



**PRELIMINARY GEOTECHNICAL REPORT
PROPOSED HUME SOCAL CAMPGROUND
REDEVELOPMENT AND EXPANSION PROJECT
251-ACRES OF FORESTED LAND
UNINCORPORATED SAN BERNARDINO COUNTY
CALIFORNIA**

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Attention: Mr. John Nsofor

**Subject: Preliminary Geotechnical Report
Proposed Hume SoCal Campground Redevelopment and Expansion Project
251-Acres of Forested Land
Unincorporated San Bernardino County, California**

In accordance with our July 22, 2022 authorization, Leighton and Associates, Inc. (Leighton) is pleased to present this Geotechnical Exploration Report addressing redevelopment and expansion of the subject Hume SoCal Christian Campground facility, located on approximately 251-acres of partially developed forestland within unincorporated San Bernardino County, California. The purpose of our study was to evaluate subsurface soil, groundwater and geologic hazard conditions within the areas planned improvements and provide preliminary geotechnical recommendations for project design and construction.

We appreciate the opportunity to provide our services. If you have any questions or concerns, please contact us at your convenience. The undersigned can be reached at **(866) LEIGHTON**, or directly at the phone extensions and e-mail addresses listed below.

Respectfully submitted,

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TABLE OF CONTENTS

<u>Section</u>	<u>Page</u>
1.0 INTRODUCTION	1
1.1 Site Description.....	1
1.2 Proposed Development	2
1.3 Purpose and Scope	3
2.0 GEOTECHNICAL FINDINGS.....	4
2.1 Regional Geologic Setting	4
2.2 Local Geologic Setting	4
2.3 Geologic Structure	5
2.4 Groundwater	6
2.5 Surface Water	6
2.6 Landslides and Slope Stability	6
2.7 Site Soil Properties	6
2.7.1 Expansive Soil.....	6
2.8 Hydrocollapse and Compressible Soils.....	7
2.9 Soil Corrosivity	7
2.9.1 Soil Compressibility.....	8
2.9.2 Shear Strength	8
2.9.3 Excavation Characteristics	8
2.9.4 Infiltration	9
3.0 GEOLOGIC HAZARDS.....	10
3.1 Liquefaction Potential.....	10
3.2 Seismically-Induced Settlement.....	10
3.3 Seismically-Induced Lateral Displacements.....	10
3.4 Seismically-Induced Landsliding	10
3.5 Rockfalls	11
3.6 Flooding	11
3.7 Debris Flows	11
3.8 Seiches and Tsunamis.....	12
3.9 Dam Inundation	12
4.0 SEISMIC HAZARDS	13
4.1 Surface Fault Rupture.....	13
4.2 Seismicity and Ground Motion	14
5.0 SUMMARY OF FINDINGS AND CONCLUSIONS.....	15
6.0 PRELIMINARY RECOMMENDATIONS	16
6.1 Earthwork.....	16
6.1.1 Site Preparation	16
6.1.2 Overexcavation and Recompaction	16

6.1.3	Subgrade Preparation	17
6.1.4	Fill Placement and Compaction.....	17
6.2	Preliminary Foundation Design Parameters	17
6.2.1	Shallow Foundations.....	17
6.2.2	Deep Foundation.....	18
6.3	Slab-On-Grade	20
6.4	Seismic Design Parameters.....	21
6.5	Earth Retaining Structures.....	21
6.6	Cement Type	22
6.7	Corrosion Protection Measures	23
6.8	Pavement Design	24
6.9	Surface Drainage.....	25
6.10	Additional Geotechnical Services	26
6.11	Plan Review.....	26
7.0	CONSTRUCTION CONSIDERATIONS	27
7.1	Rippability and Oversize Materials.....	27
7.2	Trench Backfill	27
7.3	Recreational Ponds	27
7.4	Foundation Setback.....	28
7.5	Temporary Excavations and Shoring Design.....	28
7.6	Rockfalls	29
8.0	LIMITATIONS.....	30

TABLES

Table 1. – Summary of Corrosion Potential.....	8
Table 2. – Preliminary Geotechnical Parameters for LPILE Analysis.....	20
Table 3. – 2022 CBC Based Seismic Design Parameters.....	21
Table 4. – Equivalent Fluid Pressures.....	22
Table 5. – Soil Corrosivity as a Function of Resistivity	23
Table 6. – Summary of Corrosivity Testing	24
Table 7. – Pavement Sections	25
Table 8. – Minimum Foundation Setback from Slope Faces	28

ATTACHMENTS

- Figure 1 – Site Location Map
- Figure 2 – Regional Geology Map
- Figure 3 – Regional Fault and Historical Seismicity Map
- Figure 4 – Flood Hazard Zone Map
- Figure 5 – Seismic Hazard Map
- Figure 6 – Dam Inundation Map
- Figure 7 – Retaining Wall Backfill and Subdrain Detail (EI < 50)

Plate 1 – Geotechnical Map

Plate 2A – Geotechnical Cross-Sections A-A' thru M-M'

Plate 2B – Geotechnical Cross-Sections N-N' thru Z-Z'

Appendix A – References

Appendix B1 – Auger Boring Summary

Appendix B2 – Field Exploration

Appendix C – Laboratory Test Data

Appendix D – General Earthwork and Grading Recommendations

Appendix E - GBA Geotechnical Insert

1.0 INTRODUCTION

1.1 Site Description

The boundaries of the site are roughly rectangular in shape and encompassing approximately 251 acres of forested mountain terrain, situated within the San Bernardino National Forest, unincorporated San Bernardino County, California. The regional site location is shown on attached Figure 1, *Site Location Map*. The property is developed sparsely as a 300-occupant campground facility, including various existing building structures, paved access roads, recreational ponds, pools, lawn areas and other appurtenant improvements. Similar undeveloped forest land borders the site on the north and northwest. Nearby local communities include Green Valley Lake, Arrowbear Lake and Running Springs to northeast, southeast, and southwest of the property, respectively. The site is accessed via Green Valley Lake Road, which transects through the property on the north, and forms its eastern boundary on the south. Based on a review of historical aerial photographs, construction of Green Valley Road and improvements on the site commenced sometime between 1951 and 1955.

Site Coordinates:

Latitude: N34.2268°

Longitude: W-117.0941°

USGS 7.5 Topographic Map

Keller Peak

December 2021

A majority of the site terrain is manifest in a series of relatively narrow ridgelines and valleys with steeply sided slopes. Surface elevations range between approximately 6,400 and 6,600 feet above mean sea level (msl). Local Areas of relatively flatter terrain exist as elevated paleo-stream terraces. Additional relatively localized flat terrain also occur within certain narrow valley-bottom areas resulting from more recent alluvial deposition. The main existing campground facilities are centrally located on the property, haven taken advantage of an area of naturally flatter surface relief. Existing building pads, access roads and minor slopes have been created by limited grading modification.

Our site reconnaissance, geologic mapping and auger borings reveal a large part of the site is mantled by thin deposits of residual soil, colluvium and/or alluvium. Large areas of bedrock are also exposed in surface outcrop, consisting of a Cretaceous-age granitic formation which is hard to very hard, massive, and slightly weathered. The general subsurface distribution of soil and bedrock units is depicted in a series of 20-scale cross sections prepared as transecting areas of planned improvements, attached as Plates 2A and 2B.

1.2 **Proposed Development**

We understand construction will be performed in five phases of work. Upon completion, the improvements will accommodate approximately 3,000 additional occupants. Based on a set of preliminary plans forwarded to our office, dated January 9, 2023, the phase numbers, phase names and general locations on the property are referred to herein as follows:

- Phase 1 – Junior High School Camp (central)
- Phase 2 – High School Camp (SE)
- Phase 3 – Adult Lodge (W)
- Phase 4 – Elementary School Camp (NE)
- Phase 5 – Wildwood Camp (NW)

The proposed location and layout of improvements for each phase are depicted on attached Plate 1, *Geotechnical Map*. Construction types will vary from phase to phase, including demolition and replacement of certain existing buildings, and conversions and/or additions to others. New structures will include multi-story dormitories, large open-ceiling chapels, dining facilities, pools, gymnasiums, snack shops, restrooms, maintenance facilities, parking lots, grass quads, outdoor amphitheaters and recreational ponds. We expect appurtenant improvements will include access roads with embankment fills and/or cut slopes, drainage control structures and underground utilities.

Given existing site geologic conditions, and preference to limit disturbance of natural terrain by grading, many lighter-load structures of limited footprint can be founded on systems of drilled rock-socketed piles and grade-beams. Floors may consist of raised and/or slab-on-grade construction. For larger buildings, especially in areas of formerly graded flat pads where existing buildings will be demolished, re-grading and use of conventional spread footings and slab-on-grade construction should be considered. Where building footprints will span sloped terrain, a combination of conventional spread footing and drilled rock-socketed pile foundations may be considered.

As of the date of this report, no grading or structural plans defining foundation design or load demands of structures have been prepared. Geotechnical recommendations for design are thus considered preliminary, and subject to revision or value engineering once plans become available.

1.3 **Purpose and Scope**

The purpose of our work was to evaluate site geotechnical conditions and geologic hazards pertinent to the proposed development, determine engineering properties of soil and bedrock, and provide preliminary geotechnical recommendations to aid in project planning. To accomplish these tasks we reviewed readily available published geologic reports and maps, conducted site geologic reconnaissance mapping and advanced small-diameter auger borings to determine the thickness of surface soils, depth to competent bedrock, and collection of samples for physical and chemical laboratory testing. We prepared this report to summarize the findings of our study, and aid in the preparation of project grading and foundation design plans. A list of documents consulted as part of our study are noted in attached Appendix A, *References*.

The scope of work performed is as follows:

- **Background Review:** Reviewed published geology reports/maps, and historical aerial photographs and topographic maps and Lidar imagery readily available and pertinent to the site.
- **Field Exploration:** For the purpose of site soil and bedrock exploration, assessment of groundwater conditions, and collection of samples for laboratory testing, we advanced fifty-five (55) small-diameter auger borings into select locations each of the five development phases, and collected representative samples of surficial bedrock boulders for coring and testing. A summary of subsurface boring data is attached in Appendix B1, *Auger Boring Summary*. A summary of our field exploration is attached in Appendix B2, *Field Exploration*.
- **Geotechnical Laboratory Testing:** Conducted certain laboratory tests on collected soil and bedrock samples including Modified Proctor Compaction (maximum dry density and optimum moisture content), R-Value, remolded direct shear, unconfined compressive strength, and corrosivity suite (sulfate and chloride content, resistivity and pH). The results of our laboratory testing are attached in Appendix C, *Laboratory Test Data*.
- **Analyses and Report Preparation:** Analyzed collected field data to determine soil properties for design. Prepared this report presenting our preliminary geotechnical findings, conclusions and recommendations.

2.0 GEOTECHNICAL FINDINGS

2.1 Regional Geologic Setting

From a regional perspective, the site exists within the eastern (San Bernardino Mountains) segment of the Transverse Ranges Geomorphic Province of California. The province is defined by an east-west trending series of steep mountain ranges and valleys trending obliquely to the typical northwesterly trend of coastal California. The regional extent of the province spans easterly from the San Bernardino Mountains as far as the offshore islands of San Miguel, Santa Rosa, and Santa Cruz. The San Bernardino Mountains are considered one of the most rapidly rising regions on earth. Over the past 11 million years they have been uplifted to heights over 11,000 feet above mean sea level (msl), largely in response to intense north-south tectonic compression and southward displacement along the San Andreas Fault. The bulk of bedrock underlying the San Bernardino Mountains, and the site, are composed of Mesozoic age granitoid and gneissose bedrock associated with the southern California Batholith, see attached Figure 2, *Regional Geology Map*, and Figure 3, *Regional Fault and Historical Seismicity Map*.

2.2 Local Geologic Setting

Based on the findings of our subsurface exploration and review of published geologic reports/maps, the site is underlain by units of relatively thin surficial deposits including artificial fill and colluvium/alluvium. Granitic bedrock occurs extensively in surface outcrop and at shallow depths below the surface soils. Due to the relative thin and variable thickness of the surficial units, they are not mappable at the scale of the attached geologic maps. While the soil units are excluded from Plate 1, they are shown in attached 20-scale cross sections to provide a generalized understanding of local conditions and relationship to improvement concepts.

A brief description of soil and bedrock units encountered on the site is presented below.

- Artificial Fill (undocumented) (Map Symbol: Afu): The deposits of fill encountered on the site typically consist of a loose to medium dense light yellowish-grey to tan coarse-grained sand, derived from weathered bedrock and containing roots and scattered cobbles. The fill thickness ranges from approximately 1 to 5 feet where encountered. The fill was identified as occurring in local slopes adjacent existing roadways, or as underlying certain leveled pads beneath existing buildings. No reports documenting placement of fill under engineered control were available. Thus, this unit is herein defined as undocumented. Where a grading solution is preferred to provide structural support of new improvements, the undocumented fill unit will require complete removal and replacement.

- Quaternary Alluvium/Colluvium (Map Symbol: Qal/col): Deposits of Alluvium and Colluvium are grouped together for the purposes of this report. The colluvium unit is defined as a thin mantle of soil typically covering slopes, derived from upslope erosion. The soils were found to consist of loose light grey to tan coarse-grained silts and sands with roots concentrated along their contact with underlying bedrock. The diameter of roots generally increases with depth below the surface. The thickness of the colluvium is generally less than 2 feet where encountered. Alluvium consist of loose sediments concentrated along canyon bottoms, and locally as deposits on elevated terrace areas. Given the potential compressible nature of the alluvium/colluvium unit, it will require complete removal and replacement in areas where a grading solution is preferred to provide structural support for new improvements.

Cretaceous Granite (Map Symbol: Kk): The entire site is underlain by bedrock assigned as Cretaceous age Granite (USGS, 1998). The crystalline igneous rock occurs widely in surface outcrop or buried beneath thin deposits of alluvium/colluvium and/or fill units. It is hard to very hard, light in color, massive, and exhibits a widely spaced joint pattern. Where exposed at the surface it is altered by differential physical and chemical weathering processes resulting in rounded shapes bounded by open joints. The depth of weathering is on the order of a few feet thick, where fracturing and jointing are more pronounced. Rock hardness quickly increases with depth beneath the surface, as does the effects of open fracturing and jointing. Where surface erosion is focused along joints, localized precarious-pedestaled boulders take form which can become dislodged and prone to downslope movement as rockfalls. Fresh bedrock is considered suitable for structural support new fill, and/or rock-socketed piles.

The depths to bedrock beneath the site surface was determined by drilling and logging a series of auger borings. The encountered thickness of surface soils and depths of bedrock are noted in attached Appendix B1, *Auger Boring Summary*. The depths to bedrock are considered representative of site conditions, however, it is possible that deeper soils may exist in areas beyond the locations of the auger borings.

2.3 Geologic Structure

Our field mapping indicates the bedrock structure is massive and absent any adverse through-going structures that would otherwise result in large-scale planes of weakness. If larger cut slopes are planned, the orientation of joint-sets should be mapped to determine any degree of potential surficial instability.

2.4 Groundwater

Local groundwater was encountered as a shallow perched condition above bedrock, confined to deposits of alluvium/colluvium adjacent to active stream channels. Sources of groundwater mainly relate to periods of precipitation in the form of rain or snow melt. Based on the encountered conditions, we expect groundwater will not pose any significant constraint to planned grading or proposed construction.

2.5 Surface Water

The most significant occurrence of surface water is limited to active stream channels on the site. Certain structures are planned to span the axes of dry and active stream channels on the site. If buildings cannot be relocated, the control of surface waters will be required to mitigate the potential impacts of current and potentially larger future flow of water and debris. Other areas of potential concentrated surface flow can be expected in side-slope swales. Through implementation of proper engineering control, the impacts of surface water are not expected to be a significant constraint to development.

2.6 Landslides and Slope Stability

No evidence of landslides or landslide deposits were observed during our site reconnaissance, nor identified during our review of historical aerial photographs. The massive nature of the granitic bedrock formation underlying the site is such that the potential for significant landslides or large-scale slope instability is considered very low. However, an assessment of slope stability should be made during and after excavations into the existing road cut are made. Slope stability analysis was not within the scope of this study. Furthermore, although no surficial signs of adverse geologic structure (adverse jointing and joint orientation) were observed in the areas of planned improvements at the time of our investigation, adverse structure could be exposed during construction. As such, a Geologist should be present at the time of any temporary or permanent cuts to observe and map exposed geologic structure, as needed.

2.7 Site Soil Properties

Based on the results of our field exploration and laboratory testing, certain engineering characteristics of on-site soils (fill and alluvium/colluvium) are discussed as follows.

2.7.1 Expansive Soil

Given the non-cohesive course-grained nature of on-site soils, no tests were performed to evaluate expansive potential.

2.8 Hydrocollapse and Compressible Soils

Based on the coarse-grained character of alluvium/colluvium soils, the relative thin presence of the soil, and recommendations for complete removal of the soil where grading is preferred, the potential for hydro-collapse is not a constraint to site development. The bedrock is hard and not prone to such hazards.

2.9 Soil Corrosivity

A screening of site soil units for corrosivity was performed to evaluate their potential effect on concrete and ferrous metals. The corrosion potential was evaluated using the results of laboratory tests on representative soil samples obtained during our subsurface evaluation. Resultant test parameters were determined for pH, minimum electrical resistivity, and chloride and soluble sulfate content. The testing results indicated an average values of 5.33 for pH, measured minimum electrical resistivity of 45,1400 ohm-cm, chloride content of 74.8 ppm, and soluble sulfate content of 157.8 ppm.

In general, several agencies list a set of conditions for soils to be considered a corrosive environment. The California Department of Transportation (Caltrans) considers a site to exhibit a corrosive environment if one or more of the following conditions exist: chloride concentration is 500 ppm or greater, sulfate concentration is 2000 ppm or greater, or pH is 5.5 or less (Caltrans, 2012).

In addition, the American Association of State Highway and Transportation Officials (AASHTO) considers a water condition detrimental potential piles where: chloride concentration is greater than 500 ppm, sulfate concentration is greater than 500 ppm, there is potential effluence of mine or industrial runoff, high organic content, a pH of less than 5.5, marine borers (biological), or piles exposed to wet/dry cycles.

The following table presents a summary of corrosion analysis, in addition to values where Caltrans and AASHTO consider an environment to be corrosive. Please note the summary is limited to ONLY the listed conditions expressed in the table. Not all conditions considered to be constituent, part or whole, to corrosion potential were considered during our exploration and as a result our assessment of onsite corrosion potential is considered preliminary. A corrosion engineer should be consulted for further analysis and recommendations.

Table 1. – Summary of Corrosion Potential

Constituent		Average Site Value (ppm)	Caltrans Values (ppm)	AASHTO Values (ppm)
Chloride Content (ppm)		74.8	500 or greater	500 or greater
Sulfate Content (ppm)		157.8	2,000 or greater	500 or greater
pH		5.33	5.5 or less	5.5 or less

Preliminary corrosion analysis indicate the site soils are generally considered not to host a corrosive environment; however, site soils in contact with buried uncoated ferrous metals is considered to pose a corrosion hazard and should be mitigated.

2.9.1 Soil Compressibility

While no in-situ samples of surficial alluvium/colluvium soils were collected for compressibility testing, their organic content and density suggest they are compressible and should not be relied upon for structural support of structures.

2.9.2 Shear Strength

Evaluation of shear strength characteristics included laboratory direct shear testing of composite remolded soils. The results of testing are attached in Appendix C, including graphs of angle of internal friction (ϕ) and cohesion (c) for use in geotechnical analysis.

2.9.3 Excavation Characteristics

Soil: It is anticipated on-site alluvium/colluvium and fill soils can be excavated with relative ease using conventional light- to heavy-duty construction equipment.

Bedrock: Bedrock ranges from moderately hard where weathered, and hard to very hard where unweathered. Its rippability depends on the degree of weathering, density of fractures and jointing. Given the interest to avoid grading, we do not anticipate that excavations of any significant depth, or modifications of the site surface other than to remove and replace existing soils are planned. If deeper cuts in bedrock are planned, we anticipate rippability will be difficult and require use of heavy grading equipment including but not limited to dozers with single-shank rippers. To further discern rippability characteristics where such grading is

planned, the use of geophysical refraction surveys may provide more specific information concerning the feasibility of bedrock excavations.

2.9.4 Infiltration

The location and types of stormwater or septic systems was not designated on the current site plan. Thus, Leighton did not evaluate the infiltration rates for site soils. We anticipate the employment of such systems would be limited to the thin on-site alluvium/colluvium soil unit, away from planned structures. The evaluation of infiltration rates should be performed once more detailed plans are available.

3.0 GEOLOGIC HAZARDS

Potential geologic hazards common to southern California and more directly constrained to an on-site condition commonly include surface fault rupture and landslides and rockfalls. A summary of these hazards is provided in the following sections.

3.1 Liquefaction Potential

Liquefaction is the phenomenon of the reduction in strength and stiffness of certain soil types that exist below the groundwater table when subjected to strong ground shaking such as typically associated with seismic activity. Soils most susceptible to liquefaction are saturated granular deposits that exhibit loose to medium dense relative density.

The San Bernardino County Geologic Hazard Overlay Maps (2010), indicate the site is not located within an area susceptible to liquefaction, and thus not warranting any required investigation for liquefaction.

3.2 Seismically-Induced Settlement

Seismically-induced settlement consists of dynamic settlement of unsaturated soil (above groundwater) and liquefaction-induced settlement (below groundwater). These settlements occur primarily within low density sandy soil due to reduction in volume during and shortly after an earthquake event. Given the relatively thin presence of soil deposits on the site, and intended use of pile foundation and/or grading solutions, the potential for seismically-induced settlement is expected to be negligible.

3.3 Seismically-Induced Lateral Displacements

On sites where a liquifiable soil mass exists, the lateral movement of a non-liquefied soil mass can occur, termed lateral spreading. The direction of mass movement is typically towards a nearby gap in local topography, such as a basin, channel or valley. As earth deposits on the subject site are not prone the occurrence of liquefaction which is instrumental for the occurrence of lateral spreading, the potential for lateral spreading is in turn considered remote.

3.4 Seismically-Induced Landsliding

Based on the geologic hazard overlay maps issued by the County of San Bernardino, and preponderance of underlying massive igneous bedrock, the potential occurrence of landslides at the site is considered low. Proposed slopes, if any, should be engineered and constructed at a gradient of 2:1 (horizontal:vertical) or flatter top facilitate overall stability.

3.5 Rockfalls

The subject site consists of rugged forested terrain with moderate to steep slopes and narrow valleys. In areas where bedrock is exposed at the surface and an advanced degree of spheroidic weathering has occurred, larger precariously perched boulders have formed, typically bounded by joints, which are subject to dislodging and sudden downslope movement. The movement is often triggered by ground-shaking during earthquakes, where the boulders are mobilized in a downslope direction. In natural areas upgradient of planned grading, rockfalls can pose a hazard to path of human life, improvements, and/or infrastructure. Leighton observed only localized areas on the site during our reconnaissance, where the potential for this hazard exists. During the development phase of the project, a qualified geologist should examine areas upslope of planned improvements in more detail, and designate boulders posing a potential rockfall hazard. The boulders should be eliminated as part of the construction process, by either removal or breaking the rock in-place.

3.6 Flooding

According to current flood hazard maps available on the ESRI ArcGIS on-line database, 2023, the site is not located within a 100-year flood zone. A map showing the site and location of nearby flood hazard zones is attached as Figure 4, *Flood Hazard Zone Map*. Based on the above, the potential for site flooding is considered low.

3.7 Debris Flows

Debris flows are typically defined as the down-gradient movement of a viscous saturated silt and sand laden slurry like mixture soils laden with cobbles, boulders, vegetation and other debris. These events are surface phenomenon typically confined to areas with vertical gradients, including slopes, side-hill swales and valley bottoms where volumes of sediment have accumulated. The impacts of potential debris flows often depend on the type and volume of soil present, water content, and degree of vegetation. Debris flows are common in recent wildfire burn areas where vegetation and root structure has been removed. The loss of soil support leads to the accumulation of moisture in the soil from rain events and a higher potential for structural failure. Debris flows are not exclusive to a post-fire areas, and can occur as a result of repeated heavy rain storms where soils become increasingly saturated by waters. Debris flows can travel a significant distance from its source area under the right conditions.

Our reconnaissance of the site revealed no direct evidence of recent surface scaring or accumulations of relatively thick soils at the toe of slopes relating to possible debris flows.

While our observations note the above, the potential for debris flows is considered a potential hazard at the site as areas surrounding planned improvements will remain undisturbed or subject to little modification. A consideration of this hazard should be evaluated a part of site development, especially where buildings are cited in canyon bottom areas or side-slopes. The mitigation of debris flow hazards can include installation of drainage diversion devices such as sandbags, K-Rails, Hydro Barriers, etc., placed along known flow paths to divert runoff.

3.8 Seiches and Tsunamis

Seiches are the occurrence of waves within enclosed water bodies in response to ground shaking. The waves have the potential to runup shorelines and overtop dams. The existing and future recreational ponds and pools on the site are considered subject to the occurrence of a seiche hazard in the event of large earthquakes.

Tsunamis are sea waves generated by the sudden displacement or movement of a sea floor. Tsunami waves can impact shorelines a great distance from their source. Based on the absence of any ocean body in proximity to the site, and significant altitude above mean sea level, the potential for a tsunami to impact the site is considered remote.

3.9 Dam Inundation

According to current flood hazard maps available on the ESRI ArcGIS on-line database, 2023, the site is not located within a 100-year flood zone. A map showing the site and location of nearby flood hazard zones is attached as Figure 4, *Flood Hazard Zone Map*. A map is also provided showing an absence of potential dam inundation hazards at the site, see Figure 6, *Dam Inundation Map*. Based on the above, the potential for site flooding is considered low.

4.0 SEISMIC HAZARDS

Seismic hazards common to southern California include surface fault rupture and strong ground shaking associated earthquake(s). Depending on site conditions, other secondary seismic effects may include liquefaction, seismically-induced settlement, dynamic settlement, lateral spreading, seismically-induced landslides, seismically-induced flooding, seiches and tsunamis. Attached Figure 5, *Seismic Hazard Map*, depicts potential state-designated seismic hazards in the site area. The following sections discuss seismic hazards and their potential effects on the subject site.

4.1 Surface Fault Rupture

The project site is located in a seismically active region of southern California crossed by multiple well-defined active and potentially active faults capable of inducing ground motions affecting wide areas including the subject site. Accordingly, to reduce seismic risk, project improvements should be designed using appropriate seismic design parameters promulgated by current codes and standards, including the “Minimum Statewide Safety Standard” in Chapter 2 of the California Geological Survey (CGS) Special Publication 117a (CGS, 2008).

Our review of available in-house and readily available literature indicates the presence of no known active faults within the site boundaries. And the site is not designated within a state-defined Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). Therefore, the potential for surface fault rupture on the site is expected to be low and a surface fault rupture hazard evaluation is not mandated for this site.

The locations of active faults within 100 miles of the site was determined using the on-line program managed by the United States Geological Survey (USGS) Earthquake Hazards Program National Seismic Hazard Maps (USGS, 2008c). The nearest active faults with historical or latest Quaternary displacement occur within the San Andreas Fault Zone (SAFZ), San Jacinto Fault Zone (SJFZ), and Johnson Valley Fault Zone (JVFZ) located approximately 7.9, 15.5 and 36.7 miles from the site, respectively. The SAFZ is the largest active tectonic plate boundary in California. The SJFZ is considered one of the most active faults in southern California. And the JVFZ is considered an important component of the system in that it has experienced several more recent surface rupture events affecting the local area. Attached Figure 3, *Regional Faults and Historical Seismicity Map*, depicts these and other major faults in the vicinity of the subject site.

