OF BORING
Shea/ Anaheim Hills Festival
Anaheim Hills, CA
PROJECT NO. 22023-01



Date(s) Drilled	10/26/22	Logged By	BF	H-2 Sheet 1 of 1
Drilling Company	2R Drilling, Inc.	Drill Bit Size/Type	8"	
Drill Rig Type	CME 75 Hollow Stem	Hammer Data	140 lbs. @ 30 Inch Drop	
Sampling Method(s)	Modified California, Bulk			
Approximate Groundwater Depth: No Groundwater Encountered.				Total Depth Drilled (ft)15.5
Comments				Approximate Ground Surface Elevation (ft)467.0 msl

Elevation (ft)	Depth (ft)	SAMPLES			Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
		Type	Number	Blows per foot						
0						SC	Surface: Parking lot, 4" of asphaltic concrete over 4" of aggregate base.			
							Artificial Fill (Af) @ 0.5': Dark olive brown (2.5Y 3/3) silty clayey SAND, damp, dense, micaceous, mottled.			
							Puente Formation, Sycamore Canyon Member (Tpsc) @ 1.5': Very dark gray to black (2.5Y 3/1 to 2.5/1) silty CLAYSTONE/ clayey SILTSTONE, damp, hard, micaceous.			B-1 @ 1-5'
	5	D-1		90/9"			@ 5': Black (2.5Y 2.5/1) clayey SILTSTONE/ silty CLAYSTONE, damp, hard, micaceous, laminated, slightly plastic.	19.0	98.7	
-460										
	10	D-2		50/6"			@ 10': Black (2.5Y 2.5/1) clayey SILTSTONE/ silty CLAYSTONE, damp, hard, micaceous, laminated, slightly plastic, beds dipping approximately 24°-34°.	18.5	96.9	
	15	D-3		50/6"			@ 15': Black (2.5Y 2.5/1) clayey SILTSTONE/ silty CLAYSTONE, damp, hard, micaceous, laminated, slightly plastic, beds dipping approximately 25°.	19.0	102.5	
-450										
							Notes: Total Depth: 15.5 Feet. No Groundwater Encountered. Backfilled with Cuttings and Tamped. Patched with Quickset Concrete and Black Dye.			
	20									
	25									

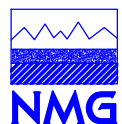
LOG OF BORING
 Shea/ Anaheim Hills Festival
 Anaheim Hills, CA
 PROJECT NO. 22023-01



Date(s) Drilled	10/26/22	Logged By	BF	H-3 Sheet 1 of 2
Drilling Company	2R Drilling, Inc.	Drill Bit Size/Type	8"	
Drill Rig Type	CME 75 Hollow Stem	Hammer Data	140 lbs. @ 30 Inch Drop	
Sampling Method(s)	Modified California, Bulk			
Approximate Groundwater Depth: Seepage at approximately 20 feet.				Total Depth Drilled (ft)25.5
Comments				Approximate Ground Surface Elevation (ft)465.0 msl

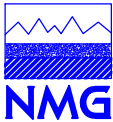
Elevation (ft)	Depth (ft)	SAMPLES			Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
		Type	Number	Blows per foot						
	0						Surface: Parking lot, 5" of asphaltic concrete over 5" of aggregate base.			
			B-1			SC-CL	Artificial Fill (Af) @ 0.8': Olive brown (2.5Y 4/4) silty clayey fine SAND to sandy CLAY, damp, dense/ hard, micaceous, trace gravel 1"-diameter, slightly plastic.			B-1 @ 1-5' CC, EI, GS
-460	5		D-1	81		CL	@ 5': Dark grayish brown (2.5Y 4/2) silty sandy CLAY, damp, hard, micaceous, small sand lenses, trace to few rootlets, FeO staining, slightly plastic.	13.2	116.0	CN, DS
	10		D-2	42			@ 10': Dark grayish brown (2.5Y 4/2) silty sandy CLAY, damp, hard, micaceous, small sand lenses, FeO staining, slightly plastic.	17.3	110.8	CN
-450	15		D-3	66		SAND STONE	Puente Formation, Sycamore Canyon Member (Tpsc) @ 15': Yellowish brown to dark yellowish brown (10YR 5/6 to 4/6) fine to medium SANDSTONE, very moist, very dense, micaceous, slightly friable.	13.7	117.8	
	20		D-4	50/6"			@ 20': Yellowish brown to dark yellowish brown (10YR 5/6 to 4/6) fine SANDSTONE, wet, very dense, micaceous, slightly friable.	20.2	107.1	
-440	25									

LOG OF BORING
 Shea/ Anaheim Hills Festival
 Anaheim Hills, CA
 PROJECT NO. 22023-01



Shea/ Anaheim Hills Festival			Anaheim Hills, CA			H-3	Sheet 2 of 2		
Elevation (ft)	Depth (ft)	SAMPLES		Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
		Type	Number						
440	25	D-5	50/6"		SANDSTONE	@ 25': Yellowish brown and gray (10YR 5/6 and 5Y 5/1) fine SANDSTONE, wet, very dense, micaceous, laminated, slightly friable, bedding dipping approximately 25°.	18.9	108.7	
						Notes: Total Depth: 25.5 Feet. Seepage at 20 Feet. Boring Caved in at 15 Feet. Backfilled with Cuttings and Tamped. Patched with Quickset Concrete and Black Dye.			
	30								
	35								
430									
	40								
	45								
420									
	50								
	55								
410									

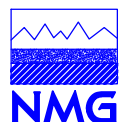
LOG OF BORING
Shea/ Anaheim Hills Festival
Anaheim Hills, CA
PROJECT NO. 22023-01



Date(s) Drilled	10/26/22	Logged By	BF	H-4 Sheet 1 of 2
Drilling Company	2R Drilling, Inc.	Drill Bit Size/Type	8"	
Drill Rig Type	CME 75 Hollow Stem	Hammer Data	140 lbs. @ 30 Inch Drop	
Sampling Method(s)	Modified California, Bulk			
Approximate Groundwater Depth: No Groundwater Encountered.				
Comments				Total Depth Drilled (ft)25.8
				Approximate Ground Surface Elevation (ft)463.0 msl

Elevation (ft)	Depth (ft)	SAMPLES			Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
		Type	Number	Blows per foot						
0							Surface: Parking lot, 4.5" of asphaltic concrete over 4.5" of aggregate base.			
-460		B-1				SC-CL	Artificial Fill (Af) @ 0.75': Light olive brown (2.5Y 5/3) silty clayey fine SAND to sandy CLAY, damp, medium dense/ very stiff, trace mica, few subangular gravel 1"-diameter, slightly plastic.			B-1 @ 1-5' CN
	5	D-1		42		SC	@ 5': Light olive brown (2.5Y 5/4) silty clayey fine SAND, damp, medium dense, micaceous, trace fine gravel, trace roots, slightly plastic.	9.8	112.6	
	10	D-2		43			@ 10': Light olive brown (2.5Y 5/4) silty clayey fine SAND, damp, medium dense, micaceous, small lenses of clean sand, slightly plastic.	11.8	117.0	
-450										
	15	D-3		90/9"		SILT STONE/ CLAY STONE	Puente Formation, Sycamore Canyon Member (Tpsc) @ 15': Dark greenish gray (GLEY 4/1) clayey SILTSTONE/ silty CLAYSTONE, moist, hard, trace mica, thinly bedded, FeO staining, plastic.	25.5	99.5	
	20	D-4		50/6"		SAND STONE	@ 20': Brownish yellow (10YR 6/6) fine SANDSTONE, moist, very dense, micaceous, mica beds, thinly bedded, friable to highly friable.	12.2	101.9	
-440										
	25									

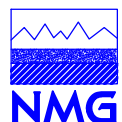
LOG OF BORING
 Shea/ Anaheim Hills Festival
 Anaheim Hills, CA
 PROJECT NO. 22023-01



Date(s) Drilled	10/26/22	Logged By	BF	H-5 Sheet 1 of 2
Drilling Company	2R Drilling, Inc.	Drill Bit Size/Type	8"	
Drill Rig Type	CME 75 Hollow Stem	Hammer Data	140 lbs. @ 30 Inch Drop	
Sampling Method(s)	Modified California, Bulk			
Approximate Groundwater Depth: No Groundwater Encountered.				Total Depth Drilled (ft)25.5
Comments				Approximate Ground Surface Elevation (ft)463.0 msl

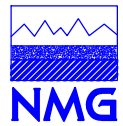
Elevation (ft)	Depth (ft)	SAMPLES			Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
		Type	Number	Blows per foot						
	0						Surface: Parking lot, 4" of asphaltic concrete over 4" of aggregate base.			
						SC-CL	Artificial Fill (Af) @ 0.7': Light olive brown (2.5Y 5/3) silty clayey fine SAND to sandy CLAY, damp, dense/ very stiff, roots at 3'.			B-1 @ 1-5' CC,EI,GS
-460		B-1								
	5	D-1		70		SM-SC	@ 5': Olive brown (2.5Y 4/3) clayey silty fine SAND to silty clayey fine SAND, damp, very dense, micaceous, trace subangular gravel 1"-diameter, trace rootlets, small lenses of clean sand.	12.7	106.0	
	10	D-2		47		CL	@ 10': Olive brown to dark olive brown (2.5Y 4/3 to 3/3) sandy silty CLAY, moist, hard, micaceous, trace subangular gravel 1/2"-diameter, small lenses of clean sand, FeO staining, plastic.	12.8	118.2	CN,DS
-450										
	15	D-3		50/6"		SANDSTONE/ SILTSTONE	Puente Formation, Sycamore Canyon Member (Tpsc) @ 15': Upper: Yellow to brownish yellow (10YR 7/6 to 6/6) fine SANDSTONE, damp, very dense, micaceous, highly friable. Lower: Gray (5Y 5/1) silty CLAYSTONE/ clayey SILTSTONE, damp, hard, laminated, plastic.	6.9	109.4	
	20	D-4		50/4"		SANDSTONE	@ 20': Light olive brown (2.5Y 5/4) fine SANDSTONE, damp, very dense, micaceous, highly friable.	5.2	108.3	
-440										
	25									

LOG OF BORING
 Shea/ Anaheim Hills Festival
 Anaheim Hills, CA
 PROJECT NO. 22023-01



Shea/ Anaheim Hills Festival			Anaheim Hills, CA			H-5	Sheet 2 of 2		
Elevation (ft)	SAMPLES			Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	Type	Number	Blows per foot						
25	D-5	50/6"			SILT STONE/ CLAY STONE	@ 25': Olive brown and very dark grayish brown (2.5Y 6/6 and 3/2) clayey SILTSTONE/ silty CLAYSTONE, damp, hard, micaceous, laminated, plastic.	19.9	97.5	
						Notes: Total Depth: 25.5 Feet. No Groundwater Encountered. Backfilled with Cuttings and Tamped. Patched with Quickset Concrete and Black Dye.			
30									
35									
40									
45									
50									
55									

LOG OF BORING
Shea/ Anaheim Hills Festival
Anaheim Hills, CA
PROJECT NO. 22023-01

