Appendix C

Geotechnical Design and Earthwork Recommendations

GEOTECHNICAL DESIGN AND EARTHWORK RECOMMENDATIONS

PROPOSED MIXED USE COMMUNITY DEVELOPMENT

VICTORIA CORPORATE CENTER PHASE II/III

2879 SEABORG AVENUE

CITY OF VENTURA, CALIFORNIA

Prepared for **RED TAIL MULTIFAMILY LAND DEVELOPMENT, LLC**
2082 MICHELSON, 4TH FLOOR
IRVINE, CALIFORNIA 92612

Prepared by **LEIGHTON AND ASSOCIATES, INC.**
56 EAST MAIN STREET, SUITE 207
VENTURA, CALIFORNIA 93001

Project Number 13582.001

August 12, 2022

Leighton 60th Anniversary
Leighton and Associates, Inc.
A Leighton Group Company

August 12, 2022

Project No. 13582.001

Red Tail Multifamily Land Development, LLC
2082 Michelson, 4th Floor
Irvine, California 92612

Attention: Mr. Brent Little

Subject: Geotechnical Design and Earthwork Recommendations
Proposed Mixed Use Community Development
Victoria Corporate Center Phase II/III
2879 Seaborg Avenue
City of Ventura, California

In accordance with our proposal dated April 15, 2022, we present this report of our geotechnical recommendations for the proposed mixed use community development located at 2879 Seaborg Avenue in the City of Ventura, California. The subject site is northeast of the intersection of S Victoria Avenue and Olivas Park Drive.

Based on the site plan by Withee Malcom Architects (2022), Leighton understands Red Tail Multifamily Land Development, LLC (Red Tail) plans to construct 4-story multi-family residences along with commercial restaurant and small retail structures, a daycare facility, self-storage areas, a play area, a recreational vehicle storage area, and associated parking area. Based on the existing, nearly flat topography it is assumed that the proposed improvements would be constructed at or near existing grade.

The purpose of this report is to summarize the geologic and geotechnical conditions at the site based on review of publicly available documents, plans provided by Red Tail, and field observations made during geotechnical explorations. Based on our review of available data, the site is underlain by alluvial fan deposits of latest Holocene age. The site is not located within an Alquist-Priolo Earthquake Fault Zone or an area mapped by the State of California as having potential for earthquake induced landsliding or liquefaction. The surrounding area is mapped in liquefaction zone and historically groundwater is mapped at shallow depths. Therefore, liquefaction was evaluated for this site as part of this study. While significant ground shaking should be anticipated at the site during the expected life of the proposed structures, standard design practices will mitigate such shaking.

Herein, we summarize our conclusions based on the field observations during exploration and review of existing geotechnical reports for the subject site. We conclude the site is considered feasible for construction of the proposed development from a geotechnical perspective; provided the recommendations for earthwork construction are incorporated in foundation design and carried out during grading.

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

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

Robert Hennessey, PE
Associate Engineer
Ext. 3023; rhennessey@leightongroup.com

Geoffrey Faneros, PG, CEG
Senior Project Geologist
Ext. 3021, gfaneros@leightongroup.com

RM/GF/RH/lr

Distribution: (1) addressee (via email)

TABLE OF CONTENTS

Section **Page**

1.0 BACKGROUND ... 1

2.0 SUMMARY OF FIELD OBSERVATIONS ... 2

3.0 FIELD EXPLORATION AND LABORATORY TESTING ... 3

3.1 Field Exploration ... 3
3.2 Laboratory Testing ... 3

4.0 GEOTECHNICAL FINDINGS ... 5

4.1 Regional Geology ... 5
4.2 Site Conditions ... 5
4.3 Groundwater ... 6
4.4 Surface Fault Rupture ... 6
4.5 Ground Shaking - 2019 CBC Site-Specific Seismic Coefficients ... 7
4.6 Secondary Seismic Hazards ... 9
4.7 Seismically Induced Settlement ... 9
4.8 Seiches and Tsunamis ... 9
4.9 Flooding ... 10
4.10 Earthquake-Induced Flooding ... 10
4.11 Expansion ... 10
4.12 Erosion ... 11

5.0 CONCLUSIONS AND RECOMMENDATIONS ... 12

5.1 General ... 12
5.2 Grading Recommendations ... 13
5.3 Foundation Design Parameters ... 14
5.4 Settlement ... 15
5.5 Lateral Earth Pressures and Retaining Wall Design Considerations ... 15
5.6 Concrete Flatwork ... 17
5.7 Trench Backfill ... 17
5.8 Preliminary Hot Mix Asphalt (HMA) Pavement Sections ... 18
5.9 Final Asphalt Cap ... 19
5.10 Surface Drainage ... 19
5.11 Infiltration ... 20
5.12 Continued Geotechnical Services ... 21

6.0 LIMITATIONS ... 22

ATTACHMENTS

Figure 1 – Site Location Map
Figure 2 – Regional Geology Map
Figure 3 – Regional Faulting and Historical Seismicity Map

Figure 4 – Seismic Hazard Zone Map
Figure 5 – Flood Hazard Zone Map
Figure 6 – Dam Inundation Map
Figure 7 – Retaining Wall Recommendations

Plate 1 – Geotechnical Map

Appendix A – References
Appendix B – Exploration Logs
Appendix C – Laboratory Test Results
Appendix D – Liquefaction Analysis
Appendix E – Infiltration Testing
Appendix F – Utility Density Tests by Gorian and Associates, Inc.
Appendix G – General Earthwork and Grading Specifications
Appendix H – GBA Important Information About This Geotechnical Engineering Report

1.0 BACKGROUND

The proposed mixed use development is located at 2879 Seaborg Avenue, northeast of the intersection of Victoria Avenue and Olivas Park Drive in the City of Ventura, California. The site location and immediate vicinity are shown on Figure 1, *Site Location Map*. The site is bound by Victoria Avenue to the west, Olivas Park Road to the south, Seaborg Avenue to the east, and the Phase 1 Corporate Center to the north.

Based on review of historic aerial photographs, the site was undeveloped land utilized for row crop agriculture, but otherwise no development, structures, or pavements are located within the site. Minor grading has occurred at the site associated with the past agricultural activities and the grading and installation of wet and dry utilities as documented by Gorian and Associates, Inc. (Gorian 2018). There are reportedly previous agricultural buildings/structures that were located at areas adjacent to Olivas Park Drive (Gorian, 2003).

The currently proposed development for the site is 4-story multi-family residences along with commercial restaurant and small retail structures, a daycare facility, self-storage areas, a play area, a recreational vehicle storage area, and associated parking area. Based on the existing, nearly flat topography it is assumed that the proposed improvements would be constructed at or near existing grade.

2.0 SUMMARY OF FIELD OBSERVATIONS

The site location is 2879 Seaborg Avenue. The property is approximately 13.5± acres in area and was previously row crop farmland with farm-related structures adjacent to Olivas Park Road. The site is currently vacant. There are no structures, pavements or other hardscape improvements within the site boundaries. North-south overhead utility lines are within the east portion of the site.

The ground surface at the project site overall very gently decreases in elevation from north to south (towards Olivas Park Rd). The topography suggests the existing grade elevations range from about 77 feet to 65 feet mean sea level (msl) from north to south, respectively.

Based on our review of available data, the site is underlain by alluvial fan deposits of latest Holocene age. The site is not located within an Alquist-Priolo Earthquake Fault Zone or a liquefaction hazard zone as defined by the California Geological Survey (CGS). While significant ground shaking should be anticipated at the site during the expected life of the proposed structures, standard design practices will mitigate such shaking.

3.0 FIELD EXPLORATION AND LABORATORY TESTING

3.1 Field Exploration

On July 12, 2022, Leighton and Associates, Inc. performed subsurface field exploration at the site that consisted of six (6) hollow-stem auger borings (LB-1 through LB-6) and one infiltration test at LB-6. The borings were excavated to depths of 5 to 50 feet below existing ground surface (bgs). The percolation test was completed on July 13, 2022. Descriptions of the earth materials encountered during our field exploration are presented on the exploration logs included in Appendix B and the infiltration test results are in Appendix E.

Prior to the field exploration, the area of our borings were marked, and Underground Service Alert (USA) was notified for utility clearance. During excavation of our borings, relative undisturbed and bulk samples were obtained for geotechnical laboratory testing. Each boring was logged in the field by an engineer from our technical staff under supervision of a State-certified engineering geologist and Professional Engineer. Collected soil samples were reviewed in the field and described in accordance with the Unified Soil Classification System (USCS).

Collected samples were sealed and packaged for transportation to our laboratory. After completion of excavation, the borings were backfilled with bentonite grout.

The approximate locations of the explorations are shown on Plate 1, *Geotechnical Map*.

3.2 Laboratory Testing

Laboratory testing of representative soil samples from this exploration are included in Appendix C, Laboratory Test Results. The laboratory testing was performed to evaluate the general engineering characteristics of the near-surface onsite soils. Geotechnical laboratory testing included:

- Maximum dry density and optimum moisture (ASTM D1557);
- Moisture and Density (ASTM D2216 and D2937);
- Percent Passing No. 200 Sieve, (ASTM D 1140);
- Direct Shear (Consolidated Undrained).

- Sulfate Content (DOT California Test 417).
- Chloride Content (DOT California Test 422).
- pH Test (DOT California Test 643).
- Soil Resistivity (DOT California Test 643).
- Expansion Index (ASTM D4829).
- Atterberg Limits (ASTM D 4318)

4.0 GEOTECHNICAL FINDINGS

4.1 Regional Geology

The subject site is near the northeast edge of the Oxnard Plain, a nearly flat lying portion of the Ventura Basin. The basin is a broad, east-west trending downwarp that has been collecting sediments for the past 65 million years. The basin forms a part of the Transverse Ranges geomorphic province that is a belt of east-west trending geologic structures stretching from offshore at Point Conception to the San Andreas fault 150 miles to the east. The folds and faults of the Transverse ranges are responsible for the uplift of the Santa Monica Mountains to the east and the Santa Ynez and Topatopa Mountains to the north. Active tectonic movements, especially faulting, constitute one of the primary geologic hazards of the region.

The Ventura Basin is filled by several tens of thousands of feet of Miocene age and younger (less than 25 million years) sediments deposited at a time when the relative sea level was higher than today and shallow marine conditions existed farther to the east of the present-day Ventura County beaches. On top of its thick section of marine deposits, a layer of deltaic sediments of the Saugus and San Pedro Formations derived from the rising mountains to the east was laid down. Deposition then changed to an alluvial and floodplain system during the Quaternary time (less than 1.8 million years old) as the sea retreated westward.

4.2 Site Conditions

Minor grading has occurred at the site associated with past agricultural activities and the grading and installation of wet and dry utilities (Gorian 2018). Plate 1, *Geotechnical Map*, presents the utility as-graded areas, excavation bottom elevations, and the approximate locations of field density tests. The density test tables by Gorian (2018) are provided in Appendix F.

The site is relatively flat and covered in grass and weeds that were recently mowed. There are no structures, pavements or other hardscape improvements within the site boundaries. North-south overhead utility lines are within the east portion of the site.

Leighton performed a subsurface field exploration at the site on July 12, 2022 that consisted of six (6) hollow-stem auger borings (LB-1 through LB-6). The borings were advanced to depths ranging from 5 to 50 feet, at locations within the footprints of the proposed buildings and parking areas, as shown on Plate 1, *Geotechnical*

Map. Based upon our review of geotechnical literature (Appendix A) and our subsurface exploration (Appendix B), the site is underlain by alluvial fan deposits of latest Holocene age. Descriptions of the earth materials encountered during our field exploration are summarized below and detailed descriptions are presented on the exploration logs included in Appendix B.

Undocumented Artificial Fill (Map Symbol af): Undocumented fill was previously mapped by Gorian (2003) and is associated with past agricultural buildings/structures and construction access to Olivas Park Drive. The depth of the materials is unknown but anticipated to be relatively thin. There may be septic or other sewage disposal systems within the areas.

Compact Artificial Fill (Map Symbol caf): Compact artificial fill is within the areas of the wet and dry utility installations as documented by Gorian (2018).

Holocene-age Alluvial Fan Deposits (Map Symbol Qhf): Latest Holocene-age alluvial fan deposits were encountered in all of the hollow-stem auger borings. The alluvial soils encountered are interbedded very loose to medium dense silty sands and soft to stiff clay and sandy clay.

4.3 Groundwater

Groundwater was not encountered during the site exploration to the maximum depth of 51.5 feet at boring LB-2. However, perched groundwater was observed in sandy materials overlying stiff clay at 15 feet below ground surface. A seismic hazard report by the California Geological Survey (2002a) indicates that historic groundwater beneath the site has been about 15 feet in depth.

4.4 Surface Fault Rupture

No active or potentially active faults have been previously mapped across the project site (Bryant and Hart, 2007), nor is the site located within a current Alquist-Priolo Earthquake Fault Zone (CGS, 2002a).

The closest mapped potentially active fault that could affect the site through ground shaking is the Oak Ridge fault, located approximately 4,490 feet northwest of the subject site. The Oak Ridge fault is a northeast-southwest trending, south-dipping reverse fault zone that nearly parallels the Santa Clara River from the town of Piru to the offshore area of the Santa Barbara Channel (Yeats, 1989; CSG, 1990; Fisher, 2005). Within the Santa Clara River Valley, river alluvium and landslide deposits largely conceal the fault trace. There are few surface exposures. The known fault locations are interpreted mainly from subsurface (well) data; inferred surficial locations are primarily from geomorphic features such as lineaments, faceted spurs, and offset landslide deposits. The fault trace would probably be visible between Saticoy and Santa Paula, but fluvial processes in the Santa Clara River have obscured its trace (CGS, 1990).

The most recent rupture along the fault is mainly late Quaternary, and therefore defined by the state geologist as potentially active. Splays of the fault observed at the Bardsdale Cemetery (about 17.5 miles northeast of the project site) and in the offshore portion of the fault zone are interpreted as having offset during the Holocene, and therefore, considered active (Yeats, 1988; CGS, 1990; Fisher, 2005).

Additional active faults close to the project site were evaluated using the United States Geological Survey (USGS) Earthquake Hazards Program National Seismic Hazard Maps (USGS, 2008c). In addition to the Oak Ridge fault, the closest active faults to the site with the potential for surface fault rupture include the Ventura-Pitas Point fault, the Wright fault, and the Simi-Santa Rosa fault, located approximately 2.4 miles, 6 miles, and 6.6 miles, respectively. The San Andreas fault, which is the largest active fault in California, is approximately 42 miles northeast of the site.

Based on the absence of faults known or mapped across the site, the potential for fault ground rupture at the site is considered low. Major regional faults with surface expression in proximity to the site are shown on Figure 3, *Regional Fault and Historic Seismicity Map*.