4.2 **Seismicity and Ground Motion**

The intensity of ground shaking at a given location depends primarily upon the earthquake magnitude, the distance from the source, and the site response characteristics. Peak horizontal ground accelerations are generally used to evaluate the intensity of ground motion. Using the SEAOC/OSHDP Seismic Design Maps Tool (<https://seismicmaps.org/>) to obtain seismic design parameter values from the United States Geological Survey (USGS), the peak ground acceleration for the Maximum Considered Earthquake (MCE_G) adjusted for the Site Class effects (PGA_M) is 0.71g. Based on the USGS online unified hazard tool program (USGS, 2023a), the modal seismic event is Moment Magnitude (M_w) 8.08 at a distance of 7.7 miles.

5.0 SUMMARY OF FINDINGS AND CONCLUSIONS

Presented below is a summary of findings based on the results of our site geotechnical exploration.

- The site is not located in a state-designated Alquist-Priolo Earthquake Fault Zone, nor fault hazard zone depicted on San Bernardino County Hazard Overlay Maps. The nearest known significant active faults are the San Andreas Fault Zone (SAFZ), San Jacinto Fault Zone (SJFZ), and Johnson Valley Fault Zone (JVFZ) located approximately 7.9, 15.5 and 36.7 miles from the site, respectively. An earthquake along any of these or other similar more distant faults in the southern California area, are capable of generating moderate to strong ground shaking at the site.
- The site is not located within an area shown as susceptible to liquefaction or landslides.
- The site is underlain by locally by up to approximately 4 to 5 feet of undocumented artificial fill. The fill typically underlies existing building pads and some slopes. The fill is not considered suitable for support of new buildings and will require removal and replacement in areas of planned grading.
- No areas of side-hill seepage or springs were encountered on the site. The occurrence of shallow perched groundwater conditions was encountered locally in thin scattered deposits of alluvium confined to valley bottoms associated with active stream flow.
- The shallow depth of bedrock on the site will permit use of rock-socked piers to support light-weight structures. On sloping ground it is possible that a combination of piers with raised floors and grading for slab-on-grade floors will be feasible. For larger relatively flat pad areas, especially where existing buildings are present, a grading solution is recommended for entire support of new buildings.

6.0 PRELIMINARY RECOMMENDATIONS

Preliminary geotechnical recommendations intended to assist in preparation of grading and foundation plans are presented in the following sections. The recommendations are considered sufficient at this preliminary stage to develop project plans in accordance with 2022 CBC requirements. The following recommendations may be superseded by more restrictive requirements of the architect, structural engineer, and/or local building official.

The client is also directed to attached Appendix E, *GBA Geotechnical Insert*, for more information concerning this report.

6.1 Earthwork

All site grading should be performed in accordance with the project specifications of appropriate design professionals. Attached are geotechnical specifications for use as a guideline in developing plans for earthwork grading, see Appendix D, *General Earthwork and Grading Recommendations*.

6.1.1 Site Preparation

Vegetation, debris, and other deleterious materials should be removed and disposed of offsite prior to the commencement of grading operations. Existing underground improvements, including utility lines, should be identified prior to the start of grading and abandoned or relocated as necessary. Trenches resulted from removal of existing improvements should be excavated to competent materials and properly backfilled under the observation and testing of the geotechnical engineer.

6.1.2 Overexcavation and Recomaction

Proposed foundations may be constructed on bedrock or a minimum of 2 feet of compacted fill. Existing near surface fill and alluvial/colluvial soils below the proposed foundation should be removed to expose bedrock and recompacted. The foundation for the proposed structures may then be constructed on undisturbed, competent bedrock or properly compacted structural fill, in order to reduce the potential for differential settlement. If fill soils or unsuitable native soils are exposed at the bottom of the excavation, they should be removed and replaced with engineered structural fill of uniform thickness. The compacted fill should extend a minimum 3 feet beyond outside edges of the foundation. If any of the planned structures are supported on a deep foundation system, then overexcavation is not required.

Pavement areas, driveway, concrete flatwork, and slab on grade should be underlain by a minimum 1 foot of compacted fill. Local conditions may be encountered which may require additional removals and recompaction. The exact extent of removals can best be determined during grading by the geotechnical engineer when direct observation and evaluation of materials are possible.

6.1.3 Subgrade Preparation

Prior to placing fill materials, the subgrade should be scarified to a minimum depth of 6 inches, moisture conditioned, and proof-rolled. Any soft and/or unsuitable materials encountered at the bottom of the excavations should be removed and replaced with fill material.

6.1.4 Fill Placement and Compaction

The onsite soils to be used as compacted structural fill should be free of organic material, construction debris or oversized material larger than 6 inches. Imported fill soils, if any, should be approved by the geotechnical engineer prior to placement as fill. Fill soils should be placed in loose lifts not exceeding 8 inches, moisture-conditioned as necessary to at least two percent above moisture optimum and compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM D 1557.

6.2 Preliminary Foundation Design Parameters

New structures may be supported on shallow footings or Cast-in-Drilled-Hole (CIDH) concrete piles. Driven piles are not recommended due to the hardness of bedrock. Note that these values are preliminary, and supplemental field data including drill cores should be collected during pile drilling to confirm design parameters.

6.2.1 Shallow Foundations

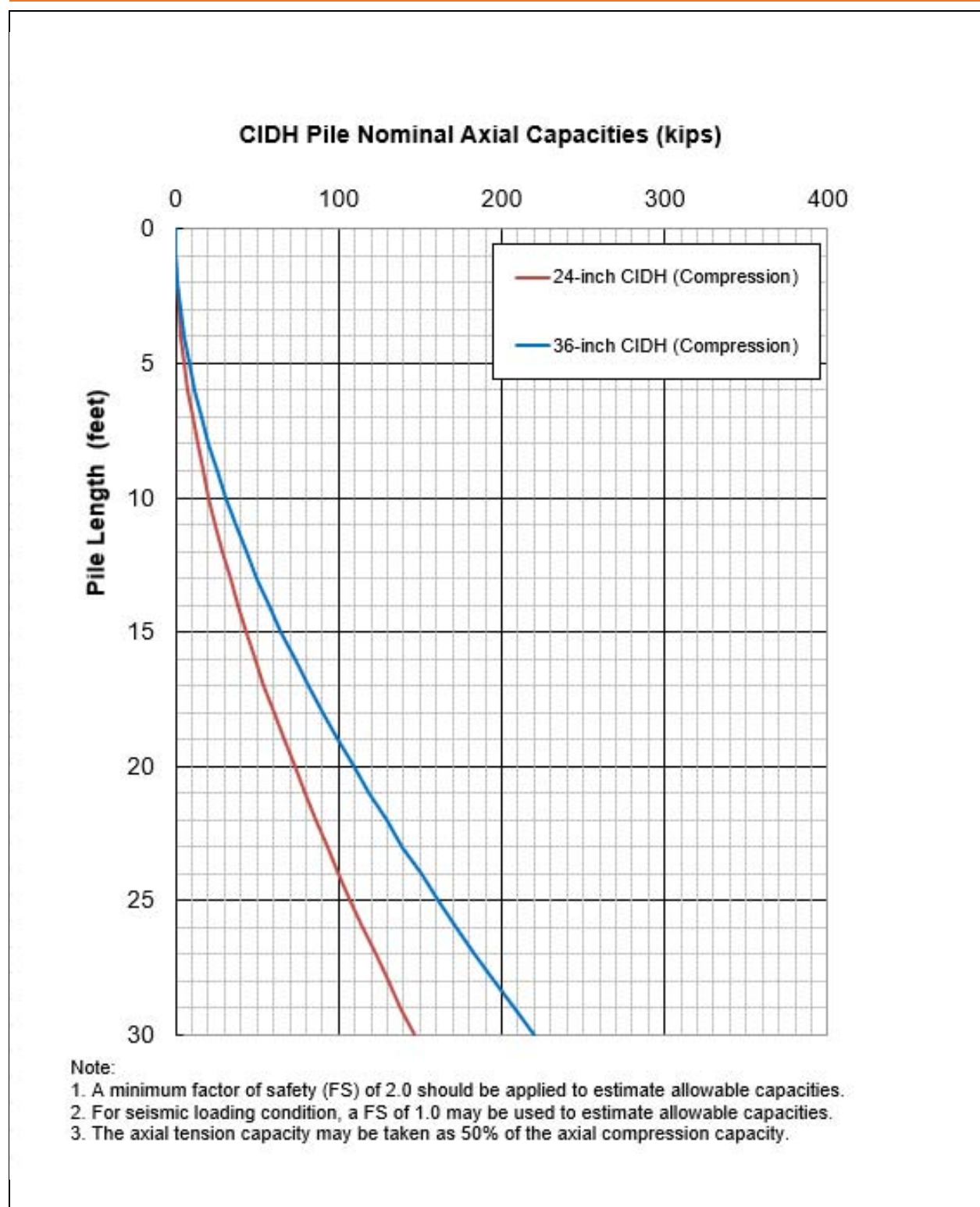
Following site grading recommended in Section 6.1, new structures may be supported on a conventional shallow foundation system. Footings may be designed using an allowable bearing capacity of 2,500 psf based on a minimum width of 12 inches and embedment depth of 12 inches below the lowest adjacent grade. This value may be increased by 250 psf for each additional foot of width or each additional foot of embedment to a maximum value of 4,000 psf. In addition, the bearing capacity may be increased by one-third for wind or seismic loading. The allowable bearing capacity incorporates a factor of safety of 3.0 and is based on a total static settlement of 1 inch and differential settlement of ½ inch over a

horizontal distance of 30 feet. Since settlement is a function of footing size and contact bearing pressure, differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists. The settlement estimate should be reviewed by Leighton Consulting when final foundation plans and loads for the proposed structures become available.

Resistance to lateral loads will be provided by a combination of friction between the soils and foundation interface and passive pressure acting against the vertical portion of the foundation. A friction coefficient of 0.40 may be used at the soil-concrete interface for calculating the sliding resistance. A passive pressure based on an equivalent fluid pressure of 390 pounds per cubic foot (pcf) may be used for calculating the lateral passive resistance. The lateral passive resistance can be taken into account only if it is ensured that the soils against embedded structures will remain intact with time. The above values do not contain an appreciable factor of safety, so the structural engineer should apply the applicable factors of safety and/or load factors during design.

6.2.2 Deep Foundation

The axial capacities of 24-inch and 36-inch CIDH piles for frequently applied dead plus live load is presented in the figure below. A factor of safety of 2.0 may be used to obtain the allowable static pile capacities and a factor of safety of 1.0 may be used for transient loads such as wind or seismic forces. Uplift capacities may be taken as equal to 50 percent of the downward capacities. To avoid group effects, the pile on-center spacing should be at least three times its largest dimension.



Pile design to provide lateral load resistance is recommended to be performed based upon the idealized subsurface profile and design parameters presented in the following table. If piles are used in a group with a pile cap, the passive pressure indicated in Section 6.5 may be used to calculate lateral resistance of the pile cap.

Table 2. – Preliminary Geotechnical Parameters for LPILE Analysis

Depth Range (feet)	Material	LPILE Model	Effective Unit Weight (lbs/ft ³)	Cohesion (lbs/ft ²)	ø (degrees)	ε ₅₀	k ⁽¹⁾ (lbs/in ³)
0 - 5	Fill Soils, Alluvium/Colluvium	SAND	120	0	32	---	30
Below 5	Granitic Bedrock	SAND	130	0	38	---	225
Notes: (1) Modulus of Subgrade Reaction P-multipliers dependent upon center-to-center (CTC) spacing. P-mult. = 1.0 for CTC = 3 oriented perpendicular to loading.							

6.3 Slab-On-Grade

Concrete slabs-on-grade subjected to special loads should be designed by the structural engineer. Where conventional light floor loading conditions exist, the following minimum recommendations for conventional slabs-on-grade should be used. More stringent requirements may be required by local agencies, the structural engineer, the architect, or the CBC.

- A minimum slab thickness of 5 inches. Slab reinforcement should be designed by the structural engineer but as a minimum should consist of No. 3 rebar placed at 24 inches on center in each direction and provided with adequate concrete cover.
- A vapor barrier, 10-mil or thicker, should be placed below slabs where moisture-sensitive floor coverings or equipment is planned. The vapor barrier should be properly sealed at all joints and any penetrations.
- To reduce the potential for excessive cracking, concrete slabs-on-grade should be provided with construction or weakened plane joints at frequent intervals. Joints should be laid out to form approximately square panels.
- Subgrade for slab-on-grade should be thoroughly wetted prior to placement of rebar and concrete.

Our experience indicates that use of reinforcement in slabs can generally reduce the potential for drying and shrinkage cracking. Some cracking should be expected as the concrete cures. Minor cracking is considered normal; however, it is often aggravated by

a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. The use of low slump concrete can reduce the potential for shrinkage cracking.

6.4 Seismic Design Parameters

Design parameters for minimum seismic load based on the 2022 California Building Code are included in Table 3 below.

Table 3. – 2022 CBC Based Seismic Design Parameters

Categorization/Coefficient	Design Value
Site Latitude	34.2268°
Site Longitude	-117.0941°
Site Class	C
Mapped Spectral Response Acceleration at Short Period (0.2 sec), S_s	1.50g
Mapped Spectral Response Acceleration at Long Period (1 sec), S_1	0.60g
Short Period (0.2 sec) Site Coefficient, F_a	1.2
Long Period (1 sec) Site Coefficient, F_v	1.4
Adjusted Spectral Response Acceleration at Short Period (0.2 sec), S_{MS}	1.80g
Adjusted Spectral Response Acceleration at Long Period (1 sec), S_{M1}	0.84g
Design Spectral Response Acceleration at Short Period (0.2 sec), S_{DS}	1.20g
Design Spectral Response Acceleration at Long Period (1 sec), S_{D1}	0.56g
Mapped Geometric Mean MCE_G Peak Ground Acceleration, PGA	0.59g
Site Coefficient, F_{PGA}	1.2
PGA adjusted for Site Class, $PGA_M = F_{PGA} * PGA$	0.708g

6.5 Earth Retaining Structures

The following recommendations may be used for design and construction of retaining structures at the site. We recommend that any permanent earth retaining structures be backfilled with onsite or import soil with Expansion Index (EI) of not greater than 50 (per ASTM D 4829).

Table 4. – Equivalent Fluid Pressures

Condition	Level Backfill	2:1 Backfill
Active	37 pcf	89 pcf
At-Rest	57 pcf	63 pcf
Passive	390 pcf (Maximum of 3,900 psf)	
Seismic	20 pcf	48 pcf

Retaining walls retaining more than 6 feet of soil should consider a seismic earth pressure increment with an inverted triangular distribution in addition to the active earth pressure provided above. The seismic earth pressure increment is included in Table 3. The above values do not contain an appreciable factor of safety, so the structural engineer should apply the applicable factors of safety and/or load factors during design. Retaining walls should include provisions for wall drainage with a backdrain system in accordance with the recommendations provided on attached Figure 7, *Retaining Wall Backfill and Subdrain Detail (EI < 50)*, and the materials behind the wall should be free-draining (i.e., less than 5 percent passing the No. 200 sieve).

Cantilever walls that are designed for a deflection at the top of the wall of at least 0.001H, where H is equal to the wall height, may be designed using the active earth pressure condition. Rigid walls that are not free to rotate, walls that are braced at the top, and walls that provide indirect support for foundations should be designed using the at-rest condition.

Lateral load resistance will be provided by the sliding resistance at the base of the foundation and the passive pressure developed along the front of the foundation. A frictional resistance coefficient of 0.40 may be used at the concrete and soil interface.

In addition to the above lateral forces due to retained earth, the appropriate loads due to surcharges should be considered in the design of retaining structures.

6.6 Cement Type

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates and/or pH values of less than 5.5. Section 4.3 of ACI 318 (ACI, 2018) provides specific guidelines for the concrete mix-design when the soluble sulfate content of the soil exceeds 0.1 percent by weight or 1,000 parts per million (ppm).

The results of laboratory testing indicated concentrations of soluble sulfate greater than 0.1 percent but less than 0.2 percent. The test results indicate a sulfate Exposure Class

designation of “S1” appears to be appropriate for the project site based upon criteria presented in ACI 318. If the concrete is expected to be in contact with reclaimed water, Type V cement and a water/cement ratio of 0.45 should be used. Samples should be collected from materials imported to the site as well as soils the compacted fill subgrade upon completion of grading to confirm the initial findings of this exploration.

6.7 Corrosion Protection Measures

In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor for corrosion to ferrous metals. The following table presents an approximate relationship between soil resistivity and soil corrosiveness.

Table 5. – Soil Corrosivity as a Function of Resistivity

Soil Resistivity (ohm-cm)	Classification of Soil Corrosiveness
< 900	Very severe corrosion
900 to 2,300	Severely corrosive
2,300 to 5,000	Moderately corrosive
5,000 to 10,000	Mildly corrosive
10,000 to >100,000	Very mildly corrosive
Note: ¹ ASTM STP 1013 titled <i>Effects of Soil Characteristics on Corrosion</i> (February, 1989).	

In addition to resistivity, the concentration of chloride ions can also be used to evaluate corrosive potential to steel, either in the form of reinforcement protected by concrete cover or plain steel substructures, such as steel pipes. As a general guideline, the chloride threshold adopted by Caltrans is a concentration of 500 ppm or greater as determined by California Test 532. Concentrations of chloride ions above the stated concentration or other characteristics such as soil resistivity or redox potential may warrant special corrosion protection measures. The results of corrosivity testing are summarized in the following table.

Table 6. – Summary of Corrosivity Testing

Test Parameter	Test Results						General Classification of Hazard
	HA-5a	HA-15	HA-19	HA-27	HA-37	HA-17	
	0 to 1'	1' to 2'	1' to 1.5'	2' to 2.5'	1.5' to 2'	3' to 3.5'	
Water-Soluble Sulfate in Soil (ppm)	72	283	219	98	99	176	Negligible to moderate
Water-Soluble Chloride in Soil (ppm)	144	42	53	41	44	125	Non-corrosive
pH	5.68	5.33	5.37	5.88	5.33	5.40	Moderately Acidic
Minimum Resistivity (saturated, ohm-cm)	47,000	50,500	20,600	39,600	23,000	45,000	Very mildly corrosive to buried ferrous pipes

The results of the resistivity tests indicate that the near surface soil exhibits very mildly corrosive potential to buried ferrous metals per ASTM STP 1013. The samples tested for water-soluble chloride content indicate a very low potential for corrosion of steel in concrete due to the chloride content of the soil.

Based on the available laboratory testing, the onsite soils are not considered to have low corrosion potential to ferrous metals. The corrosion information presented in this report should be provided to the underground utility subcontractors. We recommend that additional testing for general corrosion potential to ferrous materials and water-soluble sulfates in soils is performed during grading.

6.8 Pavement Design

Driveways and parking areas can be constructed using conventional asphalt concrete (AC) over aggregate base (AB). We have designed the pavement sections using a design R-value of 50 for different Traffic Indices (TI) and the minimum pavement thickness is presented in Table 6. The pavement design was performed using the method in *Caltrans Highway Design Manual*.

Table 7. – Pavement Sections

Traffic Index	Flexible Pavement (inches)	
	AC	AB
5 or less	3	4½
6	4	4½
7	4½	5
8	5	5½

Concrete pavement, if used, may consist of 6 inches of Portland Cement Concrete (PCC) over 4 inches of AB. The PCC pavement sections should be provided with crack-control joints spaced no more than 10 feet on-center each way, to control where cracks develop.

All pavement construction should be performed in accordance with the *Standard Specifications for Public Works Construction*. Field inspection and periodic testing, as needed during placement of the base course materials, should be undertaken to ensure that the requirements of the standard specifications are fulfilled. Prior to placement of aggregate base, the subgrade soil should be processed to a minimum depth of 8 inches, moisture-conditioned, as necessary, and recompact to a minimum of 90 percent relative compaction. Localized areas of loose soils may be encountered that require deeper removal and recompaction. The actual extent of the removal depth will be best determined during construction when direct observation of the subgrade soils can be made.

Aggregate base should be moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction.

Aggregate base and asphalt materials should conform to Sections 200-2 and 203, respectively, of the *Standard Specifications for Public Works Construction*. PCC should conform to Section 201 of the *Standard Specifications for Public Works Construction*.

6.9 Surface Drainage

Positive drainage of surface water away from structures is very important. Water should not be allowed to pond adjacent to buildings or new site improvements. Water should not be allowed to flow directly over the slope faces. Positive drainage may be accomplished by providing drainage away at a minimum of 2 percent for earthen surfaces for a lateral distance of at least five feet and further maintained by a swale or drainage path at a gradient of at least 1 percent. Where necessary, drainage paths may be shortened by the use of area drains and collector pipes. Eave gutters are recommended and should

reduce water infiltration into the subgrade materials. Downspouts should be connected to appropriate outlet devices.

Irrigation of landscaping should be controlled to maintain, as much as possible, consistent moisture content sufficient to provide healthy plant growth without over watering. Where improvements are located within valley bottoms, use of diversion structures should be considered to control anticipated flood waters. Sub-slab drainage galleries

6.10 Additional Geotechnical Services

Geotechnical observation and testing should be provided during the following activities:

- Upon completion of site clearing, where applicable;
- During site earthwork;
- Compaction of all fill materials;
- During installation of temporary shoring, wherever needed;
- After foundation excavations and prior to placement of concrete;
- Foundation pile and retaining wall construction;
- Utility trench backfilling and compaction; and
- When any unusual conditions are encountered.

6.11 Plan Review

Final project grading and foundation plans should be reviewed by Leighton as part of the design development process to ensure that recommendations in this report are incorporated in project plans.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 Rippability and Oversize Materials

Based on the results of our auger borings, existing surface soils are expected to be readily rippable using conventional equipment in good working condition. The rippability of weathered near-surface bedrock is expected to be difficult but feasible using heavy equipment, such as dozer with single-shank ripper tooth. Oversized material may be generated from bedrock removals.,

7.2 Trench Backfill

Utility trenches should be backfilled with compacted fill in accordance with Sections 306-1.2 and 306-1.3 of the Standard *Specifications for Public Works Construction*, (SSPWC, “Greenbook”), current edition. Utility trenches can be backfilled with onsite material free of rubble, debris, organic and oversized material up to 3 inches in largest dimension. Prior to backfilling trenches, pipes should be bedded in and covered with either:

- (1) **Granular Bedding:** a) ½-inch open grade aggregate; or b) a uniform sand material with a Sand Equivalent (SE) greater-than-or-equal-to ($\geq$) 30, passing the No. 4 U.S. Standard Sieve (or as specified by the pipe manufacturer).
- (2) **CLSM:** Controlled Low Strength Material (CLSM) conforming to Section 201-6 of the SSPWC. CLSM bedding should be placed to 1 foot (0.3 m) over the top of the conduit, and vibrated.

Pipe bedding should extend at least 4 inches below the pipeline invert and at least 12 inches over the top of the pipeline. The bedding and shading sand is recommended to be densified in place by vibratory, lightweight compaction equipment.

Trench backfill over the pipe bedding zone may consist of native and clean fill soils. All backfill should be placed in thin lifts (appropriate for the type of compaction equipment), moisture conditioned above optimum, and mechanically compacted to at least 90 percent relative compaction, relative to the ASTM D 1557 laboratory maximum density.

7.3 Recreational Ponds

We anticipate recreation ponds will be constructed using down-gradient gravity dam structures consisting of reinforced concrete founded on bedrock. The structures should be designed by a specialty engineer experienced in dam design, and incorporate

elements to facilitate pass-through seepage conditions. The need for collection of future rock-rippability and other geotechnical studies can be precluded to aid in dam design.

7.4 Foundation Setback

We recommend a minimum horizontal setback distance from the face of slopes for all foundations, footings, and other settlement-sensitive structures as indicated on Table 8 below. The minimum recommended setback distance from the face of retaining wall is equal to 1.5 times the height of the retaining wall. This distance is measured from the outside bottom edge of the footing, horizontally to the slope or retaining wall face, and is based on the slope or wall height. However, the foundation setback distance may be revised by the geotechnical consultant on a case-by-case basis if the geotechnical conditions are different than anticipated.

Table 8. – Minimum Foundation Setback from Slope Faces

Slope Height	Setback
less than 5 feet	5 feet
5 to 15 feet	7 feet

Potential distress to improvements (retaining walls, fences, etc.) may be mitigated by use of deepened footings or grade beam systems for support.

In addition, open or backfilled utility trenches that parallel or nearly parallel structure footings should not encroach within an imaginary 2:1 (horizontal to vertical) downward sloping line starting 9 inches above the bottom edge of the footing and should also not be located closer than 18 inches from the face of the footing. Deepened footings should meet the setbacks as described above. Also, over-excavation should be accomplished such that deepening of footings to accomplish the setback will not introduce a cut/fill transition bearing condition.

Where pipes cross under footings, the footings should be specially designed. Pipe sleeves should be provided where pipes cross through footings or footing walls and sleeve clearances should provide for possible footing settlement, but not less than 1 inch around the pipe.

7.5 Temporary Excavations and Shoring Design

All temporary excavations should be performed in accordance with project plans, specifications, and all OSHA requirements. OSHA allows the sides of unbraced excavations, up to a maximum height of 20 feet, to be cut to a $\frac{3}{4}H:1V$ (horizontal:vertical)

slope for Type A soils, 1H:1V for Type B soils, and 1½H:1V for Type C soils. The soil type should be verified or revised based on geotechnical observation and testing during construction, as soil classifications may vary over short horizontal distances. Excavations 5 feet or deeper should be laid back or shored in accordance with OSHA requirements before personnel are allowed to enter.

Typical cantilever shoring should be designed using an active earth pressure presented in Table 3. If excavations are braced at the top and at specific design intervals, the active pressure may then be approximated by a rectangular soil pressure distribution with the pressure per foot of width equal to $24H$, where H is equal to the depth of the excavation being shored. These lateral earth pressures are for a drained condition. For an undrained condition, hydrostatic pressure should be included.

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

7.6 Rockfalls

Precariously perched bedrock boulders subject to potential dislodging and downslope movement as rock falls, exist locally on the site. Removal of the boulders can be accomplished using a variety of hand tools, or equipment such as excavators or dozers., is We recommended the removals of boulders be performed in conjunction with grading activities, and prior to construction of improvements located downslope of the hazard. A specialized contractor may be necessary to perform these activities.

If removal of the rock is not feasible due to access limitations or other reasons, the hazard may be mitigated by breaking up the rock in place to reduce the potential for dislodgement.

In some instances, boulders with a large flat side may be flipped onto the flat surface where they no longer pose a rolling potential and can then be left in place per the recommendations of a State Certified Engineering Geologist.

Possible ground disturbance resulting from these operations would likely consist of trampled brush and creation of rubble piles.

In areas where no mitigation is feasible, it is possible that barrier trenches, deflection walls or earthen berms may be constructed.

8.0 LIMITATIONS

Leighton's work was performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, express or implied, is made as to the conclusions and professional opinions included in this report.

As in many projects, conditions revealed in excavations may be at variance with preliminary findings. If this occurs, the changed conditions must be evaluated by the geotechnical consultant and additional recommendations be developed as needed.

The identification and testing of hazardous, toxic or contaminated materials was outside the scope of Leighton's work. Should such materials be encountered at any time, or their existence be suspected, all measures stipulated in local, County, State and Federal regulations, as applicable, should be implemented.

This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the necessary design consultants for the project and incorporated into the plans; and that the necessary steps are taken to see that the contractors carry out such recommendations in the field.

