Appendix

Appendix C Geotechnical Investigation Irvine Operations Support Facility

Appendix

This page intentionally left blank.

Geotechnical Investigation and Design Recommendations, Irvine Operations Support Facility, 6427 Oak Canyon, Irvine, California

Prepared For

CITY OF IRVINE
c/o Griffin Structures

May 4, 2021

GMU Project No. 21-031-00

23241 Arroyo Vista
Rancho Santa Margarita | CA 92688
949.888.6513 | FX: 949.888.1380 | www.gmugeo.com

TRANSMITTAL

CITY OF IRVINE
c/o Griffin Structures
2 Technology Drive
Irvine, CA 92618

DATE: May 4, 2021

PROJECT: 21-031-00

ATTENTION: Mr. Tom Ottenstein

SUBJECT: Geotechnical Investigation and Design Recommendations, Irvine Operations Support Facility, 6427 Oak Canyon, Irvine, California

DISTRIBUTION:

Addressee: electronic copy

Lionakis
Attn: Brandon Rachac and Steven Kendrick (electronic copy)

23241 Arroyo Vista
Rancho Santa Margarita | CA 92688
949.888.6513 | FX: 949.888.1380 | www.gmugeo.com

TABLE OF CONTENTS

Description **Page**

INTRODUCTION

PURPOSE

This report summarizes the results of our geotechnical investigation for the proposed Irvine Operations Support Facility (IOSF) Site Project located at 6427 Oak Canyon, Irvine, California. The purpose of our investigation was to determine the nature of the subsurface soils, evaluate their in-place characteristics, and provide geotechnical recommendations with respect to site clearing, remedial grading, and design and construction of foundations and slabs for the proposed new structures and associated exterior site improvements. The scope of work was in accordance with our Agreement for Professional Consulting Services with the City of Irvine, previously approved on June 12, 2019.

SCOPE

The scope of our geotechnical investigation, as outlined in our December 24, 2020 proposal, was as follows:

1. Researched background information pertaining to the site, including information in your files, published geologic maps by CGS and/or USGS, and any available project plans and documents.

2. Marked five (5) hollow-stem auger (HSA) drill hole locations during our initial site visit and contacted Underground Service Alert (USA/Dig Alert) in order to provide advance notification of the subsurface drill holes planned within the subject site.

3. Performed a field subsurface exploration program consisting of advancing two HSA drill holes to a depth of approximately 51 feet, one HSA drill hole to a depth of approximately 21.5 feet, and two HSA drill holes to a depth of 11.5 feet within/near the footprint of the proposed prefabricated metal structures and above-ground tanks. Logged all field exploration work and obtained soil samples for geotechnical laboratory testing.

4. Performed laboratory testing on the soil samples obtained from the drill holes. Testing included in-situ moisture content and density, particle size distribution, maximum density and optimum moisture content, expansion index, shear strength characteristics, consolidation with one time rate, R-value, and full chemical analysis.

5. Interpreted and evaluated the field and laboratory data collected from our investigation and incorporated it with the previous data. Performed geotechnical engineering design analyses which included geologic hazards and seismicity study, settlement analysis, bearing capacity, lateral earth pressure, liquefaction analysis, seismic analysis in accordance with the American Society of Civil Engineers (ASCE) 7-16 standards, and pavement analysis.

6. Reviewed the reference (1) conceptual site plan showing the planned site improvements.

7. Prepared this formal geotechnical report for the proposed IOSF Site Project presenting our final geotechnical conclusions and recommendations to support the proposed new structures and associated exterior site improvements.

LOCATION

The IOSF Site Project is located at 6427 Oak Canyon within the City of Irvine, California. The general location of the project site is shown on Plate 1 – Location Map.

SITE DESCRIPTION

Currently, the subject site is occupied by a dog park and an operations support facility for the City of Irvine consisting of six buildings, a fueling station, and several storage and shade structures. The majority of the facility buildings are either completely or partially surrounded by either concrete flatwork or asphalt pavement while the dog park is covered by dirt and grass with trees along the western side. The Operations Support Facility building within the southeast portion of this site is surrounded by landscaping that consists of lawns and planter areas that contain shrubs and trees. The subject site is relatively flat, with only minor changes in grade.

PROJECT DESCRIPTION

The City of Irvine is planning to remove the existing dog park and a portion of the adjacent parking lot to the south to construct a fueling station with fuel islands and above ground tanks and a new parking area covered by solar panel canopies. Other parking areas within the site will also be reconfigured with solar panel canopies. Site improvements will also include a new canopy to replace the existing canopy along the west side of the Operations building, new pre-fabricated metal structures within the northeast and northwest portions of the site, new yard lighting throughout the site, a new dog path trail with lighting along the perimeter of the site, and new security gates.

A new detention basin will be constructed to treat surface runoff prior to discharging to a permanent drainage device. Infiltration at the site is not permitted due to the site being located within the El Toro Groundwater Plume area. Final grades of all improvements, excluding the detention basin, are planned to be near existing grades. The planned improvements are shown on Plate 2 – Geotechnical Map which uses the reference (1) concept site plan as the base map.

SUBSURFACE EXPLORATION

GMU conducted a field investigation program to characterize the subsurface soils in the vicinity of the proposed structures and site improvements. A total of five (5) hollow-stem auger (HSA) exploratory borings were performed to a maximum depth of 51 feet below ground surface (bgs). Relatively undisturbed Modified California samples and Standard Penetration Test (SPT) samples were obtained from the drill holes alternating every 5 feet for visual classification and laboratory testing. Groundwater was not encountered during our investigation.

The locations of our drill holes are shown on the attached Plate 2 – Geotechnical Map. The logs of our drill holes are included in Appendix A.

LABORATORY TESTING

Laboratory testing was performed on bulk and relatively undisturbed samples collected from the exploratory drill holes during our recent subsurface exploration. Testing on soil samples included the following:

- In-situ moisture and density;
- Sieve analysis;
- Maximum density and optimum moisture content;
- Expansion index;
- Consolidation;
- Direct shear tests;
- R-value; and
- Corrosion (pH, resistivity, chlorides, soluble sulfates)

The results of our laboratory testing are summarized on Table B-1 included in Appendix B.

GEOLOGIC FINDINGS

REGIONAL GEOLOGIC SETTING

The general location of the site is positioned in the southeastern portion of the Central Block of the Los Angeles Basin within an area known as the Tustin Plain (CDMG, 1980). Locally, the site exists on a series of coalescing alluvial fans between the Santa Ana Mountains and the San Joaquin

Hills. Review of the available logs, documents, and literature indicates the site is underlain predominantly by engineered fill (Qaf) and younger alluvial fan deposits (Qyfa) (USGS, 2006).

SUBSURFACE MATERIALS

Engineered Fill (Qaf)

Fill soils were encountered in all of the borings drilled within the site and consist of dark brown to brown, damp to moist, medium dense silty to clayey sands, and soft to firm sandy clays. The fills were placed as part of the previous grading operations and were observed to be approximately 3 to 3.5 feet in depth. However, deeper engineered fill may exist in local areas. The fine-grained fill soils largely possess medium to high plasticity/expansion characteristics.

Younger Alluvial Fan Deposits (Qyfa)

Younger alluvial fan deposits were encountered within the drill holes to the maximum depth explored (51 feet). The alluvial deposits encountered consisted mainly of light brown to yellowish brown, crudely stratified, firm to stiff sandy clays, and medium dense to dense silty sands, clayey sands, and poorly graded sands. The soils are generally dry to moist. Moisture contents and dry unit weights varied as summarized on Table B-1 of Appendix B.

LOCAL GROUNDWATER

No static groundwater was encountered within our drill holes to the maximum depth explored (51 feet). This is in general agreement with the depth of historically high groundwater provided in the reference Seismic Hazard Zone Report for the Tustin Quadrangle (CDMG, 2001) which indicates that historic high groundwater is in excess of 40 feet below the ground surface.

It should be noted that seasonal fluctuations in the groundwater level may occur. However, given that no groundwater was encountered to a depth of 51 feet below existing ground surface for this investigation, and historic high groundwater is in excess of 40 feet below the ground surface, it is anticipated that present and/or future groundwater is not expected to have an impact on the proposed construction.

SEISMIC CONDITIONS

Faulting and Seismicity

The site is not located within an official Alquist-Priolo Earthquake Fault Zone (Jennings, 1994; Hart and Bryant, 2007), and no known active faults are shown crossing the site on the reviewed geologic maps. The site is, however, located within close proximity to several surface faults that are presently zoned as active or potentially active by the California Geological Survey (CGS). The nearest known active fault is the San Joaquin Hills blind thrust fault which is located approximately 1 mile from the site and capable of generating a maximum earthquake magnitude (Mw) of 7.1. The site is also located within 10 miles of the Newport-Inglewood fault which is capable of generating a maximum earthquake magnitude (Mw) of 7.5.

Most of southern California is subject to some level of ground shaking (ground motion) because of movement along active and potentially active fault zones in the region. Given the proximity of the site to several active and potentially active faults, the site will likely be subject to earthquake ground motions in the future. The level of ground motion at a given site resulting from an earthquake is a function of several factors including earthquake magnitude, type of faulting, rupture propagation path, distance from the epicenter, earthquake depth, duration of shaking, site topography, and site geology.

Seismic Hazard Zones

According to the reference Seismic Hazard Zone map for the Tustin 7.5-Minute Quadrangle, the subject site does not lie within an area that is susceptible to earthquake-induced liquefaction or landsliding. However, a liquefaction zone is located on the west side of Jeffrey Road, approximately 1 mile northwest of the subject site.

GEOTECHNICAL ENGINEERING FINDINGS

STATIC SETTLEMENT/COMPRESSIBILITY

The proposed grades of the new prefabricated structures and site improvements are planned to be essentially at the same elevations as existing grades. Therefore, static settlement of the site will only be induced by introducing new structure loads to the existing grades and subsurface soils. The underlying alluvial deposits encountered were found to be medium dense/soft to dense/stiff and are considered susceptible to consolidation. Static settlement at the site was analyzed for new fill over in-situ alluvial deposits condition under our recommended bearing capacity utilizing the approximate preliminary assumed structure foundation loads by means of our consolidation laboratory test from the subject site. Calculated total static settlements under the anticipated foundation loads is approximately 1-inch with a differential settlement of 0.5 inch over a span of 40 feet.

LIQUEFACTION AND EARTHQUAKE-INDUCED SETTLEMENTS

Liquefaction

The subject site is not located within a zone of potential liquefaction per the Seismic Hazard Zone Map for the Tustin Quadrangle (CDMG, 2001). However, it is adjacent to a liquefaction zone located approximately 1 mile northwest of the subject site. Therefore, a liquefaction analysis was performed.

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are loose to moderately dense, saturated granular soils with poor drainage, such as silty sands or sands and gravels capped by or containing seams of impermeable sediment.

When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause increased hydrostatic pressure that induces liquefaction. Liquefaction can cause softening, which can result in large cyclic deformations. In loose granular soils, softening can also be accompanied by a loss of shear strength that may lead to large shear deformations or even flow failure under moderate to high shear stresses, such as beneath a foundation or sloping ground (NCEER/NSF, 1998).

Loose granular soil can also settle (compact) during liquefaction and as pore pressures dissipate following an earthquake. Very limited field data is available on this subject; however, in some cases, settlement on the order of 2 to 3 percent of the thickness of the liquefied zone has been measured.

Youd and Idriss, et al. (2001) methodology was used to evaluate the liquefaction resistance of subsurface soils within the site from our drill hole data. Our liquefaction analysis was based on the ASCE 7-16 ground motion criteria. The California Geological Survey (CDMG, 2001) groundwater data, which provides a historical high groundwater depth in excess of 40 feet, was used in our analysis.

Our liquefaction analysis determined that the potential for liquefaction at the subject site is low for a design groundwater table at or deeper than 40 feet below ground surface. The results of our liquefaction analysis are included in Appendix C.

Earthquake-Induced Settlement

If near-surface soils vary in composition both vertically and laterally, strong earthquake shaking can cause non-uniform compaction of soil strata, resulting in movement of the near-surface soils. But because the subsurface soils encountered at the site do not appear to change in thickness or consistency abruptly over short distances, we judge the probability of significant differential compaction at the site to be low. The total and differential earthquake-induced settlements are expected to be less than 1-inch and ½-inch, respectively. The results of our earthquake-induced settlement analysis are included in Appendix C.

Lateral Spreading

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvial material toward an open or "free" face such as an open body of water, channel, or excavation. In soils, this movement is generally due to failure along a weak plane and may often be associated with liquefaction. As cracks develop within the weakened material, blocks of soil displace laterally towards the open face. Cracking and lateral movement may gradually propagate away from the face as blocks continue to break free. Generally, failure in this mode is analytically unpredictable since it is difficult to determine where the first tension crack will occur.

Since the liquefaction potential is considered low at the site, and there are no creeks or open bodies of water within an appropriate distance from the site for lateral spreading to occur, the probability of lateral spreading occurring at the site during a seismic event is very low.

TSUNAMI, SEICHE, AND FLOODING

The site is not located on any State of California – County of Orange Tsunami Inundation Map for Emergency Planning. The potential for the site to be adversely impacted by earthquake-induced tsunamis is considered to be negligible since the site is located several miles inland from the Pacific Ocean coast at an elevation exceeding the maximum height of potential tsunami inundation.

The potential for the site to be adversely impacted by earthquake-induced seiches is considered to be negligible due to the lack of any significant enclosed bodies of water located in the vicinity of the site.

According to the County of Orange FEMA Flood Insurance Rate Map, the site is located within "Zone X", an area of minimal flood hazards. The potential for the site to be adversely impacted by significant flooding is considered very low.

SOIL EXPANSION

According to the 2019 CBC, soils meeting all four of the following provisions shall be considered expansive, except that tests for compliance with Items 1, 2, and 3 shall not be required if the test prescribed in Item 4 is conducted:

1. Plasticity index (PI) of 15 or greater (ASTM D4318).
2. More than 10 percent of the soil particles pass the #200 sieve (ASTM D422).
3. More than 10 percent of the soil particles are less than 5 micrometers in size (ASTM D422).
4. Expansion index greater than 20 (ASTM D422).

One expansion index (EI) test was performed on the near surface soils at the site. The expansion index of the tested soil was 113, which indicates a high expansion potential. Therefore, the shallow soils within the site have a potential for expansion and special design considerations will be required for design of the proposed improvements. Test results are provided in Appendix B.

SOIL CORROSION

Based on the test results for pH, soluble chlorides, sulfate, and minimum resistivity obtained during this investigation (presented in Appendix B), the on-site soils should be considered to have:

- A low minimum resistivity (severely corrosive to ferrous metals).
- A negligible sulfate exposure to concrete per the ACI 318 Table 19.3.1.1 (Exposure Class S_0).
- A low chloride content (i.e., less than 400 ppm).

Further corrosivity testing is recommended below proposed structures and improvements upon completion of precise grading and prior to construction to confirm the preliminary results provided herein.