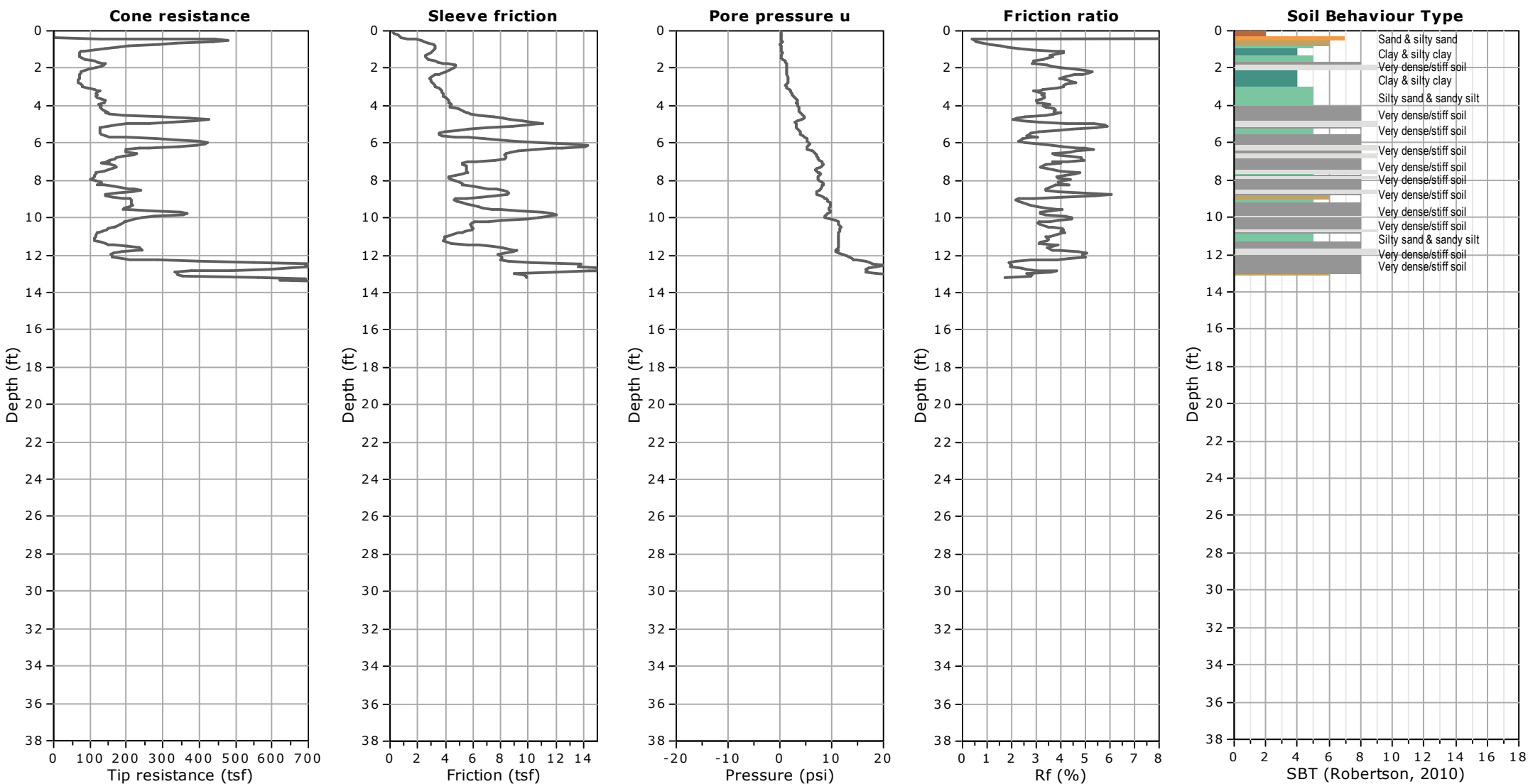




Project: NMG Geotechnical
Location: 300 S. Festival Dr, Anaheim, CA

CPT-1

Total depth: 13.58 ft, Date: 10/26/2022



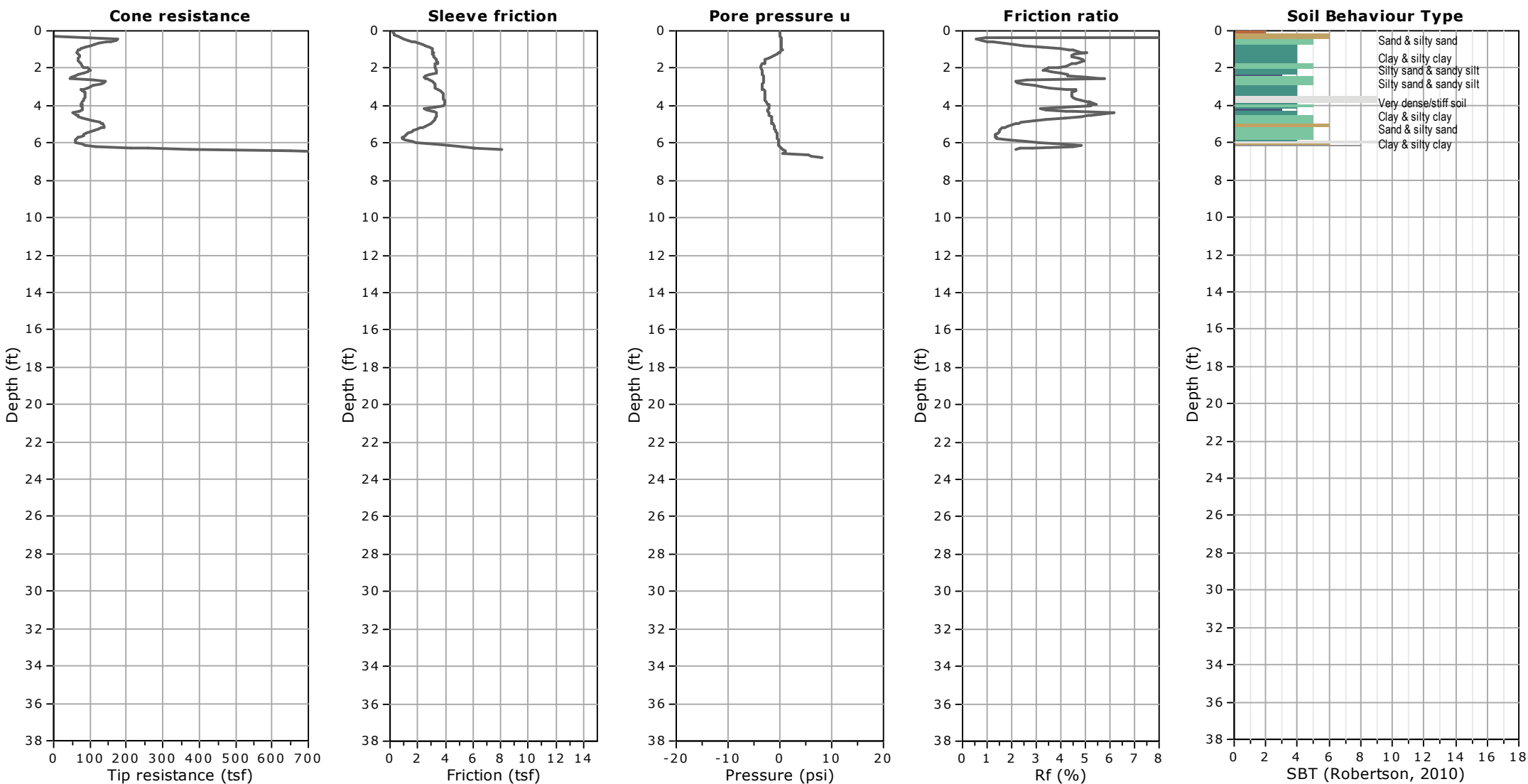


Project: NMG Geotechnical

Location: 300 S. Festival Dr, Anaheim, CA

CPT-2A

Total depth: 6.77 ft, Date: 10/26/2022



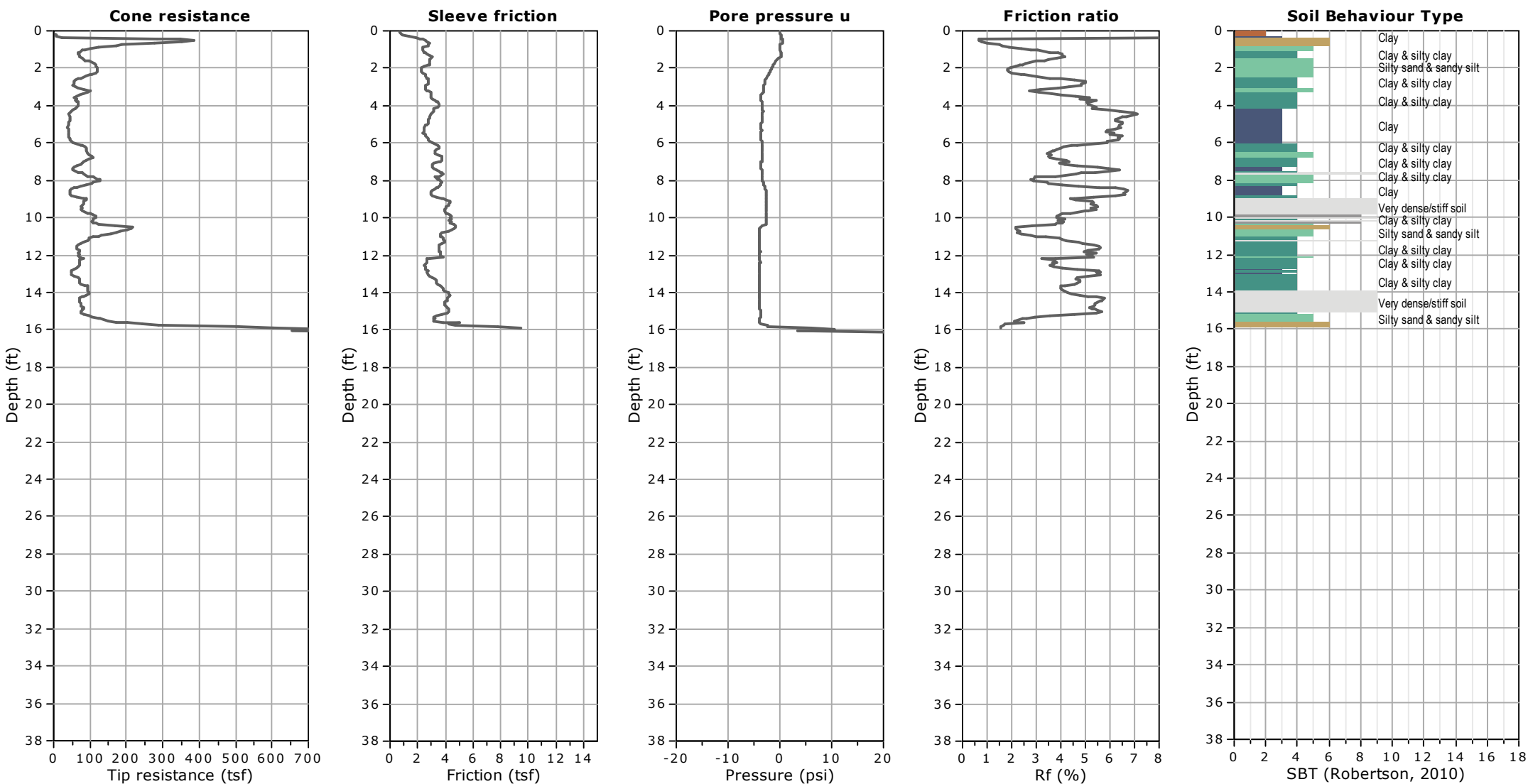


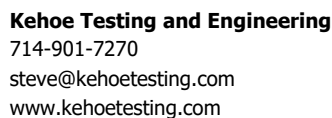
Project: NMG Geotechnical

Location: 300 S. Festival Dr, Anaheim, CA

CPT-3

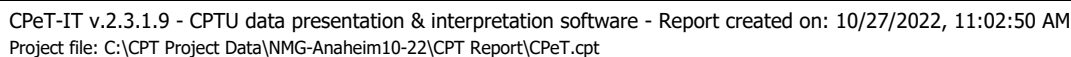
Total depth: 16.29 ft, Date: 10/26/2022





Location: 300 S. Festival Dr, Anaheim, CA

Total depth: 37.34 ft, Date: 10/26/2022



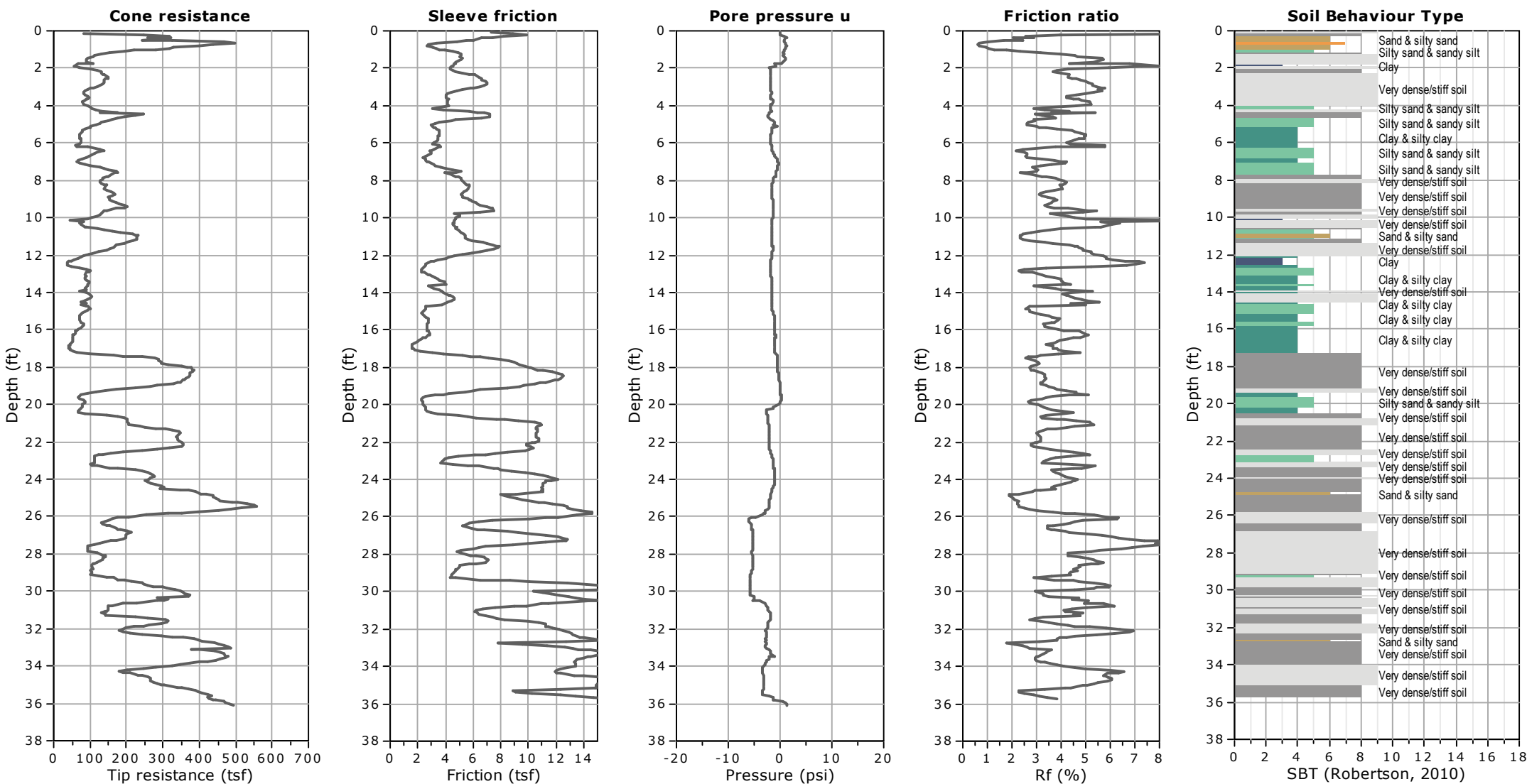


Project: NMG Geotechnical

Location: 300 S. Festival Dr, Anaheim, CA

CPT-4A

Total depth: 36.10 ft, Date: 10/26/2022



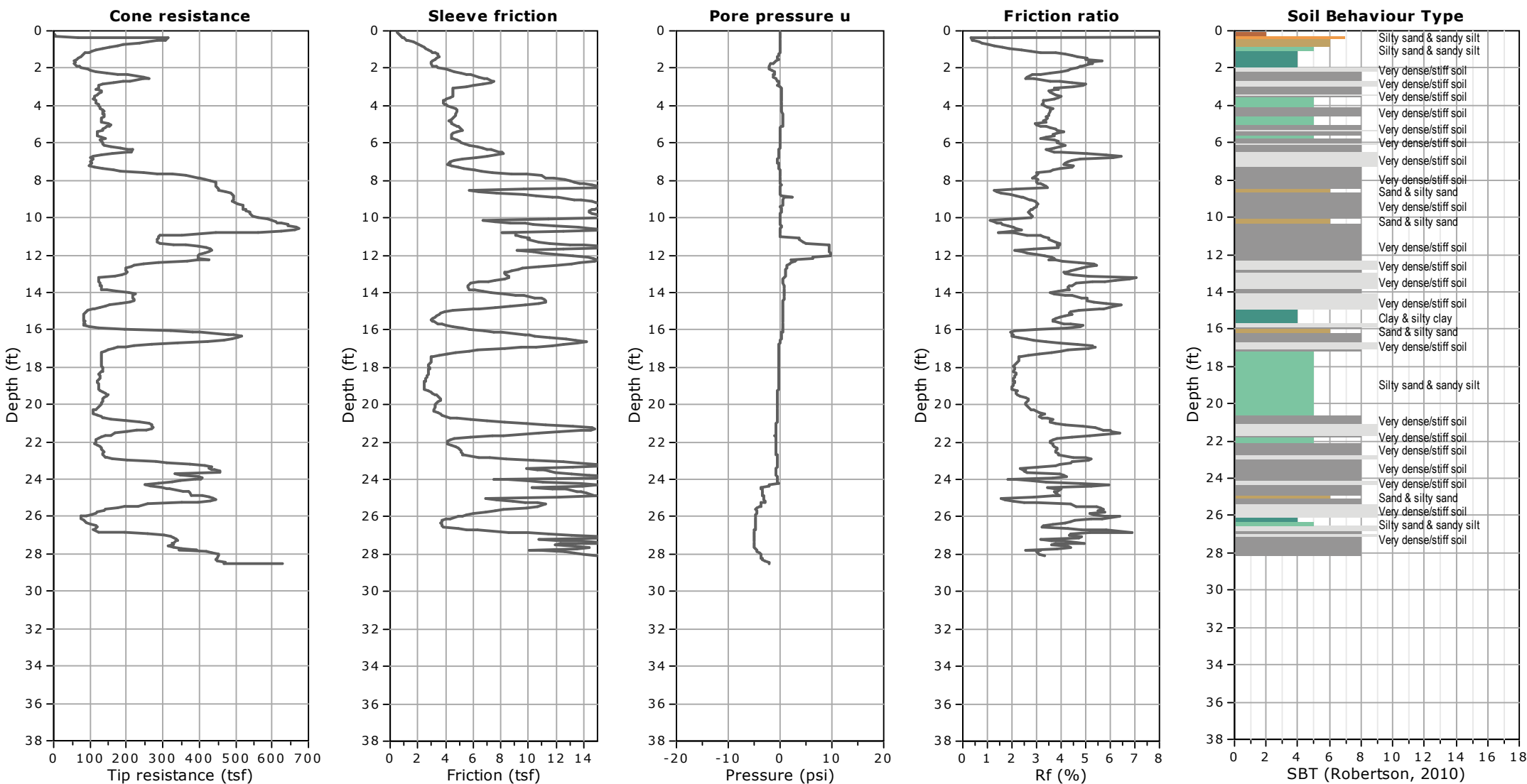


Project: NMG Geotechnical

Location: 300 S. Festival Dr, Anaheim, CA

CPT-5

Total depth: 28.55 ft, Date: 10/26/2022



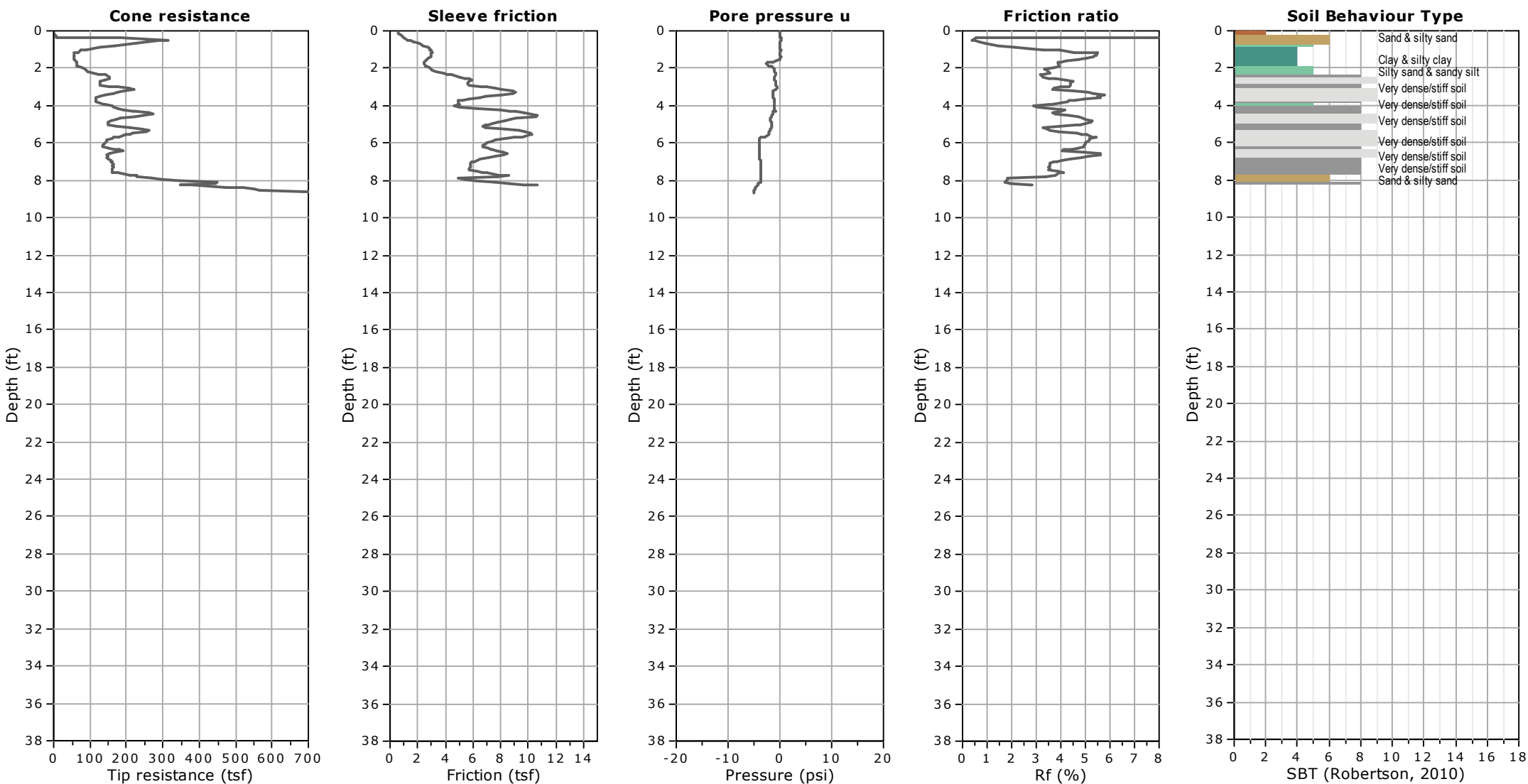