4.5 Ground Shaking - 2019 CBC Site-Specific Seismic Coefficients

The site is located within a seismically active region, as is all of Southern California in general. 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. Accordingly, the project should be designed in accordance with all applicable current codes and standards utilizing the appropriate seismic design parameters to reduce seismic risk as defined by California Geological Survey (CGS) Chapter 2 of Special Publication 117a (CGS, 2008). Through compliance with these regulatory requirements and the utilization of appropriate seismic design parameters selected by the design professionals, potential effects relating to seismic shaking can be reduced.

The following parameters should be considered for design under the 2019 CBC:

2019 CBC Parameters (CBC or ASCE 7-16 reference)	Value 2019 CBC
Site Latitude and Longitude: 34.24474°, -119.213367°	
Site Class Definition (1613.2.2, ASCE 7-16 Ch 20)	D
Mapped Spectral Response Acceleration at 0.2s Period (1613.2.1), S_s	1.935 g
Mapped Spectral Response Acceleration at 1s Period (1613.2.1), S_1	0.724 g
Short Period Site Coefficient at 0.2s Period (T1613.2.3(1)), F_a	1.0 g
Long Period Site Coefficient at 1s Period (T1613.2.3(2)), F_v	1.7* g
Adjusted Spectral Response Acceleration at 0.2s Period (1613.2.3), S_{MS}	1.935 g
Adjusted Spectral Response Acceleration at 1s Period (1613.2.3), S_{M1}	1.231* g
Design Spectral Response Acceleration at 0.2s Period (1613.2.4), S_{DS}	1.290 g
Design Spectral Response Acceleration at 1s Period (1613.2.4), S_{D1}	0.821* g
Mapped MCE_G peak ground acceleration (11.8.3.2, Fig 22-9 to 13), **PGA**	0.852 g
Site Coefficient for Mapped MCE_G PGA (11.8.3.2), F_{PGA}	1.100
Site-Modified Peak Ground Acceleration (1803.5.12; 11.8.3.2), PGA_M	0.938 g

* Per Table 11.4-2 of Supplement 1 of ASCE 7-16, this value of F_v may only be used to calculate T_s [that note is not included in Table 1613A.2.3(2)]; note that S_{D1} and S_{M1} are functions of F_v. In addition, per Exception 2 of 11.4.8 of ASCE 7-16, special equations for C_s are required. This is in lieu of a site-specific ground motion hazard analysis per ASCE 7-16 Chapter 21.2.

** Site Class D, and all of the resulting parameters in this table, may only be used for structures without seismic isolation or seismic damping systems.

Based on the 2019 CBC Table 1613.2.3(2) footnote c., F_v should be determined in accordance with Section 11.4.8 of ASCE 7-16, since the mapped spectral response acceleration at 1 second is greater than 0.2g for Site Class D; in accordance with Section 11.4.8 of ASCE 7-16, a site-specific seismic analysis is required. However, the values provided in the table above may be utilized if design is performed in accordance with Exception (2) in Section 11.4.8 of ASCE 7-16, with special requirements for the seismic response coefficient (C_s), and F_v is only used for calculation of T_s. This exception does not apply (and the values in the table above would not be applicable) for proposed with seismic isolation or seismic damping systems. The project structural engineer should review the seismic parameters. A site-specific seismic ground motion analysis can be performed upon request.

Hazard deaggregation was estimated using the USGS Interactive Deaggregations utility. The results of this analysis indicate that the predominant modal earthquake has a magnitude of approximately 7.53 (M_W) at a distance on the order of 5.71 kilometers for the Maximum Considered Earthquake (2% probability of exceedance in 50 years).

4.6 Secondary Seismic Hazards

Ground shaking can induce “secondary” seismic hazards such as liquefaction, dynamic densification, and differential subsidence along ground fissures. The site is **not** mapped within a state designated Liquefaction hazard zone as shown on Figure 4, *Seismic Hazard Map*. However, the area surrounding the site is mapped in a liquefaction zone and historically there is shallow groundwater at this site. Therefore, liquefaction analysis was performed for the site.

Perched groundwater was encountered at a depth of approximately 15 feet within the sand materials overlying stiff clay. Groundwater was not encountered within the clay strara or in sands below the clays. Historical high groundwater is documented at a depth of about 15 feet below ground surface (CGS, 2002a).

4.7 Seismically Induced Settlement

Strong ground motion during earthquakes tends to rearrange looser soils particles into a more compact arrangement, especially in granular soil deposits. The cumulative effects of soil particles rearrangement during earthquake ground shaking will result in settlement. In general, a poorly graded granular deposit is more susceptible to settlement than a fine-grained or well-graded soil. Liquefiable sands below the clay layers were identified at a depth of about 30 and 45 feet.

The combined seismically induced settlement at the site due to dry dynamic settlement (soils above groundwater) and liquefaction settlement was calculated to be approximately 1.5 inches (Appendix D). The differential settlement may be assumed to be about 0.75 inches over a distance of 30 feet.

4.8 Seiches and Tsunamis

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. Based on the absence of an enclosed water body near the site, the risk from a seiche is considered negligible. According to the Tsunami Inundation Map for the Oxnard Quadrangle (CGS, 2009), the inland location of project side is not within a tsunami inundation area.

4.9 Flooding

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2010), the site is not located within a flood hazard area (Figure 5, *Flood Hazard Zone Map*).

4.10 Earthquake-Induced Flooding

Earthquake-induced flooding can be caused by failure of dams or other water-retaining structures as a result of earthquakes. Based on our review of available information, the southern portion of the project site **is** located within known dam inundation zones during hypothetical failure of the Castaic Dam (DWR, 2018a) and Pyramid Dam (DWR 2018b). Refer to Figure 6, *Dam Inundation Map*. Details include the following:

- **Castaic Dam –** Maximum inundation depth of less than or equal to 2 feet to 10 feet, with a flood wave arrival time of 4 hours and 31 minutes.
- **Pyramid Dam -** Maximum inundation depth of less than or equal to 2 feet to 5 feet, with a flood wave arrival time of 5 hours and 30 minutes.

4.11 Expansion

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

The surficial soils at the site were anticipated to have a medium expansion potential based on findings from a previous report (Gorian, 2003). Expansion testing of samples obtained from this study indicate medium to high expansion. The high expansion range is above 91, therefore the one samples with a result of 96 is on the lower end of the high expansion range. As such we recommended that medium expansion as the basis of the design for this site. The expansion index results conducted on the soil samples and classification are summarized in the following table. Expansion test results are attached in Appendix B.

Sample No.	Boring	Soil Type	Expansion Index (EI)	Classification	Reference
LB-1	B-1	Olive lean clay (CL)	83	Medium	Leighton, this report
LB-5	B-1	Brown lean clay (CL)	96	High	Leighton, this report
B-2	@ 0 – 1'	Grayish brown sandy silty clay	75	Medium	Gorian, 2003

Expansion Index should be verified for each building pad upon completion of rough grading.

4.12 Erosion

The site is not considered highly subject to mechanical erosion, runoff, and sedimentation due to the cohesive nature of the site soils and relatively flat site. Based on proposed erosion control measures and surface drainage improvements upon completion of the project, the loss of site materials due to erosion (water and wind) is considered low.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 General

Based on our review of the preliminary plans provided by you and the results of our field investigation performed for this report, the proposed multiuse commercial and residential development is considered feasible from a geotechnical viewpoint, provided the recommendations presented in this report are implemented and confirmed prior to construction. A summary of our main findings and conclusions are as follows:

- Observation of our geotechnical explorations indicate soils are characterized as medium to high expansion fine grained sandy clay throughout the property.
- Compacted documented fill as shown on Plate 1 is located throughout the property. The fill was placed over utilities lines previously installed onsite. The fill was placed under the observation and testing by Gorian (2018). Provided the utilities lines do not need to be moved this compacted fill can remain in place.
- The site is very densely covered in vegetation and has been used for agricultural purposes in the past. In order to create a suitable uniform bearing surface for support of new construction the upper five feet of soil should be overexcavated and recompacted.
- Perched groundwater was encountered at a depth of approximately 15 feet. The perched groundwater was observed in the sandy soil layers overlying stiff clay. Groundwater was not encountered within or below the clay layer to the depth explored of 51.5 feet. Groundwater is not anticipated to pose a constraint to construction.
- The site is **not** located within an Alquist-Priolo Earthquake Fault Hazard Zone and no active faults are known to cross the site.
- The site is **not** mapped within a landslide or liquefaction by the State of California. However due to historic high groundwater and perched groundwater at the site, liquefaction was analyzed for the site.
- Seismic induced settlement is anticipated to be approximately 1.5 inches. Differential settlement is estimated to be 0.75 inches over a distance of 30 feet.

- Leighton completed infiltration testing at LB-6 within at a depth of 5-feet and measured an infiltration rate of 1.1 in/hr. In accordance with Ventura County Standards, we recommend a site suitability factor of safety of 1.25 be incorporated with this infiltration rate.

- Near surface soils range from medium to high expansion potential. Medium to high expansive soils tend to be difficult to either dry or moisten during grading operations. In addition, when placing the onsite soils as compacted fill the moisture should be 2 percentage points above optimum.

- Typical continuous foundations should be designed in accordance with Table 1809.7 of the Ventura Building Code considering medium expansion (51<EI<91).

5.2 Grading Recommendations

The site is covered in dense vegetation and prior land use has disturbed the surficial soils. As such, the following recommendations should be anticipated for development of this site:

- All undocumented fill, vegetation, organic material or deleterious debris should be removed from the site prior to placement of any fill.

- In order to create a suitable uniform bearing surface for new construction, the upper five feet of soil should be overexcavated and recompacted.

- After soils are excavated to a depth of five feet, in-place alluvial soils, exposed in the base of the excavation, shall be deemed suitable for the addition of structural compacted fill if found to have a minimum 90 percent relative compaction (ASTM Test Method D1557). Suitability of all removal bottoms should be reviewed and evaluated by an engineering geologist or a representative of the geotechnical engineer. Additional overexcavation may be required if the exposed soils are found to not be suitable for support of new fill or the proposed development.

- After removal of soils and geotechnical acceptance of the subgrade, the exposed surfaces of the overexcavated areas should be scarified to a minimum depth of 6 inches, moisture conditioned as needed and mechanically compacted.

- The recommended overexcavation and recompaction will result in shrinkage of the existing site soils. For the purposes of earthwork estimates and budgeting, 10 percent to 15 percent can be used to estimate shrinkage.

Fill Placement and Compaction: Onsite soil may be used for compacted structural fill provided it is free of debris and organic material. Any soil to be placed as fill, whether onsite or imported material, should be reviewed and tested by Leighton as needed or required. All fill soil should be placed in thin, loose lifts, moisture conditioned, as necessary, and compacted to a minimum 90 percent relative compaction. Relative compaction should be determined in accordance with ASTM Test Method D1557. Aggregate base for pavement should be compacted to a minimum of 95 percent relative compaction.

5.3 Foundation Design Parameters

The proposed commercial and multi-family residential structures ranging from 3- to 4-stories may be founded on conventional continuous foundations based on the design parameters provided below. The proposed foundations and slabs should be designed in accordance with the structural consultants' design, the minimum geotechnical recommendations presented herein, and the applicable Ventura Building Code. In utilizing the minimum geotechnical foundation recommendations, the structural consultant should design the foundation system to acceptable deflection criteria as determined by the architect. Based on the expansion index testing performed for this study, we recommend that foundations be designed for medium expansion (51<EI<91) and Table 1809.7 of the Ventura County Building Code. The following parameters can be used for foundation design:

- Allowable vertical bearing pressure: 2,000 psf (pounds per square foot) for a minimum 24 inches embedment into compacted fill and a minimum footing width 18 inches. These allowable bearing values may be increased by 350 psf per foot increase in embedment depth and/or width to a maximum allowable bearing pressure of 3,500 psf, and are for total dead load and sustained live loads, which can be increased by one third when considering short-duration wind or seismic loads. Footing reinforcement should be designed by the project Structural Engineer. The bearing pressure may be increased by one-third for transient or temporary loads (e.g., seismic, wind).

- Lateral bearing pressure: 200 psf/foot per foot of depth and embedment to a maximum of 2,000 psf (a factor of safety of 1.5 has been applied).

- Sliding Coefficient: A sliding coefficient of 0.3 may be used for soil to structural concrete interface.
- Modulus of Subgrade Reaction: 120 pci

The footing width, depth, reinforcement, slab reinforcement, and the slab-on-grade thickness should be designed by the structural consultant based on recommendations and soil characteristics indicated herein. If exterior footings are within 5 feet horizontally of side yard swales, the footing should be embedded sufficiently to ensure embedment below the swale bottom is maintained.

5.4 Settlement

The above recommended allowable bearing capacity is generally based on a total allowable, post-construction static settlement of 1 inch for dead plus sustained live loads less-than-or-equal-to 3 kips-per-foot of wall (**not** over undocumented fill). For compacted fill thickness less-than-or-equal-to (≤) 2 feet below footings, differential settlement due to static loading is estimated at ½-inch over a horizontal distance of 30 feet. Since settlement is a function of footing sustained load, size and contact bearing pressure, differential settlement can be expected between adjacent dissimilarly loaded walls where a large differential loading condition exists. These settlement estimates can be reevaluated by Leighton Consulting, Inc. when foundation plans and actual loads for these proposed improvements become available.

5.5 Lateral Earth Pressures and Retaining Wall Design Considerations

Retaining walls should be designed for lateral earth pressures exerted on them. The magnitude of these pressures depends on the amount of deformation that the wall can yield under load. If the wall can yield enough to mobilize the full shear strength of the soil, it can be designed for "active" pressure. If the wall cannot yield under the applied load, the shear strength of the soil cannot be mobilized and the earth pressure will be higher. Such walls should be designed for "at rest" conditions. If a structure moves toward the soils, the resulting resistance developed by the soil is the "passive" resistance.

Currently no retaining walls are planned for the project. The recommendations contained her in are for you information purposes and in case retaining walls are added in the future. Walls should not be backfilled with the medium expansive onsite soils.

For design purposes, the recommended equivalent fluid pressures for walls backfilled with soils of very low expansion potential, and free draining conditions are provided in the table below (Lateral Earth Pressures). If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical engineer. Surcharge loading effects from the adjacent structures should be evaluated by the geotechnical and structural engineer.