EXCAVATION CHARACTERISTICS

Rippability

The soil materials to be encountered for the project can be excavated with conventional grading and excavation equipment.

Excavation and Trenching

We expect that the proposed corrective grading and utility trenches can be accomplished utilizing conventional excavating and trenching machines and backhoes. Significant quantities of gravels or oversize materials were not observed during our field investigation. However, zones of medium dense, sandy soils were encountered during our exploration, and these materials may be subject to caving or sloughing due to the granular nature of the uncemented soil matrix. Trench support requirements will be limited to those required by safety laws or other locations where trench slopes will need to be flattened or supported by shoring designed to suit the specific conditions exposed.

Excavation Stability

Excavations created for corrective grading and utility trenches will need to be laid back at an angle no greater than 1:1 up to a depth of 4 feet and/or shored per OSHA requirements. Below 4 feet, excavations will need to be laid back 1.5:1 as Type C soils were encountered during the investigation.

The above verbiage regarding excavation stability is presented for general guidance only. All aspects of construction stability are the responsibility of the contractor. All governing regulations in regards to excavation stability (i.e., OSHA, City of Irvine, etc.) should be followed.

Volume Change

In order to aid in the planning for the anticipated precise grading, we estimate that the change in volume of the on-site engineered fill excavated and placed as compacted fill at an average relative compaction of 90%, will result in about 2% to 5% decrease of volume or shrink.

IN-SITU SOIL MOISTURE CHARACTERISTICS

The fill and alluvial soils within the site are generally dry to very moist. Soils within the upper 5 to 10 feet have an average degree of saturation between 48 to 93 percent. Consequently, the potential for expansive soil movements to impact all improvements is high. It should be noted, however, that the moisture content within the upper several feet may vary depending on rainfall and the time of year in which grading occurs. One or more of the following measures during site grading may be required: 1) moisture conditioning, 2) locally drying back of the soils, and/or 3) mixing of the soils.

CONCLUSIONS

Based on the geologic and geotechnical findings, the following is a summary of our conclusions:

1. It is our opinion that the proposed project is feasible assuming all applicable recommendations contained herein are implemented.

2. The sandy alluvial deposits may be subject to caving or sloughing due to the granular nature of the uncemented soil matrix.

3. Groundwater is not anticipated to be a design constraint and/or encountered during the planned precise grading or during the installation of shallow underground utilities.

4. There are no known active faults crossing the subject site. The site seismicity is typical for the Irvine area. Structure design should be in accordance with the current CBC.

5. The magnitude of total static settlements beneath the proposed structures (i.e., prefabricated metal structures and aboveground tanks) are not expected to exceed 1 inch.

6. The potential for liquefaction is considered low and total earthquake-induced settlement is expected to be less than 1 inch.

7. The potential for liquefaction-induced lateral spreading is considered very low.

8. The on-site soils have a high expansion potential. Due to the potential for expansive soils, special design considerations will be required for the foundations, slabs, and flatwork associated with the proposed improvements. The previously graded site contains soils within the upper 5 to 10 feet that have an average degree of saturation between 48 and 93% indicating damp to moist conditions and a high potential for expansive soil movements.

9. The on-site soils are corrosive to ferrous metals and have a potential for chloride corrosion exposure to concrete (i.e., as defined by the CBC) and reinforcement. Special design considerations will be required for proposed improvements in contact with on-site soil.

RECOMMENDATIONS

DEVELOPMENT FEASIBILITY

Based on the geologic and geotechnical findings, it is our opinion that the proposed grading and construction shown on the reference (1) precise grading plans is feasible and practical from a geotechnical standpoint if accomplished in accordance with the City of Irvine grading and building requirements and the recommendations presented in this report. Geotechnical recommendations provided in this report include the following:

- Recommendations for corrective grading for the proposed improvements (i.e., foundations, structure pads, and pavement/flatwork areas);
- Design parameters for spread and continuous footings, slab-on-grade systems, and pole foundations to support the proposed structures and site improvements;
- Utility trench and structure excavations, and backfill recommendations; and
- Asphalt pavement and concrete flatwork recommendations.

GENERAL SITE PREPARATION AND GRADING

General

The following recommendations pertain to any required grading associated with the proposed improvements. All site preparation and grading should be performed in accordance with the City of Irvine grading code requirements and the recommendations presented in this report.

Clearing

All significant organic material such as weeds, brush, tree branches, or roots, or construction debris such as old irrigation lines, asphalt concrete, and other decomposable material should be removed from the area to be graded. No rock or broken concrete greater than 6 inches in diameter should be utilized in the fills.

Corrective Grading

Corrective grading will serve to create a firm and workable platform for construction of the proposed developments such as new prefabricated structures and associated pavement and site flatwork.

It should be noted that the recommendations provided herein are based on our subsurface exploration and knowledge of the on-site geology. Actual removals may vary in configuration and volume based on observations of geologic materials and conditions encountered during grading. The bottom of all remedial grading removals should be observed by a GMU representative to verify the suitability of in-place soil prior to performing scarification and recompaction. Corrective grading recommendations are outlined below.

Structure Foundations/Slabs: Grading recommendations for support of the new prefabricated metal structure pads, above-ground tank pads, and miscellaneous shallow spread/continuous foundations should consist of the following:

- The existing ground surfaces should be over-excavated to a depth of at least 24 inches below existing grades or to a depth of at least 18 inches below the bottoms of new footings or slabs, whichever is deeper. The lateral extent of the over-excavation should extend a minimum of 3 feet beyond the perimeter edges of the footings or slabs, where possible.
- The bottoms of the over-excavations should then be scarified to a depth of at least 6 inches, moisture conditioned to 3% above optimum moisture content, and recompacted to at least 90% relative compaction.
- Following the approval and processing of the over-excavation bottom by a representative of GMU, the onsite material may be used as fill material to achieve the planned subgrade elevation.
- The fill material should then be placed in 6- to- 8-inch-thick lifts, moisture conditioned to at least 3% above optimum moisture content, blended to achieve uniform moisture content, and compacted to achieve 90% relative compaction.

Flatwork/Pavement Areas: Grading recommendations for the support of asphalt and concrete pavement and flatwork areas should consist of the following:

- The upper 18 inches of existing fill within new pavement and flatwork areas should be removed. The removal should, at a minimum, provide for at least 1 foot of new engineered fill supporting the structural asphalt and concrete flatwork sections.
- The bottom of the removal should be scarified to a depth of 6 inches, moisture conditioned to least 3% above optimum moisture content, and recompacted to at least 90% relative compaction.

Detention Basin: The following corrective grading recommendations for the proposed stormwater detention basin are based on preliminary conceptual plans. These recommendations may require revisions after the final design of the proposed detention basin has been determined. Preliminary corrective grading recommendations for the detention basin are as follows:

- If a structure is planned for the stormwater detention basin, then the subgrade for the structure should be over-excavated 2 feet to provide a minimum of 2 feet of engineered fill under the design section for the basin. The over-excavation should extend at least 2 feet outside the footprint of the structure.
    - The bottom of the excavation should then be scarified to a depth of at least 6 inches, moisture conditioned to 3% above optimum moisture content, and compacted to at least 90% relative compaction.
    - Following the approval and processing of the over-excavation bottom by a representative of GMU, the onsite material may be used as fill material to achieve the planned subgrade elevation.
    - The fill material should then be placed in 6- to- 8-inch-thick lifts, moisture conditioned to at least 3% above optimum moisture content, blended to achieve uniform moisture content, and compacted to achieve 90% relative compaction.
- If a liner is planned for the stormwater detention basin, then only processing of the liner subgrade, as described above, is required.

FILL MATERIAL AND PLACEMENT

Suitability and Selective Grading

All on-site soil materials within the limits of grading are suitable for use as compacted fill if care is taken to remove all significant organic and other decomposable debris and to separate and selectively place and/or stockpile rock materials larger than 6 inches in diameter.

Compaction Standard and Moisture Requirements

All on-site soil material used as compacted fill, material processed in place, or used to backfill trenches should be moistened, dried, or blended as necessary to achieve a minimum of 3% over optimum moisture content (i.e., if the optimum moisture content is 12%, the compacted fill's moisture content shall be at least 15%), and densified to at least 90% relative compaction as determined by ASTM Test Method D1557. Final surface subgrade soils should be frequently watered in order to keep the soil moist until structure slabs, flatwork, or any other final improvements are installed. If the soil is allowed to dry out and deep shrinkage cracks appear, at least the upper 6 inches should be re-processed, moisture conditioned to 3% over optimum, and re-compacted.

Use of Rock or Broken Concrete

No rock or broken concrete greater than 6 inches in diameter should be utilized in the fills.

STRUCTURE SEISMIC DESIGN

The average shear wave velocity for the upper 100 feet of subsurface soils (V_{s30}) was estimated to be approximately 760 feet per second (fps) based on the empirical relationship between SPT blow counts and shear wave velocity of DH-1 and DH-5. Based on this shear wave velocity, Table 20.3-1 of ASCE 7-16 indicates that the site should be designated as Site Class D which corresponds to a "stiff soil" profile. The seismic design coefficients based on ASCE 7-16 are listed below in Table 1.

Table 1. 2019 CBC and ASCE 7-16 Seismic Design Parameters
(To be utilized as per the requirements of Section 11.4.8 of ASCE 7-16)

Seismic Item	Design Value	2016 ASCE 7-16 or 2019 CBC Reference
Site Class based on soil profile (ASCE 7-16 Table 20.3-1)	D[(a)]	ASCE 7-16 Table 20.3-1
Short Period Spectral Acceleration S_s	1.246[(a)]	CBC Figures 1613.2.1 (1-8)
1-sec. Period Spectral Acceleration S_1	0.446[(a)]	CBC Figures 1613.2.1 (1-8)
Site Coefficient F_a (2019 CBC Table 1613.2.3(1))	1.002[(a)]	CBC Table 1613.2.3 (1)
Site Coefficient F_v (2019 CBC Table 1613.2.3(2))	1.854[(b)]	CBC Table 1613.2.3 (2)
Short Period MCE* Spectral Acceleration S_{MS} $S_{MS} = F_a S_s$	1.249[(a)]	CBC Equation 16-36
1-sec. Period MCE Spectral Acceleration S_{M1} $S_{M1} = F_v S_1$	0.827[(b)]	CBC Equation 16-37
Short Period Design Spectral Acceleration S_{DS} $S_{DS} = 2/3 S_{Ms}$	0.832[(a)]	CBC Equation 16-38
1-sec. Period Design Spectral Acceleration S_{D1} $S_{D1} = 2/3 S_{M1}$	0.551[(b)]	CBC Equation 16-39
Short Period Transition Period T_S (sec) $T_S = S_{D1}/S_{DS}$	0.663[(b)]	ASCE 7-16 Section 11.4.6
Long Period Transition Period Tl (sec)	8[(b)]	ASCE 7-16 Figures 22-14 to 22-17
MCE[(c)] Peak Ground Acceleration (PGA)	0.521[(a)]	ASCE 7-16 Figures 22-9 to 22-13
Site Coefficient F_{PGA} (ASCE 7-16 Table 11.8-1)	1.100[(a)]	ASCE 7-16 Table 11.8-1
Modified MCE[(c)] Peak Ground Acceleration (PGA_M)	0.573[(a)]	ASCE 7-16 Equation 11.8-1
Seismic Design Category	D[(b)]	ASCE 7-16 Tables 11.6.1 and 11.6.2

(a) Design Values Obtained from USGS Earthquake Hazards Program website that are based on the ASCE-7-16 and 2019 CBC and site coordinates of N33.676450° and W117.764625°.
(b) Design Values Determined per ASCE Table 11.4-2 and CBC Equations 16-36 through 16-39.
(c) MCE: Maximum Considered Earthquake.

The Maximum Considered Earthquake (MCE) Peak Horizontal Ground Acceleration (PGA_M) is 0.57g as determined in accordance with ASCE 7-16. This PGA_M is primarily dominated by earthquakes with a mean magnitude of 6.6 at a mean distance of 9 miles from the site using the USGS 2014 Interactive Deaggregation website.

Since the Site Class is designated as D and the S_1 value is greater than or equal to 0.2, the 2019 CBC requires either a site-specific ground motion hazard analysis per Section 21.2 of ASCE 7-16 or the application of Exception 2 of Section 11.4.8 of ASCE 7-16. Exception 2 states that a site-specific ground motion hazard analysis is not required provided that the value of the seismic response coefficient, C_s, is conservatively calculated by the project structural engineer using Equation 12.8-2 of ASCE 7-16 for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Equation 12.8-3 for $T_L \geq T > 1.5T_s$ or Eqn. 12.8-4 for $T > T_L$.

It should be recognized that much of southern California is subject to some level of damaging ground shaking as a result of movement along the major active (and potentially active) fault zones that characterize this region. Design utilizing the 2019 CBC is not meant to completely protect against damage or loss of function. Therefore, the preceding parameters should be considered as minimum design criteria.

SHALLOW FOUNDATION AND SLAB-ON-GRADE RECOMMENDATIONS

General

The following design parameters are considered applicable for shallow foundations and slab-on-grade systems that may be constructed for the proposed prefabricated metal structures, aboveground tanks, and other miscellaneous improvements provided the grading recommendations outlined above are followed.

Soil Parameters

<u>Bearing Material:</u> New Engineered Fill

<u>Allowable Bearing Capacity:</u>

- Allowable bearing capacity: 2,500 psf for minimum footing size
- May be increased by 500 psf for each additional foot of footing depth and by 150 psf for each additional foot of footing width to a maximum of 4,000 psf
- Above value may be increased by 1/3 for temporary loads such as wind or seismic

<u>Lateral Foundation Resistance:</u>

- Allowable passive resistance: 250 psf/ft (disregard upper 6 inches, max 2,500 psf)
- Allowable friction coefficient: 0.33
- Above values may be combined without reduction and may be increased by 1/3 for temporary loads such as wind or seismic

<u>Subgrade Reaction Modulus:</u> 100 pci

Minimum Foundation Design Parameters

Minimum Footing Sizes (for designing):

- Spread (i.e., Square): 1.5 feet wide and 1.5-foot embedment below lowest adjacent soil grade (depth)
  - Due to the expansive nature of the onsite soils, grade beams to tie together the individual pad footings for canopy structures should be considered by the structural engineer.
- Continuous: 1.5 feet wide and 1.5-foot embedment below lowest adjacent soil grade (depth)

Settlement:

- Static:
  - Total: 1.0"
  - Differential: 0.5" over 40 feet
- Seismic Settlement:
  - Total: 1.0"
  - Differential: 0.5" over 40 feet

Slab Subgrade and Slab Design

Minimum Thickness:

- Prefabricated Metal Structure Slab: 6 inches
- Aboveground Tank Slab: 8 inches
- Final slab thickness should be determined by the structural engineer.