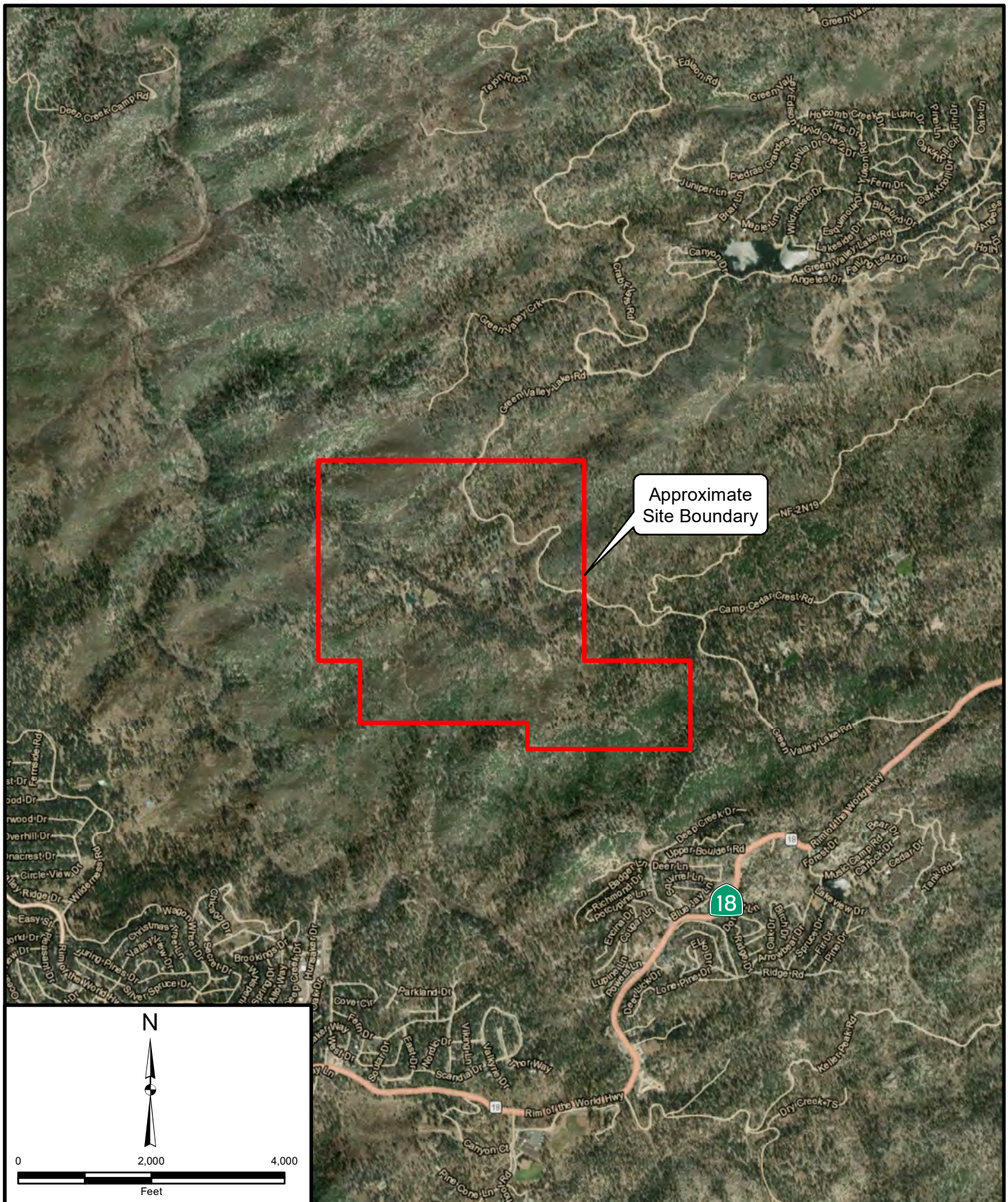
Changes in the conditions of a property can occur with the passage of time, whether due to natural processes or the work of man on the subject or adjacent properties. In addition, changes in standards of practice may occur from legislation or the broadening of knowledge. Accordingly, the findings of this report may at some future time be invalidated wholly or partially by changes outside Leighton's control.

The conclusions and recommendations in this report are based in part upon data that were obtained from a necessarily limited number of observations, site visits, excavations, samples and tests. Such information can be obtained only with respect to the specific locations explored, and therefore may not completely define all subsurface conditions throughout the site. The nature of many sites is that differing geotechnical or geological conditions can occur within small distances and under varying climatic conditions. Furthermore, changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions and recommendations presented in this report can be relied upon only if Leighton has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to verify that our preliminary findings are representative of the site.

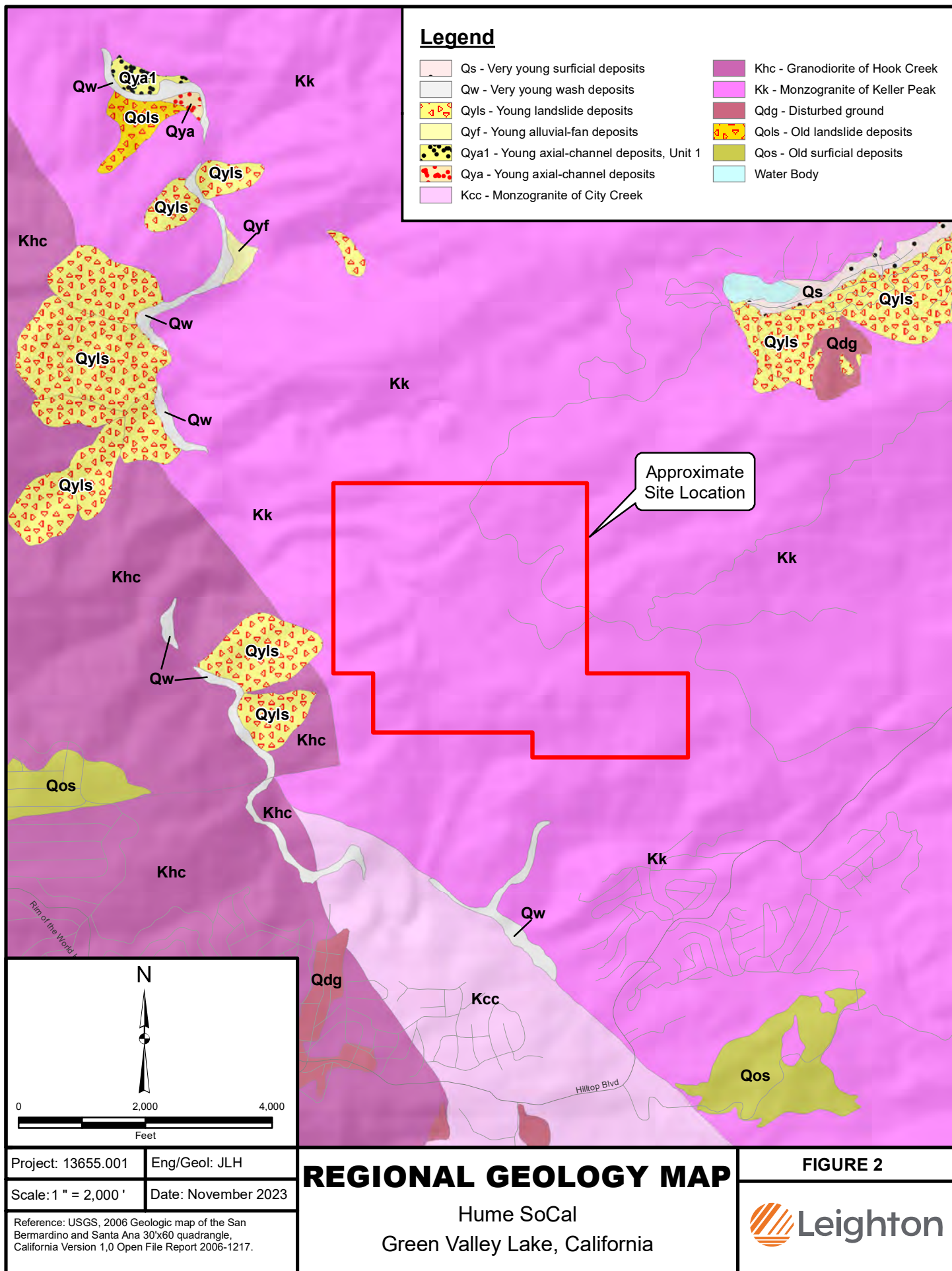
This report is intended only for the use of Kimley-Horn and Associates, Inc., and its design consultants, and only as related expressly to evaluation of the feasibility of developing the subject site with the proposed development and for preliminary planning and design.

If parties other than Leighton are engaged to provide construction geotechnical services, they must be notified that they will be required to assume complete responsibility for the geotechnical phase of the project by concurring with the findings and recommendations in this report or by providing alternative recommendations.

Any persons using this report for bidding or construction purposes should perform such independent explorations as they deem necessary to satisfy themselves as to the surface and subsurface conditions to be encountered and the procedures to be used in the performance of work on the subject site.



Project: 13655.001	Eng/Geol: JLH	SITE LOCATION MAP Hume SoCal Green Valley Lake, California	FIGURE 1
Scale: 1 " = 2,000 '	Date: November 2023		
Base Map: ESRI ArcGIS Online 2023			
Author: (mmurphy)			



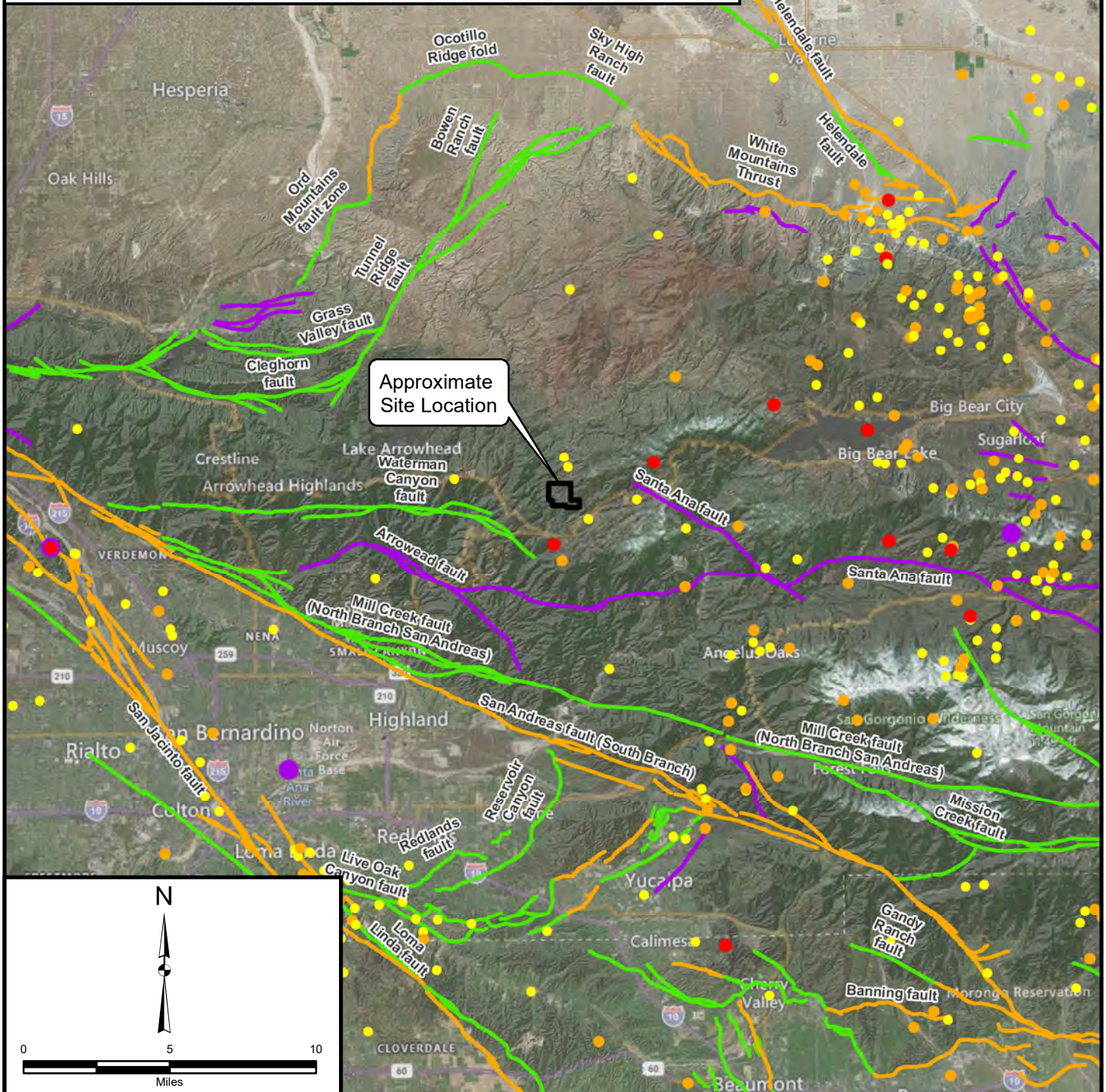
Legend

Fault activity

- Holocene (<11,700 years)
- Late Quaternary (last 700,000 years)
- Quaternary (<1.6M years)

Historical Earthquakes (≥M3.5)

- 3.5 - 3.99
- 4.0 - 4.99
- 5.0 - 5.99
- 6.0 - 6.99



Project: 13655.001

Eng/Geol: JLH

Scale: 1" = 5 miles

Date: November 2023

Base Map: ESRI ArcGIS Online 2023

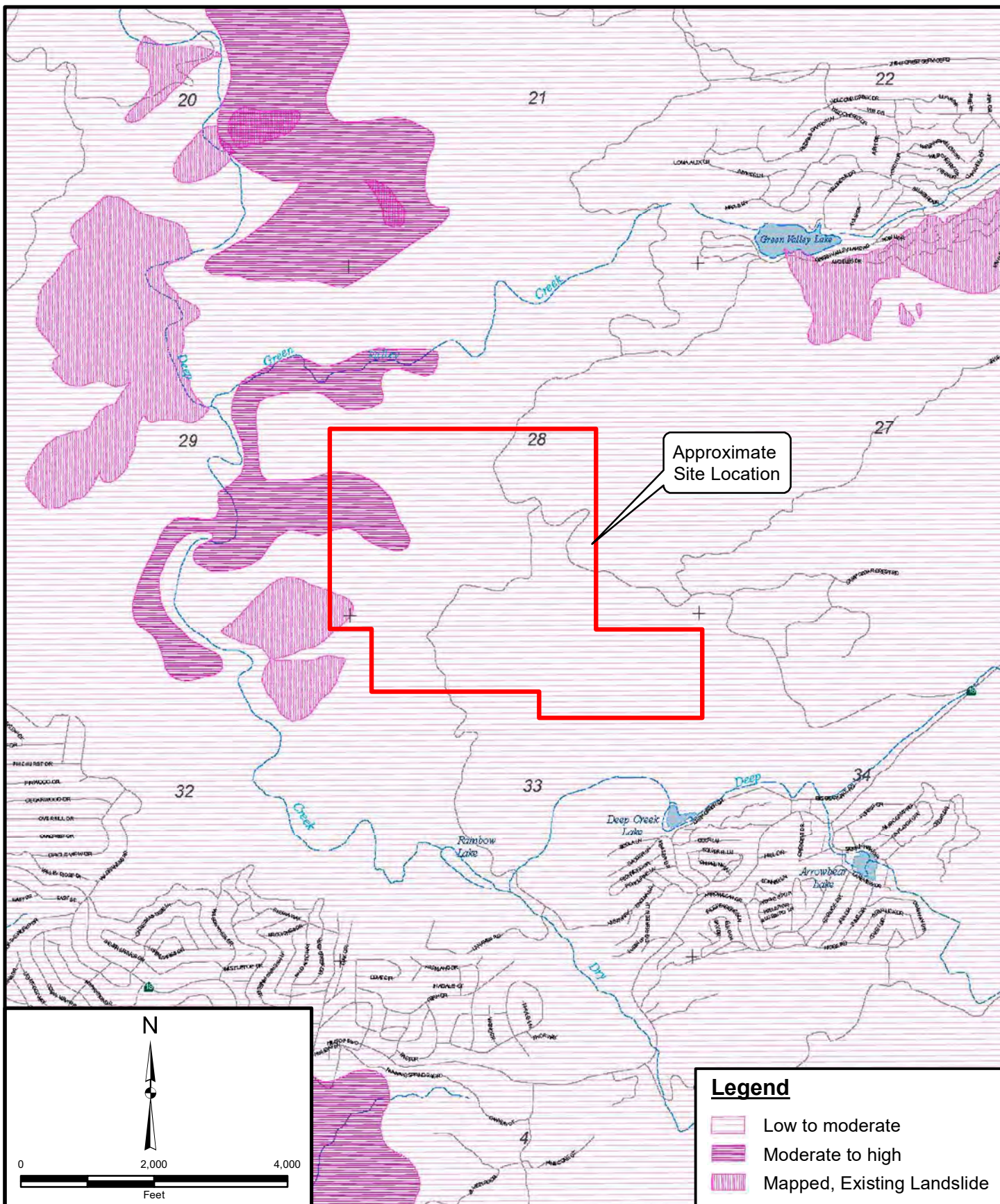
Reference: maps.conservation.ca.gov

REGIONAL FAULTS AND HISTORIC SEISMICITY MAP

Hume SoCal
Green Valley Lake, California

FIGURE 3





Legend

- Low to moderate
- Moderate to high
- Mapped, Existing Landslide

Project: 13655.001

Eng/Geol: JLH

Scale: 1" = 2,000'

Date: November 2023

Base Map: San Bernardino County Land Use Plan GENERAL PLAN
Geologic Hazard Overlays Map data originally compiled on 1:48,000
scale mosaicked maps photo-reduced from 7-1/2 minute USGS
quads by D.M. Morton, U.S. Geological Survey, 1974 and by S.S.
Tan, State Division of Mines
and Geology, 1989.

SEISMIC HAZARD MAP

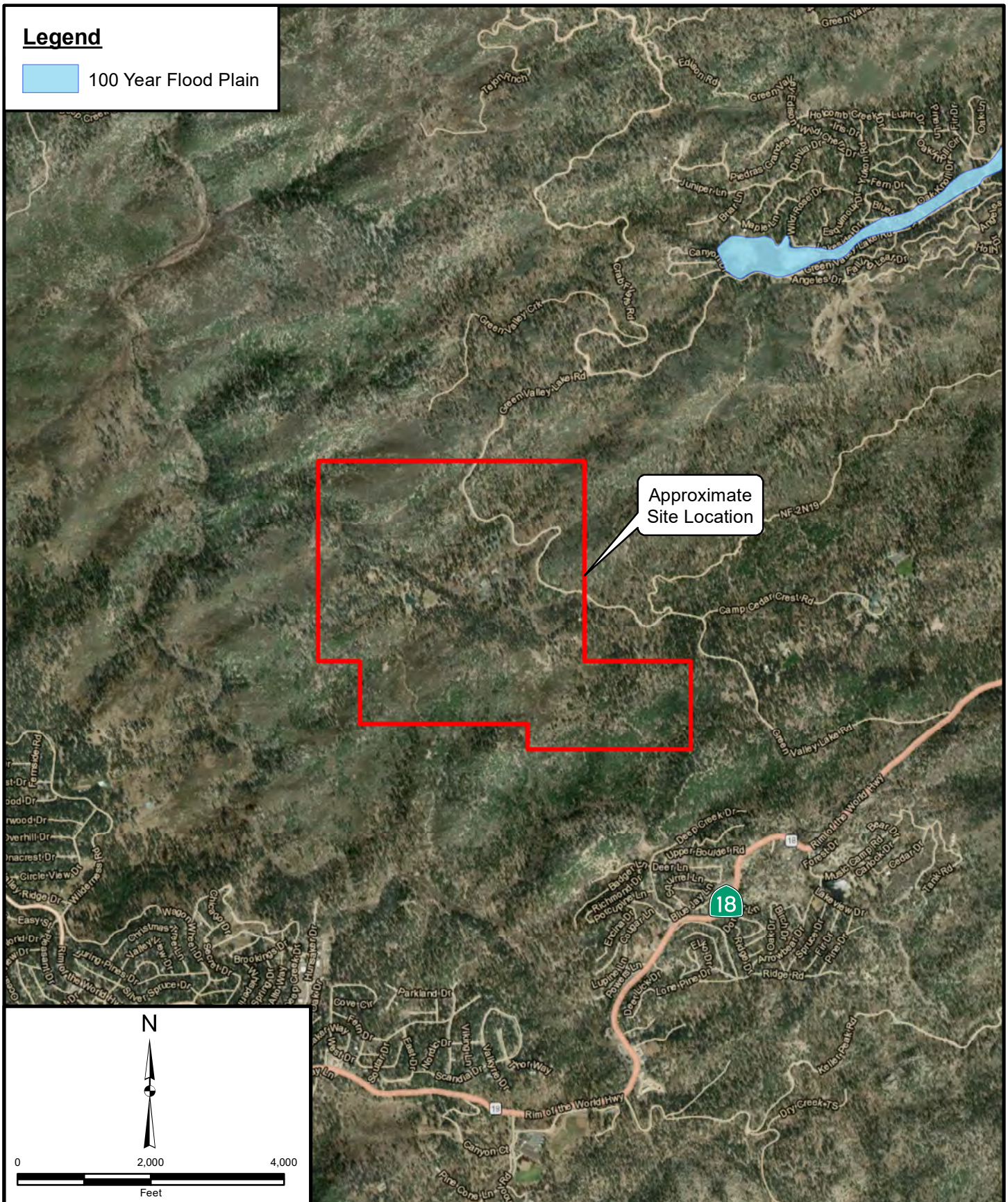
Hume SoCal
Green Valley Lake, California

FIGURE 4



Legend

 100 Year Flood Plain



Approximate
Site Location

18

N

0 2,000 4,000
Feet

Project: 13655.001

Eng/Geol: JLH

Scale: 1" = 2,000'

Date: November 2023

Base Map: ESRI ArcGIS Online 2023

Reference: FEMA, DWR

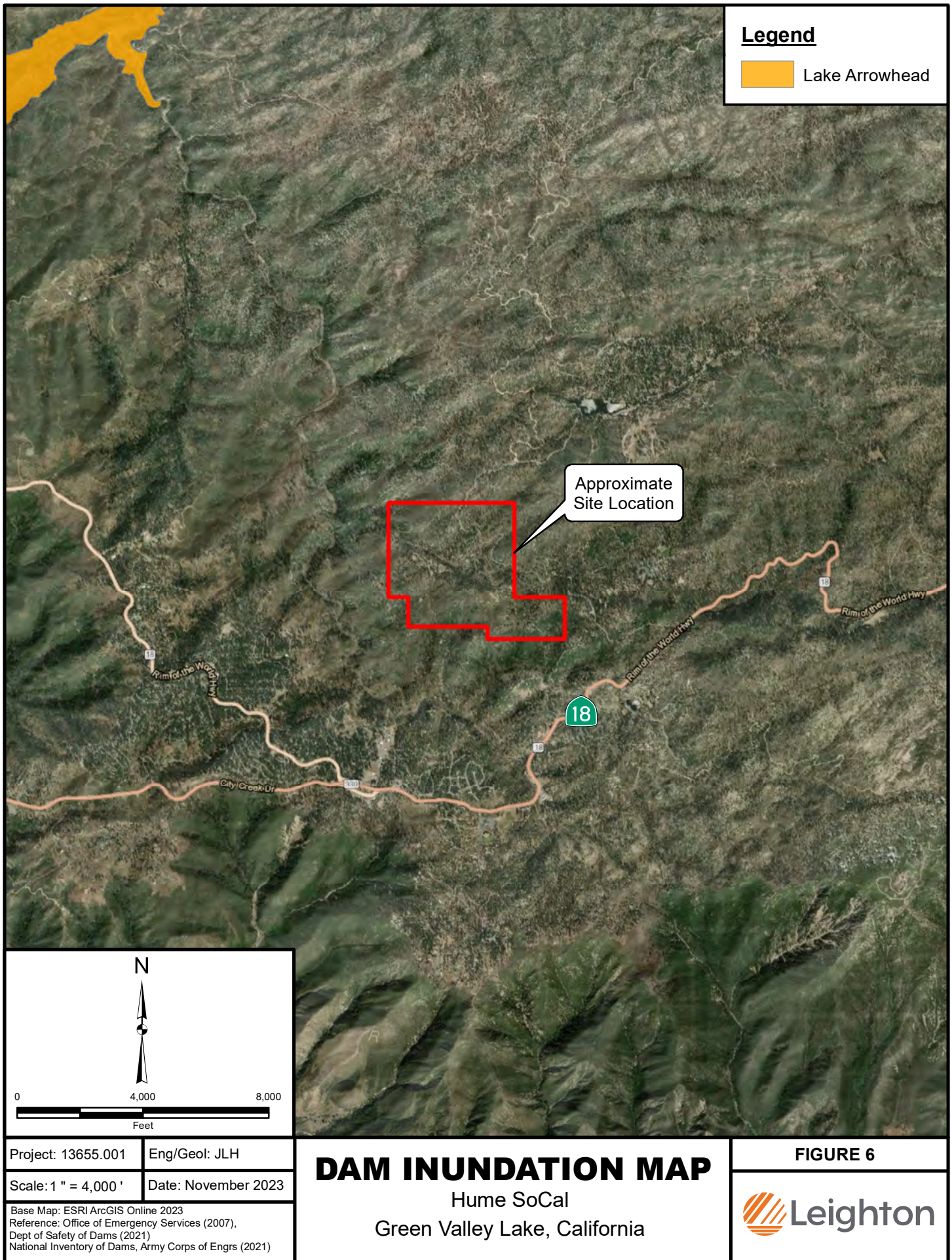
Author: (mmurphy)

FLOOD HAZARD ZONE MAP

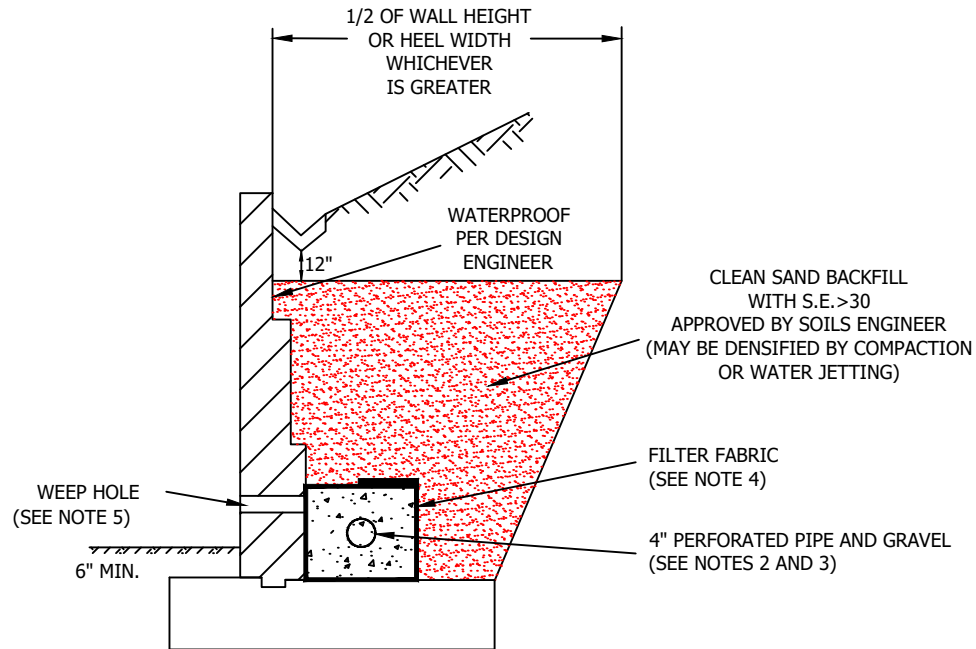
Hume SoCal
Green Valley Lake, California

FIGURE 5





SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF >50



NOTE: AS AN ALTERNATE TO CLEAN SAND BACKFILL, CLEAN GRAVEL MAY BE UTILIZED WITH APPROVED FILTER FABRIC. A SECOND ALTERNATE IS TO UTILIZE AN AGGREGATE BASE MATERIAL COMPACTED TO 90% RELATIVE COMPACTION. A SAMPLE OF THE PROPOSED BASE MUST BE APPROVED BY THE GEOTECHNICAL CONSULTANT PRIOR TO BACKFILL FOR SUITABILITY. COMPACTION SHOULD BE ACHIEVED WITHOUT DAMAGING THE WALL.