Project: NMG Geotechnical

Location: 300 S. Festival Dr, Anaheim, CA

CPT-6

Total depth: 8.66 ft, Date: 10/26/2022



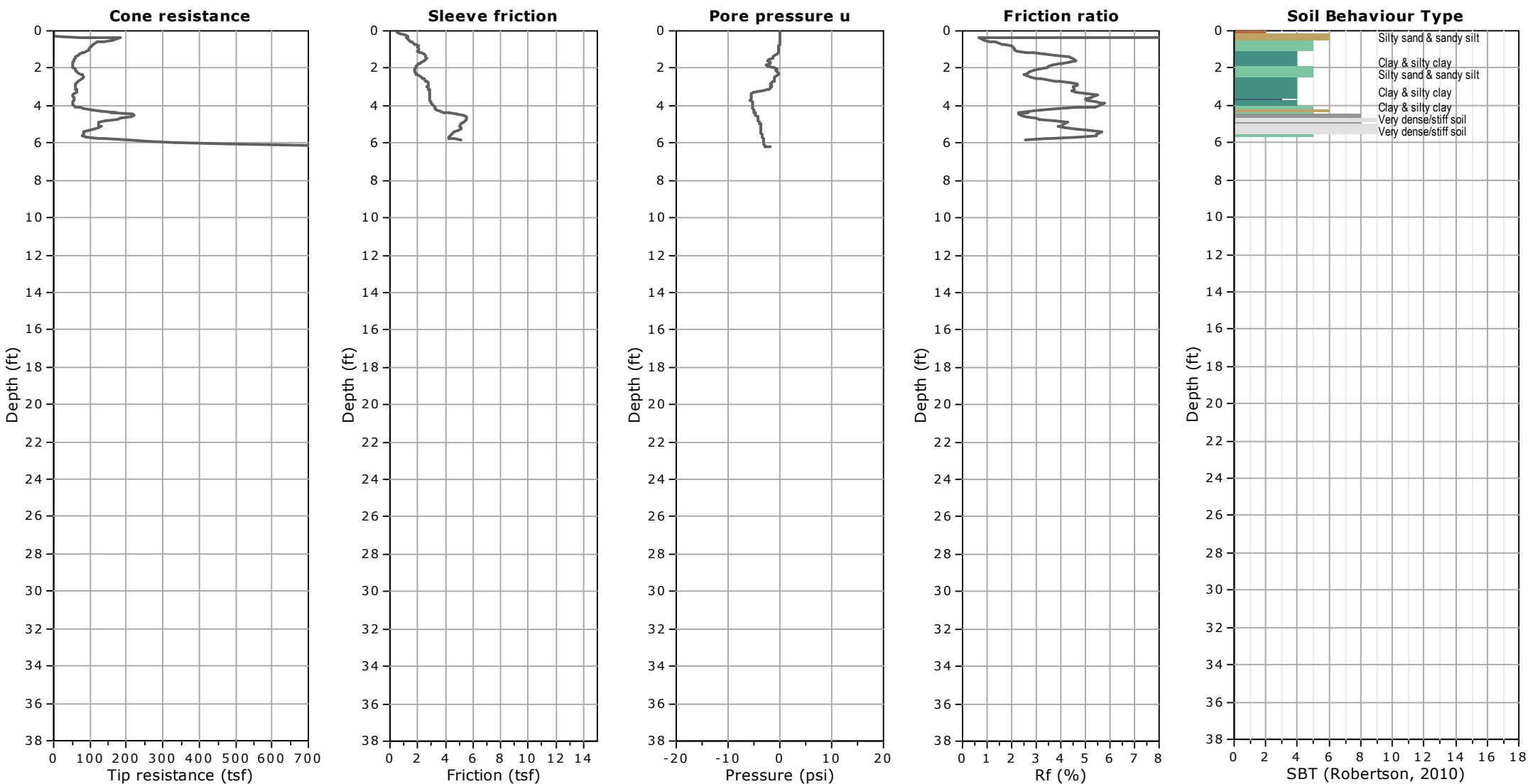


Project: NMG Geotechnical

Location: 300 S. Festival Dr, Anaheim, CA

CPT-7

Total depth: 6.24 ft, Date: 10/26/2022



NMG Geotechnical
300 S. Festival Dr.
Anaheim, CA

CPT Shear Wave Measurements

Location	Tip Depth (ft)	Geophone Depth (ft)	Travel Distance (ft)	S-Wave Arrival (msec)	S-Wave Velocity from Surface (ft/sec)	Interval S-Wave Velocity (ft/sec)
CPT-2	5.02	4.02	4.49	3.30	1361	
CPT-2A	5.09	4.09	4.55	3.96	1150	
	6.73	5.73	6.07	5.36	1132	1083
CPT-4A	5.02	4.02	4.49	3.30	1361	
	10.10	9.10	9.32	7.48	1246	1155
	15.06	14.06	14.20	12.08	1176	1062
	20.08	19.08	19.18	17.66	1086	893
	25.07	24.07	24.15	22.70	1064	986
	30.09	29.09	29.16	26.96	1082	1175
	35.01	34.01	34.07	30.92	1102	1240

Shear Wave Source Offset - 2 ft

S-Wave Velocity from Surface = Travel Distance/S-Wave Arrival
Interval S-Wave Velocity = (Travel Dist2-Travel Dist1)/(Time2-Time1)

BORINGS BY GEOSOILS, INC.