Minimum Slab Reinforcement:

- Minimum slab reinforcement shall not be less than No. 4 bars placed at 18 inches on center.
- Welded wire mesh is not recommended. Care should be taken to position the reinforcement bars in the center of the slab.
- Final reinforcement should be determined by the structural engineer.
- Final design details should be provided to our office by the design structural engineer for review.

Slab Subgrade:

- The on-site soils and subgrade soil should be moisture conditioned to a minimum of 3% over optimum moisture content.
- 4-inch section of ¾-inch gravel or crushed stone layer (i.e., to act as a capillary break) placed over engineered fill.
- Sand above the moisture retarder/barrier (i.e., directly below the slab) is not a geotechnical issue. This should be provided by the structural engineer of record or architect based on the type of slab, potential for curling, etc.

MOISTURE VAPOR TRANSMISSION

Moisture Vapor Retarder

A vapor retarder or barrier such as Stego 15 Mil Class A or equivalent should be utilized overtop of the required gravel/stone course for the prefabricated metal structure slabs. The retarder/barrier should be installed as follows:

- Below moisture-sensitive floor areas.
- Installed per manufacture's specifications as well as with all applicable recognized installation procedures such as ASTM E1643.
- Joints between the sheets and the openings for utility piping should be lapped and taped. If the retarder/barrier is not continuously placed across footings/ribs, the retarder/barrier should, as a minimum, be lapped into the sides of the footing/rib trenches down to the bottom of the trench.
- A 4-inch section of ¾-inch gravel or crushed stone layer shall be provided directly below the moisture vapor retarder/barrier to act as a moisture or capillary break.
- Punctures in the vapor retarder/barrier should be repaired prior to concrete placement.

The need for sand and/or the amount of sand above the moisture vapor retarder/barrier should be specified by the owner with approval by the structural engineer. The selection of sand above the retarder/barrier is not a geotechnical engineering issue and is hence outside our purview. However, if sand is to be placed above the retarder/barrier for this project, the sand should be placed in a dry condition.

Water Vapor Transmission Discussion

As discussed above, placement of a moisture vapor retarder/barrier below all slab areas is recommended where moisture sensitive flooring will be placed. This moisture vapor retarder/barrier recommendation is intended only to reduce moisture vapor transmissions from the soil beneath the concrete and is consistent with the current standard of the industry for construction in southern California. It is not intended to provide a "waterproof" or "vapor proof" barrier or reduce vapor transmission from sources above the retarder. Sources above the retarder/barrier include any sand placed on top of the retarder/barrier (i.e., to be determined by the project structural designer) and from the concrete itself (i.e., vapor emitted during the curing process). The evaluation of water vapor from any source and its effect on any aspect of the proposed living space above the slab (i.e., floor covering applicability, mold growth, etc.) is outside our purview and the scope of this report.

Floor Coverings

Prior to the placement of flooring, the floor slabs should be properly cured and tested to verify that the water vapor transmission rate (WVTR) is compatible with the flooring requirements.

POLE FOUNDATION DESIGN PARAMETERS

The following design parameters are considered applicable for pole foundation systems associated with the proposed site improvements (i.e., canopies, light poles, fencing, etc.) provided the grading recommendations outlined above are followed. Final depth and size of pole foundations should be determined by the project structural engineer.

Soil Parameters

<u>Bearing Material:</u> Existing Engineered Fill or Competent Alluvium

<u>End Bearing:</u>

- 1,600 psf (for minimum pole foundation depth of 2 feet)
    - May be increased by 450 psf for each additional foot of pole depth and by 90 psf for each additional foot of pole diameter to a maximum of 5,000 psf
    - One-third increase for wind or seismic loading
    - Assumes bottom of drill hole thoroughly cleaned of all loose soil prior to pour.

<u>Allowable Average Unit Skin Friction:</u>

- 150 psf
    - One-third increase for wind or seismic loading

<u>Allowable Passive Resistance:</u>

- Allowable passive resistance: 250 psf/ft of pole foundation depth
    - Disregard the upper 1 foot due to possible soil disturbance.
    - Passive may be increased by an isolated pile factor of 2 (e.g., 500 psf/ft of pole diameter per foot of depth) when center-to-center distance of poles is greater than 3 times their diameter.
    - One-third increase for wind or seismic loading.

Minimum Pole Foundation Design Parameters

Improvements > 6 feet in height:

- Minimum pole foundation diameter: 18 inches
- Minimum pole foundation depth: 4 feet (final depth to be determined by structural engineer)

Improvements ≤ 6 feet in height:

- Minimum pole foundation diameter: 12 inches
- Minimum pole foundation depth: 2 feet (final depth to be determined by structural engineer)

Construction Considerations for Pole Foundations

GMU recommends the following construction considerations for the pole foundations:

- Drilling for pole foundations should be performed under the observation of GMU to confirm the poles have been extended to the design embedment depths.
- The alluvial deposits may be subject to caving due to the granular nature of some subsurface alluvial deposits. Casing or other means of sidewall stabilization and protection may be required.
- The drill holes should be cleaned of loose soil prior to placement of rebar and concrete.

UTILITY TRENCH BACKFILL CONSIDERATIONS

General

New utility line pipeline trenches should be backfilled with select bedding materials beneath and around the pipes (pipe zone) and compacted soil above the pipe bedding. Recommendations for the types of the materials to be used and the proper placement of these materials are provided in the following sections.

Pipe Zone (Bedding and Shading)

The pipe bedding and shading materials should extend from at least 6 inches below the pipes to at least 12 inches above the crown of the pipes. Pipe bedding and shading should consist of either clean sand with a sand equivalent (SE) of at least 30, or crushed rock. If crushed rock is used, it should consist of ¾-inch crushed rock that conforms to Table 200-1.2.1 (A) of the 2021 "Greenbook." Pipe bedding and shading should also meet the minimum requirements of the City of Irvine. If the requirements of the City are more stringent, they should take precedence over the geotechnical recommendations. Sufficient laboratory testing should be performed to verify the bedding and shading meet the minimum requirements of the Greenbook and City of Irvine grading code.

Based on our subsurface exploration and knowledge of the onsite materials, the soils that will be excavated from the pipeline trenches will not meet the recommendations for pipe bedding and shading materials; therefore, imported materials will be required for pipe bedding and shading.

Granular pipe bedding and shading material having a sand equivalent of 30 or greater should be properly placed in thicknesses not exceeding 3 feet, and then sufficiently flooded or jetted in place. Crushed rock, if used, should be capped with filter fabric (Mirafi 180N, or equivalent) to prevent the migration of fines into the rock.

Trench Backfill

All existing soil material within the limits of the site are considered suitable for use as trench backfill above the pipe bedding and shading zone if care is taken to remove all significant organic and other decomposable debris, moisture condition the soil materials as necessary, and separate and selectively place and/or stockpile any inert materials larger than 6 inches in maximum diameter.

Imported soils are not anticipated for backfill since the on-site soils are suitable. However, if imported soils are used, the soils should consist of clean, granular materials with physical and chemical characteristics similar to or better than those described herein for on-site soils. Any imported soils to be used as backfill should be evaluated and approved by GMU prior to placement.

Soils to be used as trench backfill should be moistened, dried, or blended as necessary to achieve a minimum of 3% over optimum moisture content (i.e., if the optimum moisture content is 12.0%, the compacted fill’s moisture content shall be at least 15.0%), placed in lifts which, prior to compaction, shall not exceed the thickness specified in Section 306-12.3 of the 2021 “Greenbook” for various types of equipment, and mechanically compacted/densified to at least 90% relative compaction as determined by ASTM Test Method D1557. Jetting is not permitted in this trench zone.

No rock or broken concrete greater than 6 inches in maximum diameter should be utilized in the trench backfills.

DETENTION BASIN RECOMMENDATIONS

Our grading recommendations for the proposed detention basin are outlined in the “Corrective Grading” section of this report. These recommendations are based on conceptual plans provided by Lionakis, the project Civil engineer, and may require revisions once the final design of the detention basin has been determined. Based on the final design of the proposed detention basin, supplemental recommendations can be provided as necessary.

PAVEMENTS

Asphalt Pavement Design

Pavement engineering analyses were performed in accordance with the Caltrans Highway Design Manual. Topic 633 of the Caltrans Design Manual was followed to develop pavement thickness design recommendations. This design method considers the relationship between the subgrade R-value, gravel factor of the various pavement layers, and the traffic index (TI).

Pavement thickness recommendations were developed based on an assumed range of traffic indices (TI's) for a 20-year design life. A traffic engineer should review and confirm the appropriateness of the TI's used in our analysis. Based on our R-value test result and shallow soil types encountered, an R-value of 5 was used for the design.

The actual service life of the pavement can be extended with proper maintenance and rehabilitation (i.e., slurry seal every 7 years, mill-and-overlay every 12-16 years, etc.)

The following table summarizes the recommended minimum pavement thicknesses.

Table 2. Conventional Asphalt Concrete (AC) Pavement Thickness Recommendations

Location	Assumed Traffic Index	Composite Pavement Asphalt Concrete over Aggregate Base (AC over AB over subgrade)	Full-Depth Asphalt Concrete (AC over subgrade)
Passenger Vehicle Parking Stalls	4.5	4.0" AC over 9.5" AB over Properly Prepared Subgrade	7.0" AC over Properly Prepared Subgrade
Drive Aisles	5.5	4.0" AC over 12.0" AB over Properly Prepared Subgrade	8.5" AC over Properly Prepared Subgrade
Heavy Truck Areas	7.0	4.5" AC over 16.0" AB over Properly Prepared Subgrade	11.0" AC over Properly Prepared Subgrade

Implementing any of these recommendations involves:

- Grading the existing site to create sufficient depth for the recommended asphalt concrete (AC) or asphalt concrete over aggregate base (AC/AB) sections;
- Processing and re-compacting the exposed subgrade material to a depth of at least 12 inches in accordance with Standard Specifications for Public Works Construction (Greenbook) Sections 301-1.2 and 301-1.3. The required relative compaction of the subgrade is 90% minimum with a moisture content at least 3% above optimum moisture content for the composite section (AC/AB), and 95% relative compaction with a moisture content at least 3% above optimum moisture content for the full depth section (AC over subgrade). Maximum density and optimum moisture content of the subgrade should be determined by ASTM D1557;
- Placing the aggregate base (AB) section to at least 95% relative compaction and moisture conditioning to near optimum moisture content. Maximum density and optimum moisture content of the aggregate base should be determined by ASTM D1557; and
- Placing the asphalt concrete (AC) section in lifts not exceeding Greenbook minimum lift thicknesses.

All materials used and work performed should meet the current edition of the Standard Specifications for Public Works Construction (Greenbook) with all supplements, unless superseded by the recommendations provided within this report.

The AB section may be Crushed Miscellaneous Base (CMB) or Crushed Aggregate Base (CAB) meeting Greenbook Section 200-2.

We recommend using the Greenbook Type IIIC3 AC mix with PG 64-10 asphalt binder for both the AC surface and AC base course sections.

Concrete Pavement Design

Driveways, vehicular drives, and appurtenant concrete paving such as trash receptacle bays, will require PCC pavement. Assuming a T.I. of 6 to 7, a design section of 8 inches of PCC over 6 inches AB should be adequate. The AB should be compacted to a minimum of 95% relative compaction as per ASTM D1557 and moisture conditioned to near optimum moisture content.

CONCRETE FLATWORK DESIGN AND CONSTRUCTION

We recommend that the subgrade for the subject concrete flatwork be moisture conditioned to 3% above optimum moisture content (i.e., if the optimum moisture content is 12%, the compacted fill's moisture content should be at least 15%) to a depth of 18 inches below finish grade and compacted to 90% relative compaction as per ASTM D1557.

Concrete flatwork should be designed and constructed per the City of Irvine Standard Plans (such as Standard Plans 201, 204, 205, and/or 206) or the flatwork recommendations provided in Appendix D, whichever is more conservative.

CONCRETE

Due to low soil resistivity and medium chloride levels, the potential for on-site corrosion to ferrous metals and hence reinforcing steel are severe. Thus, we recommend the following:

Structural Elements (i.e., foundations, slabs, etc.)

- Cement Type: Type II/V
- Maximum Water Cement Ratio: 0.50
- Minimum Strength: 4,000 psi (geotechnical perspective only)

Utilization of CBC moderate sulfate level requirements will also serve to reduce the permeability of the concrete and help minimize the potential of water and/or vapor transmission through the concrete. Wet curing of the concrete per ACI Publication 308 is also recommended.

Non-structural Elements (i.e., flatwork, pavement, etc.)

Concrete mix design shall be selected by the concrete designer such that sulfate and chloride attack mitigations are balanced with shrinkage crack control. Concrete mix design is outside the geotechnical engineer's purview.

The aforementioned recommendations in regards to all concrete (i.e., structural and non-structural) are made from a soil's perspective only. Final concrete mix design is beyond our purview. All applicable codes, ordinances, regulations, and guidelines should be followed in regard to designing a durable concrete with respect to the potential for sulfate exposure from the on-site soils and/or changes in the environment.

CORROSION PROTECTION OF METAL STRUCTURES

The results of the laboratory chemical tests performed on soil samples collected within the subject area indicate that the on-site soils are corrosive to ferrous metals. Consequently, metal structures which will be in direct contact with the soil (i.e., underground metal conduits, pipelines, metal sign posts, metal door frames, etc.) and/or in close proximity to the soil (wrought iron fencing, etc.)

may be subject to corrosion. The use of special coatings or cathodic protection around buried metal structures has been shown to be beneficial in reducing corrosion potential. The potential for corrosion of ferrous metal reinforcing elements embedded in structural concrete will be reduced by the use of the recommended maximum water/cement ratio for concrete and additional concrete cover.

The laboratory testing program performed for this project does not address the potential for corrosion to copper piping. In this regard, a corrosion engineer should be consulted to perform more detailed testing and develop appropriate mitigation measures (if necessary). Otherwise, the on-site soils should be considered corrosive to copper.

The above discussion is provided for general guidance in regards to the corrosiveness of the on-site soils to typical metal structures used for construction. Detailed corrosion testing and recommendations for protecting buried ferrous metal and/or copper elements is beyond our purview.

PLANTERS AND TREES

Where new trees or large shrubs are to be located in close proximity to new concrete flatwork, pavement, or structure foundations, rigid moisture/root barriers should be placed around the perimeter of the flatwork to at least 2 feet in depth in order to offer protection to the adjacent flatwork against potential root and moisture damage. Existing mature trees near flatwork areas should also incorporate a rigid moisture/root barrier placed at least 2 feet in depth below the top of the flatwork, pavement, or structure foundations.

SURFACE DRAINAGE

Surface drainage should be carefully controlled during and after grading to prevent ponding and uncontrolled runoff adjacent to structures and/or other properties. Particular care will be required during grading to maintain slopes, swales, and other erosion control measures needed to direct runoff toward permanent surface drainage facilities. Positive drainage of at least 2% away from the perimeters of the structures and site pavements should be incorporated into the design. In addition, it is recommended that nuisance water be directed away from the perimeter of the structures by the use of swales and/or area drains in adjacent landscape and flatwork areas.