Conditions	Equivalent Fluid Weight[1] (pcf)	
	Level Backfill	**2:1 Slope Backfill**
Active	36	60
At-Rest	55	85
Passive[2]	330	150 (Sloping Down 2:1)

Notes:
[1]Assumes drained condition (See Figure 7)
[2] Maximum passive pressure is 2000 psf for level surface in front of the wall

Retaining walls should be founded on compacted fill. Foundations may be designed in accordance with the recommendations presented in Section 5.3 and passive resistance parameters in the table above. In combining the total lateral resistance, the passive pressure or the frictional resistance should be reduced by 50 percent. The passive resistance value may be increased by one-third when considering loads of short duration, including wind or seismic loads. The horizontal distance between foundation elements providing passive resistance should be a minimum of three times the depth of the elements to allow full development of these passive pressures. The total depth of retained earth for design of cantilever walls should be the vertical distance below the ground surface measured at the wall face for stem design or measured at the heel of the footing for overturning and sliding.

Backfill soils should be compacted to at least 90 percent relative compaction (based on ASTM Test Method D1557) and should extend horizontally to a minimum distance equal to one-half the wall height behind the walls.

5.6 Concrete Flatwork

Sidewalks/flatwork should conform to Ventura County standards for medium expansive soils. A representative of Leighton should verify subgrade soil expansion, moisture conditions and compaction prior to formwork and reinforcement placement. We recommend a minimum 8-inch deepened edge be constructed for all flatwork to reduce moisture variation in subgrade soils along concrete edges adjacent to open (unfinished) or irrigated landscape areas.

In order to reduce the potential for cracking and potential differential movement of driveways, sidewalks, patios, or other concrete flatwork, welded wire mesh reinforcement consisting of 6x6-w1.4 x w1.4 or No. 3 rebar at 24 inches on center (each way) is suggested, along with keeping subgrade soils at an elevated moisture content of at least 130 percent of optimum prior to placement of concrete.

Exterior concrete driveways, patio slabs, and swimming pool decks, often crack. Inclusion of joints at frequent intervals and reinforcement will help control the locations of the cracks, and thus reduce the unsightly appearance. Construction or weakened plane joints should be spaced at intervals of 8 feet or less for driveways, ramps, sidewalks, patio slabs, pool decks, curbs and gutters. If cracking occurs, repairs may be needed to mitigate the trip hazard and/or improve the appearance. Load transfer devices, such as dowels or keys, are recommended at joints in the paving to reduce possible offsets. Steel reinforcing may be added to the paving to reduce cracking and to prolong the life of the paving.

5.7 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”), 2021 Edition. Utility trenches may be backfilled with onsite material, provided it is free of rubble, debris, organic and oversized material up to 3 inches in largest dimension. Backfill in and above the pipe zone should be as follows:

- *Pipe Zone:* The proposed pipe should be placed on properly placed bedding materials. Pipe bedding should extend to a depth in accordance with the pipe manufacturer’s specification. The pipe bedding should extend to at least 1 foot over the top of the conduit. We recommend that the shading sand have an average sand equivalence greater than 30 per the Standard Specifications for Public Works Construction (Greenbook). Soil samples recovered from the exploratory borings were tested; the results of the testing indicate that the onsite sandy soils in the area of the waterline pipe installation are not suitable for use as bedding material (SE<30 and percent passing No. 200 > 10%).

- *Over Pipe Zone:* Above the pipe zone, trenches can be backfilled with excavated on-site soils free of debris, organic and oversized material larger than 4 inches in largest dimension. As an option, the whole or part of the trench can be backfilled with 1 1/2-sack CLSM. Oversized rock (concrete debris, cobbles and/or boulders) should either be removed from any backfill, or pulverized for use in backfill only above the pipe zone. Soil backfill over the pipe-bedding zone should be placed in thin lifts, moisture conditioned, as necessary, and mechanically compacted using a minimum standard of 95% relative compaction relative to the ASTM D 1557 laboratory maximum dry density within the right-of-way and beneath pavements. Backfill above the pipe zone (bedding) can be jetted. Backfill above the pipe zone (bedding) should be observed and tested by Leighton.

5.8 Preliminary Hot Mix Asphalt (HMA) Pavement Sections

For preliminary planning purposes, previous R-value tests of near surface soils indicated an R-value of 6, 8 and 9 (Gorian, 2008). Soils will vary throughout the site and change after rough grading earthwork is complete. In order to represent the potential post grading condition of the site roadways, a conservative R-value of 6 was selected for the preliminary pavement design calculation. Based on design procedures outlined in the current Caltrans Highway Design Manual, using an R-value of 78 for Class 2 aggregate base or crushed aggregate base course, the preliminary flexible pavement sections may consist of the following sections for Traffic Indices approved by the city.

Preliminary Asphalt Pavement Sections

Traffic Index	Asphalt Concrete (inches)	Aggregate Base (inches)
5.0	4.0	7.5
6.0	4.0	11.5
7.0	4.0	15.5

All pavement construction should be performed in accordance with the Standard Specifications for Public Works Construction (2021). 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 12 inches, moisture-conditioned, as necessary, and recompacted to a minimum of 95 percent relative compaction and kept in this condition until the pavement section is constructed.

If pavement areas are adjacent to heavily watered landscape areas, some deterioration of the subgrade load bearing capacity may result. Moisture control measures such as deepened curbs or other moisture barrier materials may be used to prevent the subgrade soils from becoming saturated. *The use of concrete cutoff or edge barriers should be considered when pavement is planned adjacent to either open (unfinished) or irrigated landscaped areas.*

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 (2021).

5.9 Final Asphalt Cap

If asphalt concrete pavement is being constructed directly upon an existing hard surfaced pavement after major construction is completed, then a tack coat should be uniformly applied in accordance with Section 302-5.3 of the 2018 *Greenbook*. Lateral cracks should be sealed. The surface should be free of water, foreign material or dust when the tack coat is applied. The contact surfaces of all cold pavement joints, curbs, gutters, manholes and the like should be painted with emulsified asphalt or paving asphalt, in accordance with Section 302-5.3 before the adjoining asphalt concrete is placed.

5.10 Surface Drainage

Inadequate control of runoff water and/or poorly controlled irrigation can lead to settlement of foundations, flatwork, walls, and other improvements. Maintaining adequate surface drainage, proper disposal of runoff water, and control of irrigation should help reduce the potential for future soil moisture problems.

Positive surface drainage should be designed in accordance with the current building code (CBC, 2019) to be directed away from foundations and toward approved drainage devices, such as gutters, paved drainage swales, or watertight area drains and collector pipes. Surface drainage should be provided to prevent ponding of water adjacent to the structures. In general, the area around the buildings should slope away from the building. We recommend that unpaved landscaped areas adjacent to the buildings be avoided. Roof runoff should be carried to suitable drainage outlets by watertight drain pipes.

5.11 Infiltration

A small diameter boring infiltration test was performed in accordance with the Ventura County Stormwater Quality Management guidelines (VCS, 2011) at a depth of approximately 5-feet below the existing ground surface in LB-6. Infiltration rates measured on July 14, 2022 were as follows:

Table 1. Small Diameter Boring Infiltration Test Results

Infiltration Test Number	Test Elevation (feet)	Infiltration Rate (inches/hour)	Native Materials
LB-6	73	1.1	CL - Sandy Clay with Silt

Our infiltration testing data is included in Appendix E of this report. We recommend that a site suitability factor of safety of 1.25 be incorporated in design of infiltration devises. The project civil engineer should also incorporate the design factor of safety in accordance with the Ventura County Stormwater Quality Management guidelines.

It should also be noted that during periods of prolonged precipitation, underlying soils tend to become saturated to increased depths/extents. Therefore, infiltration rates tend to decrease with prolonged rainfall. Periodic flows carrying silty sediments into dry wells or infiltration devices will eventually cause devices to accumulate a layer of silt, which has the potential for significantly reducing overall infiltration rate of these devices. Therefore, as a part of infiltration device maintenance, we recommend that accumulated silt soil be removed from infiltration devices by flushing and/or backwash. Stormwater infiltration should be designed in accordance with the Ventura County Technical Guidance Manual for Stormwater Quality Control Measures (VCS, 2011).

5.12 Continued Geotechnical Services

Our geotechnical conclusions and recommendations are contingent upon Leighton and Associates, Inc., providing geotechnical services during future design, earthwork and foundation construction so that the anticipated subsurface conditions can be confirmed, or such that revised conclusions and recommendations can then be made. If Leighton and Associates, Inc. is not retained during future work, then we are not liable should differing conditions become exposed once larger areas of the subsurface are excavated than the small locations excavated in the test pits of this report.

Leighton and Associates, Inc. should review site foundation, retaining wall and landscape plans when available, to comment further on geotechnical aspects of this project and check to see general conformance of final project plans to recommendations presented in this and future design level reports, or provide additional recommendations as considered necessary in accordance with current California Building Code requirements.

Geotechnical observation and testing should be provided during:

- Preparation of the residential building pads;
- Preparation of subgrade in all areas to receive fill;
- Excavation and installation of foundations;
- During placement of asphalt and compaction of base support material;
- During paving and tack coat application;
- After excavation of all footings and prior to placement of steel or concrete to confirm the footings are founded in firm, compacted fill free of loose debris; and
- Utility trench backfilling and compaction.

6.0 LIMITATIONS

This report was necessarily based in part upon data obtained from a limited number of observances, site visits, soil samples, tests, analyses, histories of occurrences, spaced subsurface explorations and limited information on historical events and observations. Such information is necessarily incomplete. The nature of many sites is such that differing characteristics can be experienced within small distances and under various climatic conditions. Changes in subsurface conditions can and do occur over time. This investigation was performed with the understanding that the subject site is proposed for residential development. The client is referred to Appendix H regarding important information provided by the Geoprofessional Business Association (GBA) on geotechnical engineering studies and reports and their applicability.

This report was prepared for Red Tail Multifamily Land Development, LLC. (Client), based on their needs, directions, and requirements at the time of our investigation. This report is not authorized for use by and is not to be relied upon by any party except our Client, and its successors and assignees as owner of the property, with whom Leighton and Associates, Inc. has contracted for the work. Use of or reliance on this report by any other party is at that party's risk. Unauthorized use of or reliance on this report constitutes an agreement to defend and indemnify Leighton and Associates, Inc. from and against any liability which may arise as a result of such use or reliance, regardless of any fault, negligence, or strict liability of Leighton and Associates, Inc.

Project: 13582.001	Eng/Geol: RH/GF
Scale: 1 " = 2,000 '	Date: August 2022

Base Map: Bing Maps 2022

Author: (mmurphy)

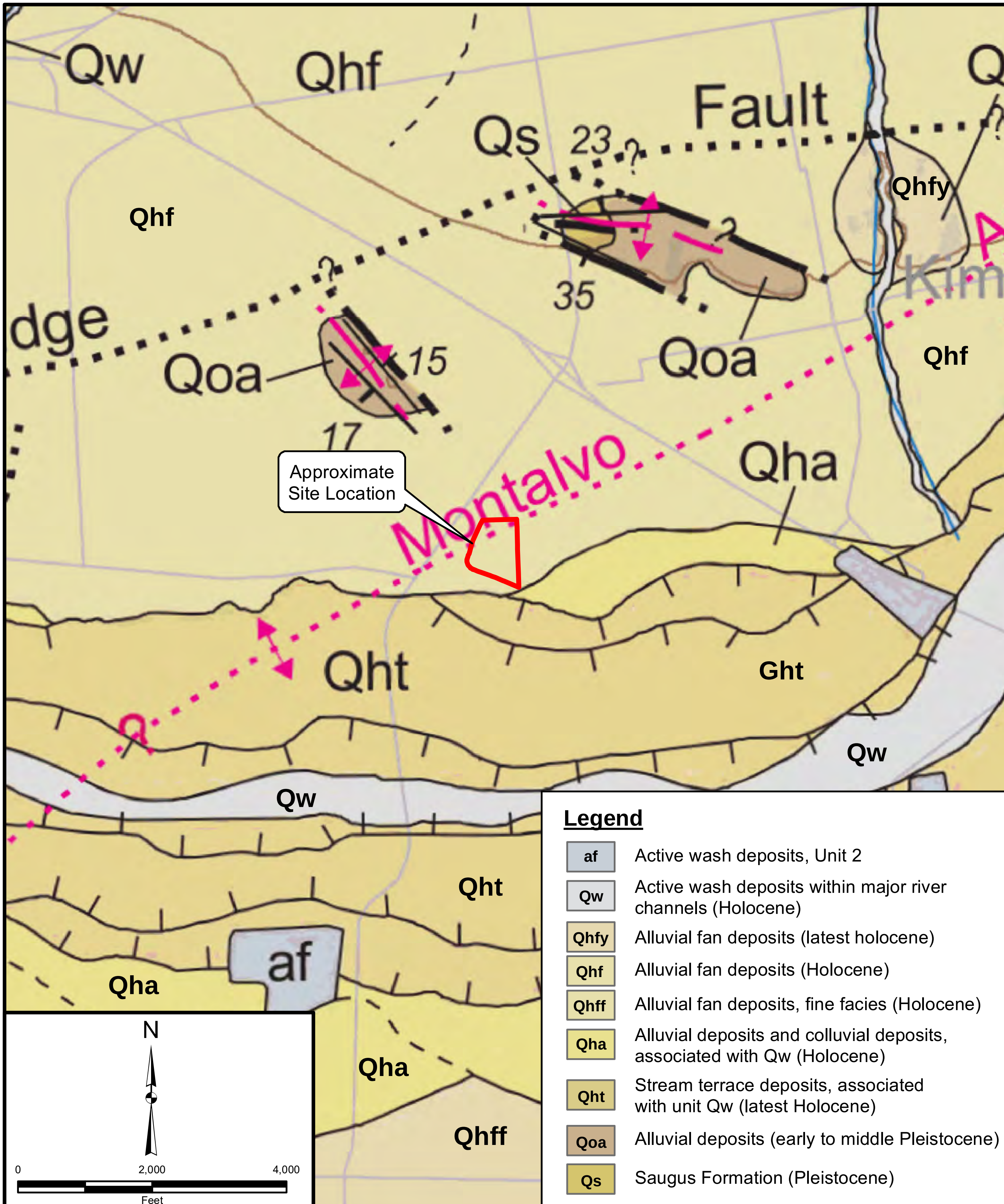
SITE LOCATION MAP

Victoria Corporate Center Phase II/III
2879 Seaborg Avenue
City of Ventura, California

FIGURE 1

Leighton

Map Saved as P:\Drafting\13582\001\Maps\13582-001_F01_SLM_2022-08-05.mxd on 8/5/2022 9:30:52 AM



Legend

af	Active wash deposits, Unit 2
Qw	Active wash deposits within major river channels (Holocene)
Qhfy	Alluvial fan deposits (latest holocene)
Qhf	Alluvial fan deposits (Holocene)
Qhff	Alluvial fan deposits, fine facies (Holocene)
Qha	Alluvial deposits and colluvial deposits, associated with Qw (Holocene)
Qht	Stream terrace deposits, associated with unit Qw (latest Holocene)
Qoa	Alluvial deposits (early to middle Pleistocene)
Qs	Saugus Formation (Pleistocene)

Project: 13582.001
Eng/Geol: RH/GF
Scale: 1 " = 2,000 '
Date: August 2022

Base Map: Gutierrez, C.I., Tan, S.S., and Clahan, K.B., 2008, Geologic Map of the East Half Santa Barbara 30' x 60' Quadrangle, California, California Geologic Survey, Scale 1:100,000.