GENERAL NOTES:

- * Waterproofing should be provided where moisture nuisance problem through the wall is undesirable.
- * Water proofing of the walls is not under purview of the geotechnical engineer
- * All drains should have a gradient of 1 percent minimum
- * Outlet portion of the subdrain should have a 4-inch diameter solid pipe discharged into a suitable disposal area designed by the project engineer. The subdrain pipe should be accessible for maintenance (rodding)
- * Other subdrain backfill options are subject to the review by the geotechnical engineer and modification of design parameters.

Notes:

- 1) Sand should have a sand equivalent of 30 or greater and may be densified by water jetting.
- 2) 1 Cu. ft. per ft. of 1/4- to 1 1/2-inch size gravel wrapped in filter fabric
- 3) Pipe type should be ASTM D1527 Acrylonitrile Butadiene Styrene (ABS) SDR35 or ASTM D1785 Polyvinyl Chloride plastic (PVC), Schedule 40, Armco A2000 PVC, or approved equivalent. Pipe should be installed with perforations down. Perforations should be 3/8 inch in diameter placed at the ends of a 120-degree arc in two rows at 3-inch on center (staggered)
- 4) Filter fabric should be Mirafi 140NC or approved equivalent.
- 5) Weephole should be 3-inch minimum diameter and provided at 10-foot maximum intervals. If exposure is permitted, weepholes should be located 12 inches above finished grade. If exposure is not permitted such as for a wall adjacent to a sidewalk/curb, a pipe under the sidewalk to be discharged through the curb face or equivalent should be provided. For a basement-type wall, a proper subdrain outlet system should be provided.
- 6) Retaining wall plans should be reviewed and approved by the geotechnical engineer.
- 7) Walls over six feet in height are subject to a special review by the geotechnical engineer and modifications to the above requirements.

RETAINING WALL BACKFILL AND SUBDRAIN DETAIL

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF >50

FIGURE 7



WILDWOOD CAMP
130 Capacity

ADULT LODGE
140 Capacity

ELEMENTARY AGE CAMP
250 to 500 Capacity

MIDDLE SCHOOL AGE CAMP
784 Capacity

HIGH SCHOOL AGE CAMP
1000 Capacity

MAINTENANCE FACILITY
5700 Sq. Ft.

LEGEND

HA-55

Approximate location auger boring

Z

Z'

Geologic Cross Section

N

0

200

400

Feet

GEOTECHNICAL MAP

Hume SoCal

Green Valley Lake, California

Leighton

Base Map: Preliminary Plan, January 9, 2023

PLATE 1

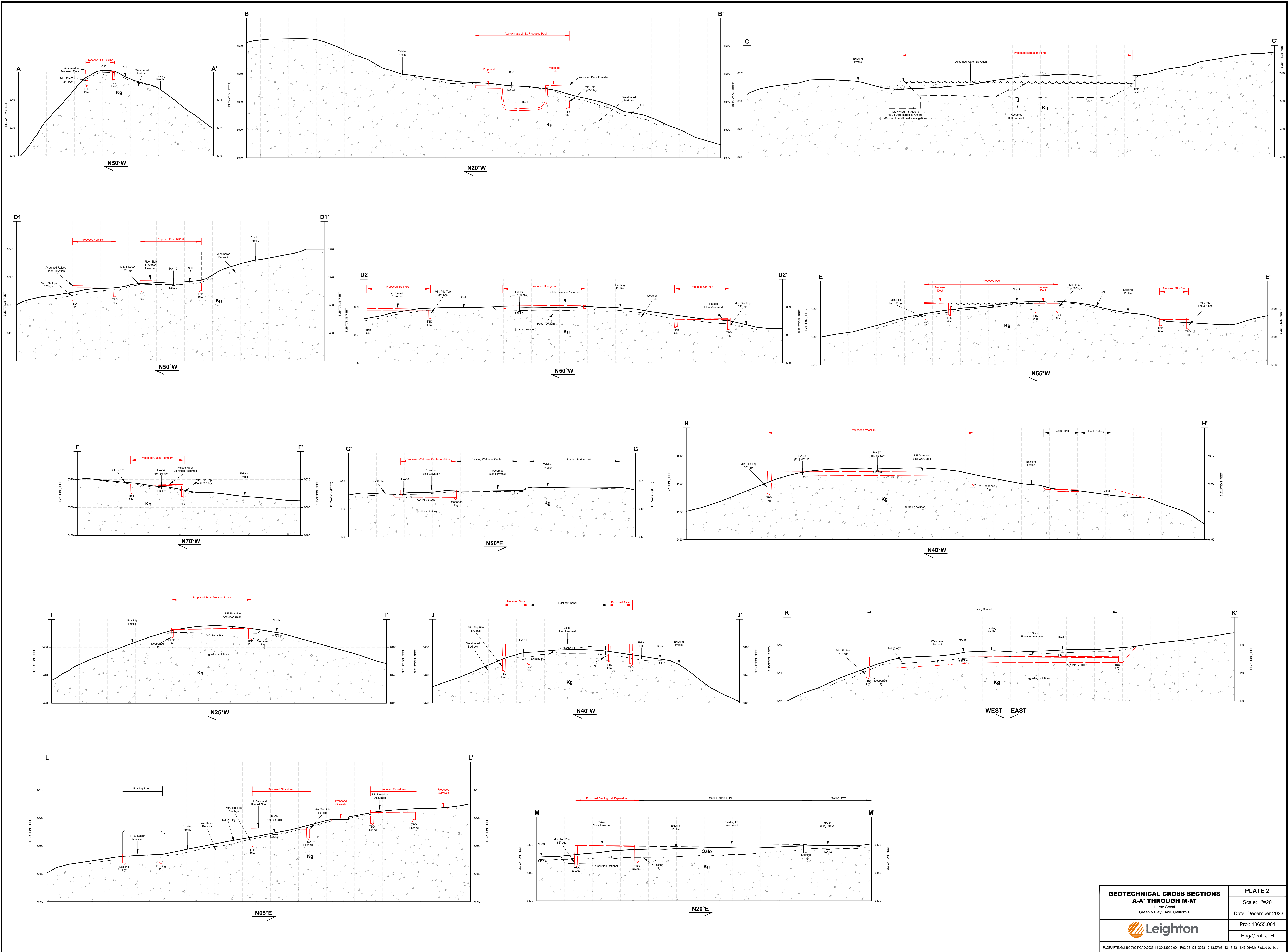
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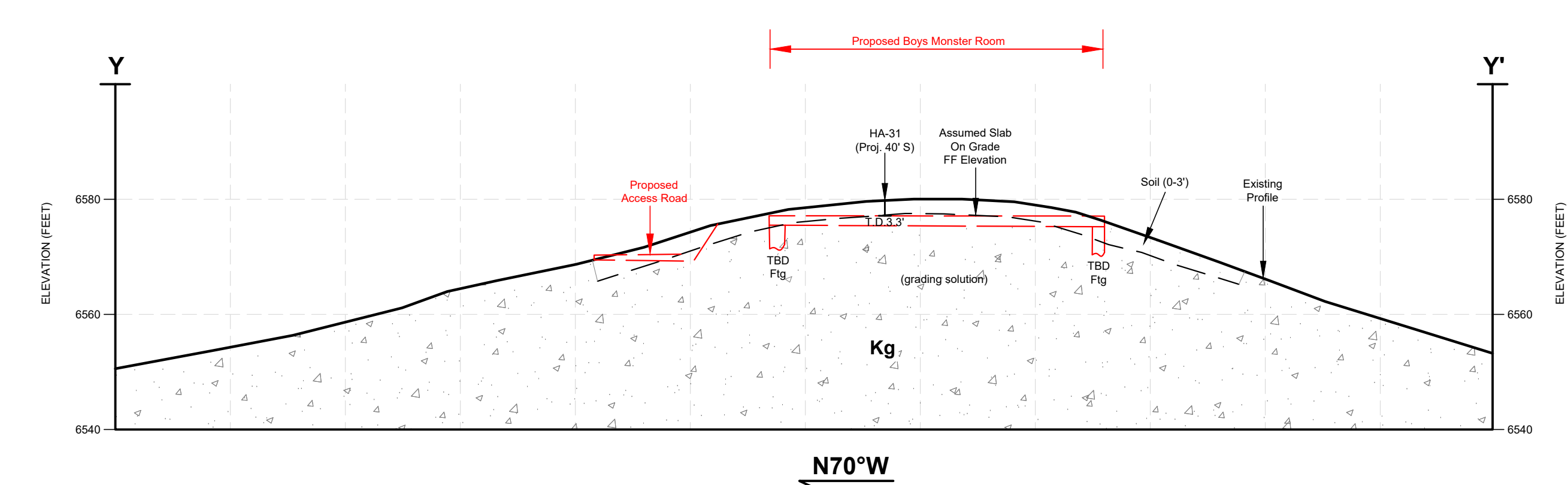
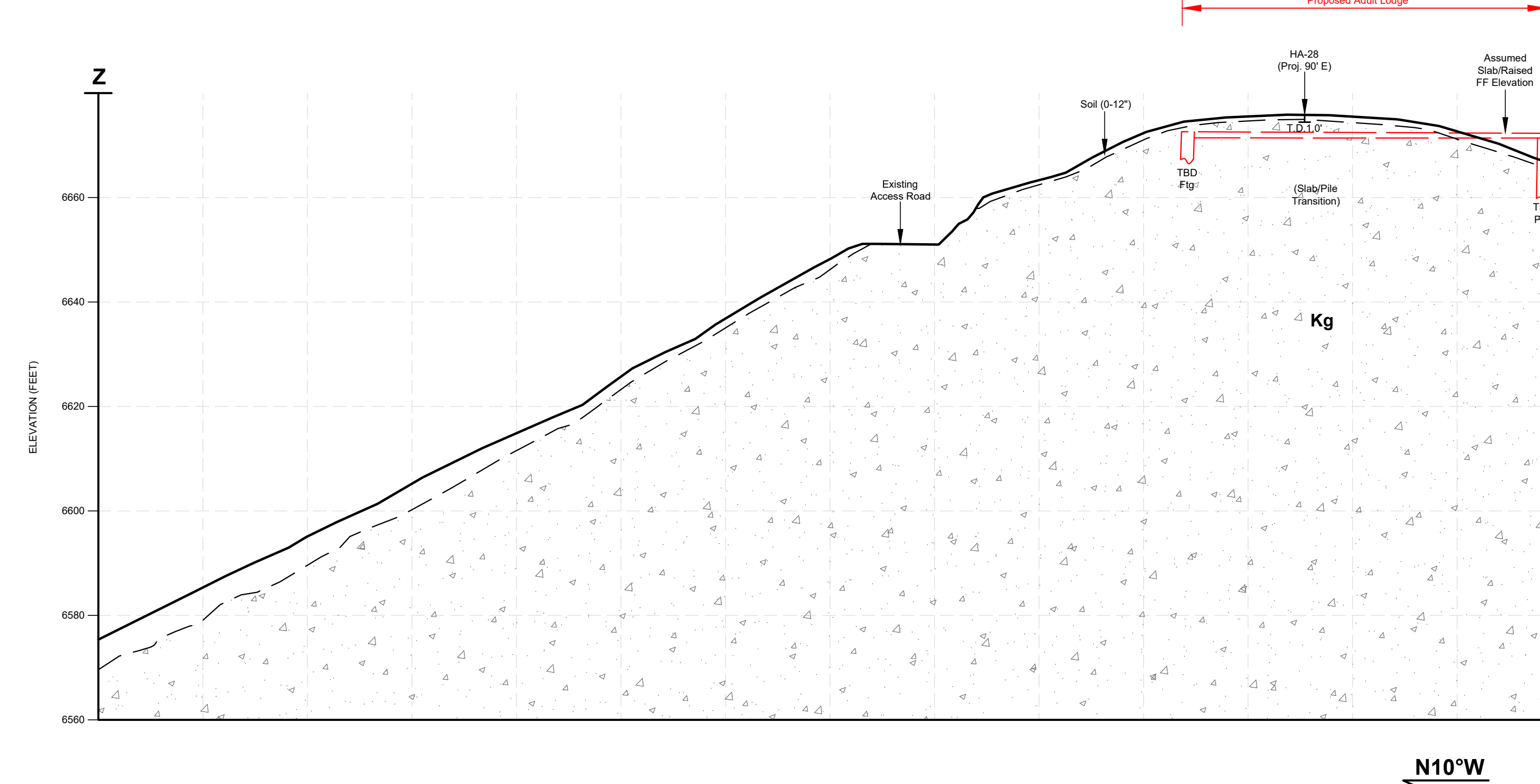
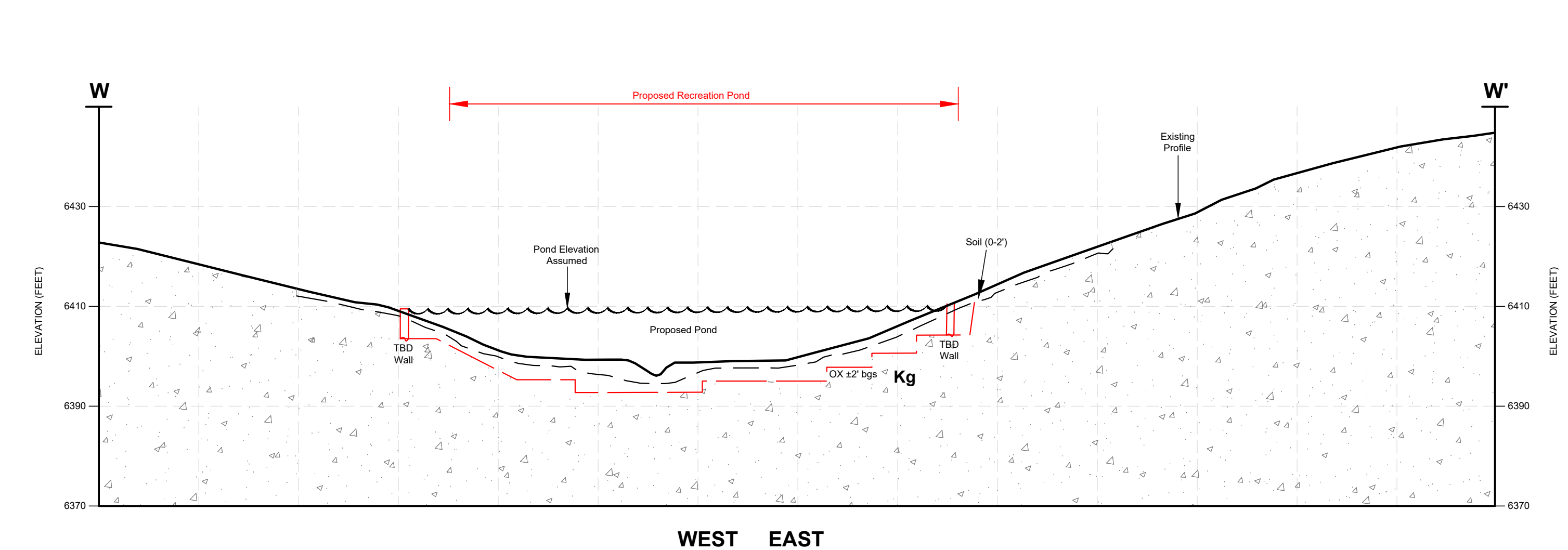
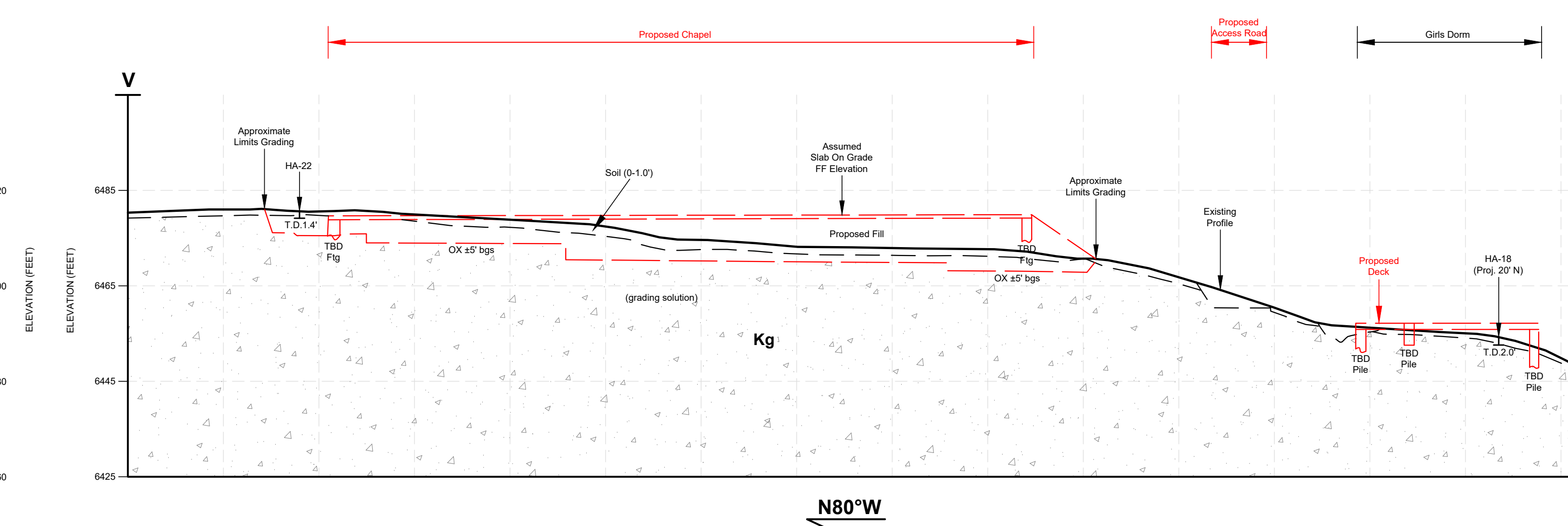
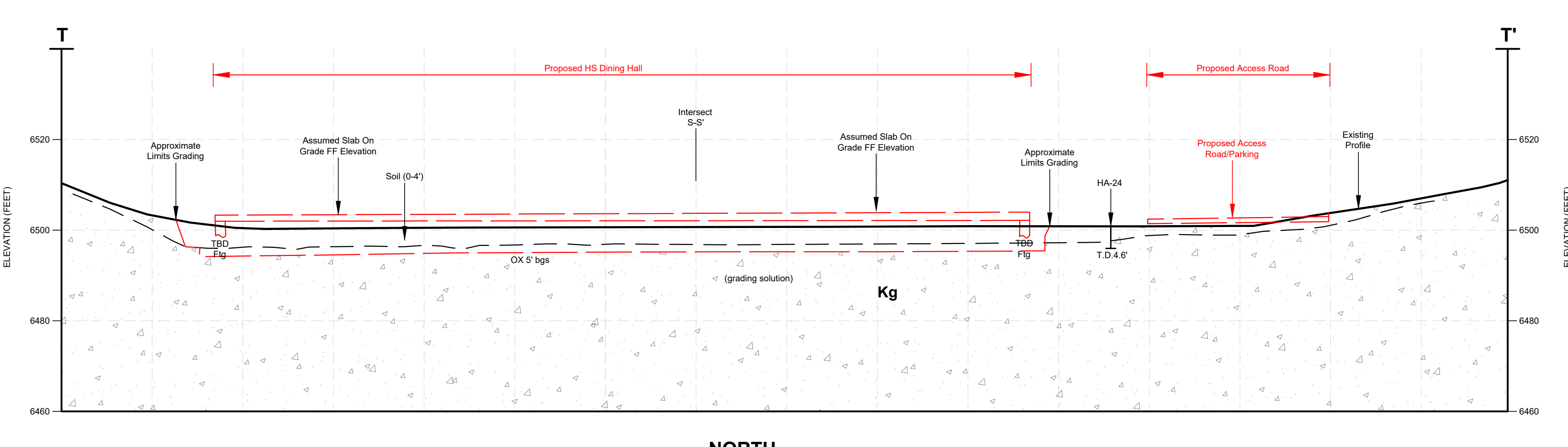
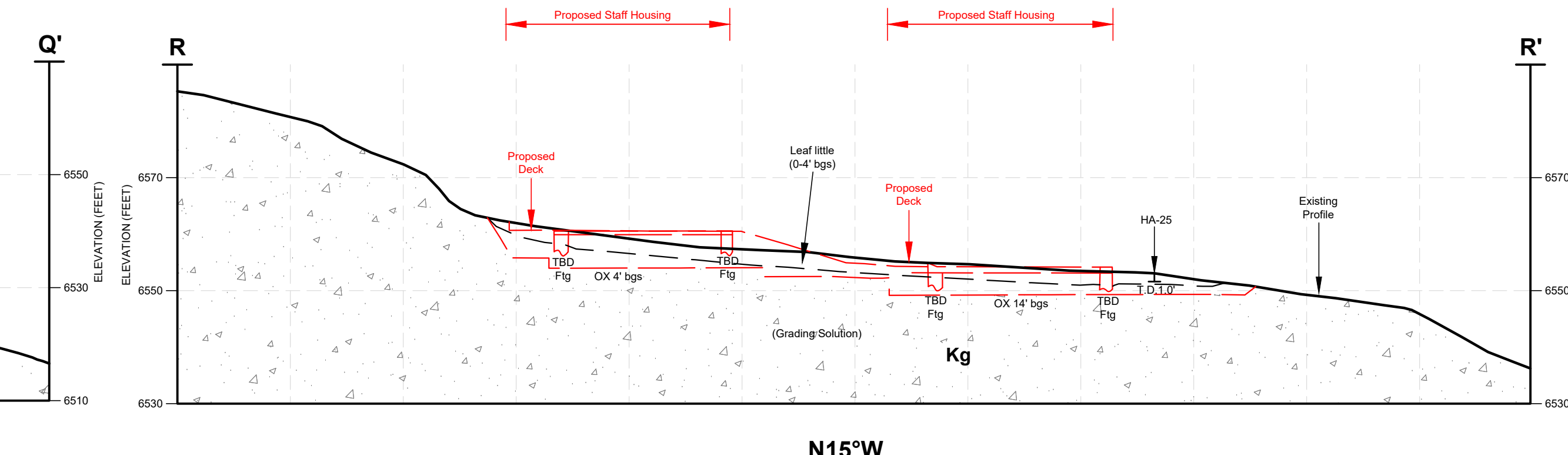
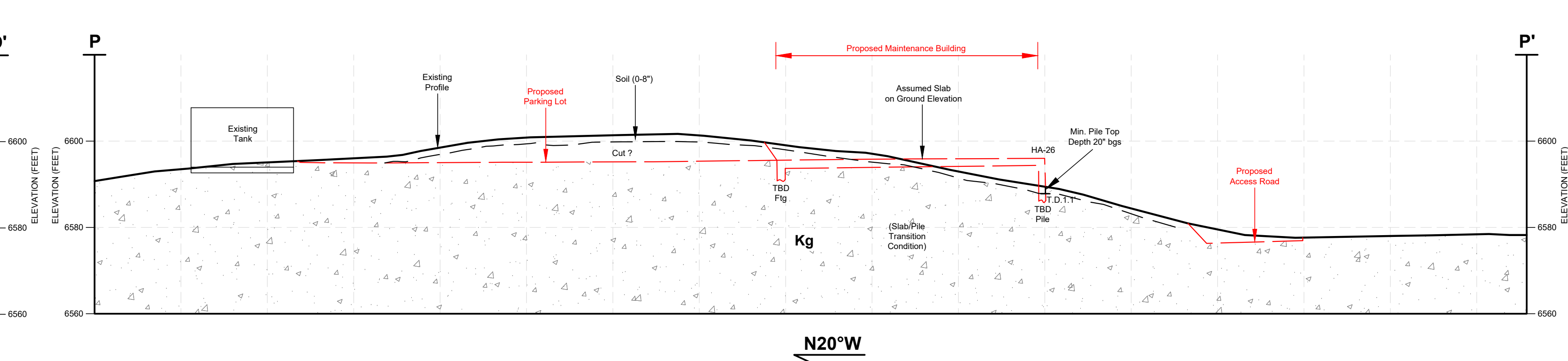
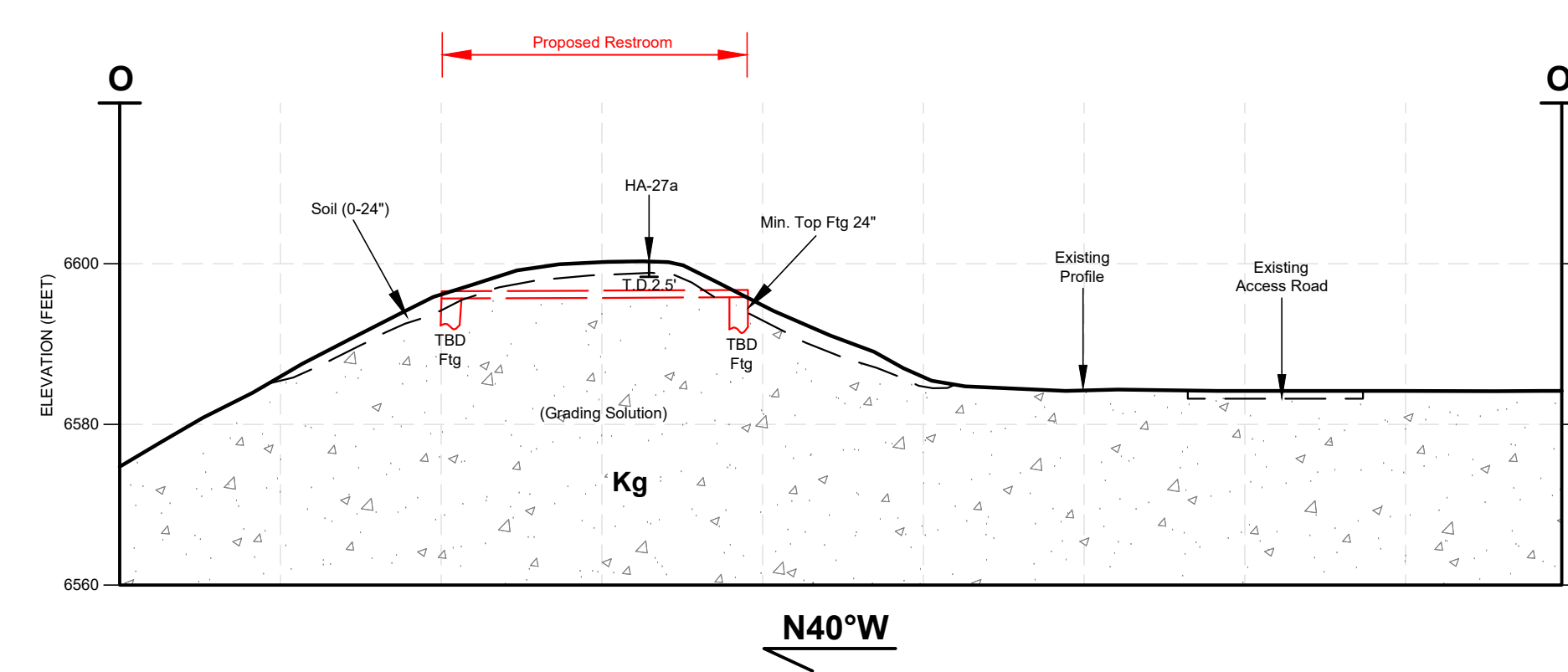
Date: December 2023