PLAN REVIEW/GEOTECHNICAL TESTING DURING GRADING/FUTURE REPORTS

Plan Review

The final precise grading plans, foundation plans, and landscape plans should be reviewed by our office to verify that the plans have incorporated the recommendations presented in this report.

Geotechnical Testing

It is recommended that geotechnical observation and testing be performed by GMU during the following stages of precise grading and construction:

- During site clearing and grubbing.
- During removal of any buried irrigation lines or other subsurface structures.
- During all phases of grading including over-excavation, temporary excavations, removals, scarification, ground preparation, moisture conditioning, proof-rolling, over-excavation, and placement and compaction of all fill materials.
- During installation of all conventional foundations and floor slab elements.
- During backfill of the detention basin and underground utilities.
- During hardscape subgrade and base placement and compaction.
- During pavement section placement and compaction.
- When any unusual conditions are encountered.

Future Reports

It is expected that a geotechnical observation report will be required following all site precise grading and construction.

LIMITATIONS

All parties reviewing or utilizing this report should recognize that the findings, conclusions, and recommendations presented represent the results of our professional geological and geotechnical engineering efforts and judgements. Due to the inexact nature of the state of the art of these professions and the possible occurrence of undetected variables in subsurface conditions, we cannot guarantee that the conditions actually encountered during grading and foundation installation will be identical to those observed and sampled during our study or that there are no unknown subsurface conditions which could have an adverse effect on the use of the property. We have exercised a degree of care comparable to the standard of practice presently maintained by other professionals in the fields of geotechnical engineering and engineering geology, and believe that our findings present a reasonably representative description of geotechnical conditions and their probable influence on the grading and use of the property.

Because our conclusions and recommendations are based on a limited amount of current and previous geotechnical exploration and analysis, all parties should recognize the need for possible revisions to our conclusions and recommendations during grading of the project. Additionally, our conclusions and recommendations are based on the assumption that our firm will act as the geotechnical engineer of record during grading of the project to observe the actual conditions exposed, to verify our design concepts and the grading contractor's general compliance with the project geotechnical specifications, and to provide our revised conclusions and recommendations should subsurface conditions differ significantly from those used as the basis for our conclusions and recommendations presented in this report.

Detailed corrosion testing and recommendations for protecting buried ferrous metal and/or copper elements are beyond our purview.

This report has not been prepared for use by other parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.

CLOSURE

We are pleased to present the results of our geotechnical investigation for this project. The Plates and Appendices that complete this report are listed in the Table of Contents.

If you have any questions concerning our findings or recommendations, please do not hesitate to contact us and we will be happy to discuss them with you.

Respectfully submitted,

Dustin R. Williams, M.Sc., PG 9883
Senior Staff Geologist

Ashley A. Varni, M.Sc., PE 89576
Project Engineer

Alan B. Mutchnick, PG, CEG 1789
Associate Engineering Geologist

dw/aav/21-031-00R (5-4-21)

REFERENCES

SITE SPECIFIC REFERENCES

(1) Proposed Concept Site Plan, prepared by Lionakis, dated March 02, 2021.

TECHNICAL REFERENCES

ASCE, 2018, ASCE 7 Hazard Tool, web site address: https://asce7hazardtool.online/.

California Building Standards Commission and International Conference of Building Officials, 2019, *2019 California Building Code.*

California Division of Mines and Geology, 1980, Geology and Engineering Geologic Aspects of the South Half Tustin Quadrangle, Orange County, California, Open File Report 81-21A.

CDMG Staff, 2001, Seismic Hazard Evaluation of the Tustin 7.5-Minute Quadrangle, Orange County, California: California Division of Mines and Geology Open File Report 97-20.

CGS Staff, 2004, Preliminary Digital Geologic Map of the Santa Ana 30'x60' Quadrangle, Southern California: California Geological Survey Open File Report 99-172, Version 2.0.

Hart, E.W., and Bryant, W.A., 2007, Fault-rupture hazard zones in California: CDMG Special Publication 42, 50p.

Idriss, I.M. and Boulanger, R.W., 2008, *Soil Liquefaction during Earthquakes*, Earthquake Engineering Research Institute," MNO-12.

Jennings, C.W., 1994, Fault activity map of California and adjacent areas: CDMG Data Map No. 6, scale 1:750,000.

Pradel, D., 1998, *Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils,* Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 124, No. 4, pgs. 364-368.

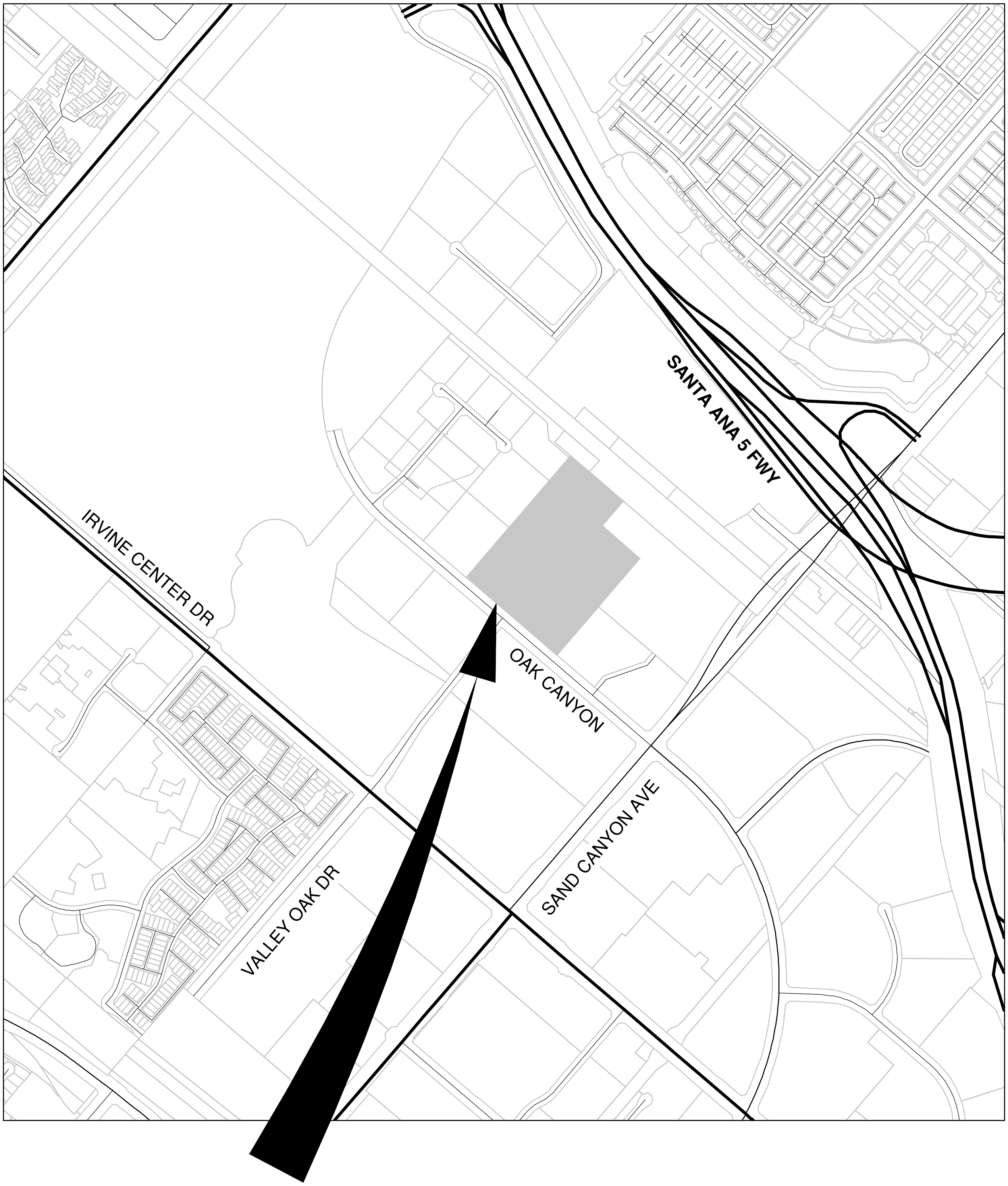
Skempton, A.W., 1986, *Standard Penetration Test Procedures and the effects in Sands of Overburden Pressure, Relative Density, Particle Size, Ageing, and Overconsolidation,* Geotechnique, Volume 36, No. 3, September 1, 1986.

Standard Specifications for Public Works Construction, by Public Works Standards, Inc., 2021, *The Greenbook 2021 Edition*.

U.S. Geological Survey, 2014 Interactive De-aggregations Program; web site address: https://earthquake.usgs.gov/hazards/interactive/.

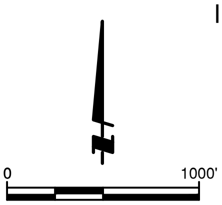
Youd, T.L. and Idriss, I.M., et al. (1997), *Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils,* National Center for Earthquake Engineering Research, Technical Report NCEER - 97-0022, January 5, 6, 1996.

Youd, T.L., Idriss, I.M., et al. (2001), *Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils*, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 10, October 2001.



IRVINE OPERATIONAL SUPPORT FACILITY

6427 OAK CANYON
IRVINE, CA 92618

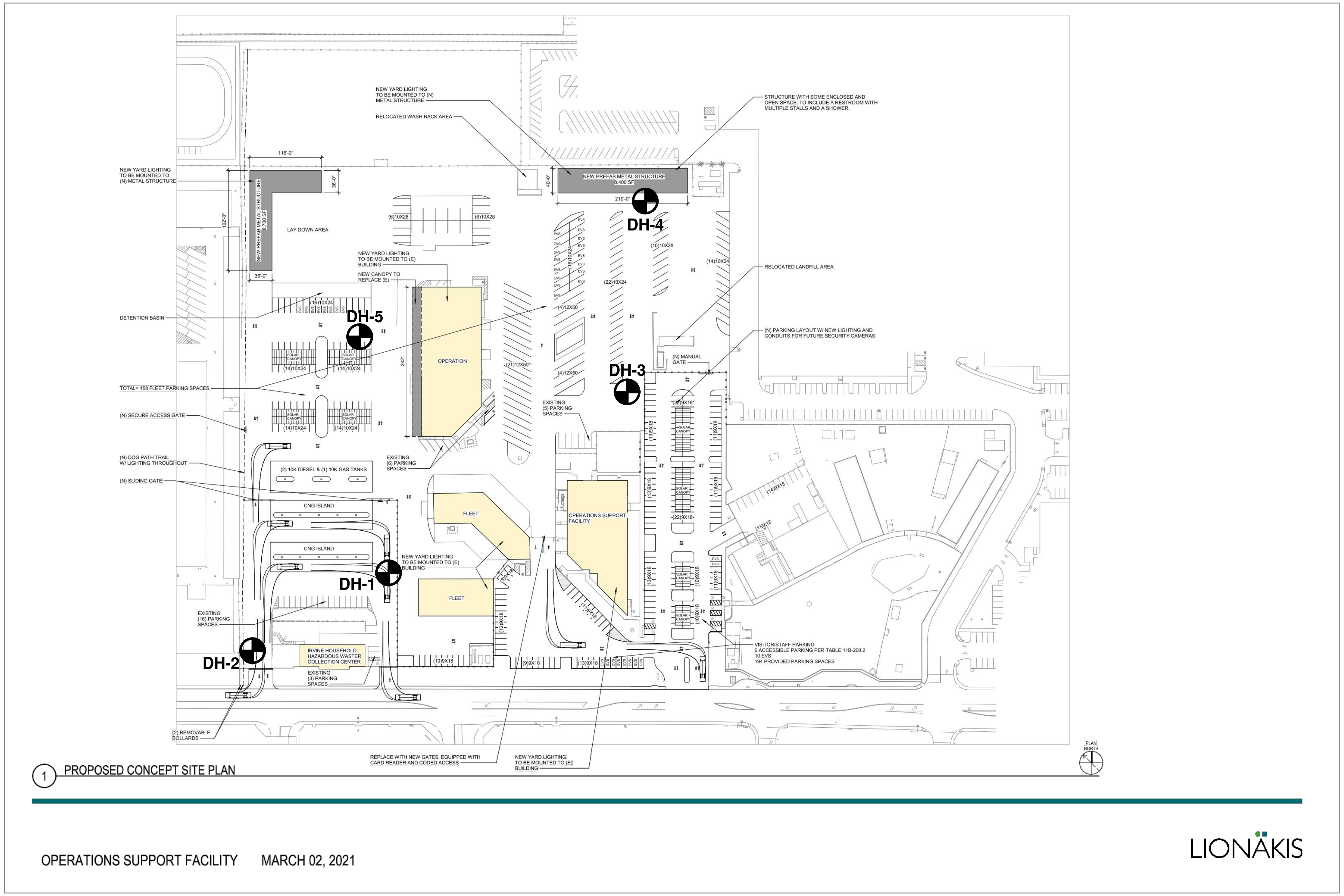


Location Map

GMU

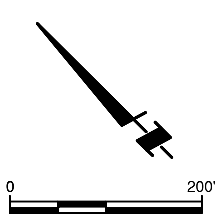
Date:	May 4, 2021	Plate
Project No.:	21-031-00	1

DRAWING: q:\2021\21-031-00\2103100_plate 1_locationmap.dwg PLOTTED: 3/15/2021 3:40 PM BY: Rosalie Chavez



GEOTECHNICAL LEGEND

DH-1 APPROXIMATE DRILL HOLE LOCATION



DRAWING: q:\2021\21-031-00\2103100_plate 2_geomap.dwg PLOTTED: 4/8/2021 12:18 PM BY: Avarni

GEOTECHNICAL MAP

GMU

Date:	May 4, 2021
Project No.:	21-031-00

Plate 2

APPENDIX A

Geotechnical Exploration Procedures and Logs

APPENDIX A

GMU GEOTECHNICAL EXPLORATION PROCEDURES AND LOGS

Our exploration at the subject site consisted of 5 drill holes. The estimated locations of the explorations are shown on Plate 2 – Geotechnical Map. Our drill holes were logged by a Certified Engineering Geologist, and California Modified, bulk, and SPT samples of the excavated soils were collected. "Undisturbed" samples were taken using a 3.0-inch outside-diameter California Modified sampler, which contains a 2.416-inch-diameter brass sample sleeve 6 inches in length. Standard penetration testing (SPT) with a 2.0-inch outside-diameter split spoon sampler without liners was performed in the borings during advancement. Blow counts recorded during sampling from the California Modified and SPT sampler are shown on the drill hole logs. The logs of each drill hole are contained in this Appendix A, and the Legend to Logs is presented as Plates A-1 and A-2.