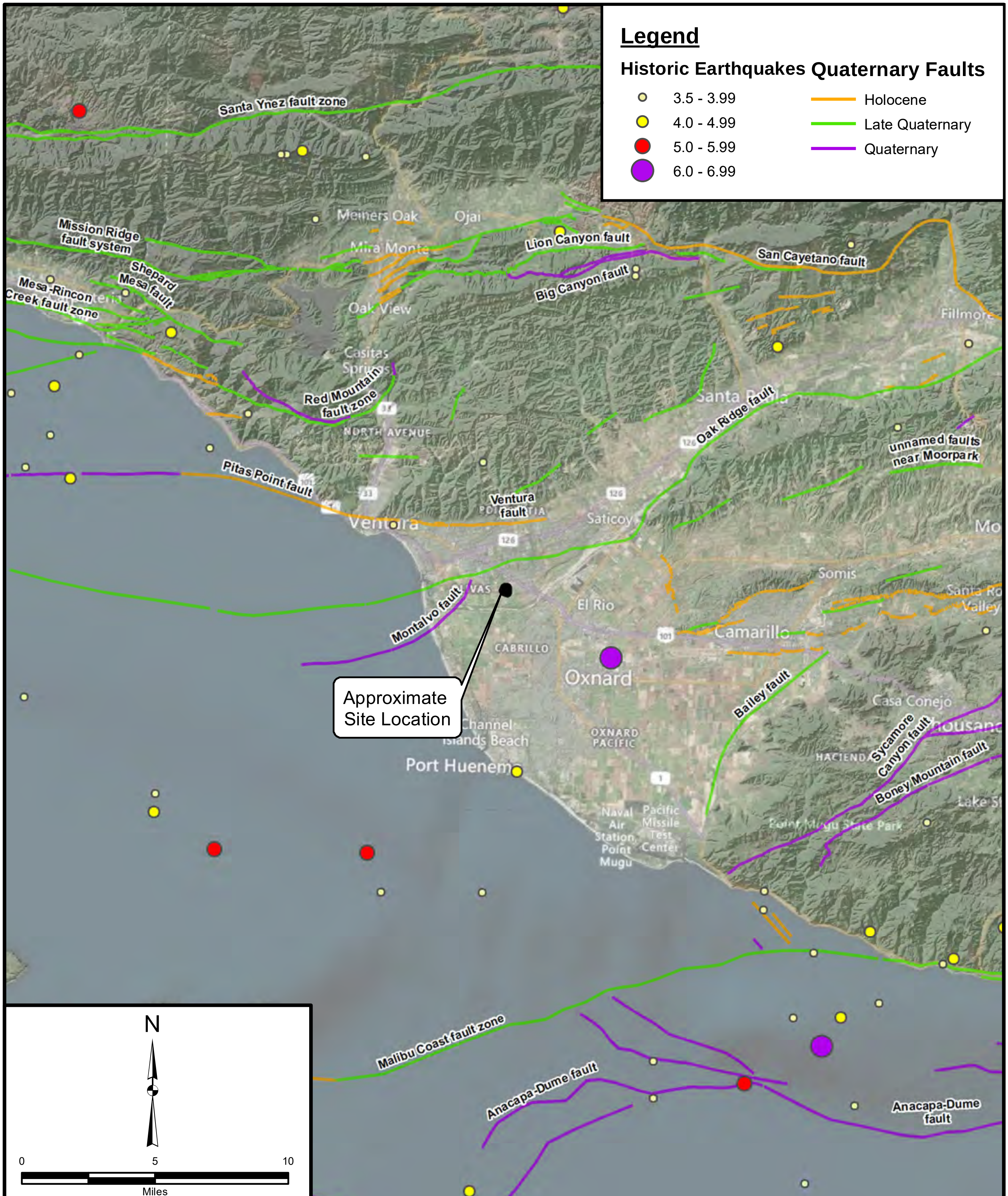
REGIONAL GEOLOGY MAP

Victoria Corporate Center Phase II/III
2879 Seaborg Avenue
City of Ventura, California

FIGURE 2

Leighton

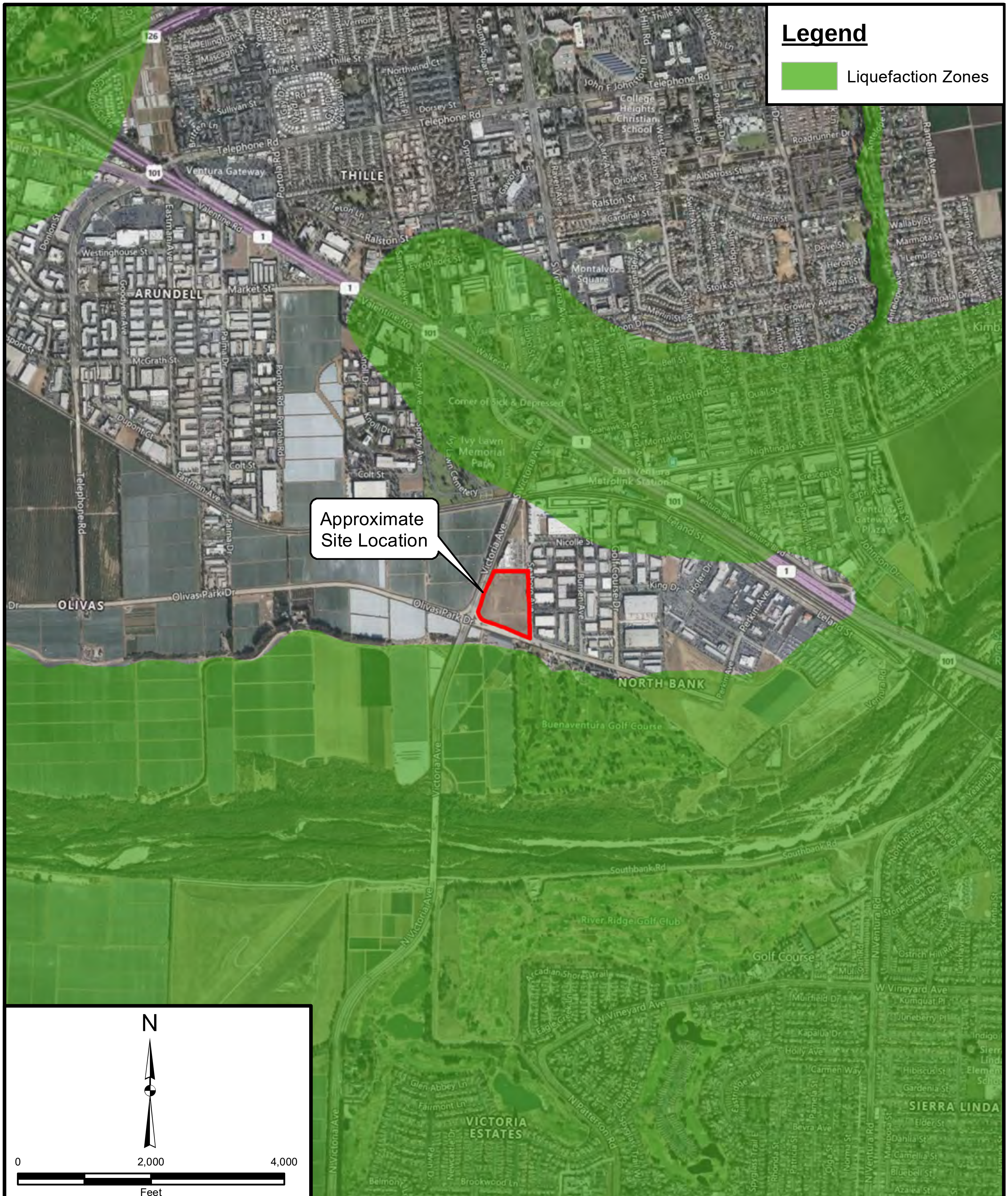
Map Saved as P:\Drafting\13582\001\Maps\13582-001_F02_RGM_2022-08-05.mxd on 8/5/2022 9:35:00 AM



Project: 13582.001	Eng/Geol: RH/GF	REGIONAL FAULT AND HISTORIC SEISMICITY MAP	FIGURE 3
Scale: 1 " = 5 miles	Date: August 2022	Victoria Corporate Center Phase II/III 2879 Seaborg Avenue City of Ventura, California	Leighton

Base Map: ESRI ArcGIS Online 2022
Reference: maps.conservation.ca.gov
Author: btran (mmurphy)

Map Saved as P:\Drafting\13582\001\Maps\13582-001_F03_RFHSM_2022-08-05.mxd on 8/5/2022 9:54:09 AM



Project: 13582.001	Eng/Geol: RH/GF
Scale: 1 " = 2,000 '	Date: August 2022

Base Map: ESRI ArcGIS Online 2022
Reference: maps.conservation.ca.gov
Author: btran (mmurphy)

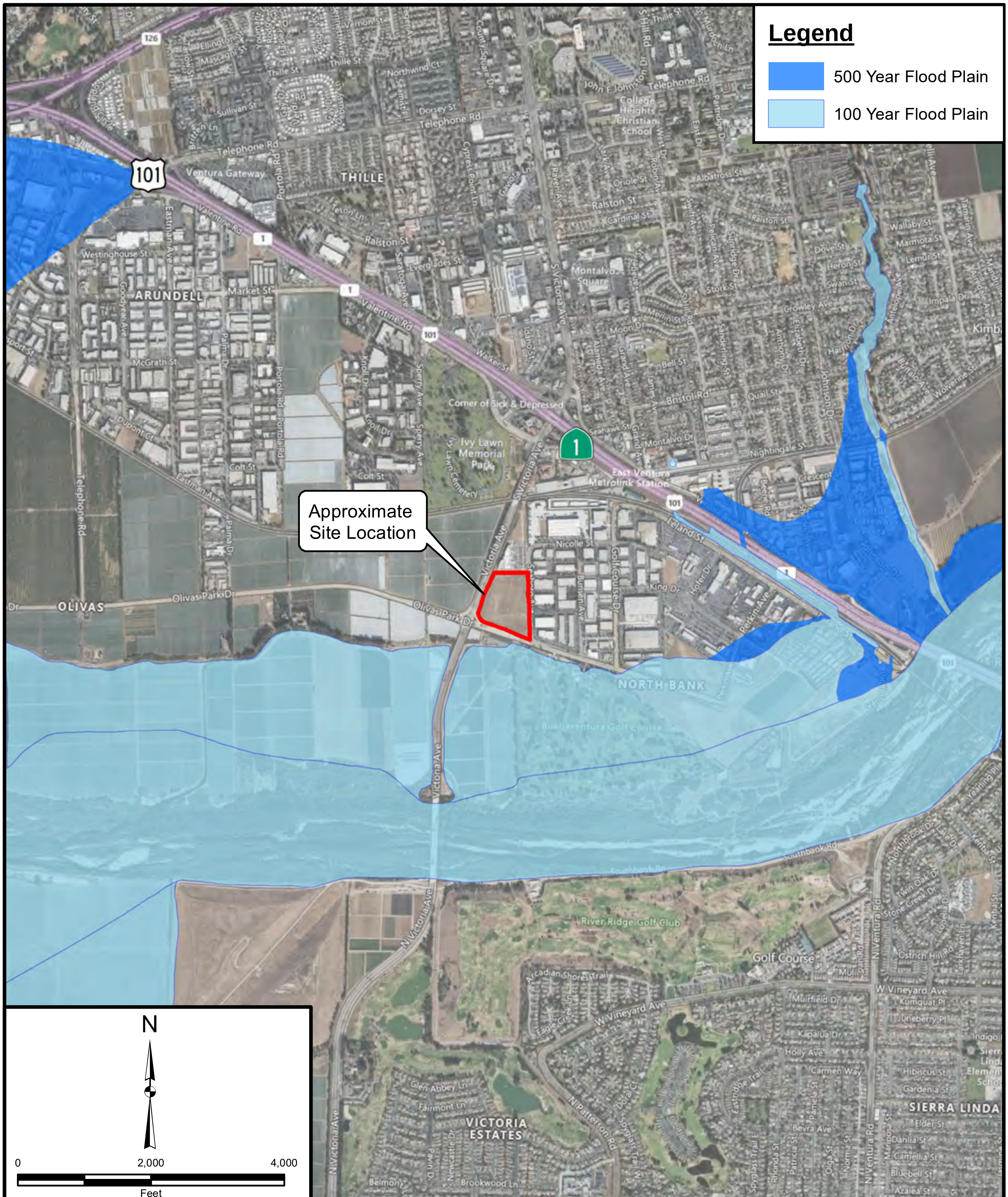
SEISMIC HAZARD MAP

Victoria Corporate Center Phase II/III
2879 Seaborg Avenue
City of Ventura, California

FIGURE 4

Leighton

Map Saved as P:\Drafting\13582\001\Maps\13582-001_F04_LHM_2022-08-05.mxd on 8/5/2022 10:03:14 AM



Project: 13582.001	Eng/Geol: RR/GF
Scale: 1 " = 2,000 '	Date: August 2022

Base Map: ESRI ArcGIS Online 2022

Reference: Federal Emergency Management Agency (FEMA), 2010, Map Number 06111C0781E, Effective Date January 20, 2010, Scale 1" = 1000'.