Proj: 13655.001

Eng/Geol: JLH

Map Saved as V:\Drafting\13655001\Map\13655-001_P01_GM_2023-12-13.mxd on 12/13/2023 11:56:21 AM





APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

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APPENDIX B1

AUGER BORING SUMMARY

Boring No.		TD (inches)	Soil (inches)	Weathered Bedrock	Harder Bedrock (inches)	Contingent Harder Bedrock (+6 inches) Top Pile Embedment	Location	Condition	Sample	Reference Cross Section	Prefered Structrual Support Solution
WILDWOOD CAMP	HA-1	16	0-10	10-16	16	22	Girls Dorm	Ridgeline Nose TOS			
	HA-2	18	0-16	16-18	18	24	Girls Restroom/Showers	Ridgeline TOS		A-A'	Piles
	HA-3	20	0-20	20-24	24	30	G. Dining Canopy	Ridgeline TOS to SE Facing 3:1	MB-1		
	HA-4	12	0-6	6-12	12	18	Girls Dorm	Ridgeline TOS to SE Facing 3:1			
	HA-5a/b	12	0-8	8-12	12	18	Access Road	Ridgeline TOS	MB-2/3		
	HA-6	6	0-6	6-8	8	14	Pool	Mid-Slope Terrace		B-B'	piles & grading
	HA-7	24	0-12	12-24	24	30	Boys Restroom/Showers	Mid-Slope Terrace			
	HA-8	28	0-20	20-28	28	34	Boys Dorm	S. Facing Slope 3:1			
	HA-9	6	6	0-6	6	12	Access Road	Existing Road level	MB-4a/b/c		
							Dining Hall			D2-D2'	grading
ELEMENTARY CAMP							Staff Restroom			D2-D2'	piles
							Girls Yort			D2-D2'	piles
	HA-10	28	0-24	24-28	28	34	Boys Restroom/Showers		MB-5	D1-D1'	piles & grading
	HA-11	20	0-16	16-20	20	26	Theater	2:1 west facing slope			
	HA-12	12	0-12		12	18	Pond	crk axis			
	HA-13	16	0-16	72"?	72"?	78?	Pond	n. side pond	refusal		
	HA-14	48	0-40	40-48	48	54	Pond	W. pond			
	HA-15	0-12	0-12	12-24	24	30	Pool	6:1 slope	MB-6	E-E'	grading
HS CAMP	HA-16						Girls Dorm				
	HA-17	54	0-42	54	54	60	Access Road		MB-7a/b		
	HA-18	24	0-24	24-36	36	42	Girls Dorm				
	HA-19								MB-8		
	HA-20	6	0-6	6	6	12	Girls Dorm	gentle slope			
	HA-21	40	0-36	36-40	60	66	Girls Dorm	4:1 slope	MB-9	U-U'	piles & grading
	HA-22	14	0-6	6-14	14	20	Chapel	False knob topo	MB-10	V-V'	grading
	HA-23	8	0-8	8	8	14	Gynmasium	toe of ridge slope		Q-Q'	grading
	HA-24	56	0-48	48-56	60	66	Dining	upper ponded channel lea area	MB-11	S-S'	grading
	HA-25	12	0-12	12	12	18	Staff Housing	Gently slopes	MB-12a/b	R-R'	grading
	HA-26	13	0-8	8-13	13	19	Maintenance Facility	4:1 slope	MB-13	P-P'	piles & grading
	HA-27a	30	0-24	24-30	30	36	Bus Pkg Restroom	1:1 to 2:1 slope	MB-14	O-O'	grading
	HA-27b	10	0-6	6-10	10	16	Bus Pkg Lot	channel axis and side slope	MB-15a		
	HA-28	12	0-12	12	12	18	Adult Lodge	nose of ridge 2:1		Z-Z'	piles & grading
	HA-29	36	0-24	24-36	36	42	Adult Lodge Pkg	saddle between peaks	MB-15b		
	HA-30	24	0-12	12-24	24	30	Boys Monster Dorm	axix of dry creek 5:1 to 2:1 slopes	MB-16	X-X'	grading
JH CAMP	HA-31	40	0-36	30-40	40	46	Boys Monster Dorm	amphatheater-like slope topo	MB-17	Y-Y'	grading
	HA-32	24	0-6	6-24	24	30	Boys Monster Dorm	saddle and W. facing 4:1 slope	MB-19		
	HA-33	52	0-42	42-48	48-52	58	Director Housing	meadow low-land	MB-20		
	HA-34	18	0-14	14-18	18	24	Director housing	terrace portion			
	HA-35	40	0-20	20-40	40	46	Guest Speaker housing	6:1 slope			
							Guest Restroom			F-F'	piles
	HA-36	12	0-12	12	12	18	Welcome Ctr Expansion	parking lot area		G-G'	grading
	HA-37						Gymnasium	existing maint/equip yard	gentle mesa slope	H-H'	piles & grading
	HA-38	24	0	0-24	24	30	Gymnasium	existing maint/equip yard	MB-21a	H-H'	piles & grading
	HA-39	48	0-48	48	48	56	Gymnasium	existing maint/equip yard	gentle mesa slope		
	HA-40	48	0-46	46-48	48	56	Gymnasium	existing maint/equip yard	top of 2:1 cut slope		
	HA-41	30	0-12	12-30	30	36	Monster Dorm East	nose of elevated bedrock ridge	existing tent cabin		
	HA-42	16	0-3	3-16			Monster Dorm West	nose of elevated bedrock ridge	MB-21b	I-I'	piles & grading
	HA-43	24	0	0-24	24	30	Dorm south	side slope 3:1	MB-22		
	HA-44	46	0-30	30-46	46	52	Chapel	level adja existing bldg	MB-24		
	HA-45	60	0-48	48-60	60	66	Chapel	level adja existing bldg	MB-23	K-K'	piles & grading
	HA-46	48	0-48	48-60	60	66	Chapel	TOP OF SLOPE			
	HA-47	36	0-36	36	36	42	Chapel	level adja existing bldg			
	HA-48	24	0-18	18-24	24	30	Chapel	sloped area perimeter			
	HA-49	24	0-3	3-24	24	30	Girls Dorm	sloped area w. 3:1	MB-25		
	HA-50	12	0-3	3-12	12	18	Girls Dorm	sloped area w. 3:1	MB-26	L-L'	piles
	HA-51	52	0-52	52	56	62	NW Chapel Deck	flat mesa	MB-27		
	HA-52a	12	0-12	12	12	18	NE Chapel Deck	flat mesa		J-J'	piles
	HA-52d	6	0	0-6	6	12	Adult Lodge Access Rd	flat mesa	MB-28		
	HA-53	18	0-8	8-18	18	24	Staff Housing	4:1 slope	MB-29	N-N'	piles
	HA-54	54	0-36	36-54	54	60	NE Dining Hall	2:1 slope	MB-30		
	HA-55	44	0-60	60	60	66	Staff Dining Hall	sloped terrace	MB-31	M-M'	grading

APPENDIX B2

FIELD EXPLORATION

Our exploration included a reconnaissance of accessible site areas to evaluate access and observe general geologic conditions and potential hazards. Once a conceptual site plan became available, our staff advanced a total of 55 small diameter borings were advanced using manual hand auger equipment and breaker bar efforts. The borings were cited in areas of planned improvements, and are generally representative of surrounding conditions. The borings were drilled until refusal in bedrock was encountered, typically less than 5 feet below the existing ground surface. Encountered soils were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). A description of soil properties encountered in each boring and other related information are documented in a spreadsheet attached as Appendix B, *Auger Boring Log Summary*. Boring locations are plotted on attached Plate 1, *Geotechnical Map*.

Bulk samples of native soil and/or artificial fill were collected from the borings. In addition to representative soils, samples of bedrock were also collected for physical strength testing. Soil samples were transported to our Irvine geotechnical laboratory for geotechnical testing. After sampling and logging, all borings were immediately backfilled with excavated soils.

Boundaries (depths) between soil or geologic units noted on logs are based on field observations and results of geotechnical laboratory tests. The resultant logs were incorporated into a series of 27 cross sections. The sections depict the existing surface profile, distribution of underlying soil and bedrock, the footprint of proposed buildings with assumed floor elevations, and conceptualized layout of foundation types suitable for the prevailing conditions.

It is possible that passage of time could alter conditions due to environmental changes, fluctuations in groundwater, or occurrence of a significant geological event such as an earthquake.

APPENDIX C

LABORATORY TEST DATA



**TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS**

Project Name: Hume SoCal Camp Tested By : GEB/GB Date: 08/16/23
Project No. : 13655.001 Checked By: J. Ward Date: 10/19/23

Boring No.	HA-5a	HA-15	HA-19	HA-27
Sample No.	MB-2a & b	MB-6	MB-8	MB-14
Sample Depth (ft)	0-12"	12-24"	12-16"	24-30"
Soil Identification:	Brownish gray SM, organics	Olive yellow SP	Olive yellow SP, organics	Olive yellow SP
Wet Weight of Soil + Container (g)	131.27	206.97	207.30	202.63
Dry Weight of Soil + Container (g)	129.32	201.06	199.07	198.23
Weight of Container (g)	52.66	57.50	59.21	61.62
Moisture Content (%)	2.54	4.12	5.88	3.22
Weight of Soaked Soil (g)	100.73	100.39	100.15	100.76

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	404	5	8	17
Crucible No.	305	6	9	7
Furnace Temperature (°C)	860	860	860	860
Time In / Time Out	8:00/8:45	8:00/8:45	8:00/8:45	8:00/8:45
Duration of Combustion (min)	45	45	45	45
Wt. of Crucible + Residue (g)	60.8511	25.7529	22.5071	22.7095
Wt. of Crucible (g)	60.8494	25.7463	22.5021	22.7072
Wt. of Residue (g) (A)	0.0017	0.0066	0.0050	0.0023
PPM of Sulfate (A) x 41150	69.95	271.59	205.75	94.64
PPM of Sulfate, Dry Weight Basis	72	283	219	98

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30	15	30	30
ml of AgNO ₃ Soln. Used in Titration (C)	1.6	0.4	0.7	0.6
PPM of Chloride (C -0.2) * 100 * 30 / B	140	40	50	40
PPM of Chloride, Dry Wt. Basis	144	42	53	41

pH TEST, DOT California Test 643

pH Value	5.68	5.33	5.37	5.88
Temperature °C	22.0	22.7	22.7	22.5



SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name: Hume SoCal Camp
Project No. : 13655.001
Boring No.: HA-5a
Sample No. : MB-2a & b

Tested By : G. Berdy Date: 08/22/23
Checked By: J. Ward Date: 10/19/23
Depth (ft.) : 0-12"

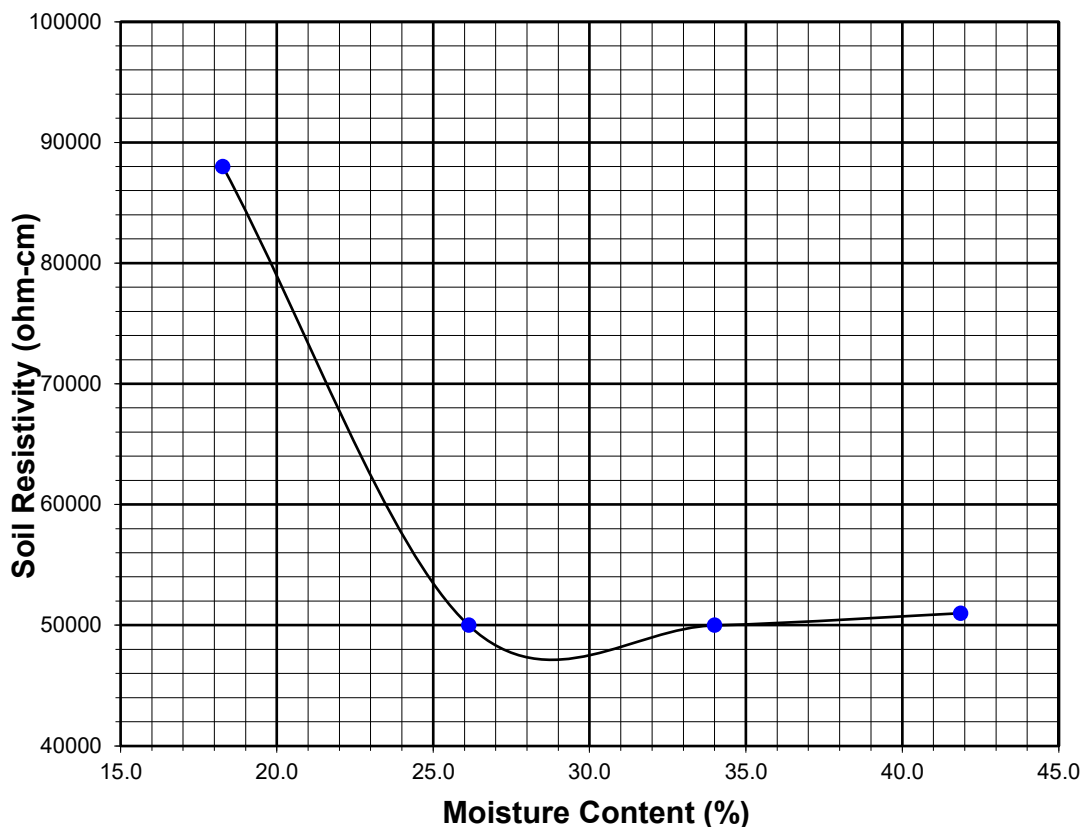
Soil Identification:* Brownish gray SM, organics

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

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	18.27	88000	88000
2	30	26.14	50000	50000
3	40	34.00	50000	50000
4	50	41.86	51000	51000
5				

Moisture Content (%) (Mci)	2.54
Wet Wt. of Soil + Cont. (g)	131.27
Dry Wt. of Soil + Cont. (g)	129.32
Wt. of Container (g)	52.66
Container No.	
Initial Soil Wt. (g) (Wt)	130.40
Box Constant	1.000
$MC = (((1 + Mci / 100) \times (Wa / Wt + 1)) - 1) \times 100$	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
47000	28.8	72	144	5.68	22.0





SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name: Hume SoCal Camp
Project No. : 13655.001
Boring No.: HA-15
Sample No. : MB-6

Tested By : G. Berdy Date: 08/22/23
Checked By: J. Ward Date: 10/19/23
Depth (ft.) : 12-24"

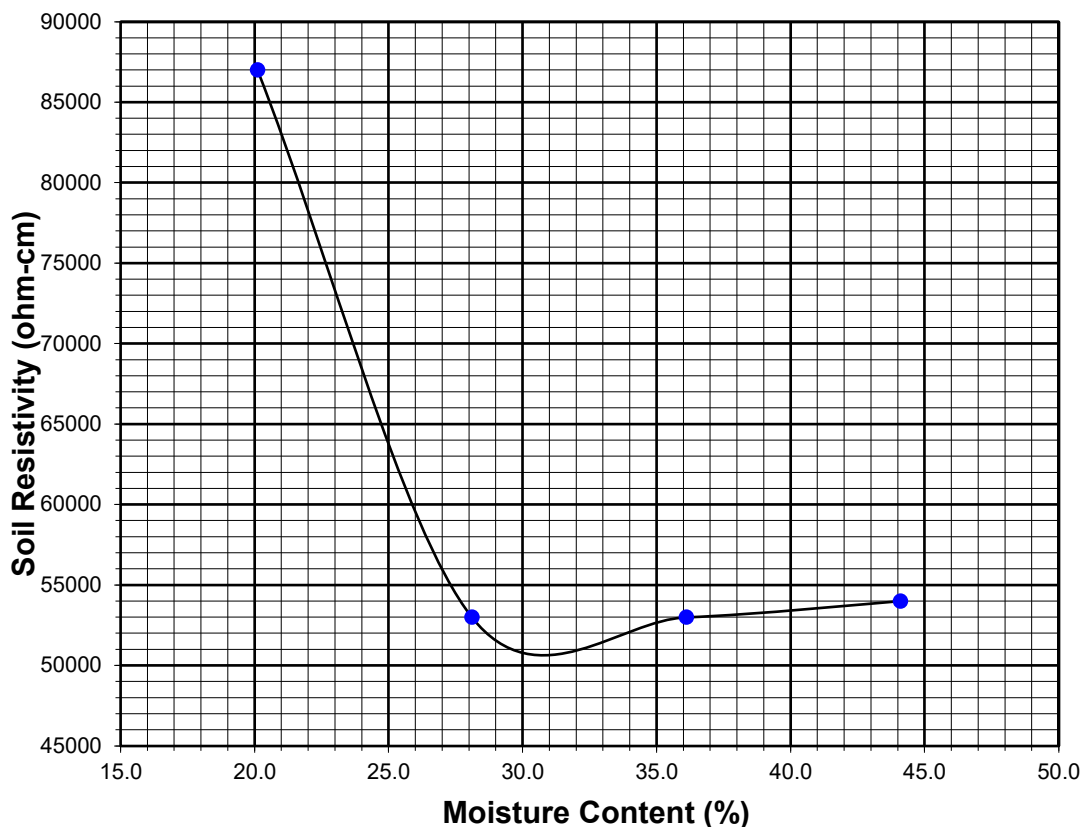
Soil Identification:* Olive yellow SP

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

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	20.11	87000	87000
2	30	28.11	53000	53000
3	40	36.11	53000	53000
4	50	44.11	54000	54000
5				

Moisture Content (%) (Mci)	4.12
Wet Wt. of Soil + Cont. (g)	206.97
Dry Wt. of Soil + Cont. (g)	201.06
Wt. of Container (g)	57.50
Container No.	
Initial Soil Wt. (g) (Wt)	130.18
Box Constant	1.000
$MC = (((1 + Mci / 100) \times (Wa / Wt + 1)) - 1) \times 100$	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
50500	30.7	283	42	5.33	22.7





SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name: Hume SoCal Camp
Project No. : 13655.001
Boring No.: HA-19
Sample No. : MB-8

Tested By : G. Berdy Date: 08/22/23
Checked By: J. Ward Date: 10/19/23
Depth (ft.) : 12-16"

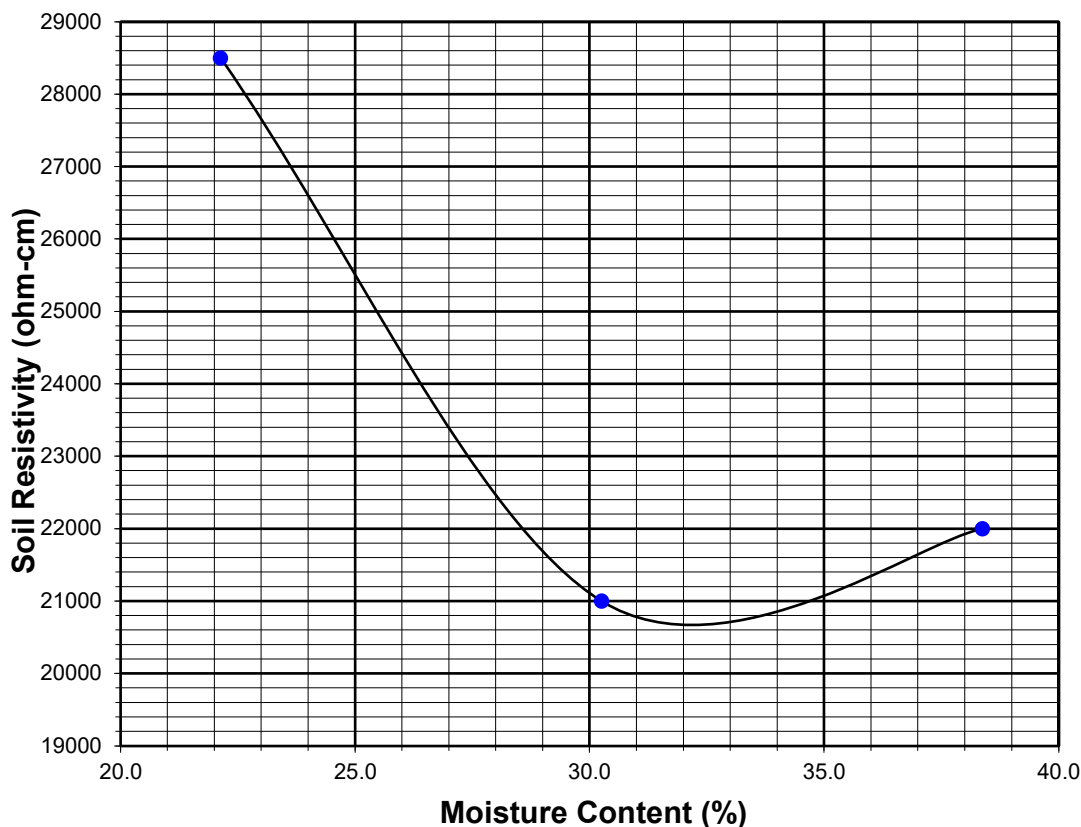
Soil Identification:* Olive yellow SP, organics

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

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	22.13	28500	28500
2	30	30.25	21000	21000
3	40	38.38	22000	22000
4				
5				

Moisture Content (%) (MCi)	5.88
Wet Wt. of Soil + Cont. (g)	207.30
Dry Wt. of Soil + Cont. (g)	199.07
Wt. of Container (g)	59.21
Container No.	
Initial Soil Wt. (g) (Wt)	130.35
Box Constant	1.000
$MC = (((1 + MC_i / 100) \times (W_a / W_t + 1)) - 1) \times 100$	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
20600	32.1	219	53	5.37	22.7





SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name: Hume SoCal Camp
Project No. : 13655.001
Boring No.: HA-27
Sample No. : MB-14

Tested By : G. Berdy Date: 08/22/23
Checked By: J. Ward Date: 10/19/23
Depth (ft.) : 24-30"

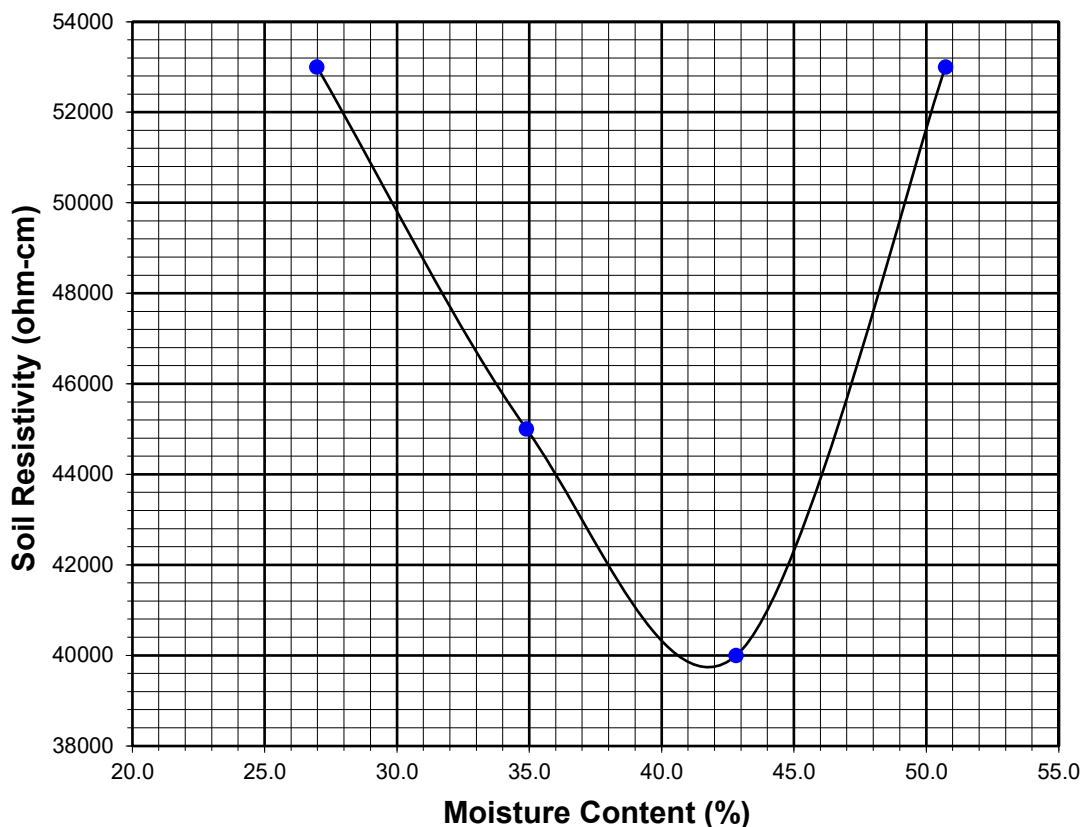
Soil Identification:* Olive yellow SP

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

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	30	26.98	53000	53000
2	40	34.89	45000	45000
3	50	42.81	40000	40000
4	60	50.73	53000	53000
5				

Moisture Content (%) (Mci)	3.22
Wet Wt. of Soil + Cont. (g)	202.63
Dry Wt. of Soil + Cont. (g)	198.23
Wt. of Container (g)	61.62
Container No.	
Initial Soil Wt. (g) (Wt)	130.36
Box Constant	1.000
$MC = (((1 + Mci / 100) \times (Wa / Wt + 1)) - 1) \times 100$	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
39600	41.8	98	41	5.88	22.5





**TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS**

Project Name: Hume SoCal Camp Tested By : G. Berdy Date: 08/16/23
Project No. : 13655.001 Checked By: J. Ward Date: 10/19/23

Boring No.	HA-37	HA-17		
Sample No.	MB-22a	MB-31		
Sample Depth (ft)	16-24"	36-40"		
Soil Identification:	Olive yellow SP, organics	Olive yellow SP, organics		
Wet Weight of Soil + Container (g)	159.39	218.25		
Dry Weight of Soil + Container (g)	151.89	211.60		
Weight of Container (g)	59.21	59.16		
Moisture Content (%)	8.09	4.36		
Weight of Soaked Soil (g)	100.12	100.12		

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	403	92		
Crucible No.	4	2		
Furnace Temperature (°C)	860	860		
Time In / Time Out	8:00/8:45	8:00/8:45		
Duration of Combustion (min)	45	45		
Wt. of Crucible + Residue (g)	21.6323	28.7231		
Wt. of Crucible (g)	21.6301	28.7190		
Wt. of Residue (g) (A)	0.0022	0.0041		
PPM of Sulfate (A) x 41150	90.53	168.71		
PPM of Sulfate, Dry Weight Basis	99	176		

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30	30		
ml of AgNO ₃ Soln. Used in Titration (C)	0.6	1.4		
PPM of Chloride (C -0.2) * 100 * 30 / B	40	120		
PPM of Chloride, Dry Wt. Basis	44	125		

pH TEST, DOT California Test 643

pH Value	5.33	5.40		
Temperature °C	22.8	22.6		



SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name: Hume SoCal Camp
Project No. : 13655.001
Boring No.: HA-37
Sample No. : MB-22a

Tested By : G. Berdy Date: 08/22/23
Checked By: J. Ward Date: 10/19/23
Depth (ft.) : 16-24"