The geologic and engineering field descriptions and classifications that appear on these logs are prepared according to Corps of Engineers and Bureau of Reclamation standards. Major soil classifications are prepared according to the Unified Soil Classification System as modified by ASTM Standard No. 2487. Since the descriptions and classifications that appear on the Log of Drill Hole are intended to be that which most accurately describe a given interval of a drill hole (frequently an interval of several feet), discrepancies do occur in the Unified Soil Classification System nomenclature between that interval and a particular sample in that interval. For example, an 8-foot-thick interval in a log may be identified as silty sand (SM) while one sample taken within the interval may have individually been identified as sandy silt (ML). This discrepancy is frequently allowed to remain to emphasize the occurrence of local textural variations in the interval.

MAJOR DIVISIONS			Group Letter	Symbol	TYPICAL NAMES
COARSE-GRAINED SOILS More Than 50% Retained On No.200 Sieve Based on The Material Passing The 3-Inch (75mm) Sieve. Reference: ASTM Standard D2487	**GRAVELS** 50% or More of Coarse Fraction Retained on No.4 Sieve	Clean Gravels	GW		Well Graded Gravels and Gravel-Sand Mixtures, Little or No Fines.
			GP		Poorly Graded Gravels and Gravel-Sand Mixtures Little or No Fines.
		Gravels With Fines	GM		Silty Gravels, Gravel-Sand-Silt Mixtures.
			GC		Clayey Gravels, Gravel-Sand-Clay Mixtures.
	SANDS More Than 50% of Coarse Fraction Passes No.4 Sieve	Clean Sands	SW		Well Graded Sands and Gravelly Sands, Little or No Fines.
			SP		Poorly Graded Sands and Gravelly Sands, Little or No Fines.
		Sands With Fines	SM		Silty Sands, Sand-Silt Mixtures.
			SC		Clayey Sands, Sand-Clay Mixtures.
FINE-GRAINED SOILS 50% or More Passe The No.200 Sieve Based on The Material Passing The 3-Inch (75mm) Sieve. Reference: ASTM Standard D2487	**SILTS AND CLAYS** Liquid Limit Less Than 50%		ML		Inorganic Silts, 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 50% or Greater		MH		Inorganic Silts, Micaceous or Diatomaceous Fine Sandy or Silty Soils, Elastic Silts.
			CH		Inorganic Clays of High Plasticity, Fat Clays.
			OH		Organic Clays of Medium To High Plasticity, Organic Silts.
HIGHLY ORGANIC SOILS			PT		Peat and Other Highly Organic Soils.

The descriptive terminology of the logs is modified from current ASTM Standards to suit the purposes of this study

ADDITIONAL TESTS

DS = Direct Shear
HY = Hydrometer Test
TC = Triaxial Compression Test
UC = Unconfined Compression
CN = Consolidation Test
(T) = Time Rate
EX = Expansion Test
CP = Compaction Test
PS = Particle Size Distribution
EI = Expansion Index
SE = Sand Equivalent Test
AL = Atterberg Limits
FC = Chemical Tests
RV = Resistance Value
SG = Specific Gravity
SU = Sulfates
CH = Chlorides
MR = Minimum Resistivity
pH
(N) = Natural Undisturbed Sample
(R) = Remolded Sample
CS = Collapse Test/Swell-Settlement

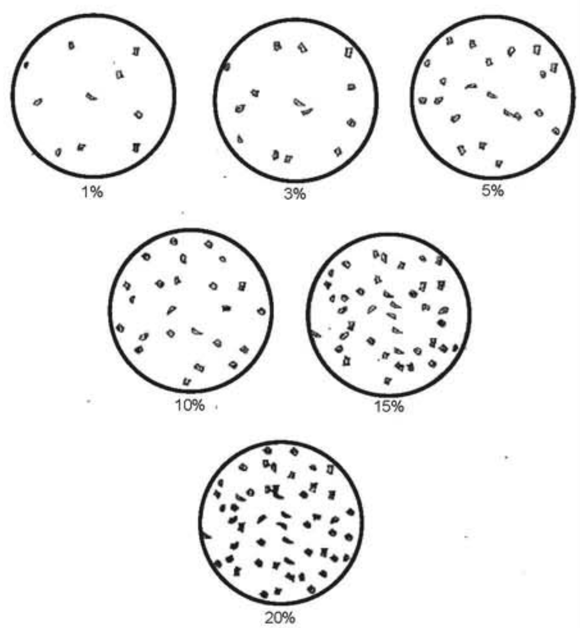
GEOLOGIC NOMENCLATURE

B = Bedding C = Contact J = Joint
F = Fracture Flt = Fault S = Shear
RS = Rupture Surface = Seepage
= Groundwater

SAMPLE SYMBOLS

Undisturbed Sample (California Sample)
Undisturbed Sample (Shelby Tube)
Bulk Sample
Unsuccessful Sampling Attempt
SPT Sample

5 10 15 Blows per 6-Inches Penetration
10: 10 Blows for 12-Inches Penetration
6/4": 6 Blows for 4-Inches Penetration
P: Push
(13): Uncorrected Blow Counts ("N" Values) for 12-Inches Penetration- Standard Penetration Test (SPT)



GMU

LEGEND TO LOGS
ASTM Designation: D 2487
(Based on Unified Soil Classification System)

Plate
A-1

P8-11/16/2012

SOIL DENSITY/CONSISTENCY

FINE GRAINED			
Consistency	Field Test	SPT (#blows/foot)	Mod (#blows/foot)
Very Soft	Easily penetrated by thumb, exudes between fingers	<2	<3
Soft	Easily penetrated one inch by thumb, molded by fingers	2-4	3-6
Firm	Penetrated over 1/2 inch by thumb with moderate effort	4-8	6-12
Stiff	Penetrated about 1/2 inch by thumb with great effort	8-15	12-25
Very Stiff	Readily indented by thumbnail	15-30	25-50
Hard	Indented with difficulty by thumbnail	>30	>50

COARSE GRAINED			
Density	Field Test	SPT (#blows/foot)	Mod (#blows/foot)
Very Loose	Easily penetrated with 0.5" rod pushed by hand	<4	<5
Loose	Easily penetrated with 0.5" rod pushed by hand	4-10	5-12
Medium Dense	Easily penetrated 1' with 0.5" rod driven by 5lb hammer	10-30	12-35
Dense	Dificult to penetrat 1' with 0.5" rod driven by 5lb hammer	31-50	35-60
Very Dense	Penetrated few inches with 0.5" rod driven by 5lb hammer	>50	>60

BEDROCK HARDNESS

Density	Field Test	SPT (#blows/foot)
Soft	Can be crushed by hand, soil like and structureless	1-30
Moderately Hard	Can be grooved with fingernails, crumbles with hammer	30-50
Hard	Can't break by hand, can be grooved with knife	50-100
Very Hard	Scratches with knife, chips with hammer blows	>100

MODIFIERS

Trace	1%
Few	1-5%
Some	5-12%
Numerous	12-20%
Abundant	>20%

GRAIN SIZE

Description		Sieve Size	Grain Size	Approximate Size
Boulders		>12"	>12"	Larger than a basketball
Cobbles		3-12"	3-12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4-3"	3/4-3"	Thumb-sized to fist-sized
	Fine	#4-3/4"	0.19-0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10-#4	0.079-0.19"	Rock-salt-sized to pea-sized
	Medium	#40-#10	0.017-0.079"	Sugar-sized to rock salt-sized
	Fine	#200-#40	0.0029-0.017"	Flour-sized to sugar-sized
Fines		passing #200	<0.0029"	Flour-sized and smaller

MOISTURE CONTENT

Dry- Very little or no moisture
Damp- Some moisture but less than optimum
Moist- Near optimum
Very Moist- Above optimum
Wet/Saturated- Contains free moisture

GMU

LEGEND TO LOGS
ASTM Designation: D 2487
(Based on Unified Soil Classification System)

Plate
A-2

Project: Irvine Operation Support Facility Project Location: Irvine, CA Project Number: 21-031-00	**Log of Drill Hole DH-1** Sheet 1 of 3

Date(s) Drilled	**2/19/2021**	Logged By	**DW**	Checked By	**ABM**
Drilling Method	**Hollow Stem Auger**	Drilling Contractor	**2R Drilling**	Total Depth of Drill Hole	**51.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**8**	Approx. Surface Elevation, ft MSL	**169.0**
Groundwater Depth [Elevation], feet	**NA []**	Sampling Method(s)	**Open drive sampler with 6-inch sleeve, SPT, and Bulk**	Drill Hole Backfill	**Native**
Remarks				Driving Method and Drop	**140lb hammer; 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	GEOLOGICAL CLASSIFICATION AND DESCRIPTION	ORIENTATION DATA	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: NUMBER OF BLOWS / 6"	SAMPLE DATA: DRIVING WEIGHT, lbs	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
	0		**ARTIFICIAL FILL (Qaf)**		ASPHALT CONCRETE - 3.5" SANDY CLAY (CL); brown, moist, firm, fine- to medium-grained sand	Bulk			19		CP, DS, FC
165			**YOUNGER ALLUVIAL DEPOSITS (Qyfa)**		SANDY CLAY (CL); yellowish brown, moist, stiff, fine- to medium-grained sand						
	5					Drive	4 5 9	140	16	108	DS
					SILTY SAND (SM) with CLAY; yellowish brown, damp to moist, medium dense, fine- to medium-grained sand						
160											
	10		Some fine-grained sand stringers		SILTY CLAY (CL); yellowish brown, moist, stiff, some fine-grained sand	SPT	3 5 7	140			
155											
	15				SANDY CLAY (CL); yellowish brown, moist, firm, fine to medium grained sand	Drive	3 7 7	140	21	101	
150											

DH_REV3 21-031-00.GPJ GMULAB.GPJ 4/13/21

Drill Hole DH-1

GMU

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	GEOLOGICAL CLASSIFICATION AND DESCRIPTION	ORIENTATION DATA	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	NUMBER OF BLOWS / 6"	DRIVING WEIGHT, lbs	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			YOUNGER ALLUVIAL DEPOSITS (Qyfa)		SILTY SAND (SM) with some CLAY; yellowish brown, damp, medium dense, fine- to medium-grained sand, few coarse-grained sand, rare gravel		5 6 8	140			
145											
	25				POORLY GRADED SAND to SILTY SAND (SP-SM); yellowish brown, damp, medium dense, fine- to medium-grained sand		7 11 8	140	9	113	
140											
	30				SILTY SAND (SM) with some CLAY; yellowish brown, damp, loose, fine- to medium-grained sand, trace fine-grained gravel		2 3 3	140			
135											
	35		Subangular gravel		Little to no CLAY, some coarse-grained sand and gravel		10 22 25	140	5	115	
130											
	40						3 3 4	140	13		PS
					CLAYEY SAND (SC); yellowish brown, loose, damp, fine- to medium-grained sand						
125											

DH_REV3 21-031-00.GPJ GMULAB.GPJ 4/13/21

Project: Irvine Operation Support Facility
Project Location: Irvine, CA
Project Number: 21-031-00

Log of Drill Hole DH-1

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	GEOLOGICAL CLASSIFICATION AND DESCRIPTION	ORIENTATION DATA	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: NUMBER OF BLOWS / 6"	SAMPLE DATA: DRIVING WEIGHT, lbs	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			YOUNGER ALLUVIAL DEPOSITS (Qyfa)		SANDY CLAY (CL) with some SILT; yellowish brown, damp, firm, fine- to medium-grained sand		7 12 12	140	25	104	
120											
	50		0.5' zone of coarse-grained sand		Some coarse-grained sand		8 5 7	140			
					Total Depth 51.5' No Groundwater						

DH_REV3 21-031-00.GPJ GMULAB.GPJ 4/13/21

Drill Hole DH-1

Project: Irvine Operation Support Facility
Project Location: Irvine, CA
Project Number: 21-031-00

Log of Drill Hole DH-2

Date(s) Drilled	**2/19/2021**	Logged By	**DW**	Checked By	**ABM**
Drilling Method	**Hollow Stem Auger**	Drilling Contractor	**2R Drilling**	Total Depth of Drill Hole	**11.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**8**	Approx. Surface Elevation, ft MSL	**166.0**
Groundwater Depth [Elevation], feet	**NA []**	Sampling Method(s)	**Open drive sampler with 6-inch sleeve, SPT, and Bulk**	Drill Hole Backfill	**Native**
Remarks				Driving Method and Drop	**140lb hammer; 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	GEOLOGICAL CLASSIFICATION AND DESCRIPTION	ORIENTATION DATA	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: NUMBER OF BLOWS / 6"	SAMPLE DATA: DRIVING WEIGHT, lbs	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			ARTIFICIAL FILL (Qaf)		ASPHALT CONCRETE - 8"						
165					CLAYEY SAND to SANDY CLAY (SC-CL); dark brown, moist, medium dense/stiff, fine- to medium-grained sand						
			YOUNGER ALLUVIAL DEPOSITS (Qyfa)		SILTY SAND (SM) with some CLAY; yellowish brown, damp, medium dense, fine- to medium-grained sand						
	5				Few coarse-grained sand, rare gravel		3 5 9	140	14	116	
160											
	10				SANDY CLAY (CL); brown to yellowish brown, moist, stiff, fine- to medium-grained sand		4 7 11	140			
155											
					Total Depth = 11.5' No Groundwater						

DH_REV3 21-031-00.GPJ GMULAB.GPJ 4/13/21

GMU

Drill Hole DH-2

Project: Irvine Operation Support Facility Project Location: Irvine, CA Project Number: 21-031-00	**Log of Drill Hole DH-3** Sheet 1 of 2

Date(s) Drilled	**2/19/2021**	Logged By	**DW**	Checked By	**ABM**
Drilling Method	**Hollow Stem Auger**	Drilling Contractor	**2R Drilling**	Total Depth of Drill Hole	**21.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**8**	Approx. Surface Elevation, ft MSL	**175.0**
Groundwater Depth [Elevation], feet	**NA []**	Sampling Method(s)	**Open drive sampler with 6-inch sleeve, SPT, and Bulk**	Drill Hole Backfill	**Native**
Remarks				Driving Method and Drop	**140lb hammer; 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	GEOLOGICAL CLASSIFICATION AND DESCRIPTION	ORIENTATION DATA	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: NUMBER OF BLOWS / 6"	SAMPLE DATA: DRIVING WEIGHT, lbs	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			ARTIFICIAL FILL (Qaf)		ASPHALT CONCRETE - 4"				25		EI, RV
					SANDY CLAY (CL); brown, moist, stiff, fine- to medium-grained sand	Bulk					
			YOUNGER ALLUVIAL DEPOSITS (Qyfa)		CLAYEY SAND (SC); yellowish brown, damp, medium dense, fine- to medium-grained sand						
170	5				SILTY SAND (SM); yellowish brown, damp, loose, fine- to medium- grained sand, few coarse-grained sand	SPT	2 2 3	140			
165	10				SANDY CLAY to CLAYEY SAND (CL-SC); light brown, damp to moist, stiff/medium dense, fine- to medium-grained sand, few coarse-grained sand	Drive	9 10 15	140	21	103	
160	15				Becomes damp, medium dense/firm, little to no coarse-grained sand	SPT	3 3 5	140			