1989



BORING LOG GB-6

CLIENT TRAMMEL CROW W.O. 1993A DATE DRILLED 9-28-89 LOGGED BY F.A.

PROJECT ANA. FESTIVAL SURFACE ELEV. 440 ± DRIVING WT. 2700 → 25'

WATER	DEPTH (FEET)	GRAPHIC LOG		GROUP SYMB. USCS	PENE. RESIST. BLOWS/FOOT	C-CORE B-BAG	DRY DENSITY pcf	MOISTURE CONTENT (%)
			TOP SOIL: SANDY SILT (ML) LOOSE, LT BRN. DRY, ROOTED					
	5		TERRACE DEPOSIT (Qt) CLAYEY SILT (ML) STIFF TO HARD SLI MOIST. MOD. POROUS		5	C		
	10		SILTY SAND W/ CLAY (ML-SM), STIFF TO HARD, RED. BRN. MOIST		5	C		
	15				3	C, B		
			TOTAL DEPTH 15'					
			NO G.W. NO CAVING					



BORING LOG CB-7

CLIENT TRAMMEL CROW W.O. 1988A DATE DRILLED 9-30-99 LOGGED BY F.A.

PROJECT _____ SURFACE ELEV. 475 ± DRIVING WT. _____

WATER	DEPTH (FEET)	GRAPHIC LOG		GROUP SYMB. USCS	PENE. RESIST. BLOWS/FOOT	C-CORE B-BAG	DRY DENSITY pcf	MOISTURE CONTENT (%)
			TOPSOIL: SILTY SAND (SM), LOOSE, MD. BRN. DRY, ROOTED					
	5		TERRACE DEPOSIT (Qtz): SANDY SILT W/ CLAY (ML), STIFF, RED. BRN., SLI MOIST, SLI MODUS					
	10		@ 5' CLAYED SILT (ML), STIFF, RED BRN. MOIST TO V. MOIST					
	15		BEDROCK (TPSS): WELL GRAINED SANDSTONE, MASSIVE, MOD. HARD, YELL. BRN., SLI MOIST, MOD. WEATHERED					
	20		TOTAL DEPTH 15'					
			NO G.W. NO CAVINGS					

PLATE _____

BORING LOG GB-16CLIENT TRAMMEL CROW W.O. 1993A DATE DRILLED 8/29/89 LOGGED BY F.A.PROJECT ANA FESTIVAL SURFACE ELEV. 425± DRIVING WT. 2700 → 25'

WATER	DEPTH (FEET)	GRAPHIC LOG		GROUP SYMB.	PENE. RESIST.	C-CORE	DRY DENSITY	MOISTURE
				USCS	BLOWS/FOOT	B-BAG	pcf	CONTENT (%)
			TOPSOIL: SANDY SILT (ML), LOOSE, LT. BRN. DRY, ROOTED					
	5		Qt: SANDY SILT W/CLAY (ML), STIFF, RED BRN. SLT MOIST		10	C		
	10		@ 5' SANDY SILT W/CLAY (SM-SC), DENSE RED BRN. MOIST, MOD. POROUS					
	15		@ 14' SILTY COARSE SAND (SM), V. DENSE RED. BRN. MOIST		3	C		
			TOTAL DEPTH 15'					
			NO G.W. NO CAVING					

PLATE _____

BORING LOG GB-17CLIENT TOHMEI CORP W.O. 1993A DATE DRILLED 8/30/89 LOGGED BY T.A.PROJECT _____ SURFACE ELEV. 460 ± DRIVING WT. _____

WATER	DEPTH (FEET)	GRAPHIC LOG		GROUP SYMB. USCS	PENE. RESIST. BLOWS/FOOT	C-CORE B-BAG	DRY DENSITY pcf	MOISTURE CONTENT (%)
			BEDROCK (TPSS):					
			SANDSTONE, HARD, YELLOW-BRN., DR.					
			TO SL. MOIST, MOD WEATHERED					
	5		① 3' 6" SHALY INTERBED					
			BEDDING, N42 E, 36 NW					
	10		② 7' 6" SHALY INTERBED					
			BEDDING, N65 E, 36 NW					
	15							
	20							
			TOTAL DEPTH 20					
			NO G.W. NO CAVING					

BORING LOG GB-20CLIENT TRAN/FL CROW W.O. 1993A DATE DRILLED 11-30-89 LOGGED BY _____PROJECT _____ SURFACE ELEV. 460± DRIVING WT. 2700 → 25

WATER	DEPTH (FEET)	GRAPHIC LOG		GROUP SYMB.	PENE. RESIST.	C-CORE	DRY DENSITY	MOISTURE
				USCS	BLOWS/FOOT	B-BAG	pcf	CONTENT (%)
			TOPSOIL: SILTY SAND (SM), LOOSE, LT. BRN.					
	5		Qt: @ 1'-5' CLAYEY SILT (ML), STIFF, RED-BRN; MOIST, MOD. WEATHERED					
	10		@ 5' GRADES TO SILTY COARSE SAND, DENSE, SLT MOIST TO MOIST, NUMEROUS LAYES OF SAND (SW) AND SMALL CHUNKS OF BEDROCK (S.S. + SH)					
	15							
	20							
			TOTAL DEPTH 21'		4	GB		
			BR. = 21' NO G.W. NO CAVING					



BORING LOG GB-21

CLIENT TRAMMEL CROW W.O. 1993A DATE DRILLED 4-28-89 LOGGED BY F.A.PROJECT ANA FESTIVAL SURFACE ELEV. 465± DRIVING WT. _____

WATER	DEPTH (FEET)	GRAPHIC LOG		GROUP SYMB. USCS	PENE. RESIST. BLOWS/FOOT	C-CORE B-BAG	DRY DENSITY pcf	MOISTURE CONTENT(%)
			BEDROCK (TPSS):					
			SHALE, MOD HARD, GRAY, DRY TO					
			SLT. MOIST. W/FEW SANDY LENSES.					
	5		MOD. FRACTURED, MOD WEATHERED					
			@ 3' BEDDING E-W, 34 N					
			@ 5' BEDDING E-W, 35 N					
	10		@ 9' BEDDING E-W, 52 N					
			@ 9' SANDY INTERBEDS					
			@ 13' N 75 E, 30 N					
	15		14' SHALE BECOMES MOISTER					
			@ 18' SANDY INTERBEDS					
	20		E-W, 32 N					
			SANDY INTERBEDS					
			@ 22' N 85 E, 31 N					
	25		SANDY INTERBEDS					
	30							
			@ 34' BEDDING, E-W, 38 N					
	35							

PLATE _____

BORING LOG GB-21CLIENT TRAMMEL CROW W.O. 1993A DATE DRILLED 8/28/89 LOGGED BY F.A.PROJECT ANA. FESTIVAL SURFACE ELEV. 465 ± DRIVING WT. _____

WATER	DEPTH (FEET)	GRAPHIC LOG			GROUP SYMB. USCS	PENE. RESIST. BLOWS/FOOT	C-CORE B-BAG	DRY DENSITY pcf	MOISTURE CONTENT (%)
	40			<u>TOTAL DEPTH 40'</u> <u>NO G.W. NO CAVING</u>					

**BORINGS BY MEDALL, ARAGON,
WORSWICK AND ASSOCIATES**

1979

BORING LOG

Logged By: H.P.	Date Drilled: 10-24-79	Drill Rig: Bucket Auger	Boring Diameter: 24"	Boring Number: B-9
This log is a representation of subsurface conditions at the time and place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.			Boring Elevation: 435	

SAMPLE		DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE % DRY WEIGHT	DRY DENSITY LB/CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	DEPTH FEET	SOIL/ROCK TYPE	GEOLOGICAL/ENGINEERING Description and Remarks
BULK	TUBE							
							SM	TOPSOIL Silty SAND Loose, reddish brown, dry to damp.
		30	11	119				TERRACE DEPOSITS - (Qt) Silty Fine SAND Medium dense to dense, reddish brown, slightly moist to moist.
		14	12	118		5		
		20	12	118				
		15	10	114		10		
		11	8	105		15		Yellow brown with scattered gravel.
		13	7	109		20	SP	SAND

BORING LOG

Logged By: H. P.	Date Drilled: 10-24-79	Drill Rig: Bucket Auger	Boring Diameter: 24"	Boring Number: B-9
This log is a representation of subsurface conditions at the time and place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.			Boring Elevation: 435	Continued

SAMPLE								GEOLOGICAL/ENGINEERING	
BULK	TUBE	DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE % DRY WEIGHT	DRY DENSITY LB/CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	DEPTH, FEET	SOIL/ROCK TYPE	Description and Remarks	
								TERRACE DEPOSITS - (Qt) Continued	
							SP	SAND	
								Medium dense to dense, yellow brown to yellow, slightly moist to moist, fine to medium grained, slightly silty, minor amounts of scattered coarse sand and gravel.	
		23	9	113		25			
						30			
		19	10	115				Silty SAND	
								Dense, brown, slightly moist to moist, scattered coarse sand and gravel.	
								End of Boring at 31½ feet	
								No groundwater	
								No caving	
								Hole backfilled	
						35			
						40			

BORING LOG

Logged By: H. P.	Date Drilled: 10-24-79	Drill Rig: Bucket Auger	Boring Diameter: 24"	Boring Number: B-10
This log is a representation of subsurface conditions at the time and place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.			Boring Elevation: 492	

SAMPLE							GEOLOGICAL/ENGINEERING	
BULK	TUBE	DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE % DRY WEIGHT	DRY DENSITY LB./CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	DEPTH, FEET	SOIL/ROCK TYPE	Description and Remarks
							SM	<u>TOPSOIL</u> Silty SAND Loose to medium dense, reddish brown to yellow brown, dry to damp, porous.
						5		<u>TERRACE DEPOSITS - (Qt)</u> Silty SAND Medium dense to dense, yellow brown to reddish brown, slightly moist.
		42	9	120		10		<u>BEDROCK - (Tpss)</u> SANDSTONE Moderately hard to hard, yellow, damp, fine to medium grained, slightly micaceous, interbedded gray siltstone (75% sandstone - 25% siltstone) moderately weathered to fresh, minor amounts of gypsum, orange oxidized laminations and mottling, slightly weathered to fresh, hard at 10'.
						15	B EW 42N	
		38	7	114		20	B EW 40N	

BORING LOG

Logged By: H. P.	Date Drilled: 10-24-79	Drill Rig: Bucket Auger	Boring Diameter: 24"	Boring Number: B-10
This log is a representation of subsurface conditions at the time and place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.			Boring Elevation: 492	Continued

SAMPLE								GEOLOGICAL/ENGINEERING	
BULK	TUBE	DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE % DRY WEIGHT	DRY DENSITY LB./CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	DEPTH, FEET	SOIL/ROCK TYPE	Description and Remarks	
						25		BEDROCK - (Tpss) Continued	
								SANDSTONE	
								B N85E 48NW	
								Hard	
		38	15	99		30		B N85E 45NW	
								SILTSTONE AND SANDSTONE	
								Hard, brown and dark gray, dry, silicified, fine grained, highly cemented-resistant to weathering.	
						35		SANDSTONE	
								B N85E	
								Moderately hard to hard, yellow, fine to medium grained, slightly micaceous, with gray interbedded siltstone, damp, micaceous, fresh- unweathered.	
		58	12	114		40		B N85E 47NW	