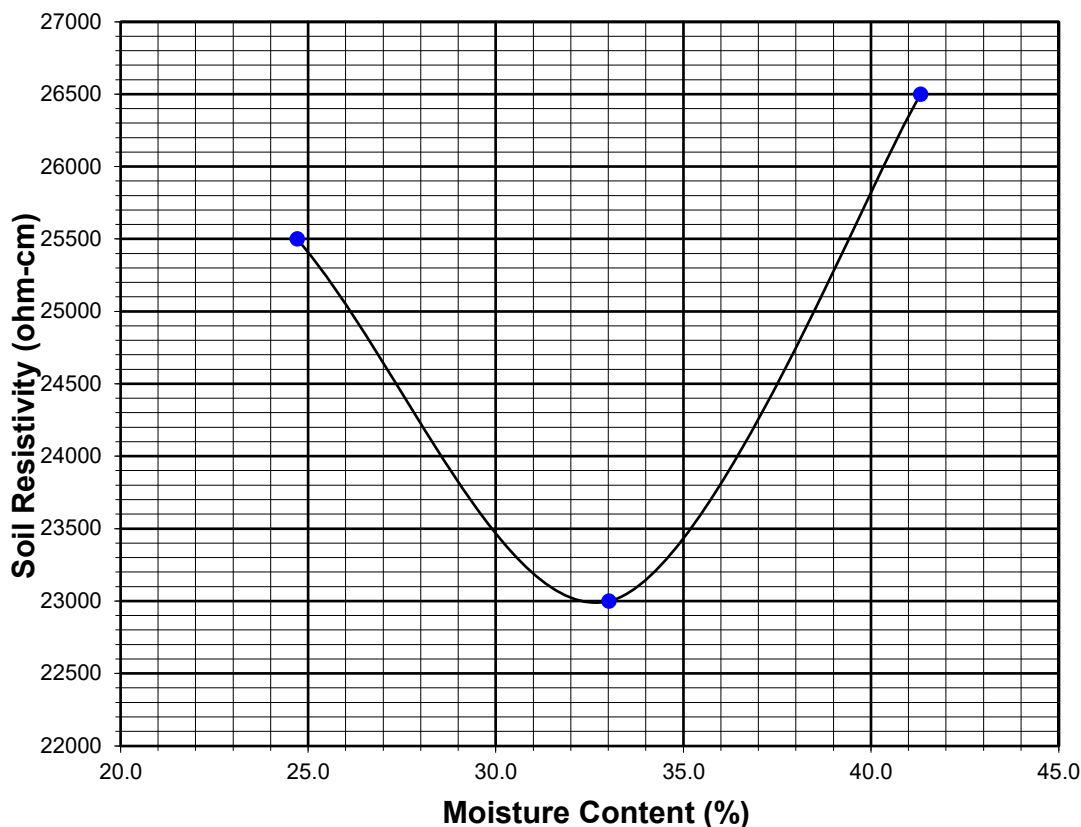
Soil Identification:* Olive yellow SP, organics

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

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	24.71	25500	25500
2	30	33.02	23000	23000
3	40	41.33	26500	26500
4				
5				

Moisture Content (%) (Mci)	8.09
Wet Wt. of Soil + Cont. (g)	159.39
Dry Wt. of Soil + Cont. (g)	151.89
Wt. of Container (g)	59.21
Container No.	
Initial Soil Wt. (g) (Wt)	130.10
Box Constant	1.000
$MC = (((1 + Mci / 100) \times (Wa / Wt + 1)) - 1) \times 100$	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
23000	32.7	99	44	5.33	22.8





SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name: Hume SoCal Camp
Project No. : 13655.001
Boring No.: HA-17
Sample No. : MB-31

Tested By : G. Berdy Date: 08/22/23
Checked By: J. Ward Date: 10/19/23
Depth (ft.) : 36-40"

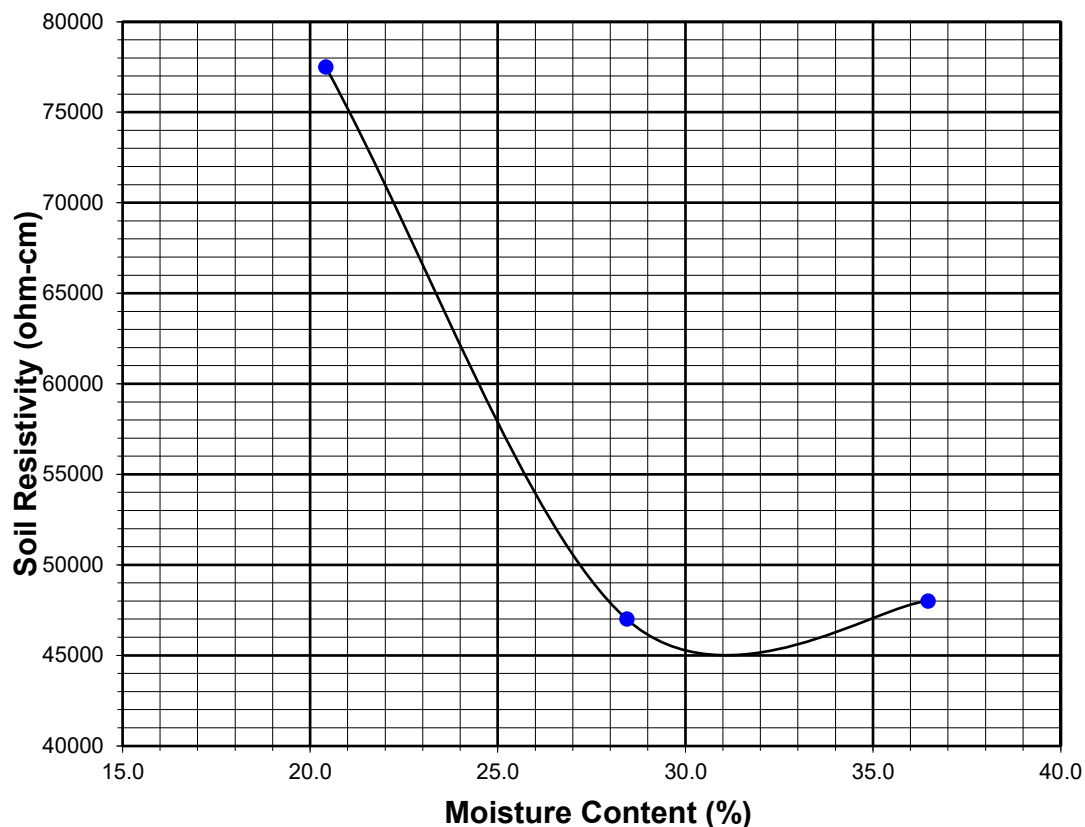
Soil Identification:* Olive yellow SP, organics

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

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	20.41	77500	77500
2	30	28.44	47000	47000
3	40	36.47	48000	48000
4				
5				

Moisture Content (%) (MCi)	4.36
Wet Wt. of Soil + Cont. (g)	218.25
Dry Wt. of Soil + Cont. (g)	211.60
Wt. of Container (g)	59.16
Container No.	
Initial Soil Wt. (g) (Wt)	130.03
Box Constant	1.000
$MC = (((1 + MC_i / 100) \times (W_a / W_t + 1)) - 1) \times 100$	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
45000	31.1	176	125	5.40	22.6





DIRECT SHEAR TEST
Consolidated Drained - ASTM D 3080

Project Name: [Hume SoCal Camp](#)

Project No.: [13655.001](#)

Boring No.: [HA-23, 25a, 25b](#)

Sample No.: [MB-11, 12a, 12b](#)

Soil Identification: [Dark brown silty sand \(SM\)](#)

Tested By: [G. Bathala](#)

Checked By: [J. Ward](#)

Sample Type: [90% Remold](#)

Depth (ft.): [Composite](#)

Date: [08/18/23](#)

Date: [10/19/23](#)

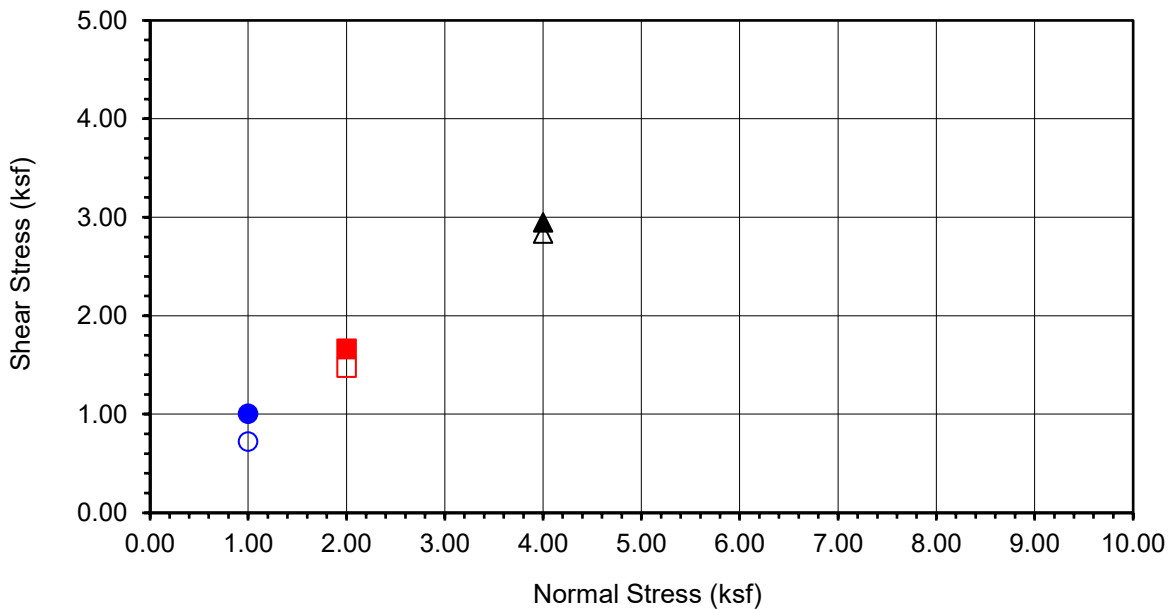
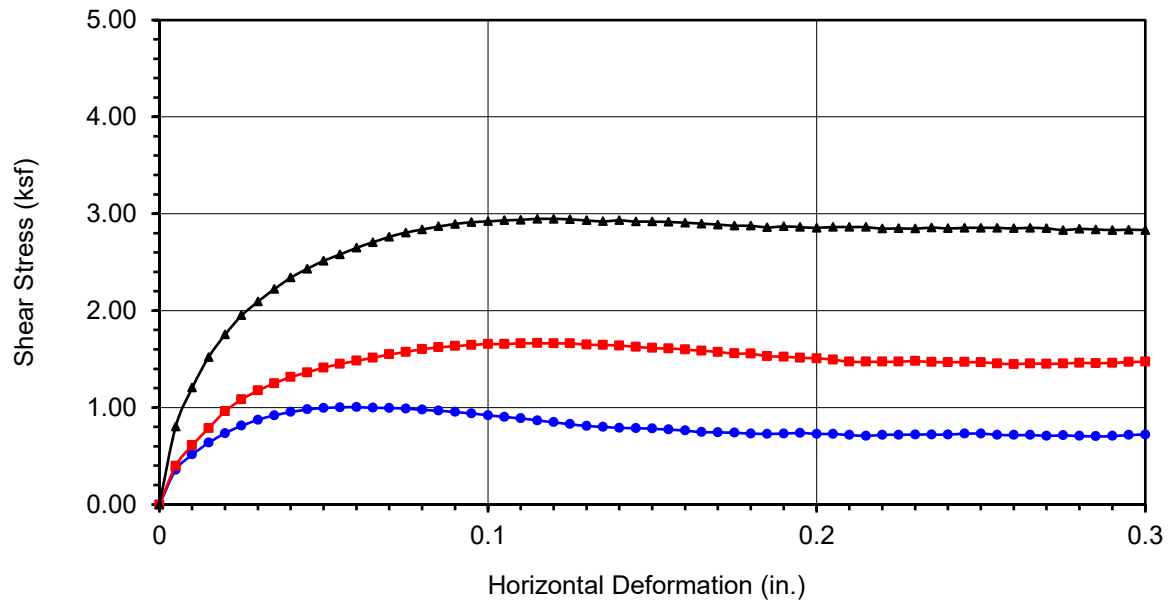
Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	190.07	190.17	189.52
Weight of Ring(gm):	45.53	45.47	44.56

Before Shearing

Weight of Wet Sample+Cont.(gm):	237.08	237.08	237.08
Weight of Dry Sample+Cont.(gm):	218.92	218.92	218.92
Weight of Container(gm):	64.19	64.19	64.19
Vertical Rdg.(in): Initial	0.2569	0.2315	0.0000
Vertical Rdg.(in): Final	0.2728	0.2531	-0.0276

After Shearing

Weight of Wet Sample+Cont.(gm):	199.95	220.06	216.26
Weight of Dry Sample+Cont.(gm):	178.70	199.11	195.60
Weight of Container(gm):	52.01	71.79	67.68
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43



Boring No.	HA-23, 25a, 25b
Sample No.	MB-11, 12a, 12b
Depth (ft)	Composite
<u>Sample Type:</u>	
90% Remold	
<u>Soil Identification:</u>	
Dark brown silty sand (SM)	

Normal Stress (kip/ft ²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft ²)	● 1.006	■ 1.666	▲ 2.949
Shear Stress @ End of Test (ksf)	○ 0.723	□ 1.474	△ 2.833
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	11.74	11.74	11.74
Dry Density (pcf)	107.6	107.7	107.9
Saturation (%)	55.9	56.1	56.4
Soil Height Before Shearing (in.)	0.9841	0.9784	0.9724
Final Moisture Content (%)	16.8	16.5	16.2



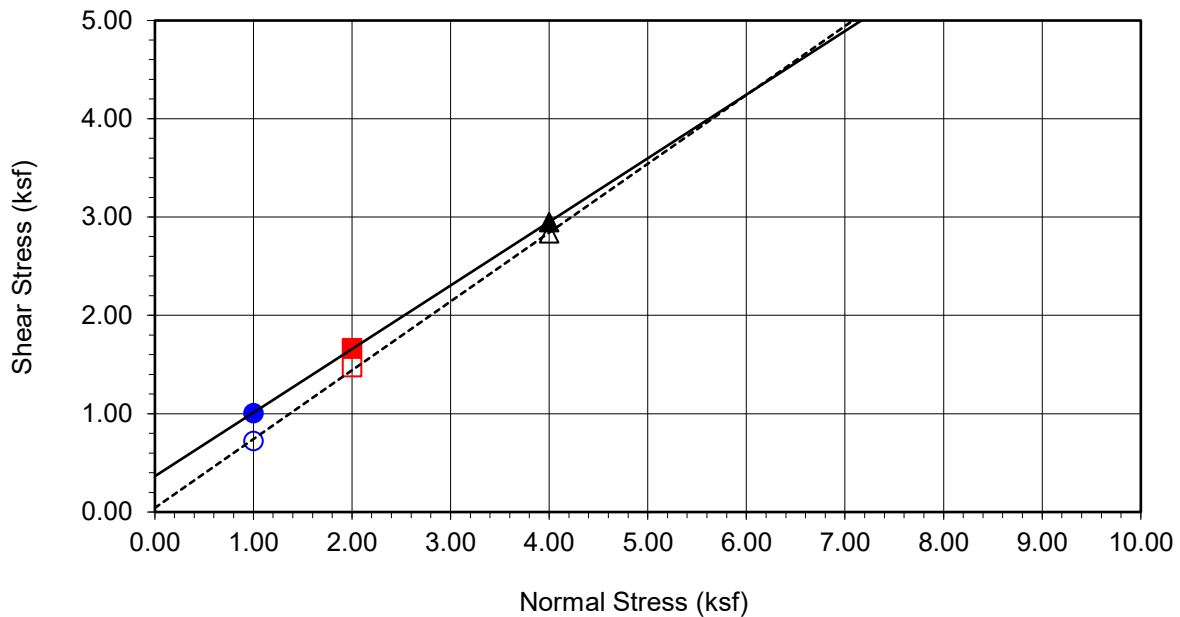
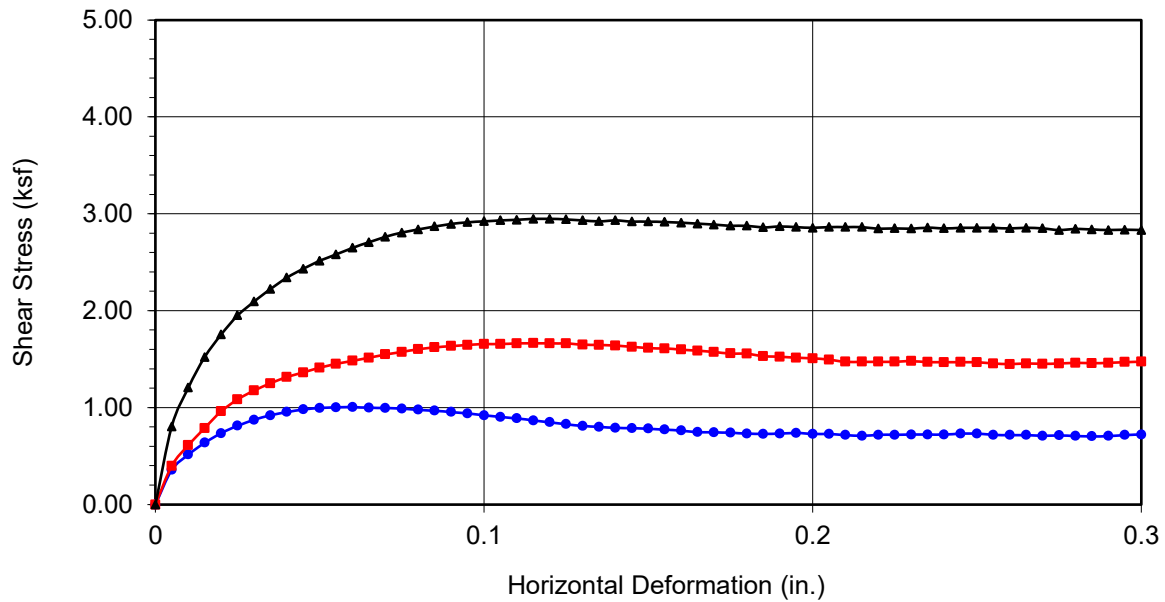
DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 13655.001

Hume SoCal Camp

08-23



Boring No.	HA-23, 25a, 25b	
Sample No.	MB-11, 12a, 12b	
Depth (ft)	Composite	
<u>Sample Type:</u> 90% Remold		
<u>Soil Identification:</u> Dark brown silty sand (SM)		
<u>Strength Parameters</u>		
	C (psf)	ϕ (°)
Peak	365	33
Ultimate	44	35

Normal Stress (kip/ft ²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft ²)	● 1.006	■ 1.666	▲ 2.949
Shear Stress @ End of Test (ksf)	○ 0.723	□ 1.474	△ 2.833
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	11.74	11.74	11.74
Dry Density (pcf)	107.6	107.7	107.9
Saturation (%)	55.9	56.1	56.4
Soil Height Before Shearing (in.)	0.9841	0.9784	0.9724
Final Moisture Content (%)	16.8	16.5	16.2



DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 13655.001

Hume SoCal Camp

08-23



DIRECT SHEAR TEST
Consolidated Drained - ASTM D 3080

Project Name: [Hume SoCal Camp](#)

Tested By: [G. Bathala](#)

Date: [08/18/23](#)

Project No.: [13655.001](#)

Checked By: [J. Ward](#)

Date: [10/19/23](#)

Boring No.: [HA-31, 32, 30](#)

Sample Type: [90% Remold](#)

Sample No.: [MB-18, 19, 30](#)

Depth (ft.): [Composite](#)

Soil Identification: [Light brown silty sand \(SM\)](#)

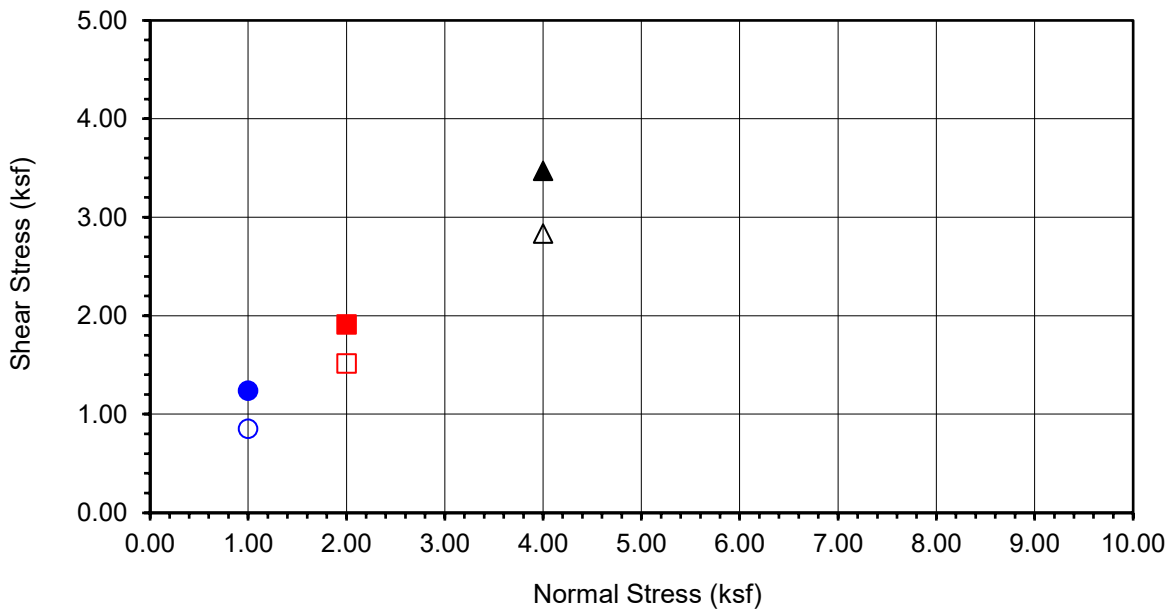
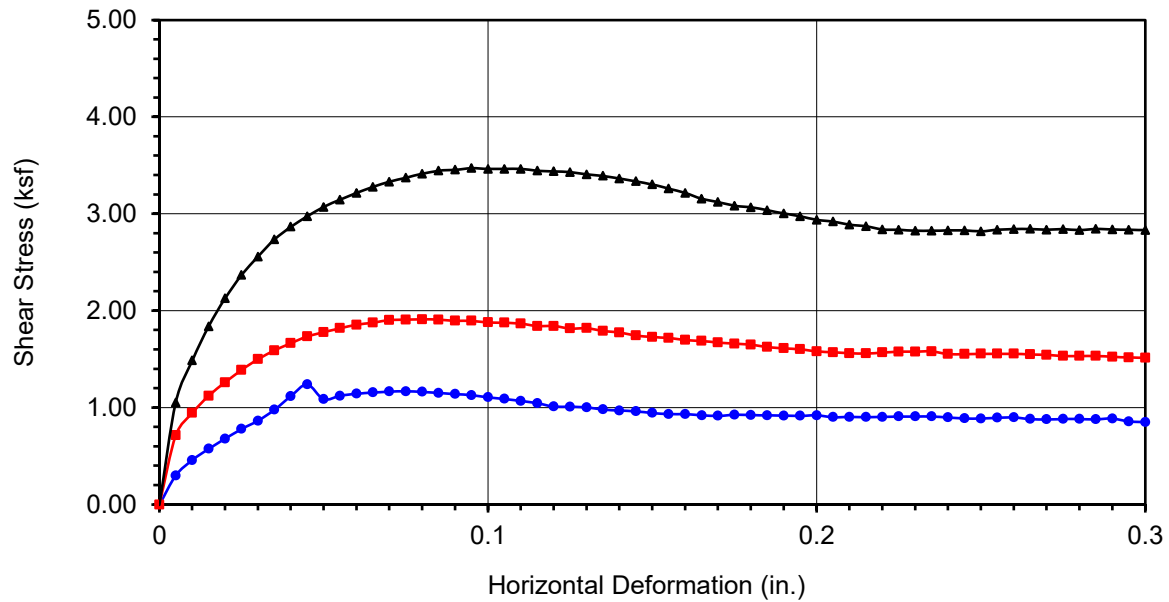
Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	191.74	190.98	192.11
Weight of Ring(gm):	45.38	44.61	45.58

Before Shearing

Weight of Wet Sample+Cont.(gm):	168.49	168.49	168.49
Weight of Dry Sample+Cont.(gm):	158.00	158.00	158.00
Weight of Container(gm):	55.81	55.81	55.81
Vertical Rdg.(in): Initial	0.0000	0.2561	0.2530
Vertical Rdg.(in): Final	-0.0109	0.2780	0.2880

After Shearing

Weight of Wet Sample+Cont.(gm):	219.68	216.64	206.79
Weight of Dry Sample+Cont.(gm):	199.25	196.91	187.37
Weight of Container(gm):	69.44	68.11	57.95
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43



Boring No.	HA-31, 32, 30
Sample No.	MB-18, 19, 30
Depth (ft)	Composite
<u>Sample Type:</u>	
90% Remold	
<u>Soil Identification:</u>	
Light brown silty sand (SM)	

Normal Stress (kip/ft ²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft ²)	● 1.239	■ 1.911	▲ 3.471
Shear Stress @ End of Test (ksf)	○ 0.852	□ 1.515	△ 2.833
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	10.27	10.27	10.27
Dry Density (pcf)	110.4	110.4	110.5
Saturation (%)	52.6	52.6	52.8
Soil Height Before Shearing (in.)	0.9891	0.9781	0.9650
Final Moisture Content (%)	15.7	15.3	15.0



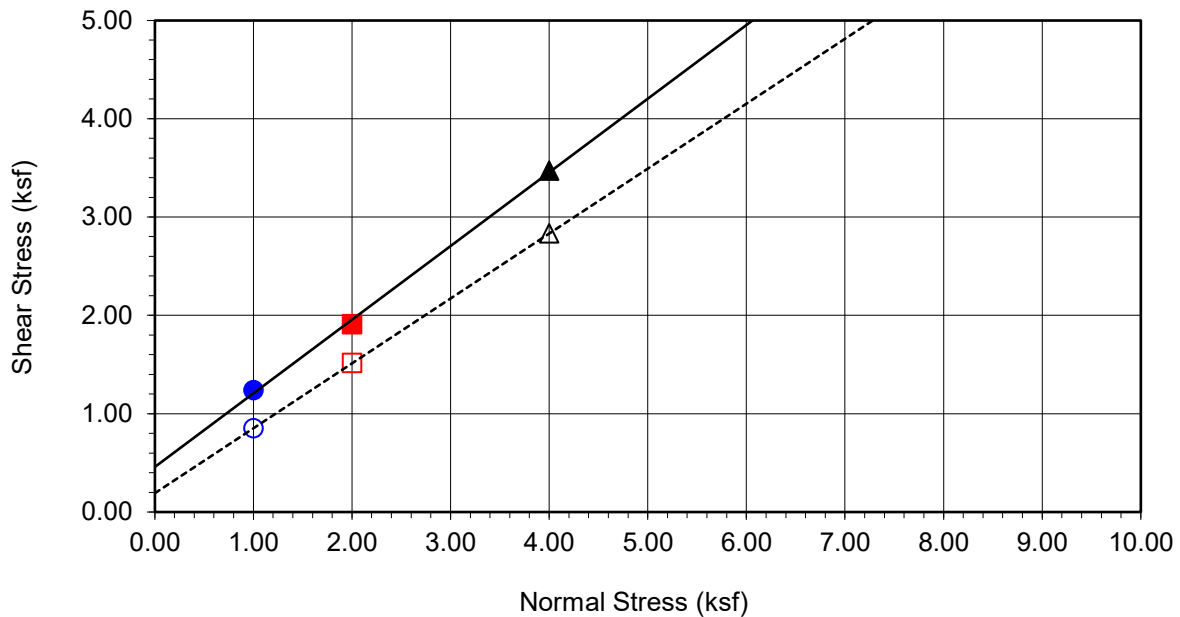
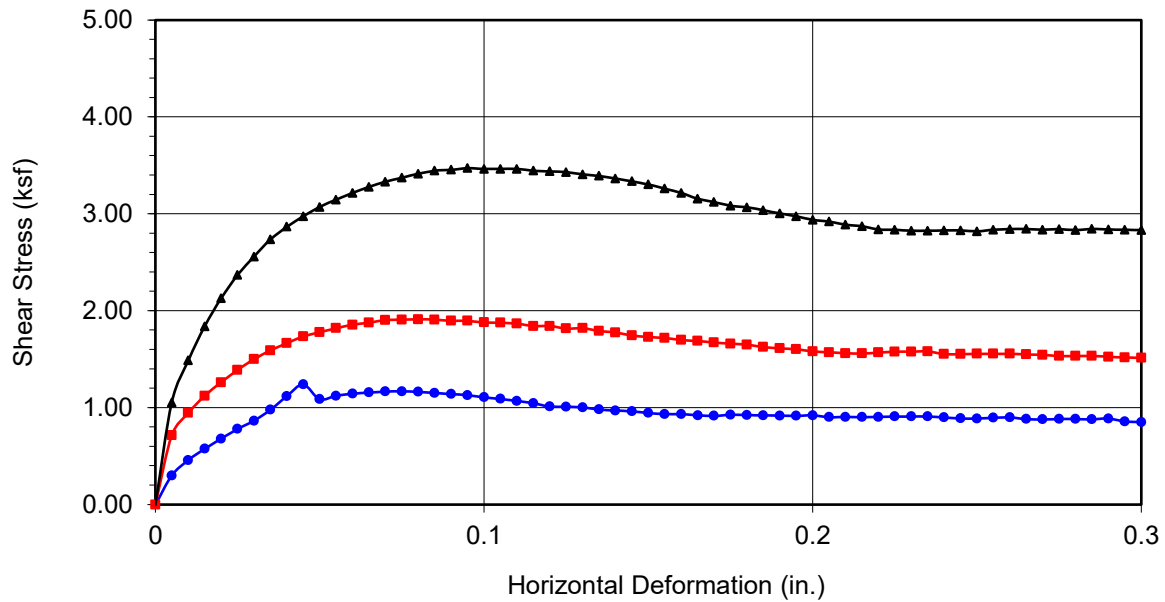
DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 13655.001