DH_REV3 21-031-00.GPJ GMULAB.GPJ 4/13/21

GMU

Drill Hole DH-3

Log of Drill Hole DH-3

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	GEOLOGICAL CLASSIFICATION AND DESCRIPTION	ORIENTATION DATA	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: NUMBER OF BLOWS / 6"	SAMPLE DATA: DRIVING WEIGHT, lbs	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			YOUNGER ALLUVIAL DEPOSITS (Qyfa)		SANDY CLAY to CLAYEY SAND (CL-SC); light brown, damp to moist, stiff/medium dense, fine- to medium-grained sand		8 10 12	140	20	101	
					Total Depth = 21.5' No Groundwater						

DH_REV3 21-031-00.GPJ GMULAB.GPJ 4/13/21

Drill Hole DH-3

Project: Irvine Operation Support Facility **Project Location: Irvine, CA** **Project Number: 21-031-00**	# Log of Drill Hole DH-4 Sheet 1 of 1

Date(s) Drilled	**2/19/2021**	Logged By	**DW**	Checked By	**ABM**
Drilling Method	**Hollow Stem Auger**	Drilling Contractor	**2R Drilling**	Total Depth of Drill Hole	**11.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**8**	Approx. Surface Elevation, ft MSL	**180.0**
Groundwater Depth [Elevation], feet	**NA []**	Sampling Method(s)	**Open drive sampler with 6-inch sleeve, SPT, and Bulk**	Drill Hole Backfill	**Native**
Remarks				Driving Method and Drop	**140lb hammer; 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	GEOLOGICAL CLASSIFICATION AND DESCRIPTION	ORIENTATION DATA	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: NUMBER OF BLOWS / 6"	SAMPLE DATA: DRIVING WEIGHT, lbs	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			ARTIFICIAL FILL (Qaf)		ASPHALT CONRETE - 3"	Bulk					
					CRUSHED MISCELLANEOUS BASE - 3"						
					CLAYEY SAND (SC); brown, moist, medium dense, fine- to medium-grained sand						
			YOUNGER ALLUVIAL DEPOSITS (Qyfa)		CLAYEY SAND (SC); yellowish brown, moist, medium dense, fine- to medium-grained sand						
175	5				SILTY SAND (SM); yellowish brown, damp, loose, fine- to medium-grained sand, few coarse-grained sand	Drive	4 5 6	140	10	107	
170	10					SPT	2 1 3	140			
			Some fine-grained sand stringers		CLAYEY SAND to SANDY CLAY (SC-CL); light brown, damp, loose/soft, fine- to medium-grained sand						
					Total Depth = 11.5' No Groundwater						

DH_REV3 21-031-00.GPJ GMULAB.GPJ 4/13/21

GMU

Drill Hole DH-4

Date(s) Drilled	2/19/2021	Logged By	DW	Checked By	ABM
Drilling Method	Hollow Stem Auger	Drilling Contractor	2R Drilling	Total Depth of Drill Hole	51.5 feet
Drill Rig Type	CME 75	Diameter(s) of Hole, inches	8	Approx. Surface Elevation, ft MSL	172.0
Groundwater Depth [Elevation], feet	NA []	Sampling Method(s)	Open drive sampler with 6-inch sleeve, SPT, and Bulk	Drill Hole Backfill	Native
Remarks				Driving Method and Drop	140lb hammer; 30" drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	GEOLOGICAL CLASSIFICATION AND DESCRIPTION	ORIENTATION DATA	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: NUMBER OF BLOWS / 6"	SAMPLE DATA: DRIVING WEIGHT, lbs	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
170			**ARTIFICIAL FILL (Qaf)**		SILTY SAND (SM) with CLAY; brown, damp, medium dense, fine- to medium-grained sand, few coarse-grained sand and fine-grained gravel	Bulk					
			YOUNGER ALLUVIAL DEPOSITS (Qyfa)		SILTY SAND (SM); yellowish brown, damp, loose, fine- to medium-grained sand						
	5					Drive	2 4 5	140	14	112	
165					Some CLAY						
	10					SPT	4 7 10	140			
160			Some fine-grained stringers		CLAYEY SAND to SANDY CLAY (SC-CL); light brown, damp to moist, medium dense/stiff, fine- to medium-grained sand						
	15					Drive	2 4 6	140	21	99	
155											

DH_REV3 21-031-00.GPJ GMULAB.GPJ 4/13/21

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	GEOLOGICAL CLASSIFICATION AND DESCRIPTION	ORIENTATION DATA	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	NUMBER OF BLOWS / 6"	DRIVING WEIGHT, lbs	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			YOUNGER ALLUVIAL DEPOSITS (Qyfa) Bulk sample collected from 20-25' which was characterized as silty sand to clayey sand		SILTY SAND (SM); yellowish brown, damp, loose, fine- to medium-grained sand		2 2 3	140			
150											
	25				Becomes dense, some CLAY		11 15 25	140	11	120	
145											
	30				CLAYEY SAND (SC); yellowish brown, damp, medium dense, fine- to medium-grained sand		4 7 9	140			
140											
	35				SILTY SAND (SM) with minor CLAY; yellowish brown, damp, medium dense, fine- to medium-grained sand		7 11 17	140	13	119	
135											
	40				Little to no CLAY		4 5 6	140			
130											

DH_REV3 21-031-00.GPJ GMULAB.GPJ 4/13/21

Log of Drill Hole DH-5

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	GEOLOGICAL CLASSIFICATION AND DESCRIPTION	ORIENTATION DATA	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	NUMBER OF BLOWS / 6"	DRIVING WEIGHT, lbs	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			YOUNGER ALLUVIAL DEPOSITS (Qyfa) Faint structure		SILTY SAND (SM) with minor CLAY; yellowish brown, damp, dense, fine- to meidum-grained sand		10 16 21	140	12	119	PS
125											
	50						5 5 6	140			
					SANDY CLAY (CL); light brown, moist, stiff, fine-grained sand						
					Total Depth = 51.5' No Groundwater						

DH_REV3 21-031-00.GPJ GMULAB.GPJ 4/13/21

Drill Hole DH-5

APPENDIX B

Geotechnical Laboratory Procedures and Test Results

APPENDIX B

GMU GEOTECHNICAL LABORATORY PROCEDURES AND TEST RESULTS

MOISTURE AND DENSITY

Field moisture content and in-place density were determined for each 6-inch sample sleeve of undisturbed soil material obtained from the drill holes. The field moisture content was determined in general accordance with ASTM Test Method D 2216 by obtaining one-half the moisture sample from each end of the 6-inch sleeve. The in-place dry density of the sample was determined by using the wet weight of the entire sample.

At the same time the field moisture content and in-place density were determined, the soil material at each end of the sleeve was classified according to the Unified Soil Classification System. The results of the field moisture content and in-place density determinations are presented on the right-hand column of the Log of Drill Hole and are summarized on Table B-1. The results of the visual classifications were used for general reference.

PARTICLE SIZE DISTRIBUTION

As part of the engineering classification of the materials underlying the site, samples were tested to determine the distribution of particle sizes. The distribution was determined in general accordance with ASTM Test Method D 422 using U.S. Standard Sieve No. 200.

EXPANSION TESTS

To provide a standard definition of one-dimensional expansion, a test was performed on typical on-site materials in general accordance with ASTM Test Method D 4829. The result from this test procedure is reported as an “expansion index”. The results of this test are contained in this Appendix B and also Table B-1.

CHEMICAL TESTS

The corrosion potential of typical on-site materials under long-term contact with both metal and concrete was determined by chemical and electrical resistance tests. The soluble sulfate test for potential concrete corrosion was performed in general accordance with California Test Method 417, the minimum resistivity test for potential metal corrosion was performed in general accordance with California Test Method 643, and the concentration of soluble chlorides was determined in general accordance with California Test Method 422. The results of these tests are contained in this Appendix B and also Table B-1.

COMPACTION TESTS

A bulk sample representative of the on-site materials was tested to determine the maximum dry density and optimum moisture content of the soil. These compactive characteristics were determined in general accordance with ASTM Test Method D 1557. The results of this test are contained in this Appendix B and also Table B-1.

CONSOLIDATION TESTS

The one-dimensional consolidation properties of "undisturbed" samples were evaluated in general accordance with the provisions of ASTM Test Method D 2435. Sample diameter was 2.416 inches and sample height was 1.00 inch. Water was added during the test at various normal loads to evaluate the potential for hydro-collapse and to produce saturation during the remainder of the testing. Consolidation readings were taken regularly during each load increment until the change in sample height was less than approximately 0.0001 inch over a two-hour period. The graphic presentation of consolidation data is a representation of volume change in change in axial load.

DIRECT SHEAR STRENGTH TEST

A direct shear test was performed on typical on-site materials. The general philosophy and procedure of the test was in accord with ASTM Test Method D 3080 - "Direct Shear Tests for Soils Under Consolidated Drained Conditions".

The test is a single shear test and is performed using a sample diameter of 2.416 inches and a height of 1.00 inch. The normal load is applied by a vertical dead load system. A constant rate of strain is applied to the upper one-half of the sample until failure occurs. Shear stress is monitored by a strain gauge-type precision load cell and deflection is measured with a digital dial indicator. This data is transferred electronically to data acquisition software which plots shear strength vs. deflection. The shear strength plots are then interpreted to determine either peak or ultimate shear strengths. Residual strengths were obtained through multiple shear box reversals. A strain rate compatible with the grain size distribution of the soils was utilized. The interpreted result of this test is shown in this Appendix B.

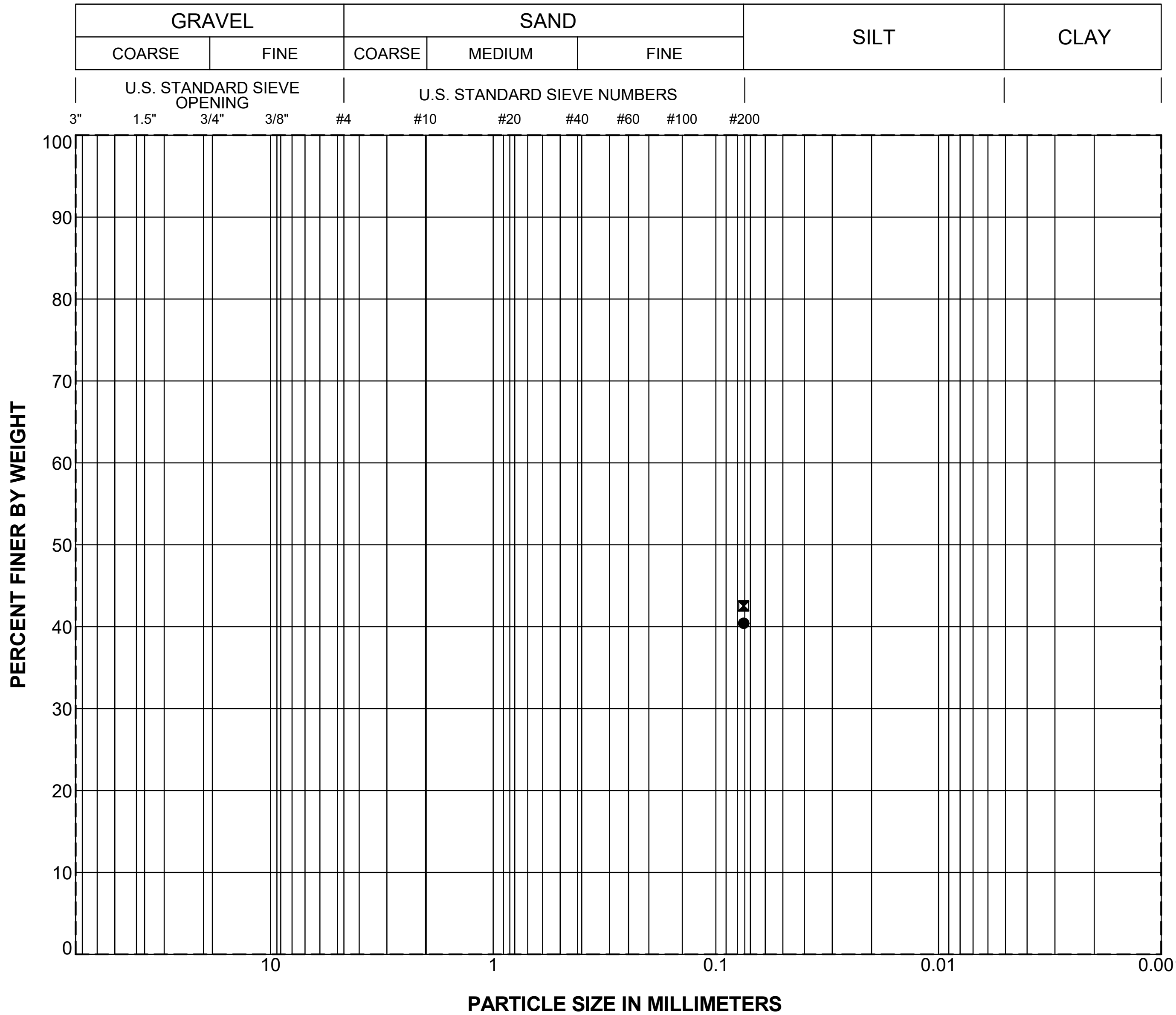
R-VALUE TESTS

Bulk samples representative of the underlying on-site materials were tested to measure the response of a compacted sample to a vertically applied pressure under specific conditions. The R-value of a material is determined when the material is in a state of saturation such that water will be exuded from the compacted test specimen when a 16.8 kN load (2.07 MPa) is applied. The results from these test procedures are reported in this Appendix B.