BORING LOG

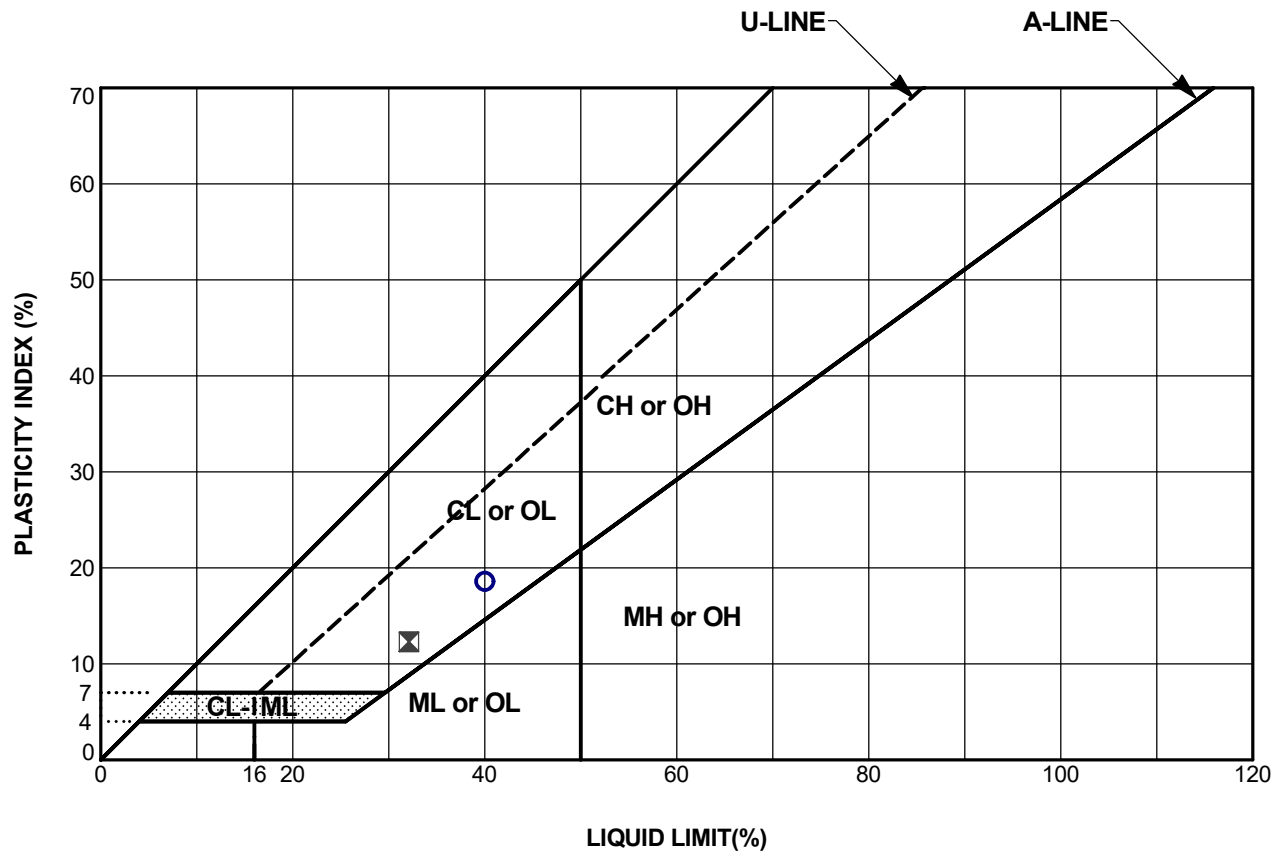
Logged By: H.P.	Date Drilled: 10-24-79	Drill Rig: Bucket Auger	Boring Diameter: 24"	Boring Number: B-10
This log is a representation of subsurface conditions at the time and place of drilling. With the passage of time or at any other location there may be consequential changes in conditions.			Boring Elevation: 492	Continued

SAMPLE								GEOLOGICAL/ENGINEERING Description and Remarks
BULK	TUBE	DRIVE ENERGY FT. KIPS/FT.	FIELD MOISTURE % DRY WEIGHT	DRY DENSITY LB./CU. FT.	SHEAR RESISTANCE KIPS/SQ. FT.	DEPTH, FEET	SOIL/ROCK TYPE	
						45		BEDROCK - (Tpss) Continued SANDSTONE
						50		End of Boring at 47 feet No groundwater No caving Hole backfilled
						55		
						60		

APPENDIX C

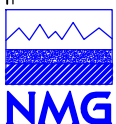
APPENDIX C
SUMMARY OF SOIL LABORATORY DATA

Boring/Sample Information						Field Wet Density (pcf)	Field Dry Density (pcf)	Field Moisture Content (%)	Degree of Sat. (%)	Sieve/ Hydrometer		Atterberg Limits		USCS Group Symbol	Direct Shear				Compaction		Expansion Index	R-Value	Soluble Sulfate Content (% by wt)	Remarks
Boring No.	Sample No.	Depth (feet)	End Depth (feet)	Elevation (feet)	Blow Count (N)					Fines Content (% pass. #200)	Clay Content (% pass. 2µ)	LL (%)	PI (%)		Ultimate		Peak		Maximum Dry Density (pcf)	Optimum Moisture Content (%)				
H-1	B-1	1.0		463.0																				
H-1	D-1	5.0		459.0	50/6"	126.6	117.8	7.5	46.8					SM	250	32	300	42.0						
H-1	D-2	10.0		454.0	50/5"	112.0	105.5	6.2	27.9															
H-1	D-3	15.0		449.0	50/4"			11.6																Disturbed
H-2	B-1	1.5		465.5																				
H-2	D-1	5.0		462.0	90/9"	117.5	98.7	19.0	72.5															
H-2	D-2	10.0		457.0	50/6"	114.7	96.9	18.5	67.4															
H-2	D-3	15.0		452.0	50/6"	121.9	102.5	19.0	79.6															
H-3	B-1	1.0	5.0	464.0						40	14	40	19	SC							51			CC
H-3	D-1	5.0		460.0	81	131.3	116.0	13.2	78.8					CL	350	30	1150	30.0						CN
H-3	D-2	10.0		455.0	42	129.9	110.8	17.3	89.5					CL										CN
H-3	D-3	15.0		450.0	66	133.9	117.8	13.7	86.1															
H-3	D-4	20.0		445.0	50/6"	128.7	107.1	20.2	95.2															
H-3	D-5	25.0		440.0	50/6"	129.2	108.7	18.9	92.6															
H-4	B-1	1.0		462.0																				
H-4	D-1	5.0		458.0	42	123.6	112.6	9.8	53.2					SM										CN
H-4	D-2	10.0		453.0	43	130.9	117.0	11.8	72.7															
H-4	D-3	15.0		448.0	90/9"	124.8	99.5	25.5	99.2															
H-4	D-4	20.0		443.0	50/6"	114.4	101.9	12.2	50.6															
H-4	D-5	25.0		438.0	80/9"	126.3	105.5	19.7	89.2															
H-5	B-1	1.0	5.0	462.0						35	12	32	12	SC							26			CC
H-5	D-1	5.0		458.0	70	119.5	106.0	12.7	58.3															
H-5	D-2	10.0		453.0	47	133.3	118.2	12.8	81.0					ML	150	30	650	30.0						CN
H-5	D-3	15.0		448.0	50/6"	117.0	109.4	6.9	34.4															
H-5	D-4	20.0		443.0	50/4"	114.0	108.3	5.2	25.2															
H-5	D-5	25.0		438.0	50/6"	116.9	97.5	19.9	73.7															



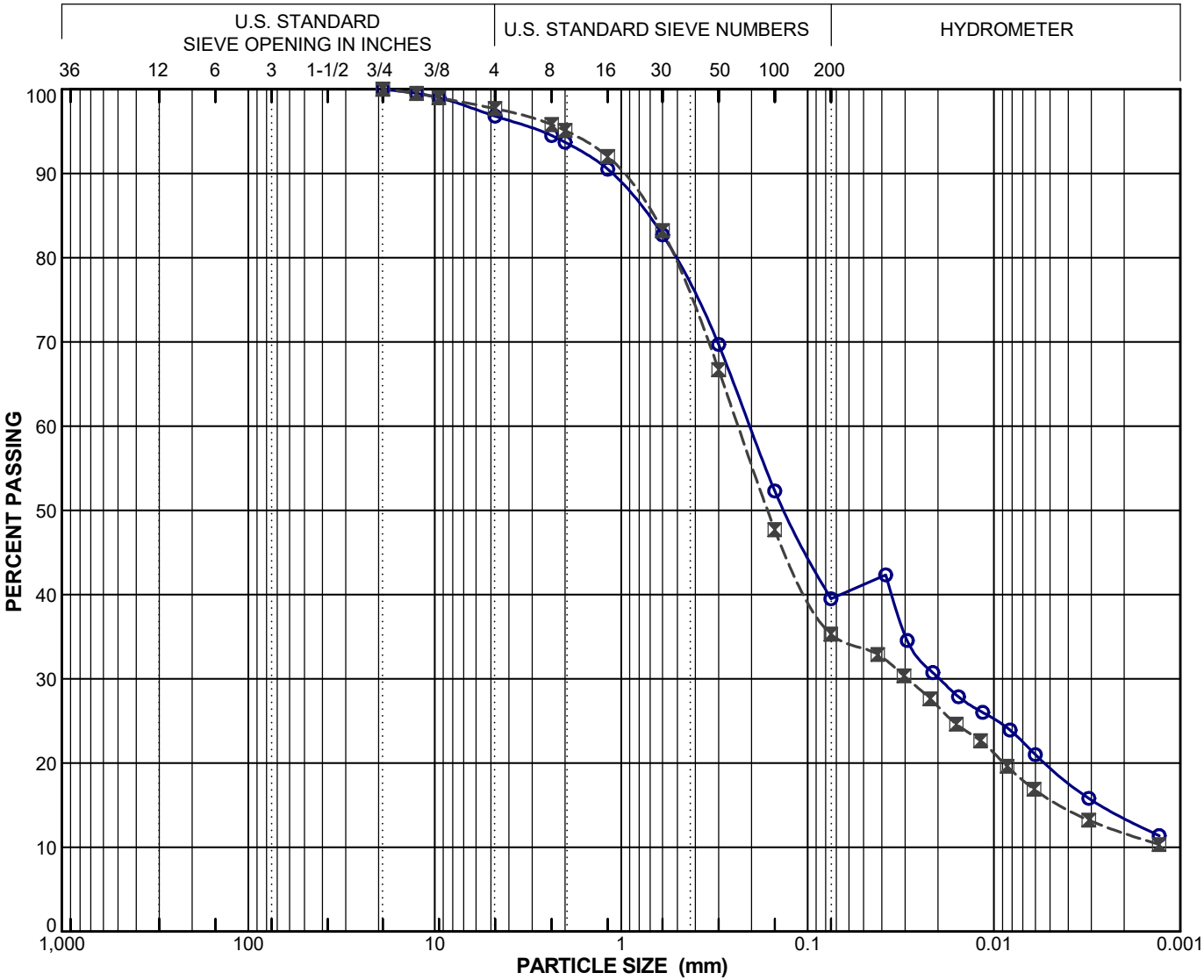
PLASTICITY CHART

Shea/ Anaheim Hills Festival
Anaheim Hills, CA
PROJECT NO. 22023-01



Geotechnical, Inc.

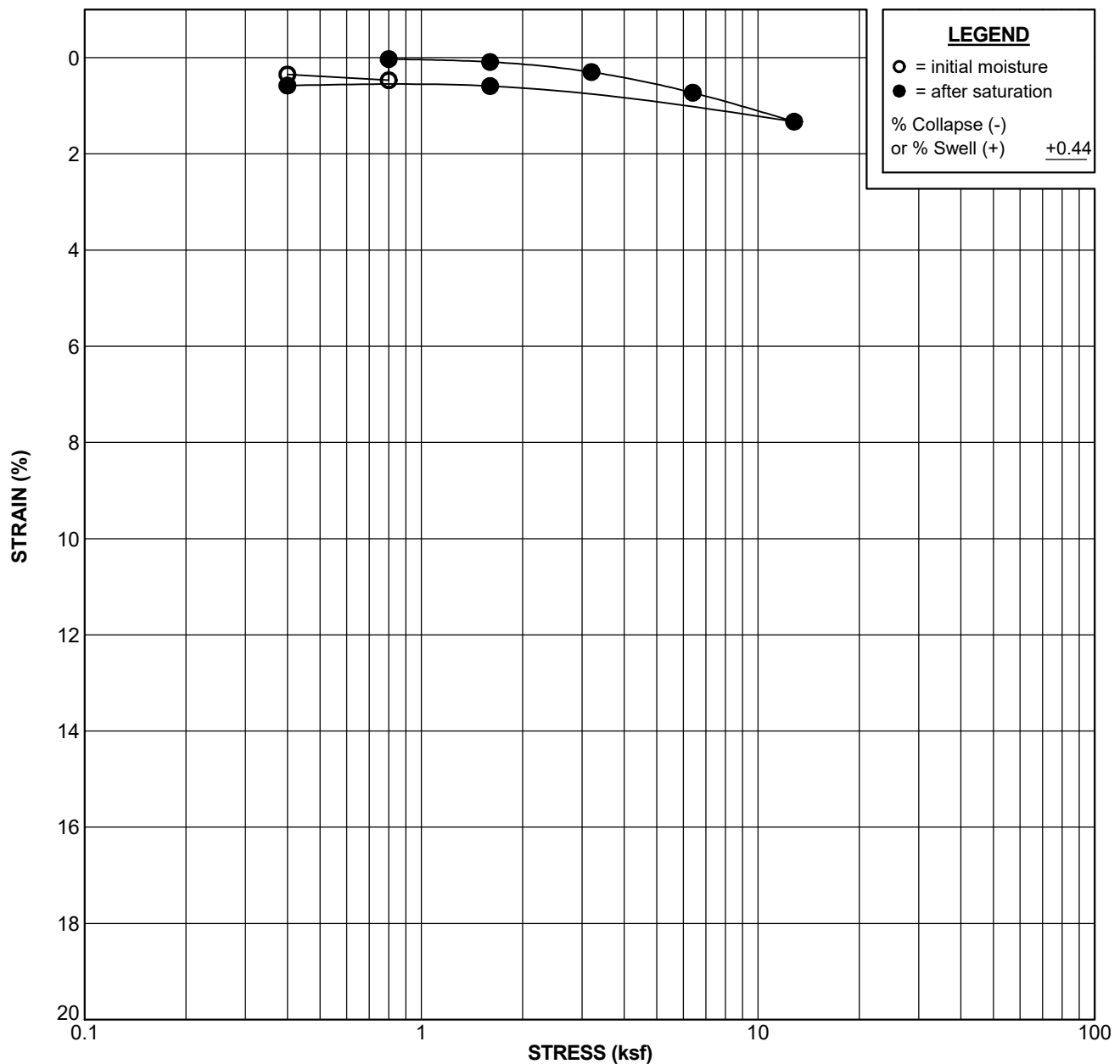
BOULDERS	COBBLES	GRAVEL		SAND			SILT OR CLAY
		coarse	fine	coarse	medium	fine	