Hume SoCal Camp

08-23



Boring No.	HA-31, 32, 30	
Sample No.	MB-18, 19, 30	
Depth (ft)	Composite	
<u>Sample Type:</u> 90% Remold		
<u>Soil Identification:</u> Light brown silty sand (SM)		
<u>Strength Parameters</u>		
	C (psf)	ϕ (°)
Peak	459	37
Ultimate	193	33

Normal Stress (kip/ft ²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft ²)	● 1.239	■ 1.911	▲ 3.471
Shear Stress @ End of Test (ksf)	○ 0.852	□ 1.515	△ 2.833
Deformation Rate (in./min.)	0.0033	0.0033	0.0033
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	10.27	10.27	10.27
Dry Density (pcf)	110.4	110.4	110.5
Saturation (%)	52.6	52.6	52.8
Soil Height Before Shearing (in.)	0.9891	0.9781	0.9650
Final Moisture Content (%)	15.7	15.3	15.0



DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 13655.001

Hume SoCal Camp

08-23



MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name: Hume SoCal Camp Tested By: A. Santos Date: 08/17/23
Project No.: 13655.001 Checked By: ACS/JHW Date: 08/18/23
Boring No.: HA-23, HA-25a, HA-25b Depth (ft.): Composite
Sample No.: MB-11, MB-12a, MB-12b
Soil Identification: Dark brown silty sand (SM)

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

Preparation Method:	☒	Moist	Scalp Fraction (%)		Rammer Weight (lb.) =	10.0
		Dry	#3/4		Height of Drop (in.) =	18.0
Compaction Method	☒	Mechanical Ram	#3/8		Mold Volume (ft ³)	
		Manual Ram	#4	6.1		

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3747	3819	3774			
Weight of Mold (g)	1808	1808	1808			
Net Weight of Soil (g)	1939	2011	1966			
Wet Weight of Soil + Cont. (g)	402.3	418.9	475.4			
Dry Weight of Soil + Cont. (g)	370.1	378.2	421.7			
Weight of Container (g)	39.7	39.2	37.2			
Moisture Content (%)	9.75	12.01	13.97			
Wet Density (pcf)	128.8	133.5	130.5			
Dry Density (pcf)	117.3	119.2	114.6			

Maximum Dry Density (pcf) **119.5**

Optimum Moisture Content (%) **11.4**

Corrected Dry Density (pcf) **121.7**

Corrected Moisture Content (%) **10.8**

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

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

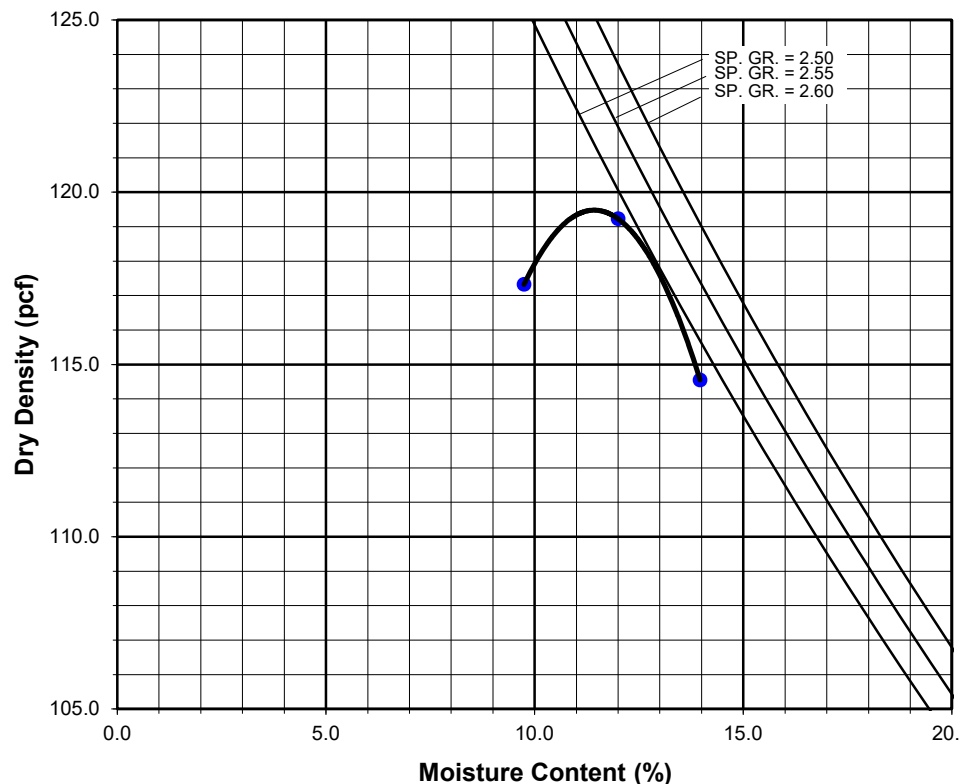
☐ **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +3/4 in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL, PL, PI





MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name: Hume SoCal Camp Tested By: A. Santos Date: 08/17/23
Project No.: 13655.001 Checked By: ACS/JHW Date: 08/18/23
Boring No.: HA-31, HA-32, HA-30 Depth (ft.): Composite
Sample No.: MB-18, MB-19, MB-30
Soil Identification: Light brown silty sand (SM)

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

Preparation Method:	☒	Moist	Scalp Fraction (%)		Rammer Weight (lb.) =	10.0
		Dry	#3/4		Height of Drop (in.) =	18.0
Compaction Method	☒	Mechanical Ram	#3/8		Mold Volume (ft ³)	
		Manual Ram	#4	8.4		

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3737	3834	3844			
Weight of Mold (g)	1808	1808	1808			
Net Weight of Soil (g)	1929	2026	2036			
Wet Weight of Soil + Cont. (g)	623.2	433.7	465.9			
Dry Weight of Soil + Cont. (g)	585.0	397.7	418.8			
Weight of Container (g)	76.7	39.0	39.8			
Moisture Content (%)	7.52	10.03	12.43			
Wet Density (pcf)	128.1	134.5	135.2			
Dry Density (pcf)	119.1	122.3	120.2			

Maximum Dry Density (pcf) **122.3**

Optimum Moisture Content (%) **10.3**

Corrected Dry Density (pcf) **125.2**

Corrected Moisture Content (%) **9.5**

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

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

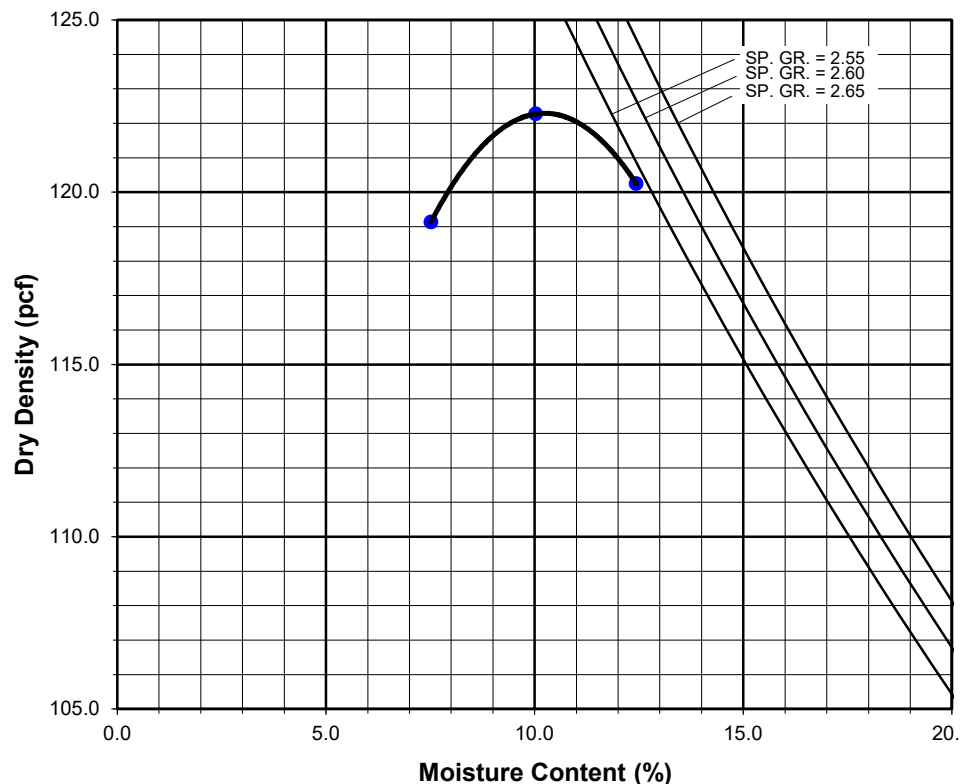
☐ **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +3/4 in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL, PL, PI





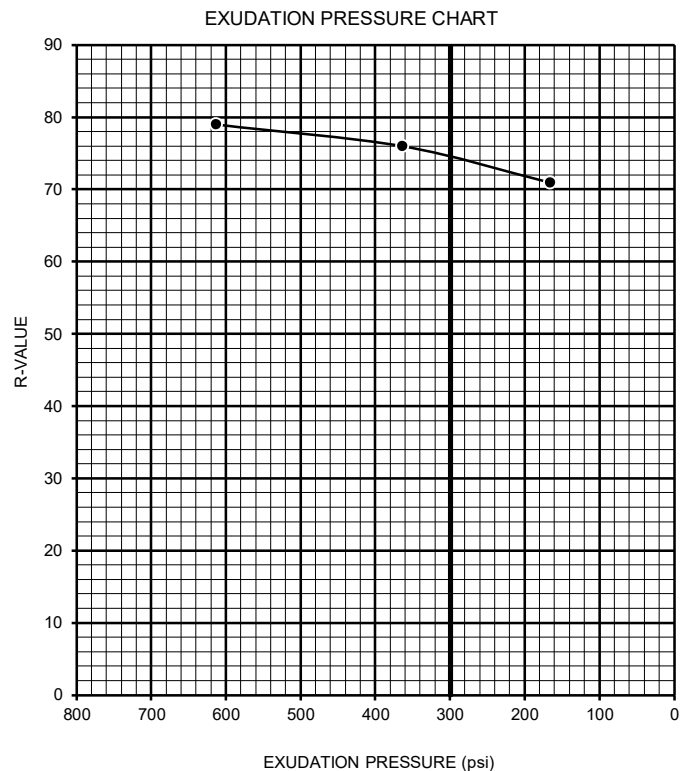
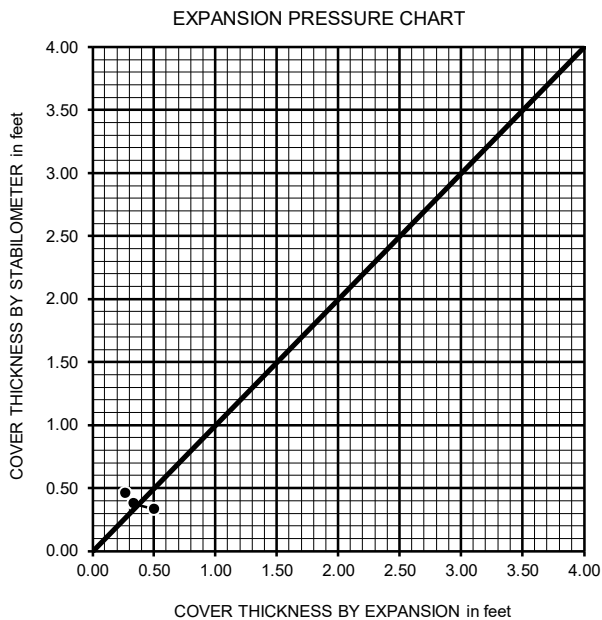
R-VALUE TEST RESULTS

DOT CA Test 301

PROJECT NAME: Hume SoCal Camp PROJECT NUMBER: 13655.001
BORING NUMBER: HA-3, HA-5b, HA-9 DEPTH (FT.): Composite
SAMPLE NUMBER: MB-1, MB-3, MB-4 TECHNICIAN: O. Figueroa
SAMPLE DESCRIPTION: Brown poorly-graded sand with silt (SP-SM) DATE COMPLETED: 8/23/2023

TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	10.5	11.1	12.0
HEIGHT OF SAMPLE, Inches	2.48	2.49	2.49
DRY DENSITY, pcf	117.0	116.8	114.9
COMPACTOR PRESSURE, psi	350	300	250
EXUDATION PRESSURE, psi	613	364	166
EXPANSION, Inches x 10exp-4	15	10	8
STABILITY Ph 2,000 lbs (160 psi)	18	20	24
TURNS DISPLACEMENT	5.35	5.65	5.92
R-VALUE UNCORRECTED	79	76	71
R-VALUE CORRECTED	79	76	71

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.34	0.38	0.46
EXPANSION PRESSURE THICKNESS, ft.	0.50	0.33	0.27



R-VALUE BY EXPANSION: 77
R-VALUE BY EXUDATION: 74
EQUILIBRIUM R-VALUE: 74



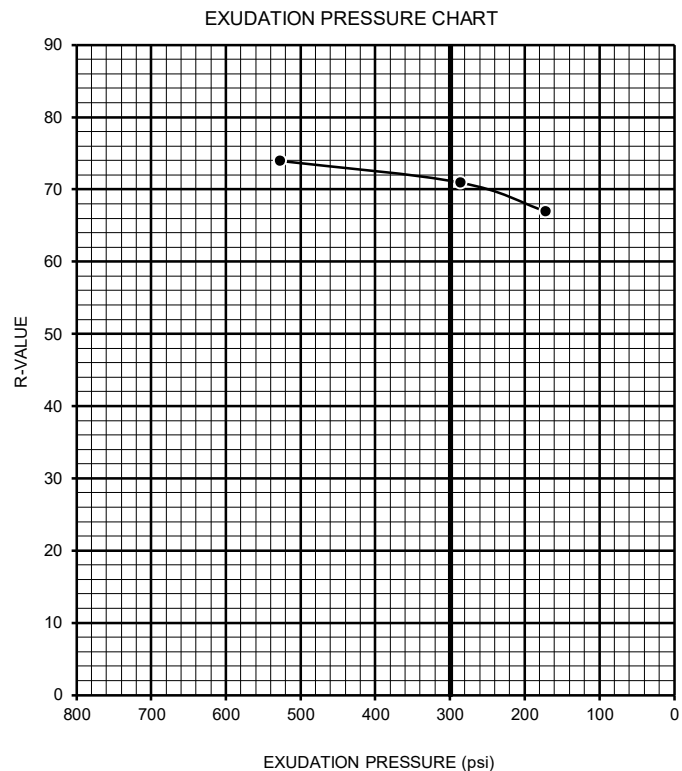
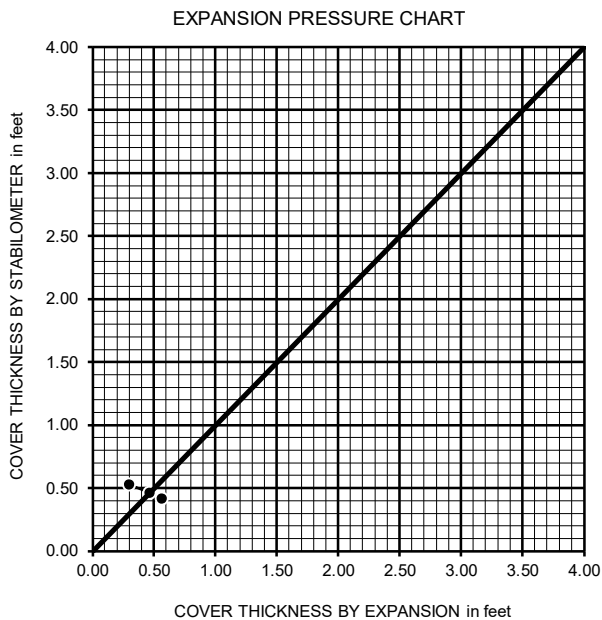
R-VALUE TEST RESULTS

DOT CA Test 301

PROJECT NAME:	Hume SoCal Camp	PROJECT NUMBER:	13655.001
BORING NUMBER:	HA-10, HA-17a, HA-17b	DEPTH (FT.):	Composite
SAMPLE NUMBER:	MB-5, MB-7a, MB-7b	TECHNICIAN:	O. Figueroa
SAMPLE DESCRIPTION:	Very dark brown silty sand (SM)	DATE COMPLETED:	8/18/2023

TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	12.5	13.4	14.1
HEIGHT OF SAMPLE, Inches	2.50	2.50	2.54
DRY DENSITY, pcf	111.8	110.5	108.9
COMPACTOR PRESSURE, psi	350	300	250
EXUDATION PRESSURE, psi	527	286	172
EXPANSION, Inches x 10exp-4	17	14	9
STABILITY Ph 2,000 lbs (160 psi)	23	26	30
TURNS DISPLACEMENT	5.25	5.38	5.41
R-VALUE UNCORRECTED	74	71	67
R-VALUE CORRECTED	74	71	67

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.42	0.46	0.53
EXPANSION PRESSURE THICKNESS, ft.	0.57	0.47	0.30



R-VALUE BY EXPANSION:	71
R-VALUE BY EXUDATION:	71
EQUILIBRIUM R-VALUE:	71



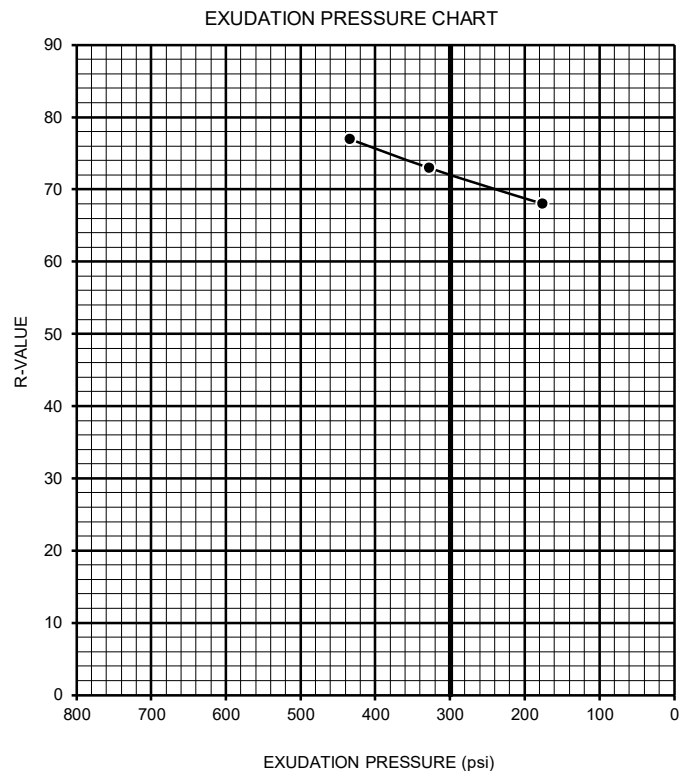
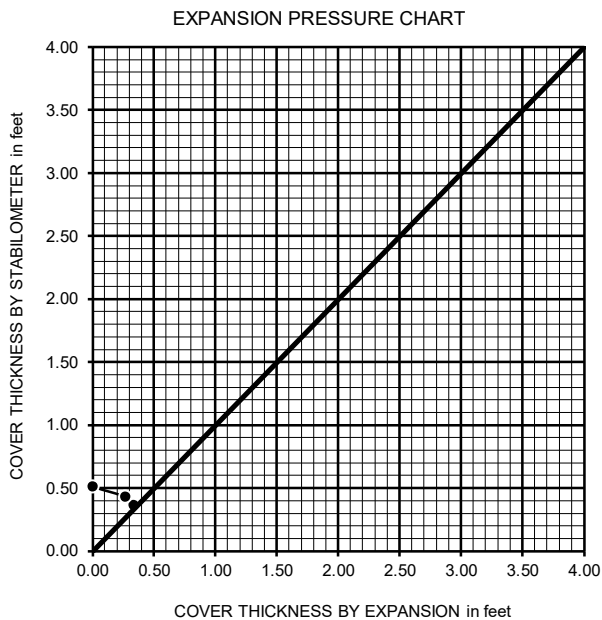
R-VALUE TEST RESULTS

DOT CA Test 301

PROJECT NAME:	Hume SoCal Camp	PROJECT NUMBER:	13655.001
BORING NUMBER:	HA-29	DEPTH (FT.):	6-36"
SAMPLE NUMBER:	MB-15	TECHNICIAN:	O. Figueroa
SAMPLE DESCRIPTION:	Dark brown silty sand (SM)	DATE COMPLETED:	8/23/2023

TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	13.0	13.4	14.3
HEIGHT OF SAMPLE, Inches	2.52	2.53	2.55
DRY DENSITY, pcf	110.9	110.2	108.0
COMPACTOR PRESSURE, psi	350	300	250
EXUDATION PRESSURE, psi	434	328	177
EXPANSION, Inches x 10exp-4	10	8	0
STABILITY Ph 2,000 lbs (160 psi)	20	23	28
URNS DISPLACEMENT	5.10	5.40	5.65
R-VALUE UNCORRECTED	77	73	68
R-VALUE CORRECTED	77	73	68

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.37	0.43	0.51
EXPANSION PRESSURE THICKNESS, ft.	0.33	0.27	0.00



R-VALUE BY EXPANSION:	78
R-VALUE BY EXUDATION:	72
EQUILIBRIUM R-VALUE:	72



16691 MILLIKAN AVE.
IRVINE, CA 92606
TEL. (949) 222-5321

Unconfined Compressive Strength

Project Name: Hume SoCal Camp Project No. 13655.001
Sampled By: Jeff Hull Sample Date: 7/21/2023

MATERIAL SAMPLE TYPE

Rock X Granite Soil Cement Concrete

FOR LAB USE ONLY DATE RECEIVED: 7/31/2023 By: Ryan Densmore

Laboratory Test Data

Laboratory No.	Age Days	Test Date	Height (in)	Diam. (in)	Area (sq. in.)	Load (lbs.)	Compr. (psi)	H/D Ratio	Corr. Factor	Compr. (psi)	Failure Type
A1	139	12/07/23	4.00	1.99	3.11	10590	3410	2.01	1.00	3410	N/A
A2	139	12/07/23	3.89	1.99	3.11	11470	3690	1.95	1.00	3690	N/A
B1	139	12/07/23	3.87	1.99	3.11	20800	6690	1.94	1.00	6690	N/A
B2	139	12/07/23	3.83	1.99	3.11	22030	7080	1.92	0.99	7010	N/A

Locations: A1, A2: Adult Lodge

B1, B2: RR at Existing Pond

APPENDIX D

**GENERAL EARTHWORK AND
GRADING GUIDE SPECIFICATIONS**

LEIGHTON AND ASSOCIATES, INC.
GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

TABLE OF CONTENTS

<u>Section</u>	<u>Page</u>
1.0 GENERAL	1
1.1 Intent	1
1.2 The Geotechnical Consultant of Record	2
1.3 The Earthwork Contractor	2
2.0 PREPARATION OF AREAS TO BE FILLED	2
2.1 Clearing and Grubbing	2
2.2 Processing	3
2.3 Overexcavation	3
2.4 Benching	4
2.5 Evaluation/Acceptance of Fill Areas	4
3.0 FILL MATERIAL	4
3.1 General	4
3.2 Oversize	4
3.3 Import	5
4.0 FILL PLACEMENT AND COMPACTION	5
4.1 Fill Layers	5
4.2 Fill Moisture Conditioning	5
4.3 Compaction of Fill	5
4.4 Compaction of Fill Slopes	5
4.5 Compaction Testing	6
4.6 Frequency of Compaction Testing	6
4.7 Compaction Test Locations	6
5.0 SUBDRAIN INSTALLATION	6
6.0 EXCAVATION	7
7.0 TRENCH BACKFILLS	7
7.1 Safety	7
7.2 Bedding & Backfill	7
7.3 Lift Thickness	7
7.4 Observation and Testing	7

Standard Details

A - Keying and Benching	Rear of Text
B - Oversize Rock Disposal	Rear of Text
C - Canyon Subdrains	Rear of Text
D - Buttress or Replacement Fill Subdrains	Rear of Text
E - Transition Lot Fills and Side Hill Fills	Rear of Text

1.0 General

1.1 Intent

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

1.2 The Geotechnical Consultant of Record

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

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

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

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

1.3 The Earthwork Contractor

The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

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

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

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing

Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

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

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

2.2 Processing

Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation

In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas

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

3.0 Fill Material

3.1 General

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

3.2 Oversize

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557).

4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557.

4.5 Compaction Testing

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

4.6 Frequency of Compaction Testing

Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations

The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

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

6.0 Excavation

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

7.0 Trench Backfills

7.1 Safety

The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations.

7.2 Bedding and Backfill

All bedding and backfill of utility trenches shall be performed in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of relative compaction from 1 foot above the top of the conduit to the surface.