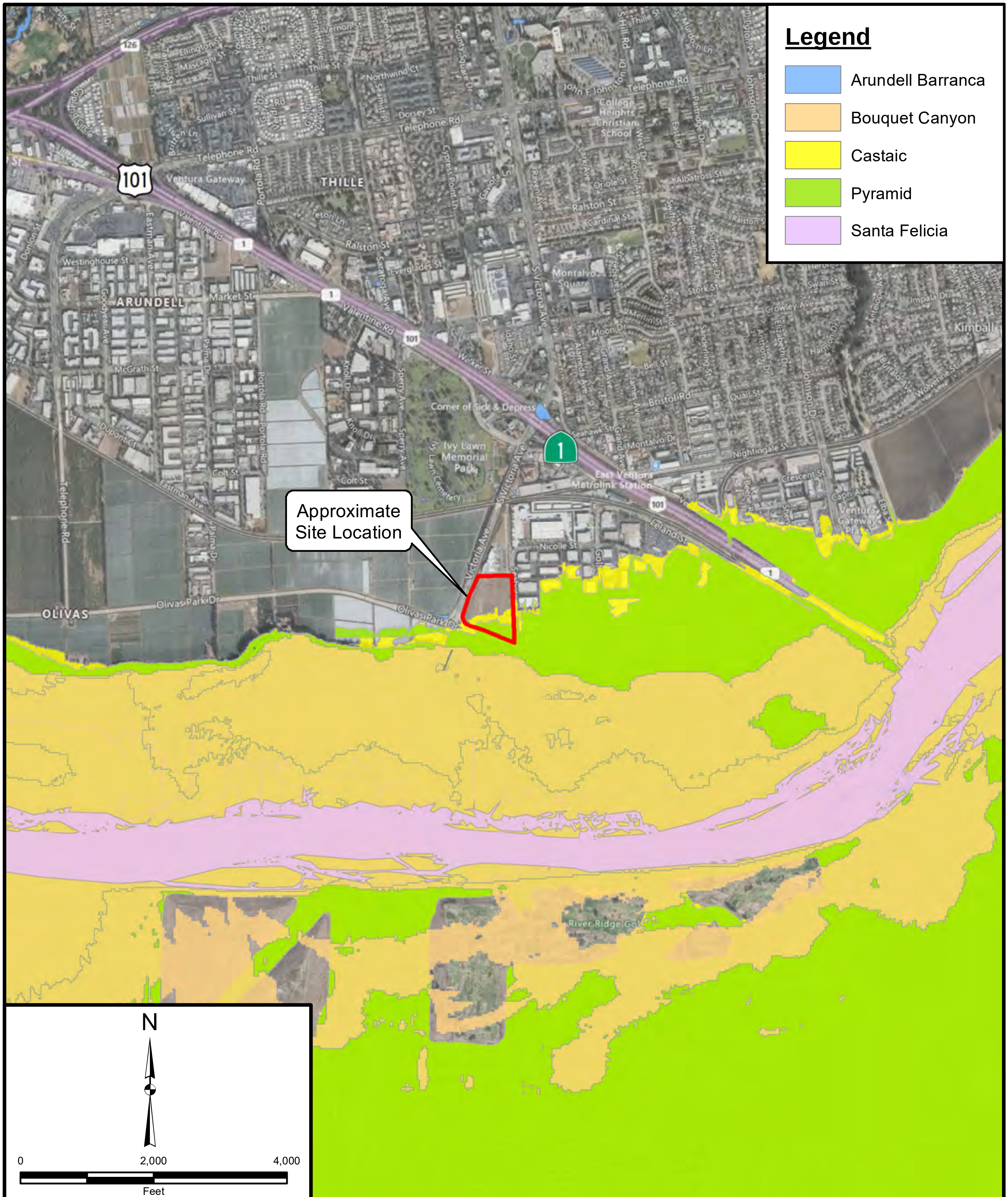
FLOOD HAZARD ZONE MAP

Victoria Corporate Center Phase II/III
2879 Seaborg Avenue
City of Ventura, California

FIGURE 5

Leighton

Map Saved as P:\Drafting\13582\001\Maps\13582-001_F05_FHZM_2022-08-05.mxd on 8/5/2022 10:09:41 AM



Project: 13582.001	Eng/Geol: RH/GF
Scale: 1 " = 2,000 '	Date: August 2022

Base Map: ESRI ArcGIS Online 2022
Reference: Dept of Safety of Dams (2021)
Author: (mmurphy)

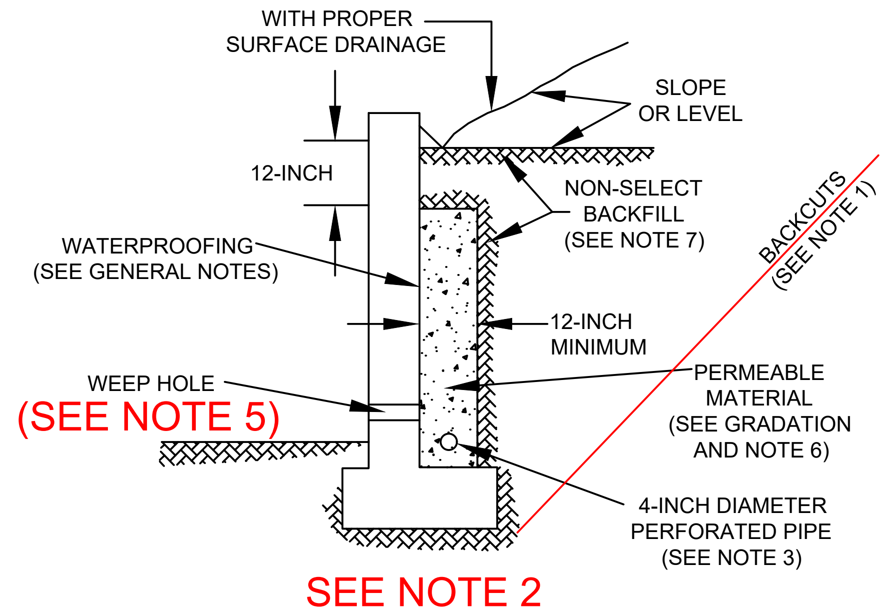
DAM INUNDATION MAP

Victoria Corporate Center Phase II/III
2879 Seaborg Avenue
City of Ventura, California

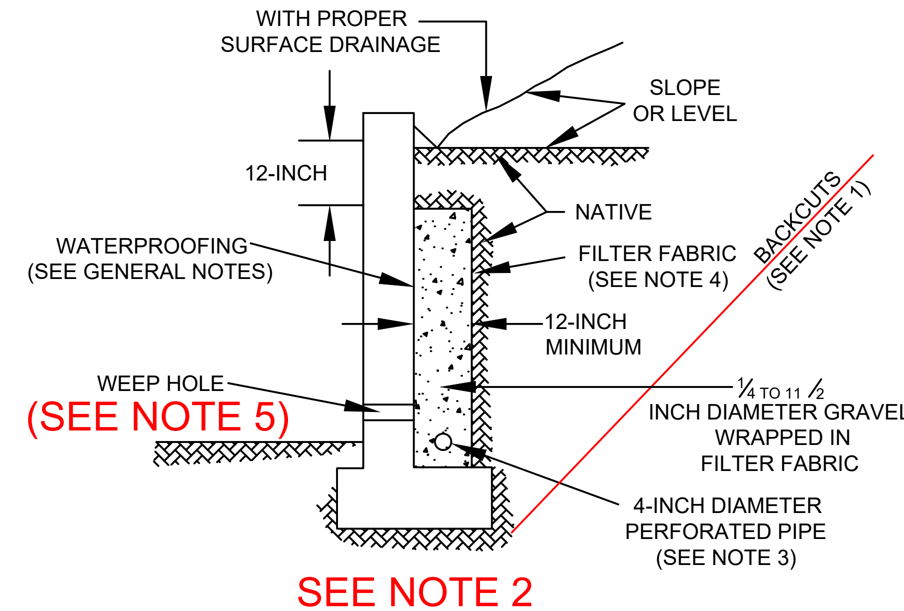
FIGURE 6

Leighton

Map Saved as P:\Drafting\13582\001\Maps\13582-001_F06_DIM_2022-08-04.mxd on 8/5/2022 10:17:29 AM



OR:



PERMEABLE MATERIAL GRADATION:

SIEVE SIZE	PERCENT PASSING
1-inch	100
3/4-inch	90-100
3/8-inch	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

RETAINING WALL BACKFILL AND DRAINAGE NOTES:

- These are schematic sections, not to scale.
- Waterproofing should be provided where moisture passing through retaining walls is undesirable. Waterproofing is not observed nor inspected by Leighton Consulting, Inc.
- All subdrains should be installed with a drainage gradient of at least 1 percent.
- Outlet portion of subdrains should be solid pipe at least 4-inches in diameter, discharging into a suitable disposal area designed by the project Civil Engineer. Subdrain pipes should be accessible for maintenance (with cleanouts, etc.).

NUMBERED NOTES KEYED TO FIGURE:

1. Backcuts: Safe backcuts, in accordance with the current California Construction Safety Orders (Article 6) are required behind retaining walls to allow for Leighton Consulting, Inc. personnel to view drainage installation and to test backfill. Site safety is the responsibility of the Contractor.
2. Foundation Bearing Surfaces: Leighton Consulting, Inc. personnel should observe foundation bearing surfaces before reinforcing steel is placed.
3. Perforated Pipes: Perforated drainpipes should be either ASTM D 1527 Acrylonitrile Butadiene Styrene (ABS) or ASTM D 1785 Polyvinyl Chloride (PVC) Schedule 40 for backfill less than 15 feet deep and Schedule 80 for deeper backfill, or approved equivalent as promulgated by the project Civil Engineer. Pipe should be installed with perforations down. Perforations should be 3/8-inch diameter placed 120° radially in two-rows at 3-inch on center (staggered). Slotted pipe can be used when backfill over the pipe is less-than 15feet deep.
4. Non-Woven Filter Fabric: Filter fabric should be Mirafi 140NC or equivalent, conforming to Section 213-5 (Table 213-5.2 (A) 90N) of the Standard Specifications For Public Works Construction (Greenbook, 2015 Edition or more current).
5. Weepholes: Weephole should be at least 3-inches in diameter and spaced no more than 10-feet on-center horizontally, at the base of retaining walls where a perforated drainpipe with gravity discharge is not provided. If exposure is permitted, weepholes should be located 12 inches above finished grade. If exposure is not permitted such as for walls adjacent to sidewalks, then a pipe under the sidewalk discharged through the curb face, or equivalent, should be provided. For basements, watertight vaults and/or reservoir walls, a proper subdrain outlet system should be provided without weepholes.
6. Permeable Material: At least one cubic-foot of permeable material or crushed rock should be placed per each horizontal foot of wall. Crushed rock should be wrapped in filter fabric as discussed in Note 4 (Mirafi 140NC or equivalent), above.
7. Backfill: All retaining wall backfill soils should have an Expansion Index (EI) <50 and should be compacted to at least 90-percent of the ASTM D 1557 laboratory maximum density, with all backfill tested by Leighton Consulting, Inc.

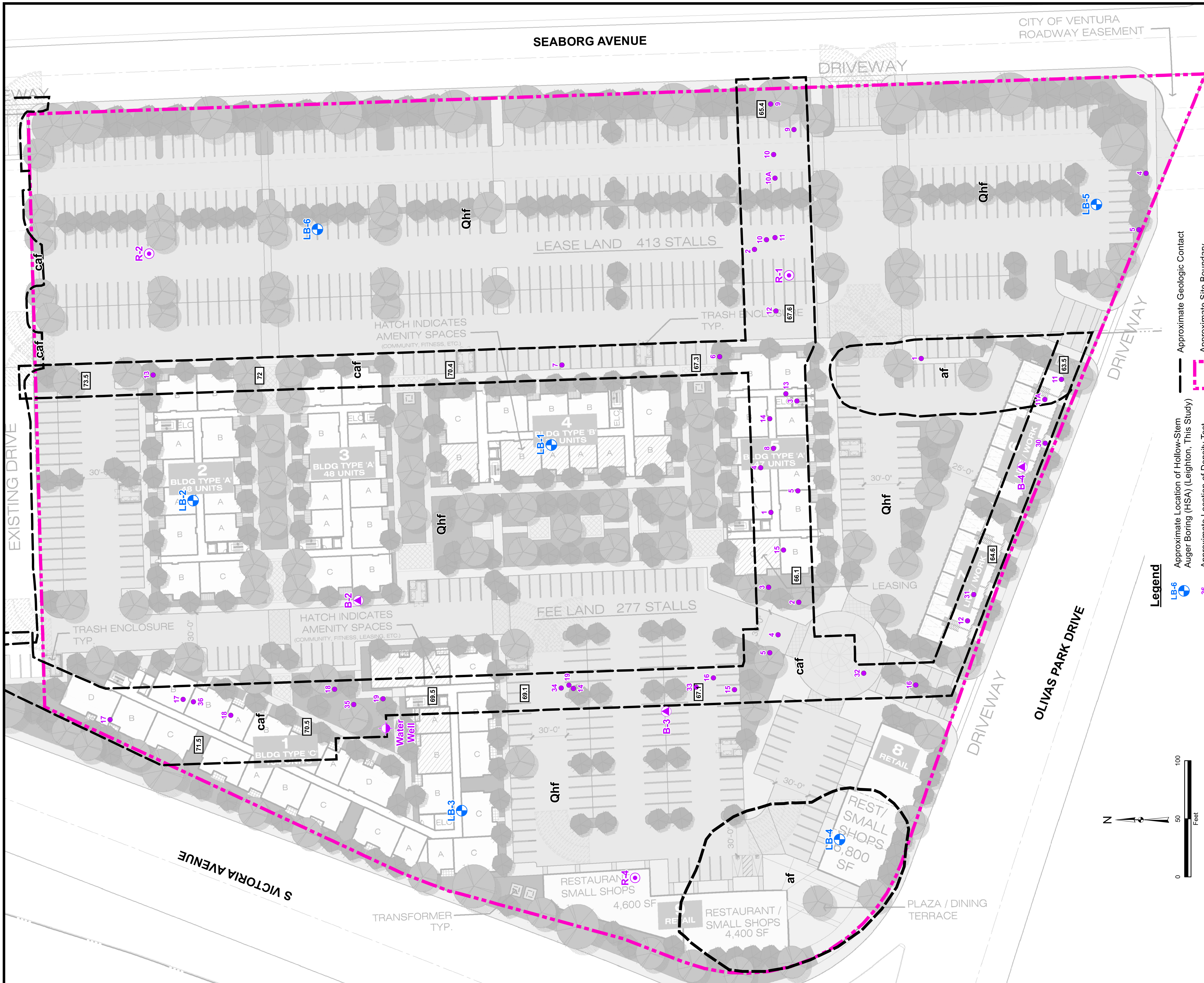
Proj: 13582.001	Eng/Geol: RM/GF
Scale: NTS	Date: August 2022
Base Map:	

RETAINING WALL BACKFILL AND SUBDRAIN DETAIL

Victoria Corporate Center Phase II/III
City of Ventura, California

FIGURE 7

Leighton



Legend

LB-6 Approximate Location of Hollow-Stem Auger Boring (HSA) (Leighton, This Study)

36 Approximate Location of Density Test (Gorian, 2018)

63.5 Approximate Location of Removal Bottom Elevation (Gorian, 2018)

R-4 Approximate Location of Near-Surface Sample (Gorian, 2008)

Water Well Approximate Location of Water Well (Gorian, 2008)

B-4 Approximate Location of Hollow-Stem Auger Boring (HSA) (Gorian, 2003)

Approximate Geologic Contact

Approximate Site Boundary

Geologic Units

af Undocumented Artificial Fill

caf Compact Artificial Fill (Gorian, 2018)

Qhf Holocene-age alluvial fan deposits

N

0 50 100 Feet

GEOTECHNICAL MAP

Victoria Corporate Center Phase II/III
2879 Seaborg Avenue
City of Ventura, California

Leighton

PLATE 1

Scale: 1 " = 50 feet

Date: August 2022

Proj: 13582.001

Eng/Geol: RH/GF

Base Map: Overall Site Plan by Withee Malcolm Architects, dated 7/28/2022.