Symbol	Boring Number	Sample Number	Depth (feet)	Field Moisture (%)	LL	PI	Activity PI/-2 μ	C _u	C _c	Passing No. 200 Sieve (%)	Passing 2 μ (%)	USCS
○	H-3	B-1	1.0 - 5.0		40	19	1.36			40	14	SC
⊠	H-5	B-1	1.0 - 5.0		32	12	1.00			35	12	SC

PARTICLE SIZE DISTRIBUTION
 Shea/ Anaheim Hills Festival
 Anaheim Hills, CA
 PROJECT NO. 22023-01

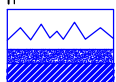




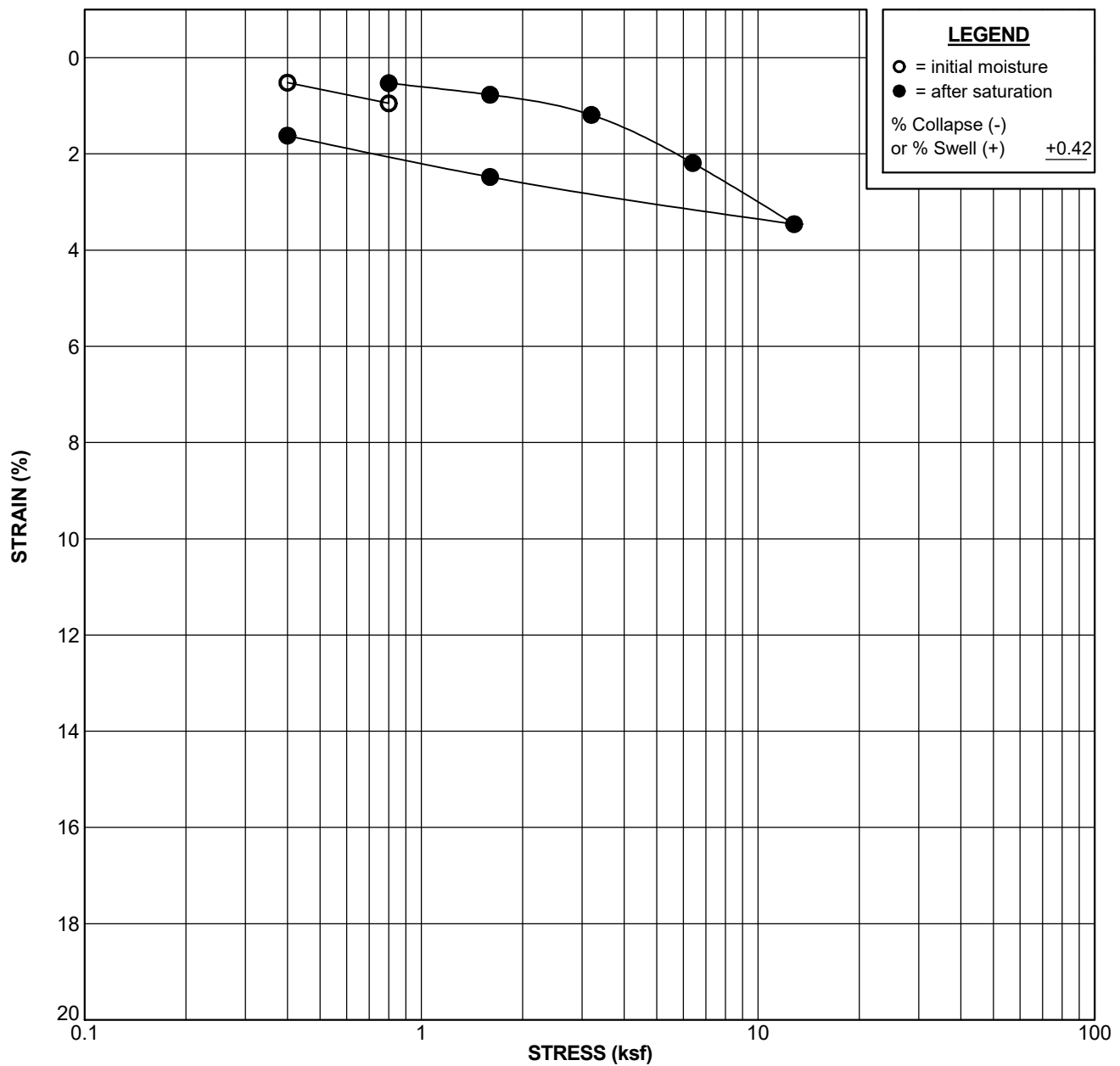
Boring No. H-3		Sample No. D-1		Depth: 5.0 ft	
Sample Description: (Af) Olive brown silty sandy CLAY				USCS: CL	
Liquid Limit:		Plasticity Index:		Percent Passing No. 200 Sieve:	
Test Stage	Moisture Content (%)	Dry Density (pcf)	Degree of Saturation (%)	Void Ratio	
Initial	14.8	112.6	62.0	0.757	
Final	19.1	113.2	81.0	0.747	

CONSOLIDATION TEST RESULTS

Shea/ Anaheim Hills Festival
Anaheim Hills, CA
PROJECT NO. 22023-01



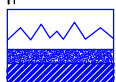
NMG Geotechnical, Inc.



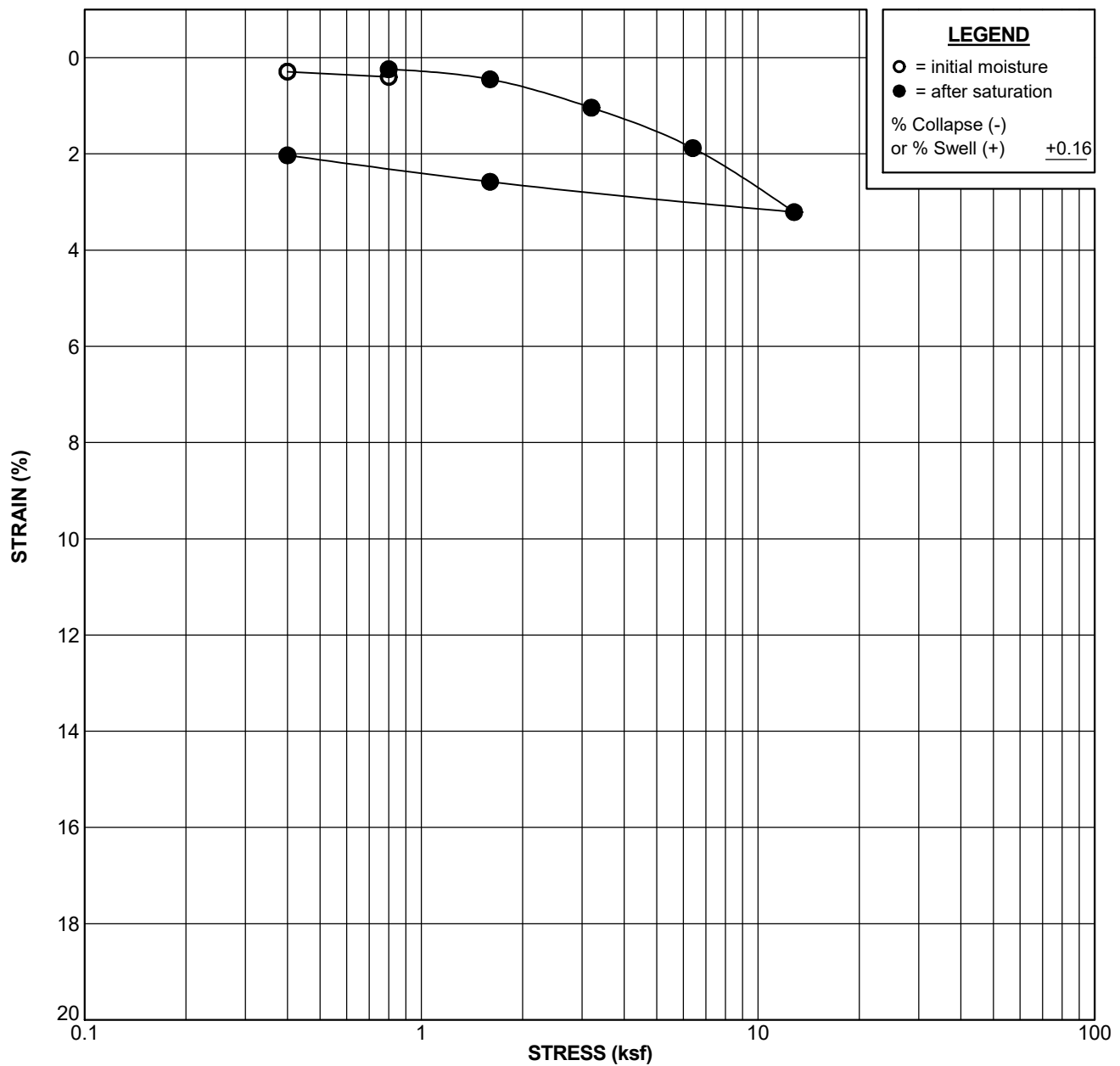
Boring No. H-3		Sample No. D-2		Depth: 10.0 ft	
Sample Description: (Af) Grayish brown sandy CLAY				USCS: CL	
Liquid Limit:		Plasticity Index:		Percent Passing No. 200 Sieve:	
Test Stage	Moisture Content (%)	Dry Density (pcf)	Degree of Saturation (%)	Void Ratio	
Initial	16.6	106.4	79.4	0.554	
Final	19.9	108.1	99.6	0.530	

CONSOLIDATION TEST RESULTS

Shea/ Anaheim Hills Festival
 Anaheim Hills, CA
 PROJECT NO. 22023-01



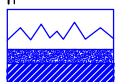
NMG Geotechnical, Inc.



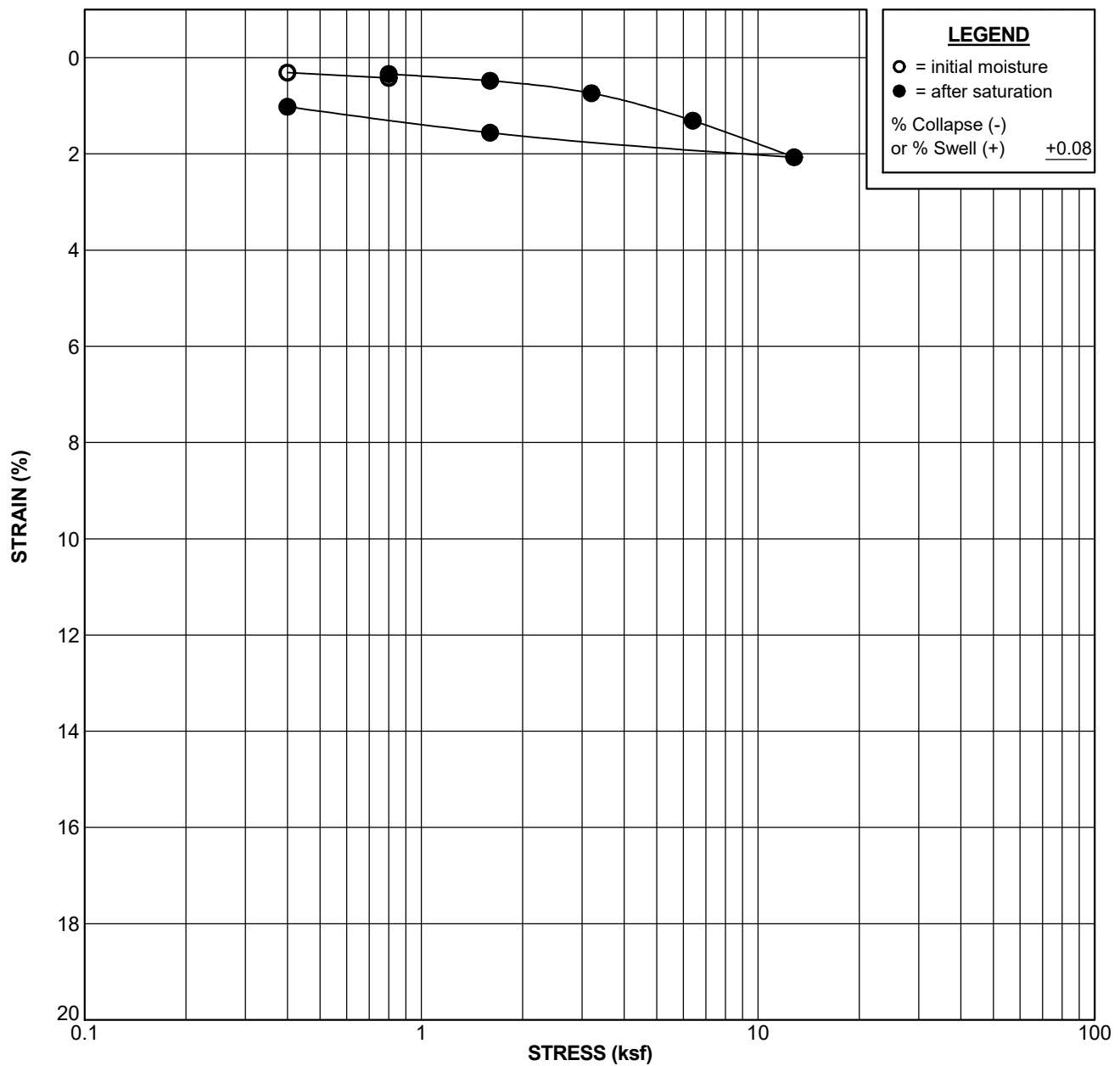
Boring No. H-4		Sample No. D-1		Depth: 5.0 ft	
Sample Description: (Af) Light yellowish brown silty SAND				USCS: SM	
Liquid Limit:		Plasticity Index:		Percent Passing No. 200 Sieve:	
Test Stage	Moisture Content (%)	Dry Density (pcf)	Degree of Saturation (%)	Void Ratio	
Initial	8.7	110.2	46.2	0.498	
Final	17.6	112.5	99.7	0.467	

CONSOLIDATION TEST RESULTS

Shea/ Anaheim Hills Festival
Anaheim Hills, CA
PROJECT NO. 22023-01



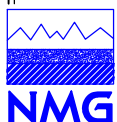
NMG Geotechnical, Inc.