The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

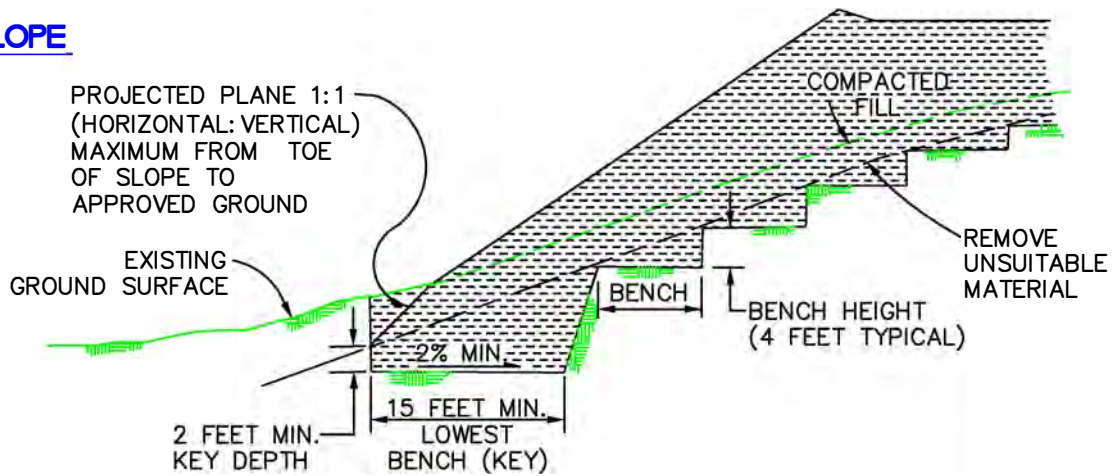
7.3 Lift Thickness

Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

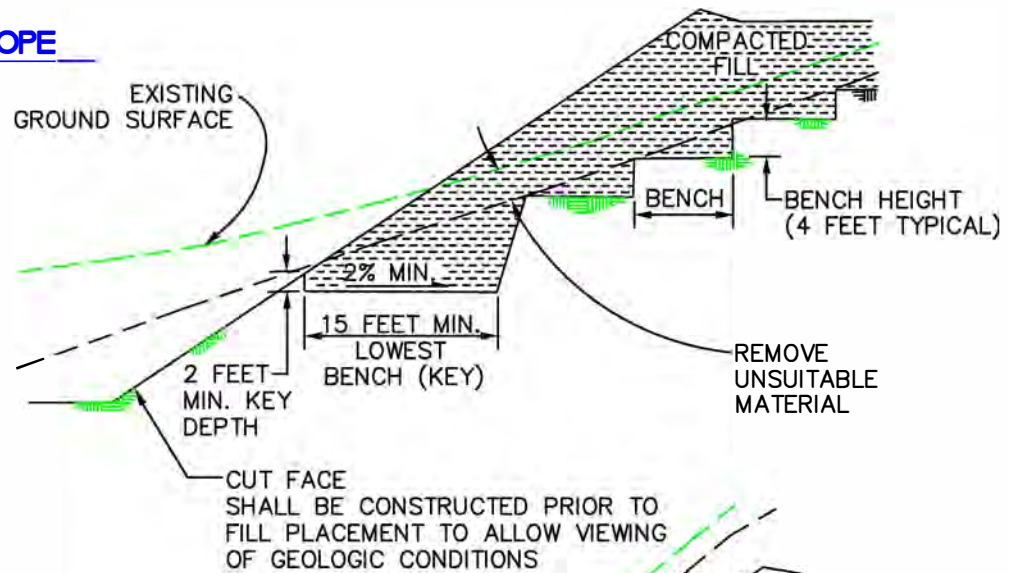
7.4 Observation and Testing

The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

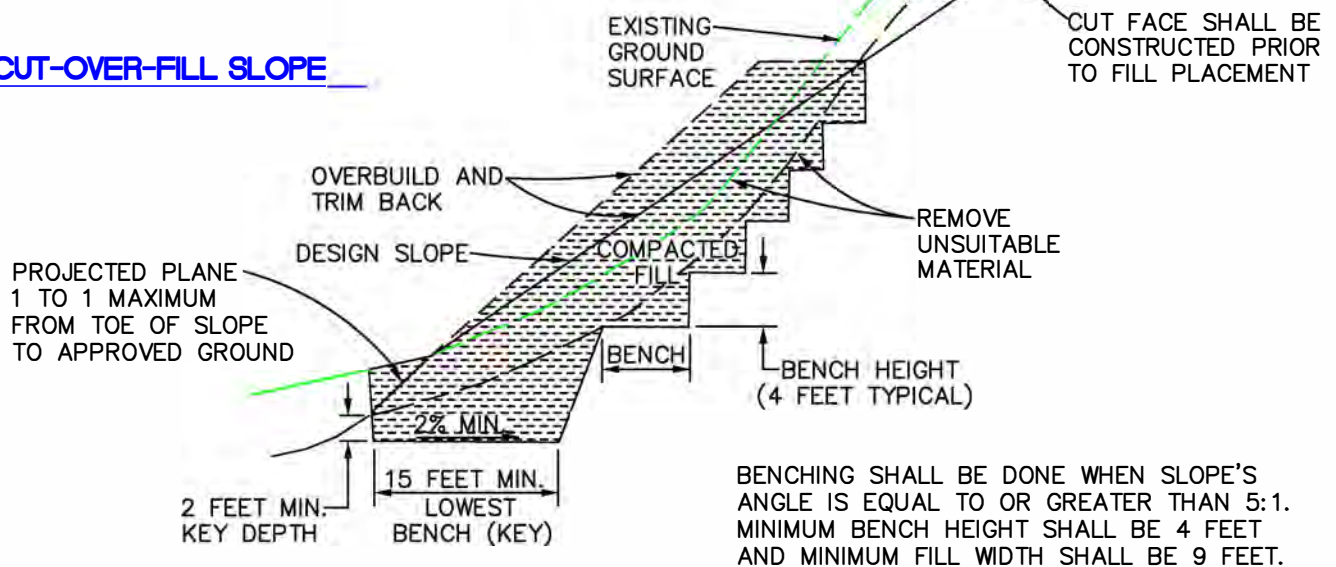
FILL SLOPE

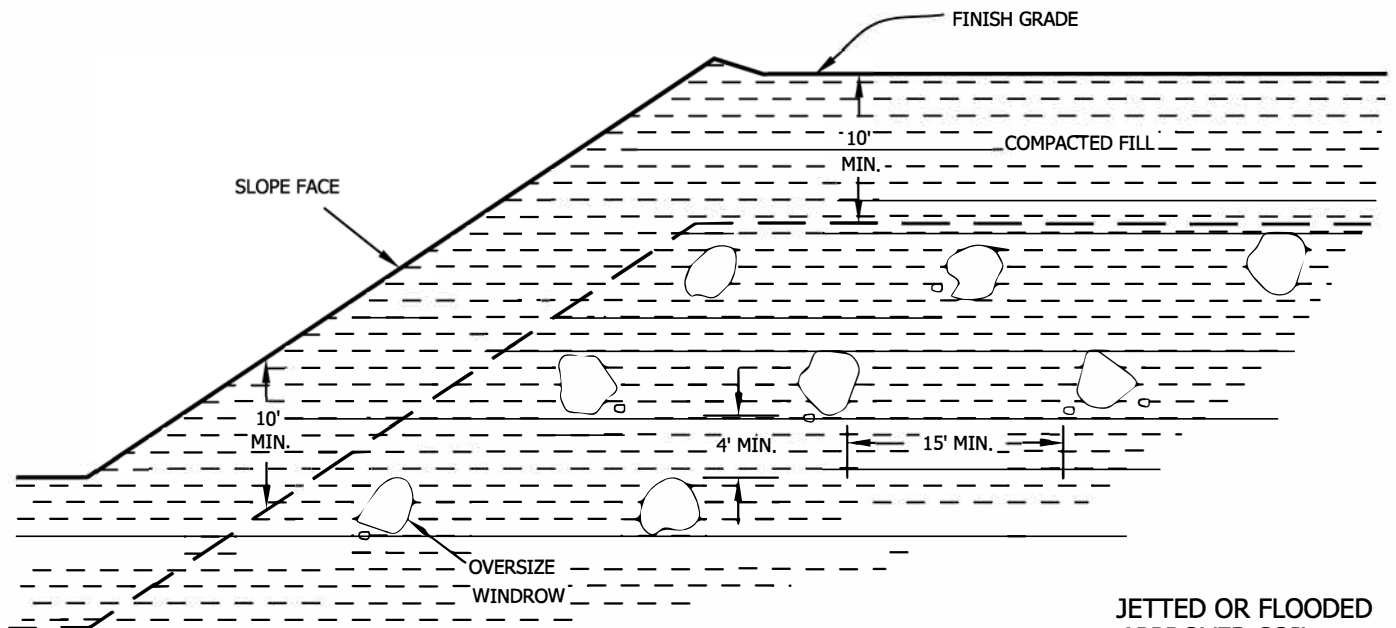


FILL-OVER-CUT SLOPE

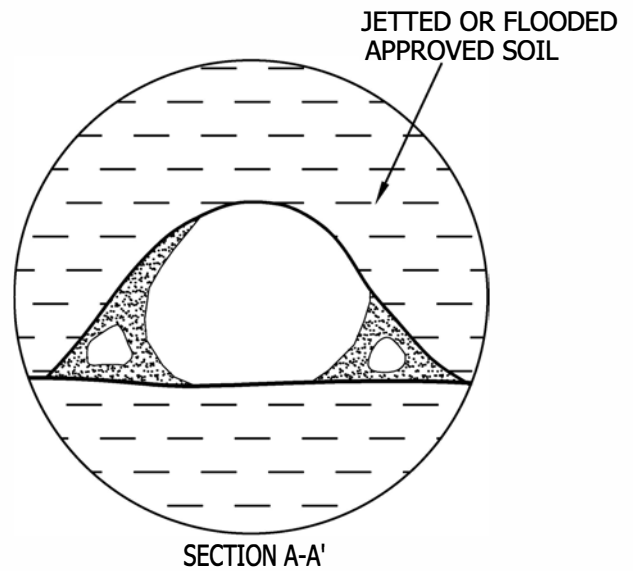


CUT-OVER-FILL SLOPE

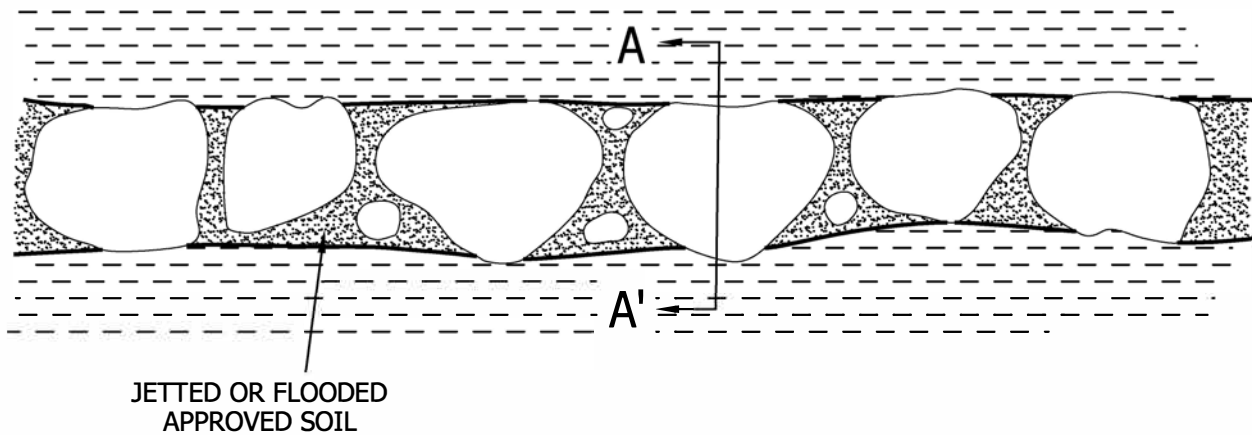




- Oversize rock is larger than 8 inches in largest dimension.
- Backfill with approved soil jetted or flooded in place to fill all the voids.
- Do not bury rock within 10 feet of finish grade.
- Windrow of buried rock shall be parallel to the finished slope face.



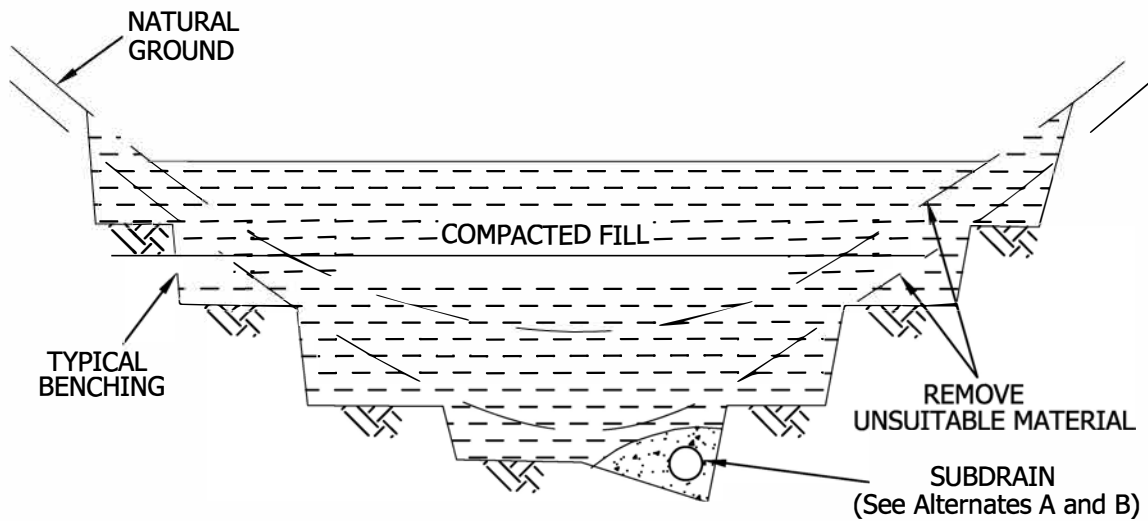
PROFILE ALONG WINDROW



OVERSIZE ROCK DISPOSAL

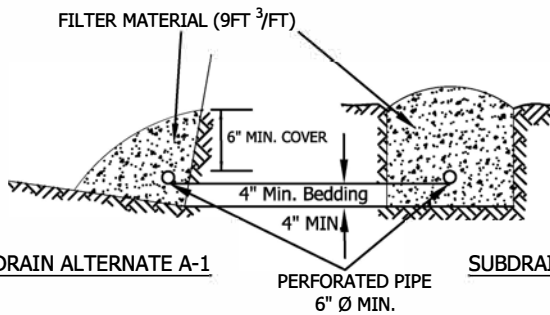
GENERAL EARTHWORK AND GRADING
SPECIFICATIONS
STANDARD DETAILS B





SUBDRAIN ALTERNATE A

PERFORATED PIPE SURROUNDED
WITH FILTER MATERIAL

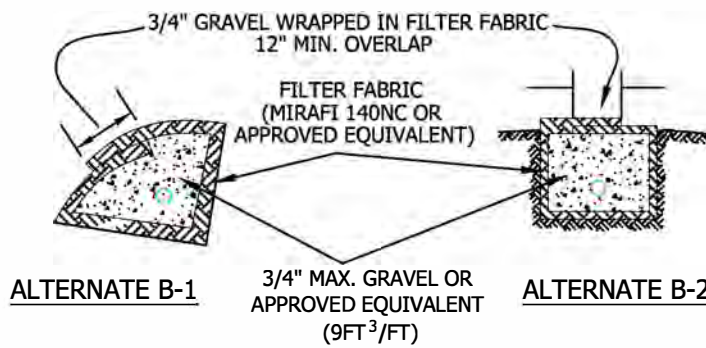


FILTER MATERIAL
FILTER MATERIAL SHALL BE CLASS 2 PERMEABLE MATERIAL PER STATE OF
CALIFORNIA STANDARD SPECIFICATION, OR APPROVED ALTERNATE.
CLASS 2 GRADING AS FOLLOWS:

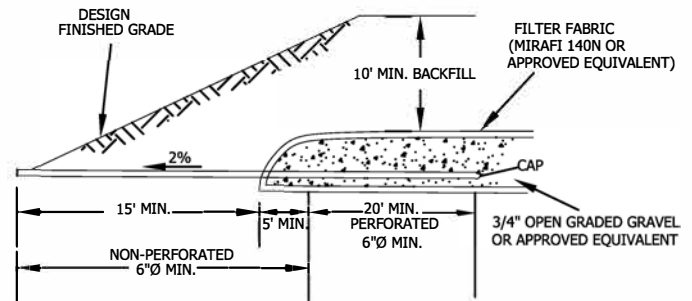
Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

SUBDRAIN ALTERNATE B

DETAIL OF CANYON SUBDRAIN TERMINAL



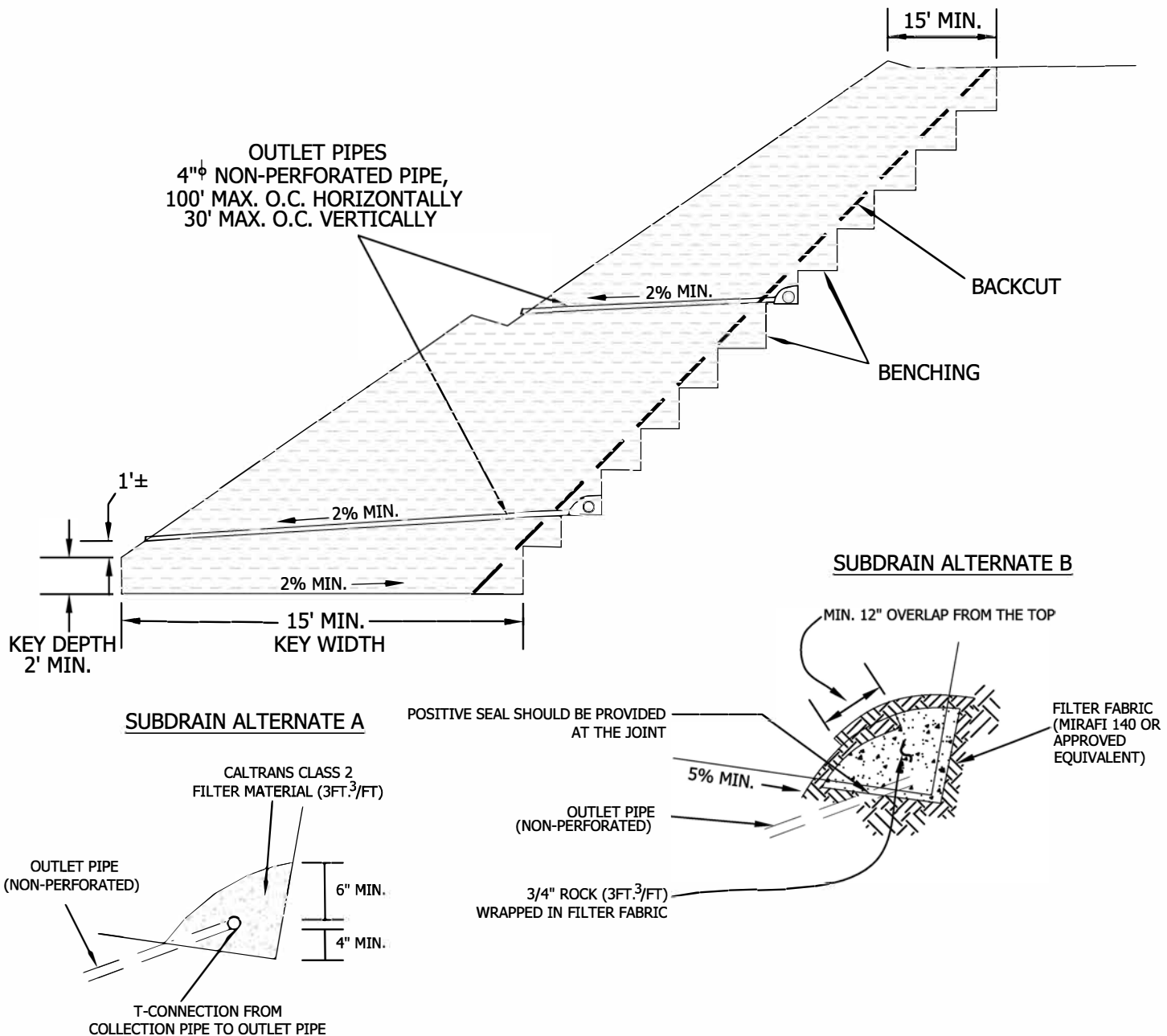
○ PERFORATED PIPE IS OPTIONAL PER
GOVERNING AGENCY'S REQUIREMENTS



CANYON
SUBDRAIN

GENERAL EARTHWORK AND GRADING
SPECIFICATIONS
STANDARD DETAILS C

Leighton
a verdantas company



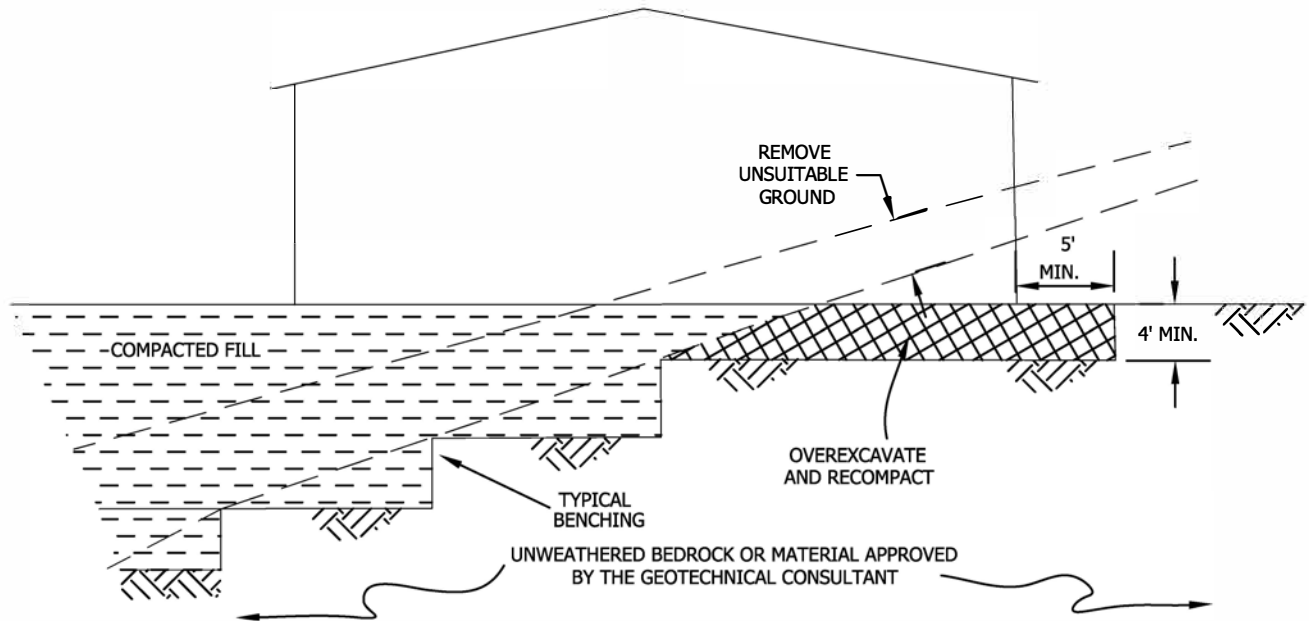
- **SUBDRAIN INSTALLATION** - Subdrain collector pipe shall be installed with perforations down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall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

**BUTTRESS OR
REPLACEMENT FILL
SUBDRAINS**

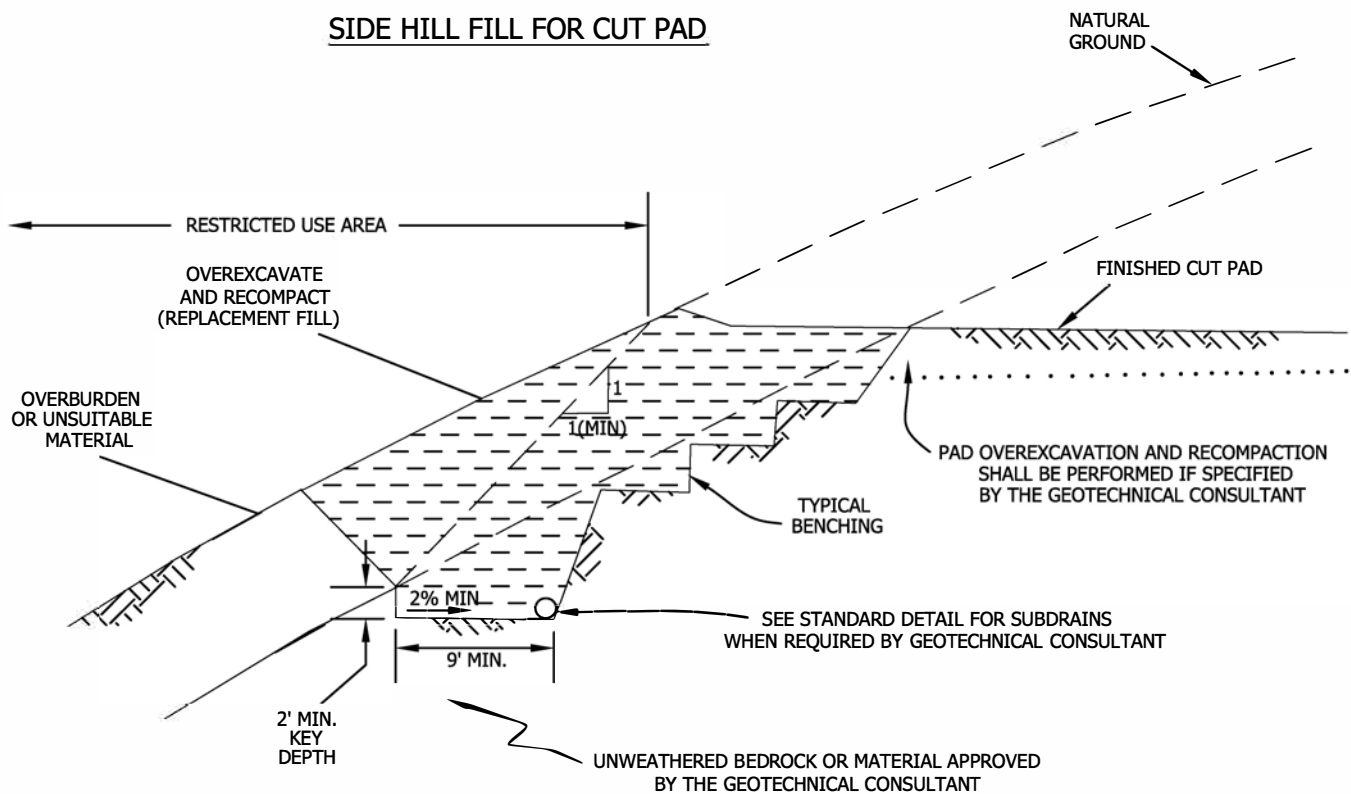
**GENERAL EARTHWORK AND GRADING
SPECIFICATIONS
STANDARD DETAILS D**

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CUT-FILL TRANSITION LOT OVEREXCAVATION



SIDE HILL FILL FOR CUT PAD



TRANSITION LOT FILLS
AND SIDE HILL FILLS

GENERAL EARTHWORK AND GRADING
SPECIFICATIONS
STANDARD DETAILS E

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APPENDIX E

GBA GEOTECHNICAL INSERT

Important Information about This Geotechnical-Engineering Report

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

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

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

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer

will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

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

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

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it.* A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept*

responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.

Most of the “Findings” Related in This Report Are Professional Opinions

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

This Report’s Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals’ plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

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

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note*

conspicuously that you’ve included the material for information purposes only. To avoid misunderstanding, you may also want to note that “informational purposes” means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled “limitations,” many of these provisions indicate where geotechnical engineers’ responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a “phase-one” or “phase-two” environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer’s services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, *proper implementation of the geotechnical engineer’s recommendations will not of itself be sufficient to prevent moisture infiltration.* Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. *Geotechnical engineers are not building-envelope or mold specialists.*



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