APPENDIX A

References

APPENDIX A

References

American Concrete Institute (ACI), 2014, Building Code Requirements for Structural Concrete (ACI-318-14) and Commentary (ACI 318-14), and ACI Standard.

Bryant, W.A. and Hart, E.W., Interim Revision 2007, Fault Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zones Maps: California Geological Survey, Special Publications 42, 42p.

California Building Standards Commission, 2019, 2019 California Building Code, California Code of Regulations, Title 24, Part 2, Volume 2 of 2, Based on 2012 International Building Code, effective January 1, 2014.

California Department of Water Resources (CDWR), 2019, California Statewide Groundwater Elevation Monitoring (CASGEM) Program, CASGEM Online system, website https://www.casgem.water.ca.gov/OSS

California Department of Water Resources (DWR), 2022, Interactive Website, Water Data Library, http://www.water.ca.gov/waterdatalibrary/

California Geological Survey (CGS; formerly California Division of Mines and Geology, CDMG), 1990, Fault evaluation report, Oak Ridge and related faults, vicinity of Fillmore and Santa Paula, Ventura County, California: Fault Evaluation Report FER- 219, 10 p.

_____, 1998, Fault evaluation report, Supplement No. 1, Oak Ridge and related faults, vicinity of Fillmore and Santa Paula, Ventura County, California: Fault Evaluation Report FER- 219, 2 p.

_____, 2002, Seismic Hazard Zone Report for the Oxnard 7.5-Minute Quadrangle, Ventura County, California, CDMG Seismic Hazard Zone Report 07, December 20, 2002.

_____, 2002b, Earthquake Zones of Required Investigation, Oxnard Quadrangle, Seismic Hazard Zones, Official Map Released December 20, 2002.

_____, 2008, Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California.

_____, 2009, Tsunami Inundation Map for Emergency Planning, State of California, County of Ventura, Oxnard Quadrangle, Scale of 1:24,000, dated February 15, 2009.

_____, 2010, Fault Activity Map of California, Compilation and interpretation by Jennings, C.W., and Bryant, W.A., Scale of 1:750,000.

_____, 2018, Earthquake Fault Zones, A Guide for Government Agencies, Property Owners / Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California, Special Publication 42, Revised 2018. Public Works Standard, Inc., 2018, Greenbook, Standard Specifications for Public Works Construction: BNI Building News, Anaheim, California.

Department of Water Resources (DWR), 2018a, Castaic Dam, Inundation Map for Sunny-day Hypothetical Failure, Maximum Depth (ft), Los Angeles and Ventura Counties, Sheet 11 of 17, Scale 1:1,000, dated December 21, 2018.

_____, 2018b, Pyramid Dam, Inundation Map for Sunny-day Hypothetical Failure, Maximum Depth (ft), Sheet 12 of 18, Scale 1:1,000, dated December 21, 2018.

Fisher, M.A., Greene, H.G., Normark, W.R., and Sliter, R.W., 2005, Neotectonics of the Offshore Oak Ridge Fault near Ventura, Southern California, Bulletin of the Seismological Society of America, Vol. 95, No. 2, pp. 739–744, April 2005.

Gorian & Associates, Inc., 2003, Preliminary Geotechnical Investigation, 10-Acre Parcel, Assessors Parcel Number 138-22-04, Northeast Corner of Victoria Avenue and Olivas Park Drive, Montalvo Area of County of Ventura, California, Work Order 2503-0-0-10, Log Number 22675, dated September 1, 2003.

_____, 2008, Geotechnical Site Update, 10-Acre Parcel, APN 138-0-220-040, Northeast Corner of Victoria Avenue and Olivas Park Drive, Montalvo Area of County of Ventura, California, Work Order 2503-0-0-100, dated June 12, 2008.

_____, 2018, Compaction Test Summary, Parking and Drive Area Improvements for Victoria Corporate Center Phase 1, 2879 Seaborg Avenue, LD-1027, City of Ventura, California.

Gutierrez, C.I., Tan, S.S., and Clahan, K.B., 2008, Geologic Map of the East Half Santa Barbara 30' x 60' Quadrangle, California, California Geologic Survey, Scale 1:100,000.

United States Geological Survey (USGS), 2008c, National Seismic Hazard Maps – Fault Parameters, https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/query_main.cfm

______, 2019 National Water Information System, Web Interface https://nwis.waterdata.usgs.gov/ca/nwis/gwlevels/?site_no=343051117223401

Ventura Countywide Stormwater Quality Management Program (VCS), 2011, Ventura County Technical Guidance Manual for Stormwater Quality Control Measures, Manual Update 2011, Errata Update 2018, dated June 29, 2018.

Withee Malcom Architects, 2022, Site Plan, Ventura Mixed Use, Victoria Corporate Center, Ventura, CA, Scale of 1" = 50', dated July 28, 2022.

Yeats, R.S., 1989, Oak Ridge Fault, Ventura Basin, California, slip rates and late Quaternary history: U.S. Geological Survey Open-File Report 89-343, 30 p., 6 plates.

APPENDIX B

Exploration Logs

GEOTECHNICAL BORING LOG LB-1

Project No.	13582.001	**Date Drilled**	7-12-22
Project	Red Tail Multifamily - Victoria Corporate	**Logged By**	RM
Drilling Co.	Martini	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	70'
Location	See Plate 1 Geotechcial Map	**Sampled By**	RM

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
70	0								@Surface-5': SILTY SAND (SM), light to dark brown, low moisture, trace of lean clay, trace rootlets.	
65	5			B-1 R-1	10 13 18	81	15	SM SM	@5'-10': SILTY SAND (SM), medium dense, dark brown, slightly moist, fine sands.	
60	10			S-1	Push 1 1			SP	@10'-16.5': Poorly Graded SAND (SP), very loose, light brown, slightly moist, trace silt.	
55	15			R-2	2 3 5	101	25	SP		
	16.5								@16.5': Lean CLAY (CL), medium soft, light brown mottled w/ dark brown, slightly moist.	
	19								@19'-20': SILTY SAND (SM), very loose, light brown, moist, little very fine sand.	
50	20			S-2	2 2 2			CL	@20'-21.5': Lean CLAY (CL), soft, dark brown mottled w/ light gray and white, trace of very fine sand	
	21.5								**TOTAL DEPTH = 21.5 FEET** **NO GROUNDWATER ENCOUNTERED DURING DRILLING**	
45	25									
40	30									

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

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

Leighton

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

GEOTECHNICAL BORING LOG LB-2

Project No.	13582.001	**Date Drilled**	7-12-22
Project	Red Tail Multifamily - Victoria Corporate	**Logged By**	RM
Drilling Co.	Martini	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	76'
Location	See Plate 1 Geotechcial Map	**Sampled By**	RM

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
75	0			B-1				SM	@Surface-5': SILTY SAND (SM), light brown, slightly moist, trace of lean clay, trace rootlets.	CR, DS, EI, MD
70	5			S-1	5 4 5			SM	@5'-8': SILTY SAND (SM), loose, light yellow brown mottled w/ dark brown and white calcium, slightly moist, few to little very fine sand, trace mica.	
				R-1	12 14 23	102	10	SM	@8'-9': SILTY SAND (SM), medium dense, dark brown, slightly moist to moist, mottled w/ oxidation, laminated layers of silt.	
									@9'-10.5': Lean CLAY (CL), very stiff, dark brown, slightly moist, trace of very fine sand.	
65	10			S-2	5 8 9			CL	@10.5'-13': Lean CLAY (CL), very stiff, dark brown, slightly moist.	
				R-2	3 4 7	97	21	CL	@13'-15.5': CLAYEY SAND (SC), medium dense, light yellow brown, moist, very fine sand.	-200
60	15			S-3	Push Push 1		33	CL	@15.5'-16.5': Lean CLAY (CL), very soft, light brown, moist, trace of very fine sand.	AL
									@16.5'-20.5': Lean CLAY (CL), very soft, light brown, moist.	
				R-3	3 5 8	96	28	CL		
55	20			S-4	1 1 1			CL	@20.5'-25': Lean CLAY (CL), very soft, light brown mottled w/ gray, moist, few to little very fine sand.	
50	25			R-4	7 8 10	96	27	CL	@25'-26.5': Lean CLAY (CL), stiff, dark brown mottled w/ calcium, moist, few to little very fine sand.	
	30								@29'-30.5': Poorly Graded SAND (SP), loose, light yellow brown, fine to medium coarse sand.	

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

TYPE OF TESTS:

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

Leighton

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

GEOTECHNICAL BORING LOG LB-2

Project No.	13582.001	**Date Drilled**	7-12-22
Project	Red Tail Multifamily - Victoria Corporate	**Logged By**	RM
Drilling Co.	Martini	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	76'
Location	See Plate 1 Geotechcial Map	**Sampled By**	RM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
45	30			S-5	6 4 4			SP	@30.5'-31.5': Lean CLAY (CL), stiff, light yellow brown,saturated, very fine sand. Note: No standing water.	-200
40	35			R-5	4 9 11	111	17	SP	@34'-35': Poorly Graded SAND w/ Gravel (SP)g, medium dense, very fine to medium coarse sand, fine to medium gravel. @35'-36.5': Poorly Graded SAND (SP), medium dense, light brown, trace clay, very fine to medium coarse sand.	-200
35	40			S-6	Push 3 4			CL	@40'-41.5': Lean CLAY (CL), medium soft, dark brown, few to little very fine sand.	-200
30	45			R-6	6 12 13	106	21	CL	@44'-46': Lean CLAY (CL), very stiff, trace of very fine sand, saturated. @46'-51.5': Poorly Graded SAND w/ CLAY (SP-SC), medium dense, light brown, very fine to coarse sand, saturated.	-200
25	50			S-7	5 13 16			SP	**TOTAL DEPTH = 51.5 FEET** **NO GROUNDWATER ENCOUNTERED DURING DRILLING**	
20	55									
	60									

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

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

Leighton

GEOTECHNICAL BORING LOG LB-3

Project No.	13582.001	**Date Drilled**	7-12-22
Project	Red Tail Multifamily - Victoria Corporate	**Logged By**	RM
Drilling Co.	Martini	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	72'
Location	See Plate 1 Geotechcial Map	**Sampled By**	RM

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N / S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				SM	@Surface-5': SILTY SAND (SM), light brown, slightly moist, trace of lean clay, trace rootlets.	
70										
	5			R-1	6 12 18	96	13	SM	@5'-6.5': SILTY SAND (SM), medium dense, light gray mottled w/ oxidation, low moisture, trace of very fine sand. @6.5': SILTY SAND (SM), medium dense, dark brown mottled w/ oxidation, low moisture, trace of very fine sand.	
65										
									@9.5': Lean CLAY (CL), medium soft, dark brown, mottled w/ oxidation,	
	10			S-1	2 2 3			SM	@10': SILTY SAND (SM), loose, light yellow brown, saturated. Note: No sample recovery. Soil description above is from soil found in the shoe.	
60										
	15			R-2	3 5 7	96	26	CL	@15': Lean CLAY (CL), stiff, yellow brown mottled w/ dark brown, moist	
55										
	20			S-2	2 3 5			CL	@20'-21.5': Lean CLAY (CL), stiff, yellow brown, moist, trace of very fine sand.	
50									**TOTAL DEPTH = 21.5 FEET** **NO GROUNDWATER ENCOUNTERED DURING DRILLING**	
	25									
45										
	30									

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

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

Leighton

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

GEOTECHNICAL BORING LOG LB-4

Project No.	13582.001	**Date Drilled**	7-12-22
Project	Red Tail Multifamily - Victoria Corporate	**Logged By**	RM
Drilling Co.	Martini	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	68'
Location	See Plate 1 Geotechcial Map	**Sampled By**	RM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0								@Surface-5': SILTY SAND (SM), light brown, slightly moist, trace of lean clay, trace rootlets.	
65										
	5			B-1 R-1	9 13 13	97	20	SM SM	@5': Lean CLAY (CL), very stiff, light yellow brown, moist.	
60										
	10			S-1	1 1 2			CL	@10': Lean CLAY (CL), soft, light yellow brown, moist. Note: No sample recovery. Classification is from soil in shoe.	
55										
	15			R-2	4 8 10	101	23	SM	@15'-16': SILTY SAND (SM), medium density, light brown. @16'-21.5': Lean CLAY (CL), medium soft, light yellow brown, moist.	
50										
	20			S-2	2 3 4			CL		
									TOTAL DEPTH = 21.5 FEET **NO GROUNDWATER ENCOUNTERED DURING DRILLING** **PERCHED ZONE 15'-20'**	
45										
	25									
40										
	30									

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

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

Leighton

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

GEOTECHNICAL BORING LOG LB-5

Project No.	13582.001	**Date Drilled**	7-12-22
Project	Red Tail Multifamily - Victoria Corporate	**Logged By**	RM
Drilling Co.	Martini	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	66'
Location	See Plate 1 Geotechcial Map	**Sampled By**	RM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
65	0			B-1				SM	@Surface-5': SILTY SAND (SM), dark brown, low moisture, trace rootlets.	CR, EI
60	5			R-1	12 16 19	105	14	SM	@5': SILTY SAND (SM), dark brown, low moisture, trace rootlets.	
55	10			S-1	1 2 2			CL	@10': Lean CLAY (CL), light yellow brown mottled w/ black, moist, few to little very fine sand. Note: Perched Zone.	
50	15			R-2	2 3 3	105	25	SM	@15': Lean CLAY (CL), light yellow brown mottled w/ black, saturated, few to little very fine sand.	
45	20			S-2	4 5 6			CL	@20': Lean CLAY (CL), light brown, moist, few to little very fine sand.	
									TOTAL DEPTH = 21.5 FEET **NO GROUNDWATER ENCOUNTERED DURING DRILLING**	
40	25									
	30									

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

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

Leighton

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

GEOTECHNICAL BORING LOG LB-6

Project No.	13582.001	**Date Drilled**	7-12-22
Project	Red Tail Multifamily - Victoria Corporate	**Logged By**	RM
Drilling Co.	Martini	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	73'
Location	See Plate 1 Geotechcial Map	**Sampled By**	RM

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0			B-1				SM	@Surface-5': SILTY SAND (SM), light brown, slightly moist, trace rootlets.	
70										
	5								**TOTAL DEPTH = 5 FEET** **NO GROUNDWATER ENCOUNTERED DURING DRILLING** **INFILTRATION SYSTEM INSTALLED**	
65										
	10									
60										
	15									
55										
	20									
50										
	25									
45										
	30									