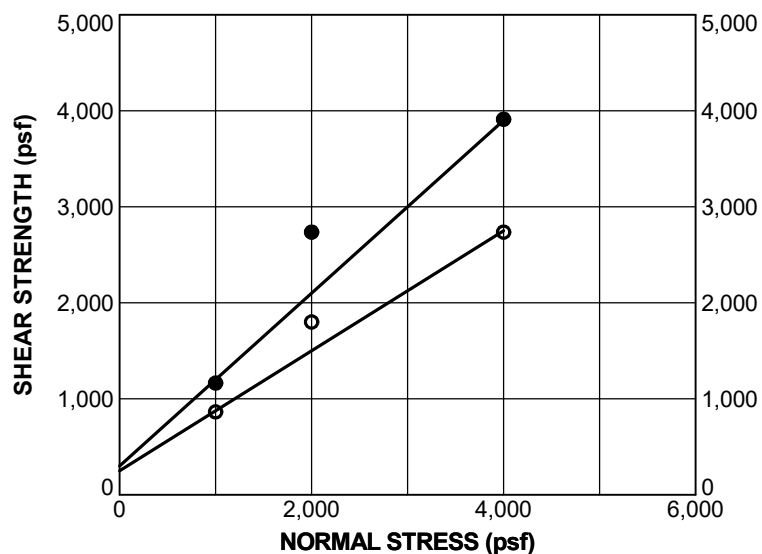
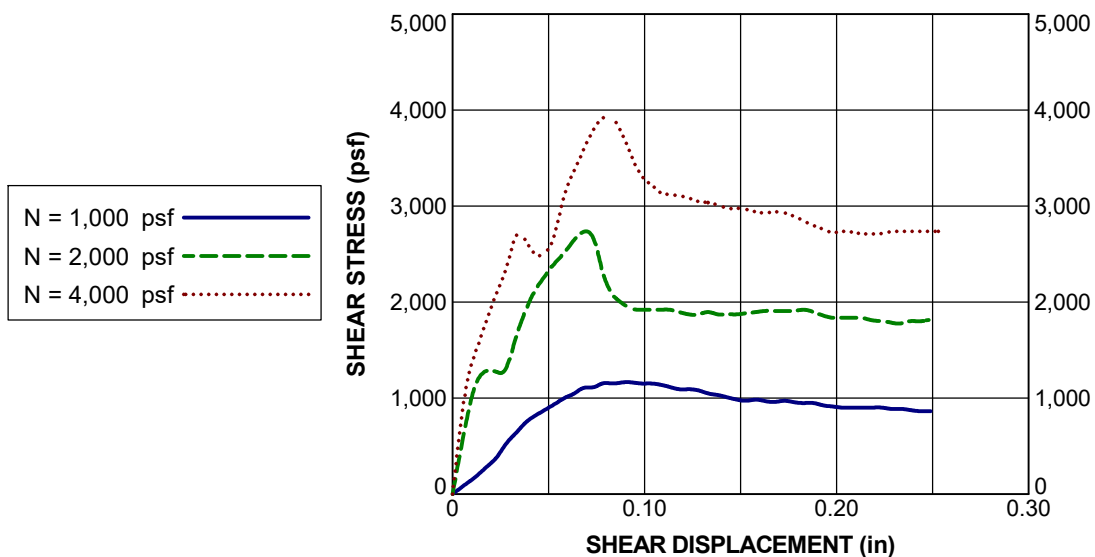
Boring No. H-5		Sample No. D-2		Depth: 10.0 ft	
Sample Description: (Af) Olive brown clayey sandy SILT				USCS: ML	
Liquid Limit:		Plasticity Index:		Percent Passing No. 200 Sieve:	
Test Stage	Moisture Content (%)	Dry Density (pcf)	Degree of Saturation (%)	Void Ratio	
Initial	13.1	117.1	62.0	0.657	
Final	15.6	118.3	75.8	0.640	

CONSOLIDATION TEST RESULTS

Shea/ Anaheim Hills Festival
Anaheim Hills, CA
PROJECT NO. 22023-01



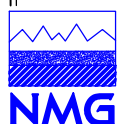
Geotechnical, Inc.



Boring No. H-1		Sample No. D-1		Depth: 5.0 ft	
Sample Description: (Tpsc) Yellowish brown silty SAND (sandstone)				USCS: SM	
Liquid Limit:		Plasticity Index:		Percent Passing No. 200 Sieve:	
Final Moisture Content (%): 20.8		Final Dry Density (pcf): 110.0		Degree of Saturation (%): 100	
Sample Type: Undisturbed		Rate of Shear (in./min.): 0.05			
SHEAR STRENGTH PARAMETERS					
Parameter		Peak ●		Ultimate ○	
Cohesion (psf)		300		250	
Friction Angle (degrees)		42.0		32.0	

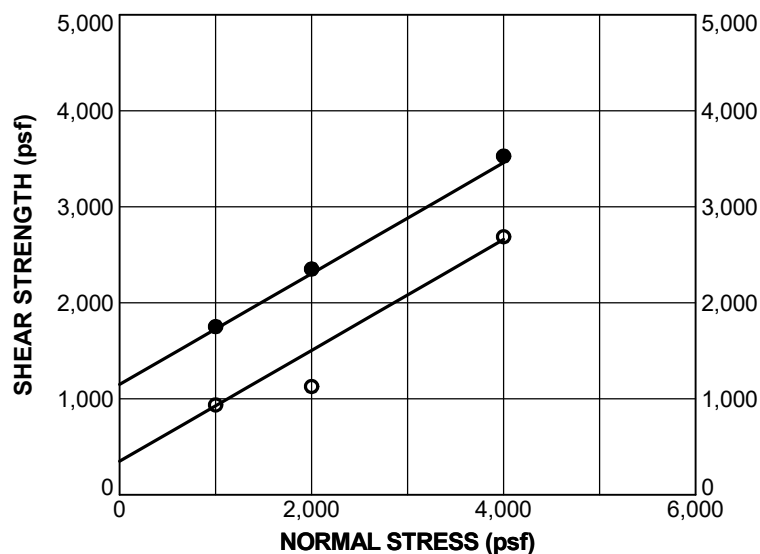
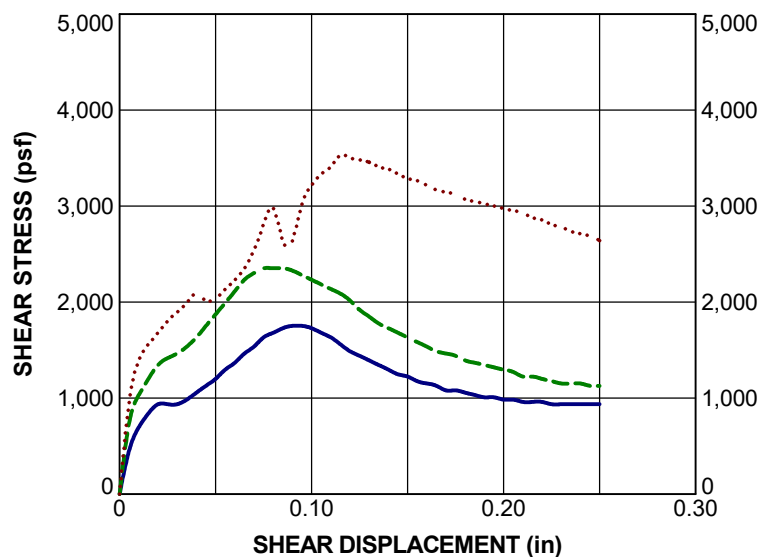
DIRECT SHEAR TEST RESULTS

Shea/ Anaheim Hills Festival
Anaheim Hills, CA
PROJECT NO. 22023-01



Geotechnical, Inc.

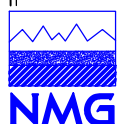
N = 1,000 psf ———
 N = 2,000 psf - - - -
 N = 4,000 psf ·····



Boring No. H-3		Sample No. D-1	Depth: 5.0 ft	
Sample Description: (Af) Olive brown silty sandy CLAY			USCS: CL	
Liquid Limit:		Plasticity Index:		Percent Passing No. 200 Sieve:
Final Moisture Content (%): 23.9		Final Dry Density (pcf): 112.4		Degree of Saturation (%): 100
Sample Type: Undisturbed		Rate of Shear (in./min.): 0.005		
SHEAR STRENGTH PARAMETERS				
Parameter		Peak ●		Ultimate ○
Cohesion (psf)		1150		350
Friction Angle (degrees)		30.0		30.0

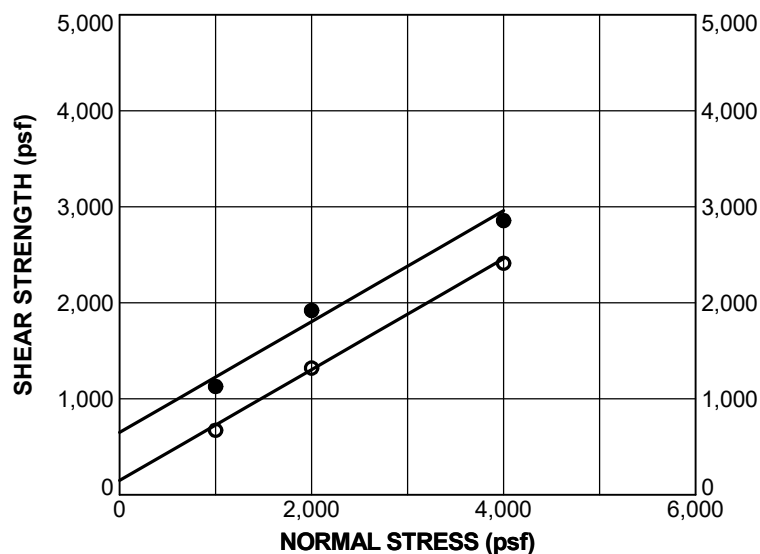
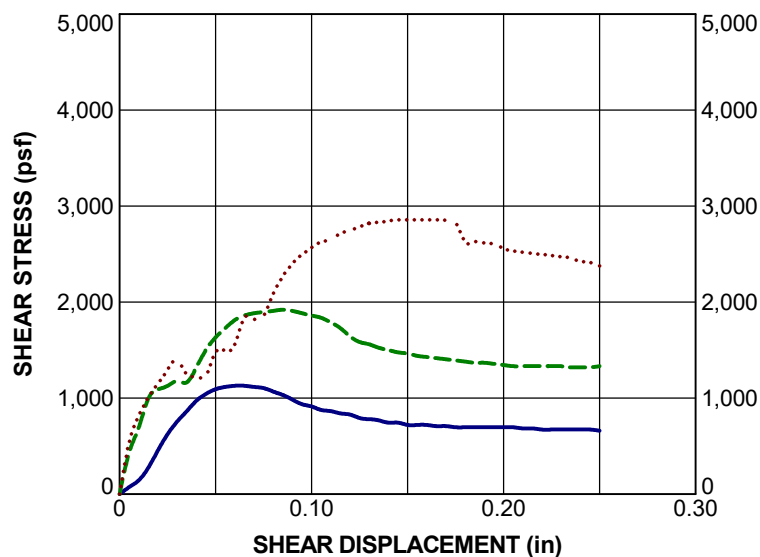
DIRECT SHEAR TEST RESULTS

Shea/ Anaheim Hills Festival
 Anaheim Hills, CA
 PROJECT NO. 22023-01



Geotechnical, Inc.