TABLE B-1
SUMMARY OF SOIL LABORATORY DATA

Sample Information			Geologic Unit	USCS Group Symbol	In Situ Water Content, %	In Situ Dry Unit Weight, pcf	In Situ Satur-ation, %	Sieve/Hydrometer				Atterberg Limits			Compaction		Expansion Index	R-Value	Chemical Test Results			
Boring Number	Depth, feet	Elevation, feet						Gravel, %	Sand, %	<#200, %	<2µ, %	LL	PL	PI	Maximum Dry Unit Weight, pcf	Optimum Water Content, %			pH	Sulfate (ppm)	Chloride (ppm)	Min. Resistivity (ohm/cm)
DH-1	0	169.0	Qaf/Qyfa	CL	19.0										119.0	12.0			8.7	252	174	879
DH-1	5	164.0	Qyfa	CL	15.6	108	77															
DH-1	15	154.0	Qyfa	CL	21.3	101	89															
DH-1	25	144.0	Qyfa	SM	9.5	113	54															
DH-1	35	134.0	Qyfa	SM	5.0	115	30															
DH-1	40	129.0	Qyfa	SM-SC	12.8					40												
DH-1	45	124.0	Qyfa	CL	16.5	112	92															
DH-2	5	161.0	Qyfa	SC	13.8	116	85															
DH-3	0	175.0	Qaf/Qyfa	CL	24.9												113	5				
DH-3	10	165.0	Qyfa	CL	21.3	103	93															
DH-3	20	155.0	Qyfa	CL	19.8	101	83															
DH-4	5	175.0	Qyfa	SM	9.9	107	48															
DH-5	5	167.0	Qyfa	SM/SC	14.3	112	79															
DH-5	15	157.0	Qyfa	CL	21.0	99	83															
DH-5	25	147.0	Qyfa	SM	10.6	120	74															
DH-5	35	137.0	Qyfa	SM	12.7	119	85															
DH-5	45	127.0	Qyfa	SM-SC	12.2	119	82			43												

GMU_TABLE_SOIL_LAB_DATA 21-031-00.GPJ FNC AB GWGN01.GDT 4/15/21

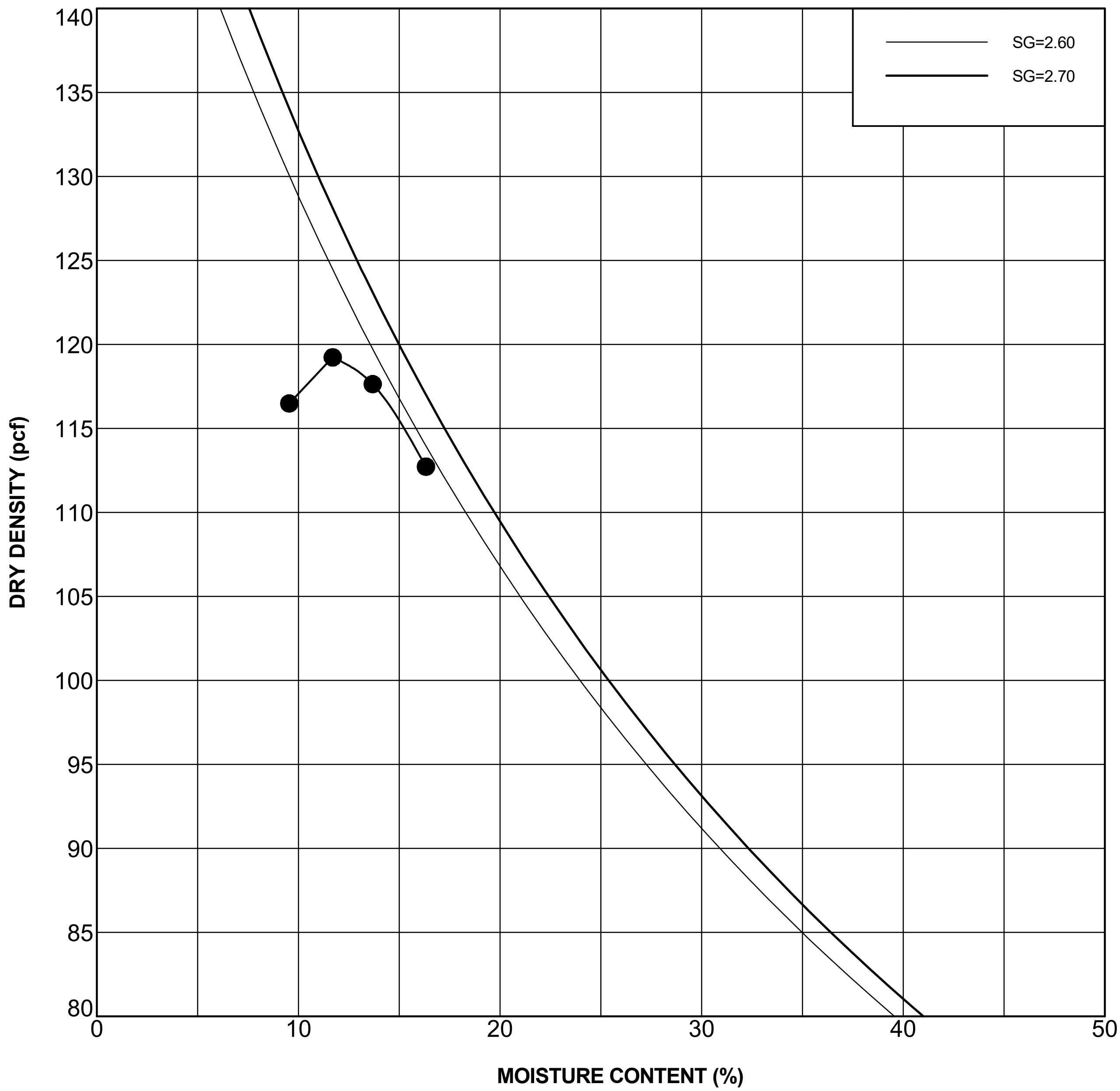


Boring Number	Depth (feet)	Geologic Unit	Symbol	LL	PI	Classification
DH-1	40.0	Qyfa	●			SILTY SAND with CLAY (SM-SC)
DH-5	45.0	Qyfa	⊠			SILTY SAND with CLAY (SM-SC)

PARTICLE SIZE DISTRIBUTION

Project: Irvine Operation Support Facility

Project No. 21-031-00



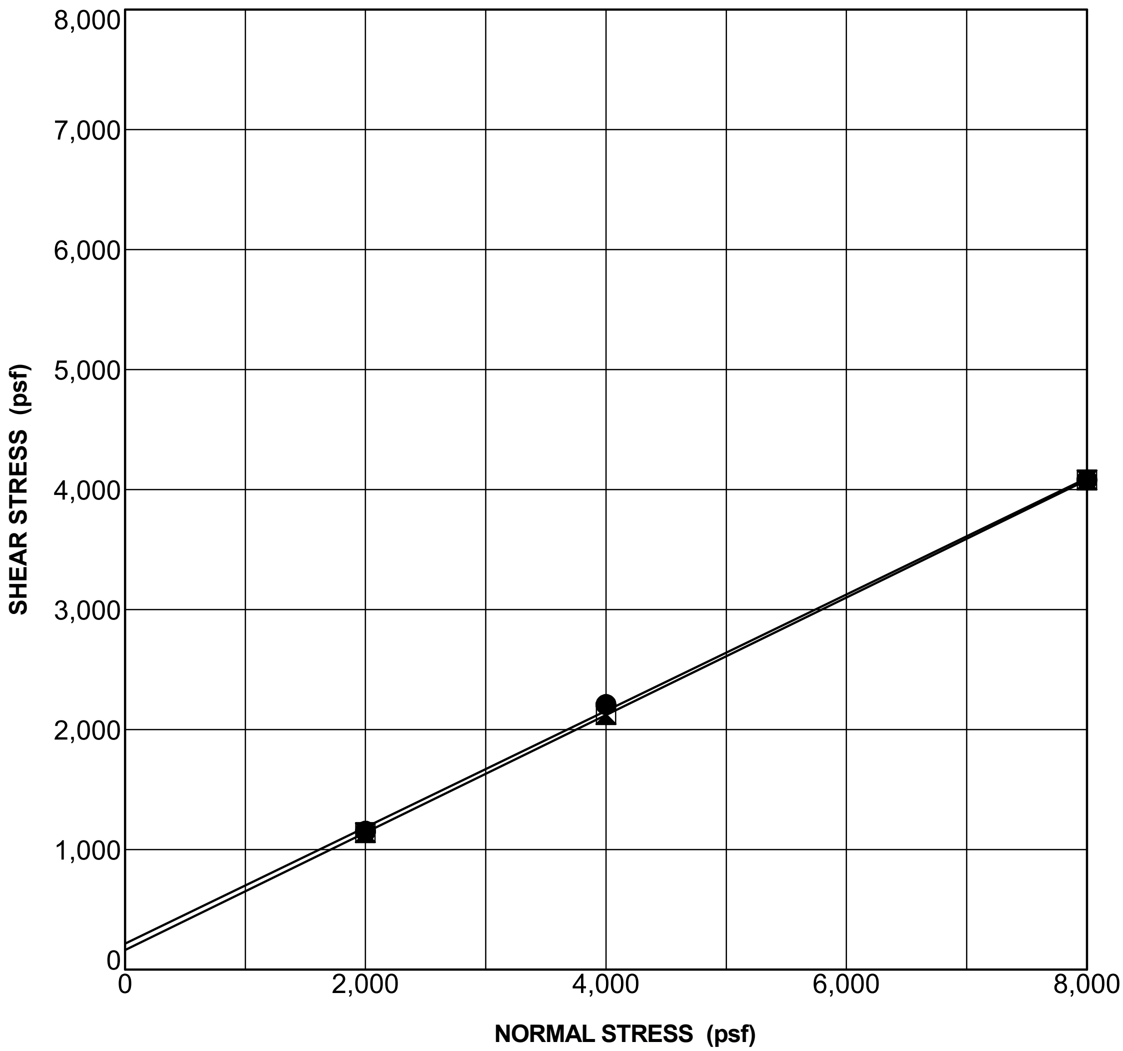
Boring Number	Depth (feet)	Geologic Unit	Symbol	Maximum Dry Density, pcf	Optimum Moisture Content, %	Classification
DH-1	0.0	Qaf/Qyfa	●	119	12	SANDY CLAY (CL)

DVTCOMP 21-031-00.GPJ 3/9/21

COMPACTION TEST DATA

Project: Irvine Operation Support Facility
Project No. 21-031-00

GMU



SAMPLE AND TEST DESCRIPTION
Sample Location: DH-1 @ 0.0 ft **Geologic Unit:** Qaf/Qyfa **Classification:** SANDY CLAY (CL)
Strain Rate (in/min): 0.005 **Sample Preparation:** Remolded
Notes: Remolded 90% compaction at optimum

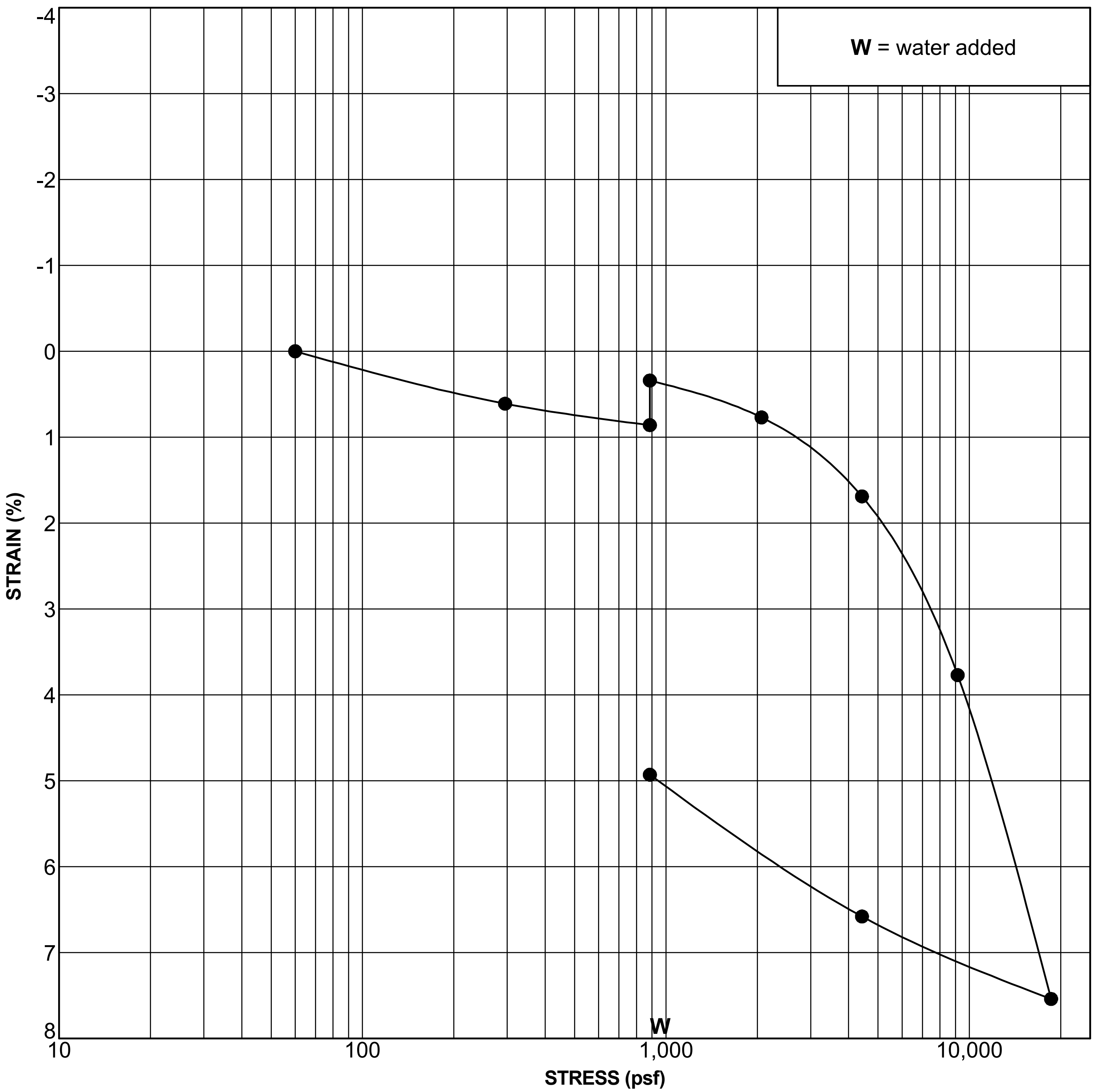
STRENGTH PARAMETERS		
STRENGTH TYPE	**COHESION (psf)**	**FRICTION ANGLE (degrees)**
● Peak Strength	216	25.9
⧗ Ultimate Strength	162	26.1

GMU_DIRECT_SHEAR 21-031-00.GPJ GM&U.GDT 3/9/21

SHEAR TEST DATA

Project: Irvine Operation Support Facility
Project No. 21-031-00

GMU



GMU_CONSOL 21-031-00.GPJ GM&U.GDT 3/9/21

Boring Number	Depth (feet)	Geologic Unit	Symbol	In Situ or Remolded Sample	% Hydro-Collapse	Classification
DH-1	5.0	Qyfa	●	In Situ	-0.52	SANDY CLAY (CL)