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

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

Leighton

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

GORIAN & ASSOCIATES, INC.
Applied Earth Sciences

Project: DPS - Victoria and Olivas, Ventura
Drill Co. and Rig Type: Jet Drilling, CME 75
Hammer: Auto, 140# Logged by: EAB/MB
Boring Diameter: 8" Surface Elevation: 73'±
Work Order: 2503-0-0-10
Report Log No.: 22675
Date: 6/30/03

Depth (ft)	Undisturbed	SPT	Bulk	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	Penetrometer (tsf)	USCS	Soil/ Lithology	Description
0			■					CL		TOPSOIL At 0 to 2½'; light grayish brown sandy silty clay (damp, stiff). Dessicated at roadway. Calcium carbonate stringers.
	■			13/ 12"	16.0	97				
								CL/: ML		ALLUVIUM At 2½' to 12'; brown sandy silty clay to clayey silt (damp to moist, medium stiff to stiff) becoming darker brown with depth.
	■			10/ 12"	19.8	97				
5										
	■			12/ 12"	25.6	100				At 6'; some pinhole porosity.
10	■			7/ 12"	25.9	95				
								ML/: SM		At 12' to 16'; brown silty sand to sandy silt (saturated, medium dense).
15	■			10/ 12	27.1	98				
										Total depth 16' No caving, Groundwater at 13½'.
20										

GORIAN & ASSOCIATES, INC.
Applied Earth Sciences

Project: DPS - Victoria and Olivas, Ventura
Drill Co. and Rig Type: Jet Drilling, CME 75
Hammer: Auto, 140#   Logged by: EAB/MB
Boring Diameter: 8"   Surface Elevation:69'±

Work Order: 2503-0-0-10
Report Log No.: 22675
Date: 6/30/03

Depth (ft)	Undisturbed	SPT	Bulk	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	Penetrometer (tsf)	USCS	Soil/ Lithology	Description
0								CL		TOPSOIL At 0 to 2½'; light grayish brown silty clay (damp, stiff to soft).
		■		0/ 3/ 3	28.0			CL		At 2½' to 10'; light grayish brown silty clay (moist, medium stiff). At 4'; becoming darker.
5		■		0/ 2/ 4	29.4					At 5; SMC = 2/35/63
		■		0/ 0/ 0	31.1					At 8½' to 8'9"; becoming wet.
10		■		1/ 0/ 1	27.9			ML/ CL		At 10' to 15'; brown clayey silt with sand (wet, very soft). At 10'; SMC = 21/53/26
		■		0/ 0/ 2	28.7					At 12½'; SMC = 17/52/31
15		■		1/ 1/ 3	26.6			ML		At 15' to 20½'; brown sandy silt (saturated, soft to medium stiff). At 15'; SMC = 22/61/17
		■		1/ 3/ 4	23.3					
20		■		1/ 1/ 2	27.7			CL		At 20'; SMC = 14/33/53 At 20½' to 27'; dark gray silty clay with some sand (saturated, soft to medium stiff).
		■		0/ 2/ 3						At 22½'; SMC = 12/40/48
25		■		4/ 3/ 4						At 25'; SMC = 0/53/47
		■		10/ 22/ 25				SM		At 27' to 35'; brown silty sand (saturated, dense). Some fine gravel.
30		■		16/ 22/ 26						At 30' to 30½'; brown clayey sand lens.

GORIAN & ASSOCIATES, INC.
Applied Earth Sciences

Project: DPS - Victoria and Olivas, Ventura
Drill Co. and Rig Type: Jet Drilling, CME 75
Hammer: Auto, 140# Logged by: EAB/MB
Boring Diameter: 8" Surface Elevation:69'±

Work Order: 2503-0-0-10
Report Log No.: 22675
Date: 6/30/03

Depth (ft)	Undisturbed	SPT	Bulk	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	Penetrometer (tsf)	USCS	Soil/ Lithology	Description
				9/ 17/ 8						At 32½'; 39% < #200
										At 33'9"; sandy clay lens.
35				2/ 5/ 7				SM		No Recovery (Assume medium dense silty sand).
				6/ 8/ 14				SM/: ML		At 37½' to 41'; brown sandy silt to silty sand (saturated, medium dense to very stiff). At 37½'; 61% < #200
40				9/ 10/ 35						At 41' to 45'; becomes mostly brown silty sand (saturated, dense).
				12/ 14/ 16						
45				7/ 12/ 15						At 45'; 57% < #200
				4/ 7/ 4				ML/: CL		At 47½'; brown clayey silt (saturated, stiff). SMC = 10/59/31
50				4/ 8/ 9				ML		At 50 to 51½'; brown sandy silt (saturated, stiff to very stiff). SMC = 39/40/21
										Total depth 51½'. No caving, Groundwater at 8½'.
										SMC = %Sand/%Silt/%Clay
55										

GORIAN & ASSOCIATES, INC.
Applied Earth Sciences

Project: DPS - Victoria and Olivas, Ventura
Drill Co. and Rig Type: Jet Drilling, CME 75
Hammer: Auto, 140#   Logged by: EAB/MB
Boring Diameter: 8"   Surface Elevation: 66'±
Work Order: 2503-0-0-10
Report Log No.: 22675
Date: 6/30/03

Depth (ft)	Undisturbed	SPT	Bulk	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	Penetrometer (tsf)	USCS	Soil/ Lithology	Description
0			■					CL		TOPSOIL At 0 to 3'; light grayish brown sandy silty clay (damp, very stiff). Calcium carbonate stringers. Dessication cracks at surface of roadway.
	■			20/ 12"	17.2	109				
								CL		ALLUVUIM At 3' to 6'; medium gray to dark grayish brown silty clay (moist, stiff). Calcium carbonate stringers.
5	■			12/ 12"	19.6	100				
	■			9/ 12"	26.6	96		ML		At 6' to 9'; brown sandy silt (moist to wet, soft to medium stiff). Becoming sandier with depth.
								ML/: SM		At 9' to 16'; brown silty sand to clayey silt with sand (saturated, soft to medium stiff or medium dense). Occasional clayey lenses. Becoming more dense with depth.
10	■			6/ 12"	25.6	97				
15	■			9/ 12"	25.6	100				
										Total depth 16' No caving, Groundwater at 9'.
20										

APPENDIX C

Laboratory Test Results

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name: Red Tail Ventura — Tested By : GEB/JD — Date: 08/02/22

Project No. : 13582.001 — Checked By: J. Ward — Date: 08/08/22

Boring No.	LB-2	LB-5		
Sample No.	B-1	B-1		
Sample Depth (ft)	0-10	0-5		
Soil Identification:	Olive CL	Brown CL		
Wet Weight of Soil + Container (g)	177.45	185.93		
Dry Weight of Soil + Container (g)	166.27	177.41		
Weight of Container (g)	57.22	56.35		
Moisture Content (%)	10.25	7.04		
Weight of Soaked Soil (g)	100.23	100.37		

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	2	8		
Crucible No.	4	9, 3		
Furnace Temperature (°C)	860	860		
Time In / Time Out	16:00/16:45	16:00/16:45		
Duration of Combustion (min)	45	45		
Wt. of Crucible + Residue (g)	21.6541	47.0386		
Wt. of Crucible (g)	21.6375	47.0127		
Wt. of Residue (g) (A)	0.0166	0.0259		
PPM of Sulfate (A) x 41150	683.09	1065.79		
PPM of Sulfate, Dry Weight Basis	**761**	**1146**		

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15	15		
ml of AgNO3 Soln. Used in Titration (C)	0.3	0.5		
PPM of Chloride (C -0.2) * 100 * 30 / B	20	60		
PPM of Chloride, Dry Wt. Basis	**22**	**65**		

pH TEST, DOT California Test 643

pH Value	**7.67**	**7.69**		
Temperature °C	20.4	20.4		

SOIL RESISTIVITY TEST

DOT CA TEST 643

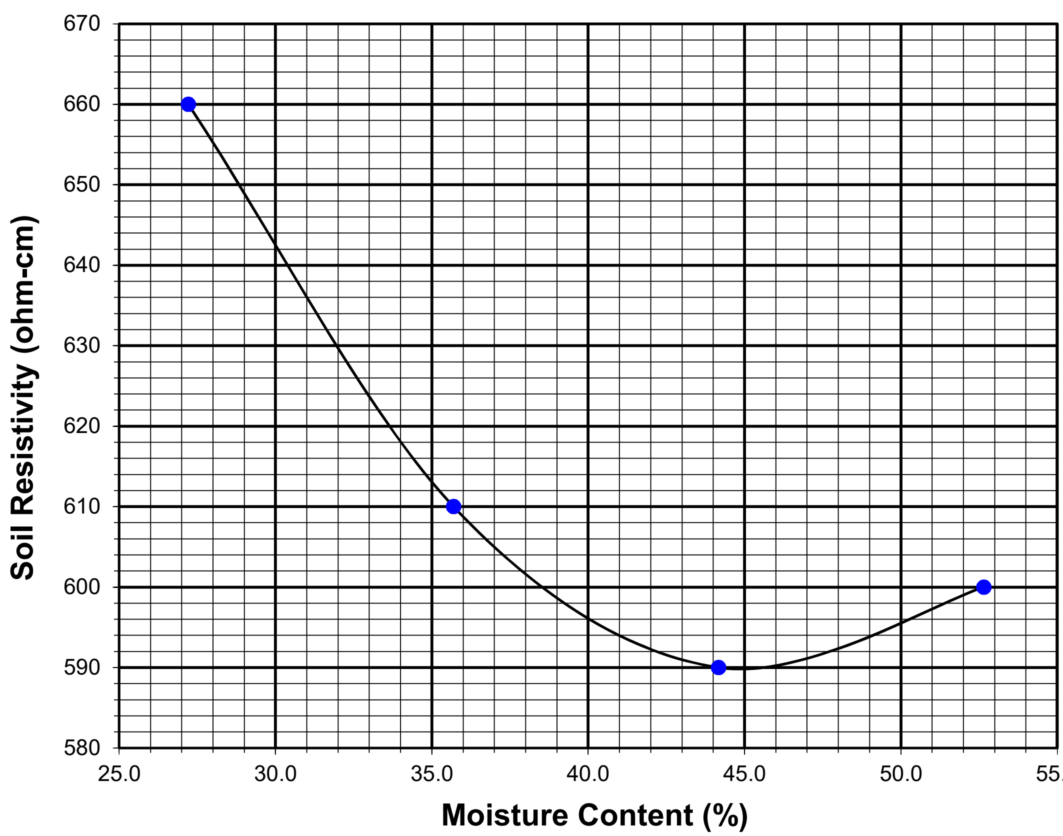
Project Name:	Red Tail Ventura		Tested By :	J. Domingo	Date: 08/05/22
Project No. :	13582.001		Checked By:	J. Ward	Date: 08/08/22
Boring No.:	LB-2		Depth (ft.) :	0-10	
Sample No. :	B-1				
Soil Identification:*	Olive CL				

*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	27.21	660	660
2	30	35.69	610	610
3	40	44.18	590	590
4	50	52.66	600	600
5				

Moisture Content (%) (MCi)	10.25
Wet Wt. of Soil + Cont. (g)	177.45
Dry Wt. of Soil + Cont. (g)	166.27
Wt. of Container (g)	57.22
Container No.	
Initial Soil Wt. (g) (Wt)	130.00
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
590	**44.8**	**761**	**22**	**7.67**	**20.4**



SOIL RESISTIVITY TEST

DOT CA TEST 643

Leighton

Project Name:	Red Tail Ventura	Tested By :	J. Domingo Date: 08/05/22
Project No. :	13582.001	Checked By:	J. Ward Date: 08/08/22
Boring No.:	LB-5	Depth (ft.) :	0-5
Sample No. :	B-1		

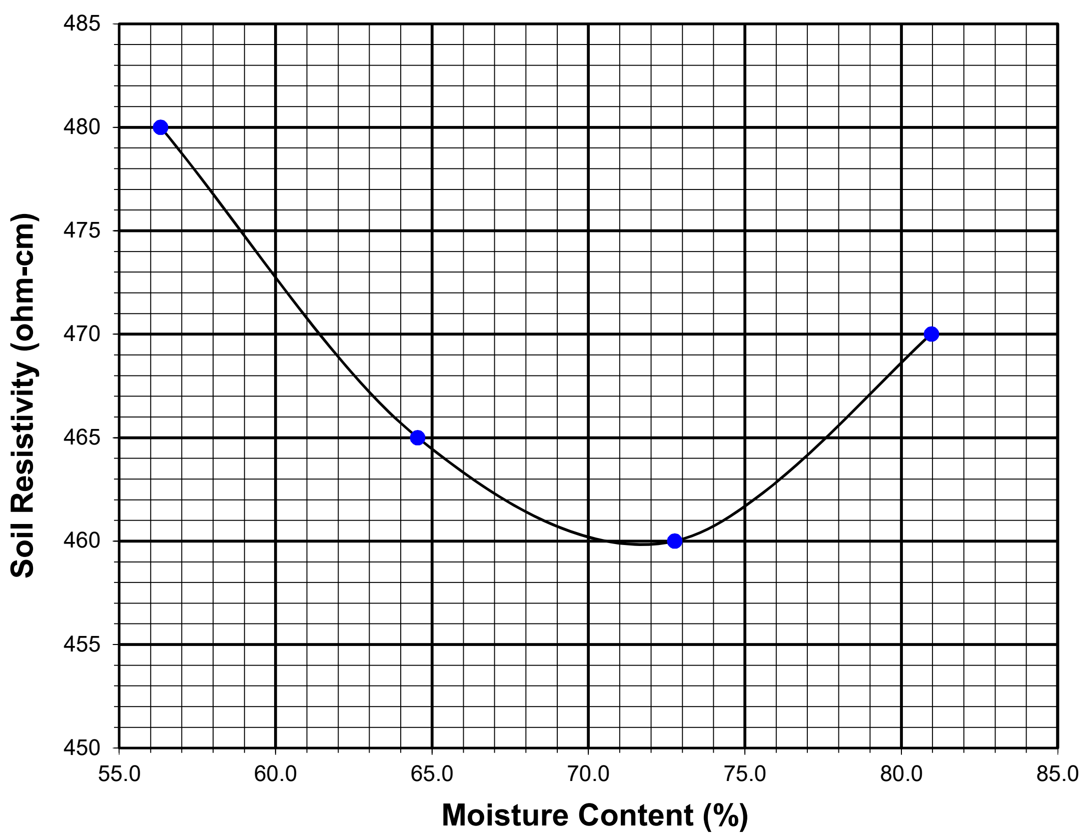
Soil Identification:* Brown CL

*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	60	56.33	480	480
2	70	64.54	465	465
3	80	72.76	460	460
4	90	80.97	470	470
5				