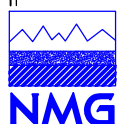
N = 1,000 psf ———
 N = 2,000 psf - - - -
 N = 4,000 psf ·····



Boring No. H-5		Sample No. D-2	Depth: 10.0 ft
Sample Description: (Af) Olive brown clayey sandy SILT			USCS: ML
Liquid Limit:		Plasticity Index:	Percent Passing No. 200 Sieve:
Final Moisture Content (%): 23.7		Final Dry Density (pcf): 111.1	Degree of Saturation (%): 99
Sample Type:		Rate of Shear (in./min.):	0.005
SHEAR STRENGTH PARAMETERS			
Parameter	Peak ●	Ultimate ○	
Cohesion (psf)	650	150	
Friction Angle (degrees)	30.0	30.0	

DIRECT SHEAR TEST RESULTS

Shea/ Anaheim Hills Festival
 Anaheim Hills, CA
 PROJECT NO. 22023-01



Geotechnical, Inc.

**Table 1 - Laboratory Tests on Soil Samples**

NMG Geotechnical, Inc.
SheaAnaheim Hills Festival
Your #22023-01, HDR Lab #22-1061LAB
10-Nov-22

Sample ID			H-3, B-1 @ 1-5'	H-5, B-1 @ 1-5'
Resistivity				
as-received	ohm-cm		20,000	10,800
saturated	ohm-cm		800	880
pH			6.7	6.6
Electrical				
Conductivity	mS/cm		0.59	0.37
Chemical Analyses				
Cations				
calcium	Ca ²⁺	mg/kg	153	96
magnesium	Mg ²⁺	mg/kg	32	30
sodium	Na ¹⁺	mg/kg	234	128
potassium	K ¹⁺	mg/kg	32	15
ammonium	NH ₄ ¹⁺	mg/kg	ND	ND
Anions				
carbonate	CO ₃ ²⁻	mg/kg	ND	ND
bicarbonate	HCO ₃ ¹⁻	mg/kg	174	122
fluoride	F ¹⁻	mg/kg	9.2	6.6
chloride	Cl ¹⁻	mg/kg	138	61
sulfate	SO ₄ ²⁻	mg/kg	1,270	794
nitrate	NO ₃ ¹⁻	mg/kg	2.0	ND
phosphate	PO ₄ ³⁻	mg/kg	ND	ND
Other Tests				
sulfide	S ²⁻	qual	na	na
Redox		mV	na	na

Resistivity per ASTM G187, pH per ASTM G51, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

APPENDIX D



Shea Prop/Festival

Latitude, Longitude: 33.8652, -117.7512



Date	11/20/2022, 11:36:16 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.841	MCE_R ground motion. (for 0.2 second period)
S_1	0.649	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.841	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.227	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	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.78	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.858	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
$SsRT$	1.841	Probabilistic risk-targeted ground motion. (0.2 second)
$SsUH$	2.017	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.317	Factored deterministic acceleration value. (0.2 second)
$S1RT$	0.649	Probabilistic risk-targeted ground motion. (1.0 second)
$S1UH$	0.712	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
$S1D$	0.881	Factored deterministic acceleration value. (1.0 second)
$PGAd$	0.955	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.78	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.913	Mapped value of the risk coefficient at short periods

Type	Value	Description
C _{R1}	0.912	Mapped value of the risk coefficient at a period of 1 s
C _V	1.468	Vertical coefficient

Unified Hazard Tool



Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the [U.S. Seismic Design Maps web tools](#) (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

^ Input

Edition

Spectral Period

Latitude

Decimal degrees

Time Horizon

Return period in years

Longitude

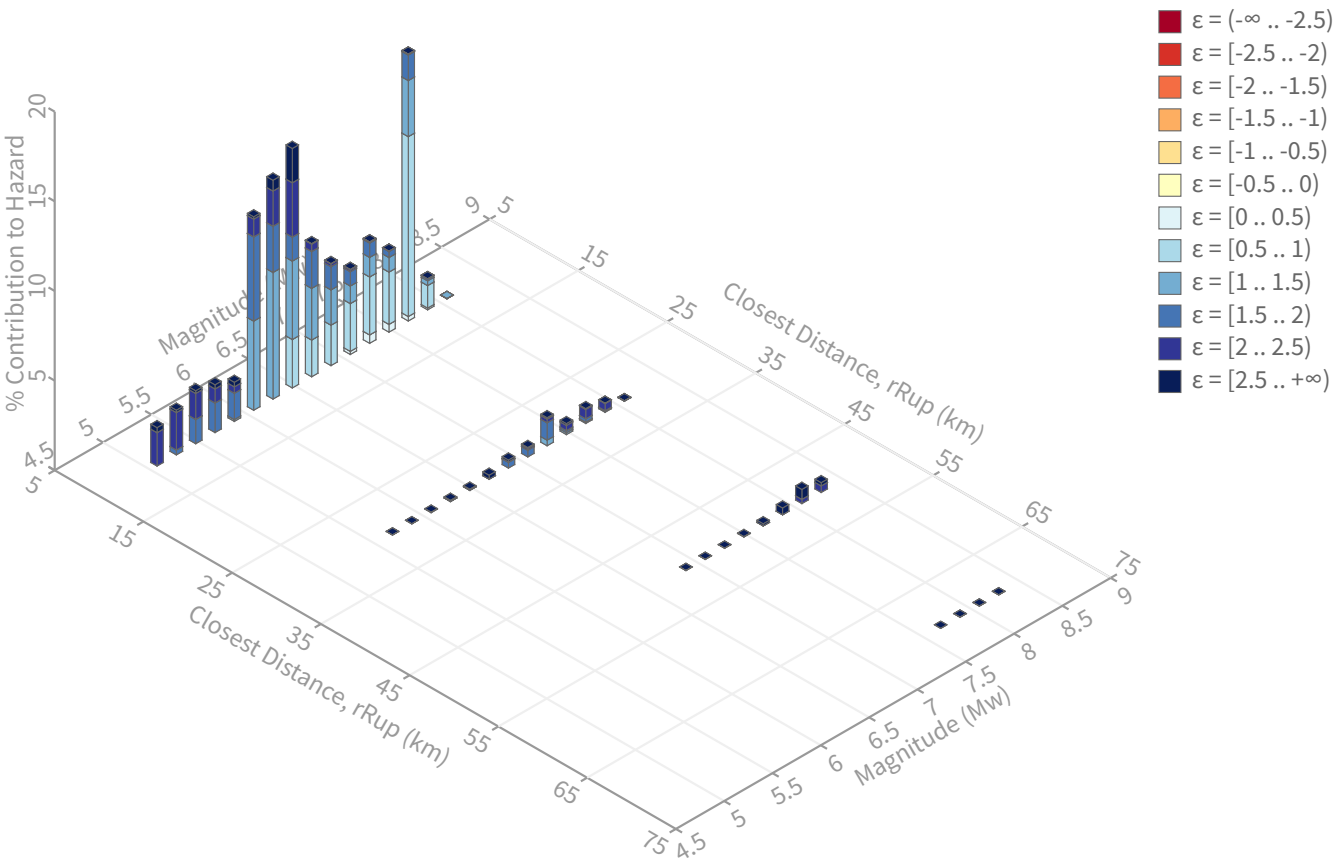
Decimal degrees, negative values for western longitudes

Site Class

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs

Exceedance rate: 0.0004040404 yr⁻¹

PGA ground motion: 0.81706111 g

Recovered targets

Return period: 2901.8505 yrs

Exceedance rate: 0.00034460769 yr⁻¹

Totals

Binned: 100 %

Residual: 0 %

Trace: 0.05 %

Mean (over all sources)

m: 6.74

r: 8.45 km

ε₀: 1.44 σ

Mode (largest m-r bin)

m: 7.72

r: 3.8 km

ε₀: 0.89 σ

Contribution: 14.88 %

Mode (largest m-r-ε₀ bin)

m: 7.72

r: 3.38 km

ε₀: 0.72 σ

Contribution: 10.03 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km

m: min = 4.4, max = 9.4, Δ = 0.2

ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)

ε1: [-2.5 .. -2.0)

ε2: [-2.0 .. -1.5)

ε3: [-1.5 .. -1.0)

ε4: [-1.0 .. -0.5)

ε5: [-0.5 .. 0.0)

ε6: [0.0 .. 0.5)

ε7: [0.5 .. 1.0)

ε8: [1.0 .. 1.5)

ε9: [1.5 .. 2.0)

ε10: [2.0 .. 2.5)

ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set	Source	Type	r	m	ϵ_0	lon	lat	az	%
UC33brAvg_FM31		System							46.29
	Whittier alt 1 [2]		3.04	7.30	0.94	117.740°W	33.888°N	21.79	14.12
	Chino alt 1 [3]		7.90	6.60	1.27	117.669°W	33.917°N	52.83	9.25
	Peralta Hills [0]		2.70	6.81	0.89	117.751°W	33.846°N	179.10	4.79
	Chino alt 1 [2]		7.90	6.24	1.32	117.676°W	33.924°N	46.98	3.50
	Chino alt 1 [1]		9.17	6.08	1.70	117.703°W	33.950°N	25.13	3.45
	Elsinore (Glen Ivy) rev [0]		15.48	6.53	2.38	117.590°W	33.829°N	105.13	3.11
	Whittier alt 1 [1]		4.76	6.86	1.25	117.702°W	33.874°N	78.31	1.71
	Compton [0]		20.15	7.25	1.67	118.043°W	33.702°N	236.24	1.18
UC33brAvg_FM32		System							36.02
	Whittier alt 2 [2]		3.03	7.51	0.88	117.737°W	33.886°N	28.43	11.72
	Chino alt 2 [1]		9.36	6.72	1.38	117.670°W	33.921°N	50.55	6.15
	Richfield [0]		6.51	6.30	1.56	117.803°W	33.889°N	298.91	3.57
	Peralta Hills [0]		2.70	7.25	0.76	117.751°W	33.846°N	179.10	3.38
	Elsinore (Glen Ivy) rev [0]		15.48	6.52	2.39	117.590°W	33.829°N	105.13	3.16
	Chino alt 2 [2]		9.36	6.73	1.39	117.670°W	33.921°N	50.55	1.77
	Compton [0]		20.15	7.31	1.64	118.043°W	33.702°N	236.24	1.28
	Whittier alt 2 [1]		3.56	7.12	1.06	117.719°W	33.878°N	63.66	1.15
UC33brAvg_FM32 (opt)		Grid							9.00
	PointSourceFinite: -117.751, 33.915		7.55	5.58	1.85	117.751°W	33.915°N	0.00	1.61
	PointSourceFinite: -117.751, 33.915		7.55	5.58	1.85	117.751°W	33.915°N	0.00	1.61
	PointSourceFinite: -117.751, 33.924		8.23	5.60	1.94	117.751°W	33.924°N	0.00	1.15
	PointSourceFinite: -117.751, 33.924		8.23	5.60	1.94	117.751°W	33.924°N	0.00	1.15
	PointSourceFinite: -117.751, 33.942		9.18	5.85	1.97	117.751°W	33.942°N	0.00	1.09
	PointSourceFinite: -117.751, 33.942		9.18	5.85	1.97	117.751°W	33.942°N	0.00	1.09
UC33brAvg_FM31 (opt)		Grid							8.69
	PointSourceFinite: -117.751, 33.915		7.57	5.56	1.86	117.751°W	33.915°N	0.00	1.57
	PointSourceFinite: -117.751, 33.915		7.57	5.56	1.86	117.751°W	33.915°N	0.00	1.57
	PointSourceFinite: -117.751, 33.942		9.15	5.87	1.96	117.751°W	33.942°N	0.00	1.11
	PointSourceFinite: -117.751, 33.942		9.15	5.87	1.96	117.751°W	33.942°N	0.00	1.11

APPENDIX E

APPENDIX E

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

1.0 General

1.1 Intent: These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 Geotechnical Consultant: Prior to commencement of work, the owner shall employ a geotechnical consultant. The geotechnical consultant shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

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

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

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

2.0 Preparation of Areas to be Filled

- 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 the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. 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.

- 2.2 Processing: Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. 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.
- 2.3 Overexcavation: In addition to removals and overexcavations 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 overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.
- 2.4 Benching: Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.
- 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 the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

- 3.1 General: Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant 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 the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.
- 3.2 Oversize: Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 12 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. 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 vertical feet of finish grade or within 2 feet of future utilities or underground construction.
- 3.3 Import: If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

- 4.1 Fill Layers: Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.
- 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 D1557-91).
- 4.3 Compaction of Fill: After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557-91). 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.

- 4.4 Compaction of Fill Slopes: In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557-91.
- 4.5 Compaction Testing: Field tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant'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).
- 4.6 Frequency of Compaction Testing: Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.
- 4.7 Compaction Test Locations: The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant 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, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

- 7.1 Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations.
- 7.2 Bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 ($SE > 30$). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum 90 percent of maximum from 1 foot above the top of the conduit to the surface, except in traveled ways (see Section 7.6 below).
- 7.3 Jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.
- 7.4 Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.
- 7.5 Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.
- 7.6 Trench backfill in the upper foot measured from finish grade within existing or future traveled way, shoulder, and other paved areas (or areas to receive pavement) should be placed to a minimum 95 percent relative compaction.