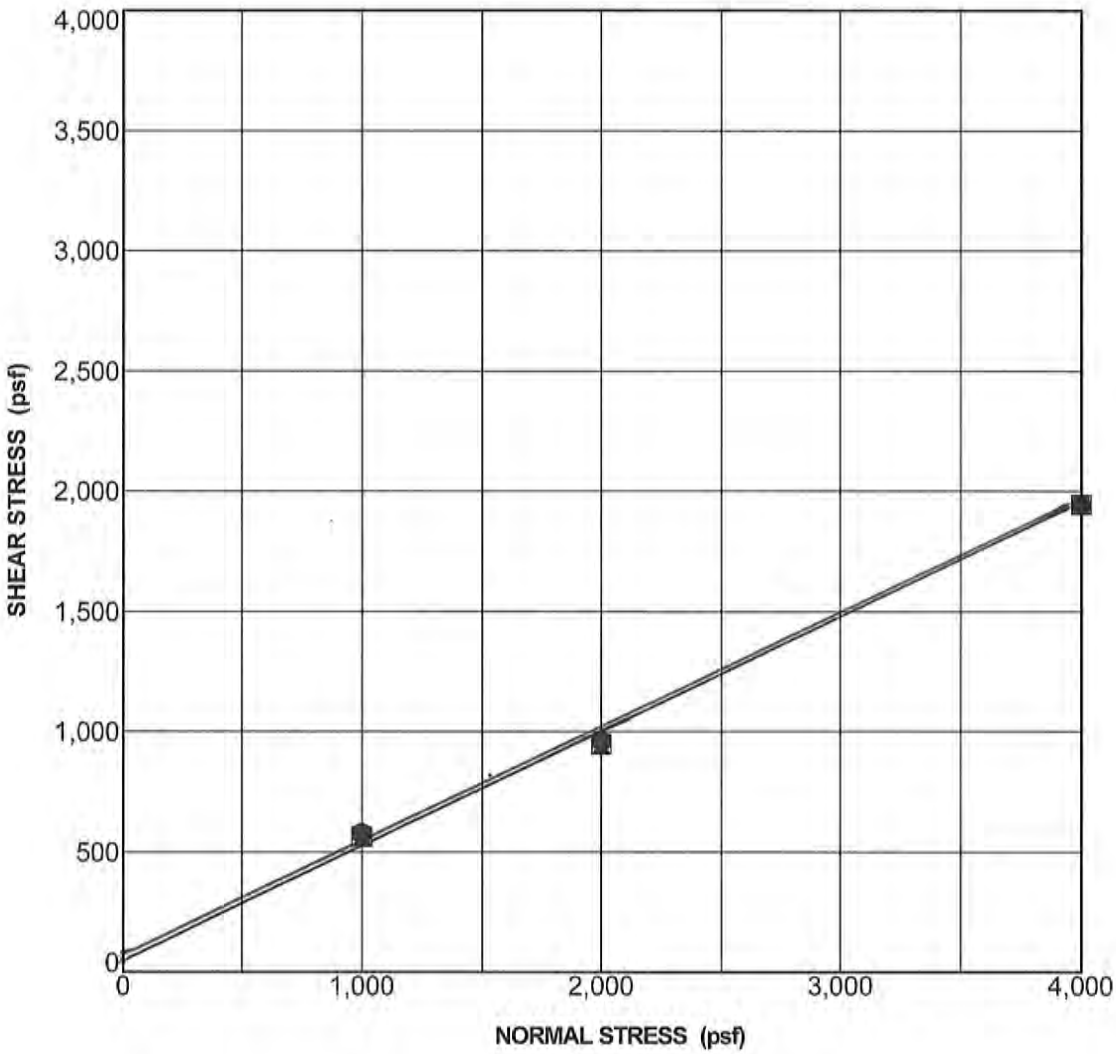
CONSOLIDATION TEST DATA

Project: Irvine Operation Support Facility
Project No. 21-031-00

GMU

APPENDIX B-1

Previous Geotechnical Laboratory Test Results by GMU



SAMPLE AND TEST DESCRIPTION
Sample Location: DH-06 @ 7.5 ft **Geologic Unit:** Qyfa **Classification:** SANDY CLAY (CL)
Strain Rate (in/min): 0.005 **Sample Preparation:** Undisturbed
Notes: Sample saturated prior and during shearing

STRENGTH PARAMETERS		
STRENGTH TYPE	**COHESION (psf)**	**FRICTION ANGLE (degrees)**
● Peak Strength	84	25.0
⊠ Ultimate Strength	66	25.0

SHEAR TEST DATA

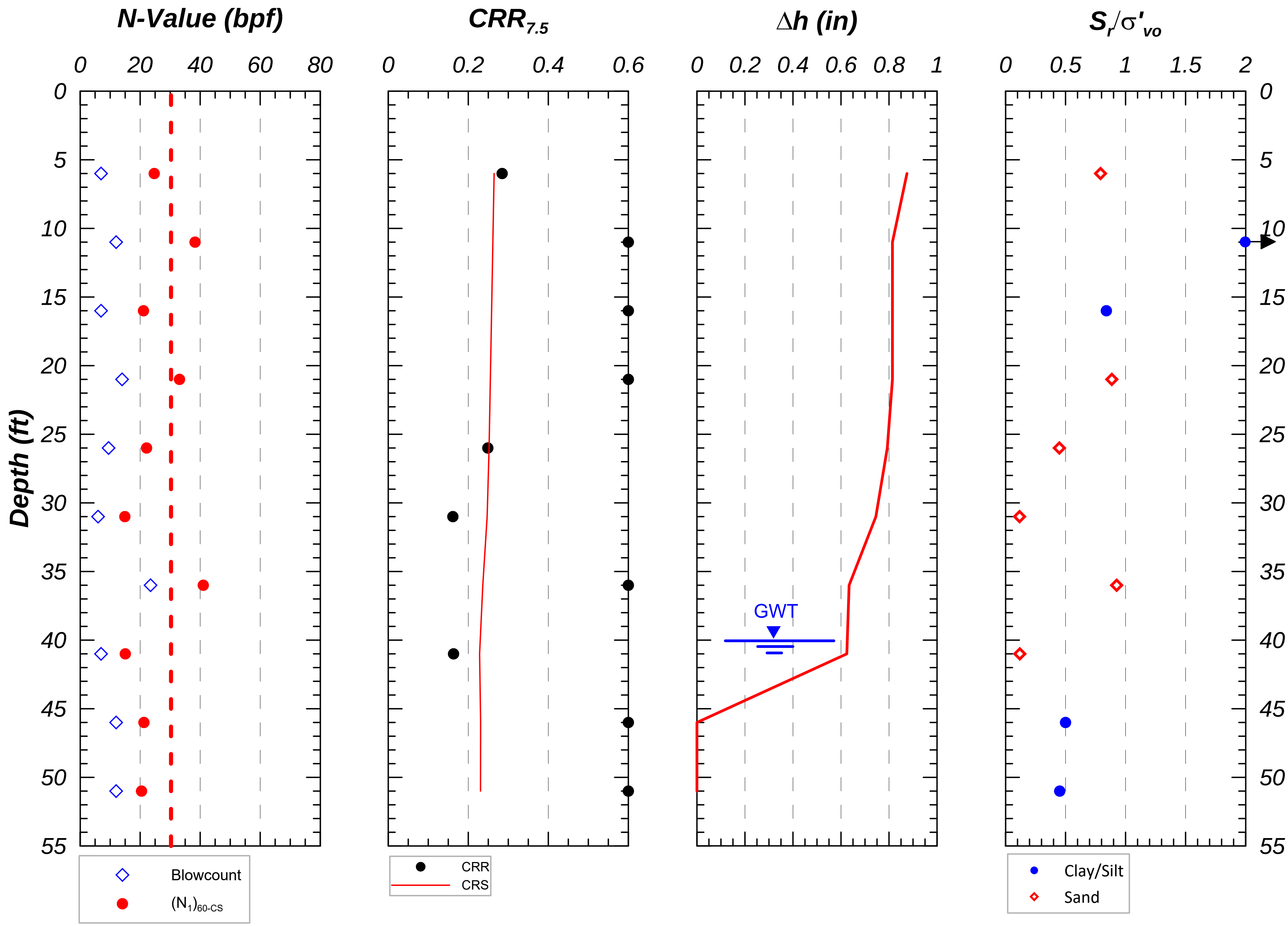
Project: Discovery Business Center Phase II

Project No. 15-176-00

GMU GEOTECHNICAL, INC.

APPENDIX C

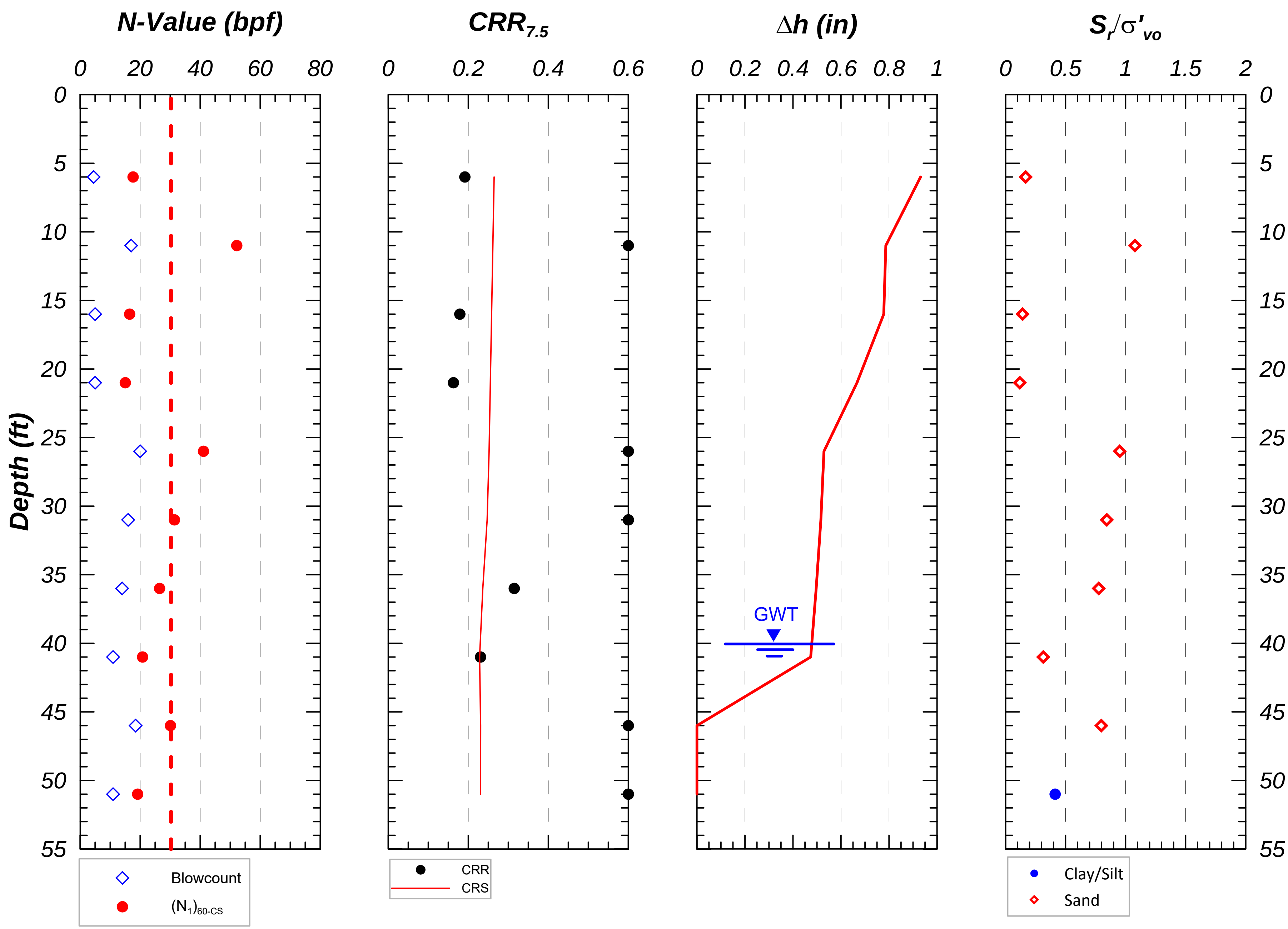
Liquefaction Analysis



Project Name OSF	Integrated SPT Method for Estimating Subsurface Stratification & Liquefaction Drill Hole DH-1
Project No. 21-031-00	Figure C-1

DH-1.xls, DH-1.grf

Date: April 2021



Project Name OSF	Integrated SPT Method for Estimating Subsurface Stratification & Liquefaction Drill Hole DH-5
Project No. 21-031-00	Figure C-2

DH-5.xls, DH-5.grf

Date: April 2021

APPENDIX D

Concrete Flatwork Recommendations

TABLE 3. FLATWORK RECOMMENDATIONS
IOSF Site Project

Description	Subgrade Preparation	Minimum Concrete Thickness (Full)	Edge Thickness	Reinforcement[2]	Joint Spacing (Maximum)	Cement Type	Sulfate Resistance
Isolated Concrete Sidewalks and Walkways (≤6 feet in width) [4]	1) 3% over optimum to 18" [1], 2) optional 2" of sand or well graded rock (i.e., Class II base or equiv.) above moisture conditioned subgrade.	4 inches	Not Required	1) No. 3 bars at 18" o.c. [2], 2) where adjacent to curbs or structures and at cold joints/ expansion joints use dowels: No. 3 bars at 24" o.c. [5]	6 feet	II/V	(3)
Concrete Walkways, Patios, Entryways and Courtyards (> 6 feet in width) [4]	1) 3% over optimum to 18"[1], 2) optional 2" of sand or well graded rock (i.e., Class II base or equiv.) above moisture conditioned subgrade.	5 inches	Where adjacent to landscape areas – 12" from adjacent finish grade. Min. 8" width	1) No. 3 bars at 18" o.c. [2] extend into thickened edge, 2) Thickened Edge: one No. 3 bar placed in long direction, 3) dowel into adjacent curbs or structures and across cold joints/ expansion joints w/No. 3 bars at 18" o.c. [5]	6 feet	II/V	(3)
Concrete Driveways, Trash Enclosures and Fire Access Lanes [4]	1) 3% over optimum to 18" [1], 2) 6 inches of sand or well graded rock (i.e., Class II base or equiv.) above moisture conditioned subgrade.	8 inches	Where adjacent to landscape areas - 12" from adjacent finish grade. Min. 8" width	1) No. 3 bars at 18" o.c. [2] extend into thickened edge, 2) Thickened Edge: one No. 3 bar placed in long direction, 3) dowel into adjacent curbs or structures and across cold joints/ expansion joints w/No. 3 bars at 18" o.c. [5]	10 feet	II/V	(3)

(1) The moisture content of the subgrade must be verified by the geotechnical consultant prior to sand/rock placement.
(2) Reinforcement to be placed both ways and at or above the mid-point of the slab (i.e., a minimum of 2.0 to 2.5 inches above the prepared subgrade).
(3) Soils having negligible levels of sulfates as defined by CBC are expected. Concrete mix design shall be selected by the concrete designer. Concrete mix design is outside the geotechnical engineer's purview.
(4) Where concrete/ flatwork is adjacent a stucco surface, a ¼" to ½" foam separation/expansion joint should be used.
(5) If dowels are placed in cored holes, the core holes shall be placed at alternating in-plane angles (i.e., not cored straight into slab).

General Note: Minor deviations to the above recommendations may be required at the discretion of the soils engineer or his representative.