Moisture Content (%) (MCi)	7.04
Wet Wt. of Soil + Cont. (g)	185.93
Dry Wt. of Soil + Cont. (g)	177.41
Wt. of Container (g)	56.35
Container No.	
Initial Soil Wt. (g) (Wt)	130.30
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
460	**71.6**	**1146**	**65**	**7.69**	**20.4**



DIRECT SHEAR TEST

Consolidated Undrained

Project Name: Red Tail Ventura
Tested By: G. Bathala
Date: 08/03/22

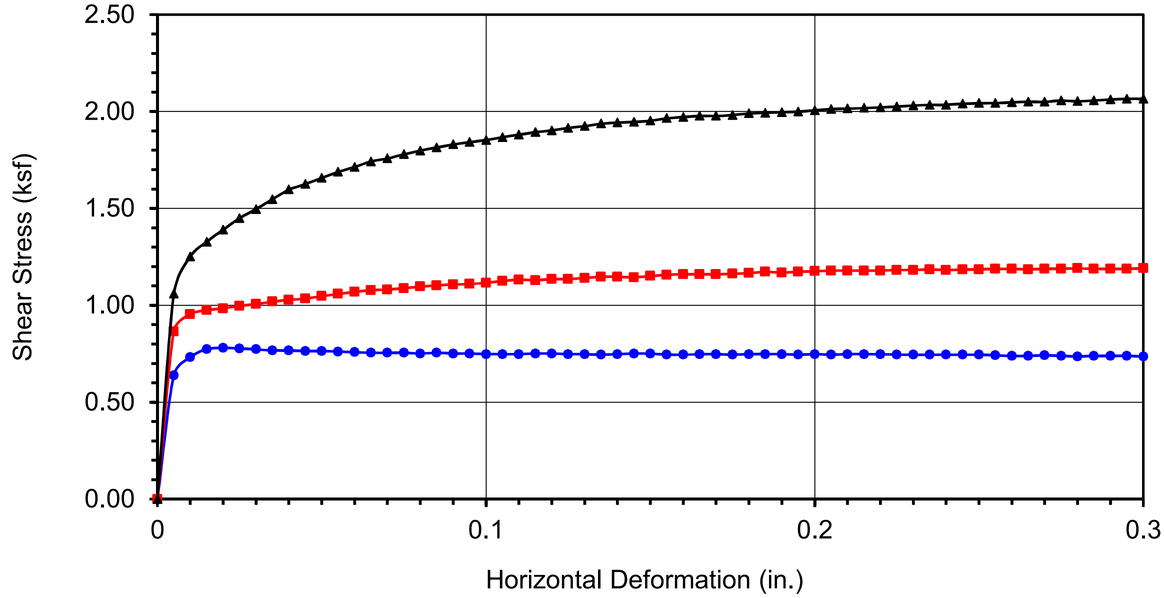
Project No.: 13582.001
Checked By: J. Ward
Date: 08/08/22

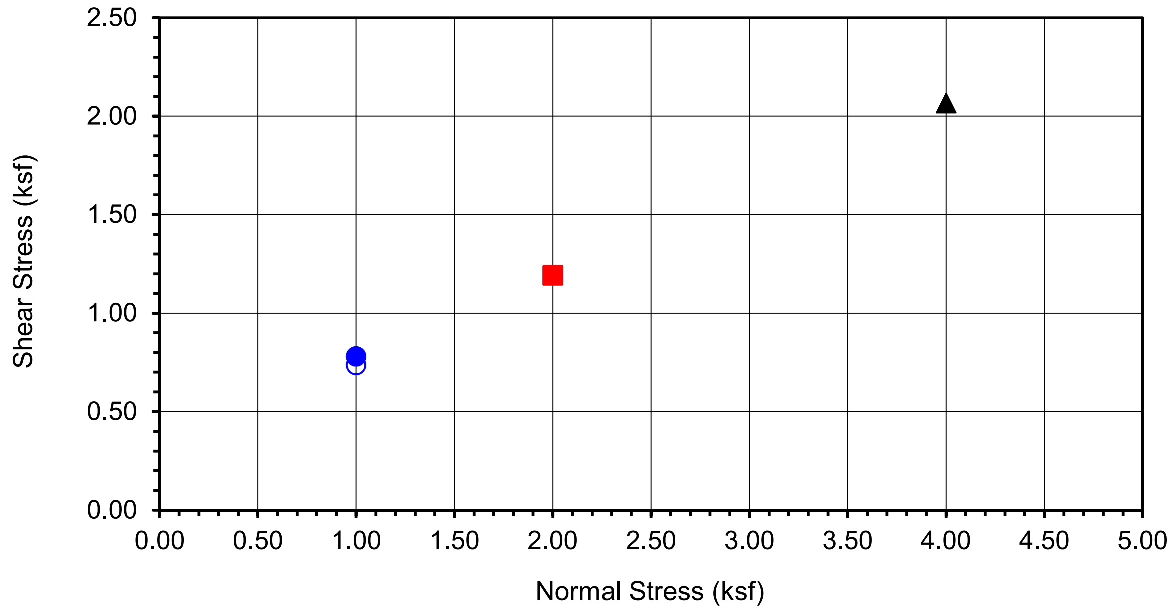
Boring No.: LB-2
Sample Type: 90% Remold

Sample No.: B-1
Depth (ft.): 0-10

Soil Identification: Olive lean clay (CL)

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	184.00	184.02	184.03
Weight of Ring(gm):	45.47	45.41	45.39
Before Shearing			
Weight of Wet Sample+Cont.(gm):	165.18	165.18	165.18
Weight of Dry Sample+Cont.(gm):	153.87	153.87	153.87
Weight of Container(gm):	65.85	65.85	65.85
Vertical Rdg.(in): Initial	0.0000	0.2451	0.2435
Vertical Rdg.(in): Final	0.0194	0.2531	0.2722
After Shearing			
Weight of Wet Sample+Cont.(gm):	209.62	208.47	206.74
Weight of Dry Sample+Cont.(gm):	179.52	179.67	179.18
Weight of Container(gm):	59.14	59.04	58.01
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**LB-2**
Sample No.	**B-1**
Depth (ft)	**0-10**

Sample Type:

90% Remold

Soil Identification:
Olive lean clay (CL)

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.780	■ 1.191	▲ 2.065
Shear Stress @ End of Test (ksf)	○ 0.736	□ 1.191	△ 2.065
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	12.85	12.85	12.85
Dry Density (pcf)	102.1	102.2	102.2
Saturation (%)	53.3	53.4	53.4
Soil Height Before Shearing (in.)	1.0194	0.9920	0.9713
Final Moisture Content (%)	25.0	23.9	22.7

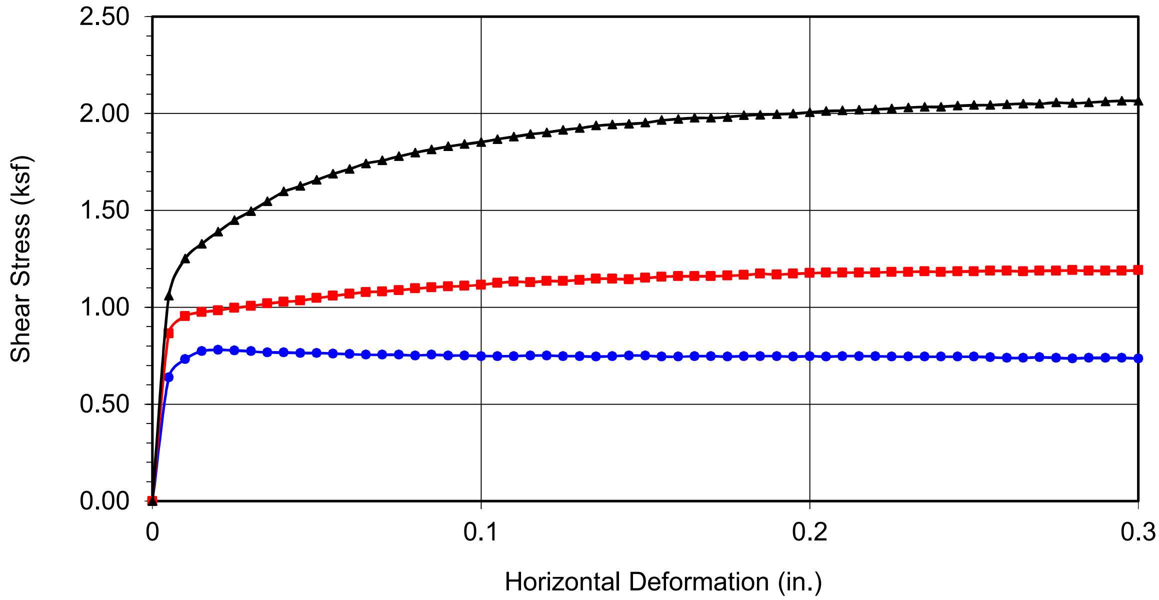
Leighton

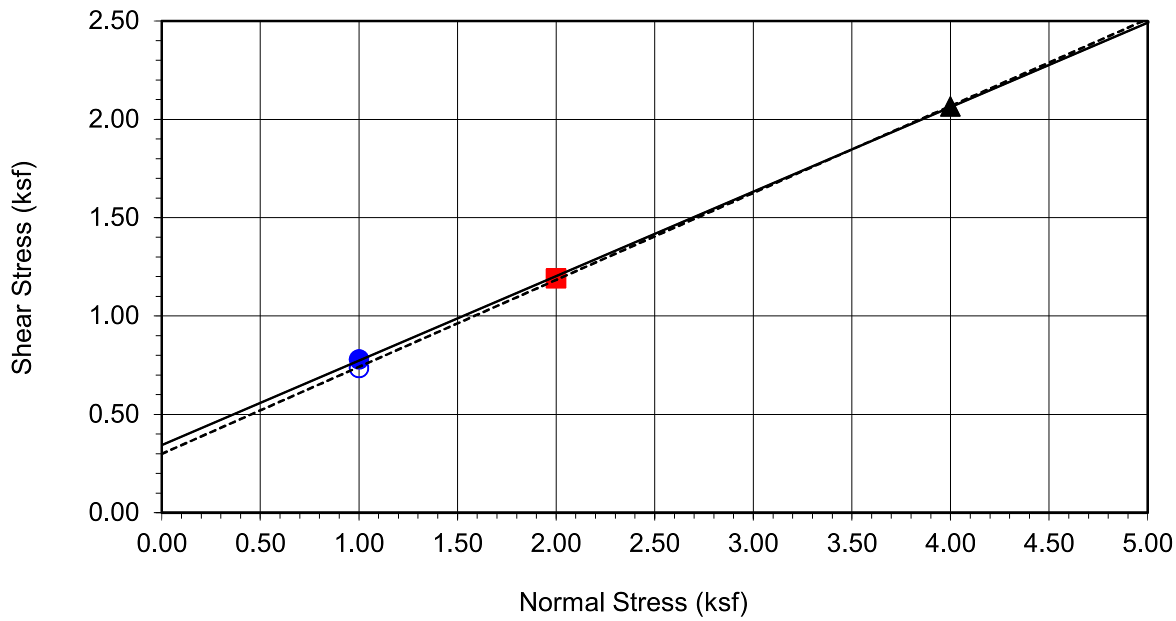
DIRECT SHEAR TEST RESULTS
Consolidated Undrained

Project No.: 13582.001

Red Tail Ventura

08-22





Boring No.	**LB-2**
Sample No.	**B-1**
Depth (ft)	**0-10**
Sample Type:	90% Remold
Soil Identification: Olive lean clay (CL)	

Strength Parameters

	C (psf)	ϕ (°)
Peak	343	23
Ultimate	299	24

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 0.780	■ 1.191	▲ 2.065
Shear Stress @ End of Test (ksf)	○ 0.736	□ 1.191	△ 2.065
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	12.85	12.85	12.85
Dry Density (pcf)	102.1	102.2	102.2
Saturation (%)	53.3	53.4	53.4
Soil Height Before Shearing (in.)	1.0194	0.9920	0.9713
Final Moisture Content (%)	25.0	23.9	22.7

Leighton

DIRECT SHEAR TEST RESULTS

Consolidated Undrained

Project No.: 13582.001

Red Tail Ventura

08-22

EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	Red Tail Ventura	Tested By:	G. Berdy	Date:	08/03/22
Project No.:	13582.001	Checked By:	J. Ward	Date:	08/08/22
Boring No.:	LB-2	Depth (ft.):	0-10		
Sample No.:	B-1				
Soil Identification:	Olive lean clay (CL)				

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

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0815
Wt. Comp. Soil + Mold (g)	578.30	448.90
Wt. of Mold (g)	190.00	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	O	O
Wet Wt. of Soil + Cont. (g)	782.00	638.90
Dry Wt. of Soil + Cont. (g)	705.80	540.45
Wt. of Container (g)	0.00	190.00
Moisture Content (%)	10.80	28.09
Wet Density (pcf)	117.1	125.2
Dry Density (pcf)	105.7	97.7
Void Ratio	0.595	0.725
Total Porosity	0.373	0.420
Pore Volume (cc)	77.2	94.1
Degree of Saturation (%) [S_{meas}]	**49.0**	104.7

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

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
08/03/22	11:17	1.0	0	0.5825
08/03/22	11:27	1.0	10	0.5810
Add Distilled Water to the Specimen				
08/03/22	11:43	1.0	16	0.6365
08/04/22	12:00	1.0	1473	0.6640
08/04/22	14:00	1.0	1593	0.6640

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**83**

Leighton

EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	Red Tail Ventura	Tested By:	G. Berdy	Date:	08/03/22
Project No.:	13582.001	Checked By:	J. Ward	Date:	08/08/22
Boring No.:	LB-5	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Brown lean clay (CL)				

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

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0950
Wt. Comp. Soil + Mold (g)	588.80	447.50
Wt. of Mold (g)	202.00	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	O	O
Wet Wt. of Soil + Cont. (g)	774.30	649.50
Dry Wt. of Soil + Cont. (g)	696.30	549.84
Wt. of Container (g)	0.00	202.00
Moisture Content (%)	11.20	28.65
Wet Density (pcf)	116.7	123.3
Dry Density (pcf)	104.9	95.8
Void Ratio	0.607	0.759
Total Porosity	0.378	0.432
Pore Volume (cc)	78.2	97.8
Degree of Saturation (%) [S_{meas}]	**49.8**	101.9

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

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
08/03/22	11:44	1.0	0	0.5540
08/03/22	11:54	1.0	10	0.5530
Add Distilled Water to the Specimen				
08/03/22	12:15	1.0	21	0.5800
08/04/22	12:00	1.0	1446	0.6490
08/04/22	14:00	1.0	1566	0.6490

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**96**

Boring No.	LB-1	LB-1	LB-2	LB-2	LB-2	LB-2	LB-2	LB-2
Sample No.	R-1	R-2	R-1	R-2	R-3	R-4	R-5	R-6
Depth (ft.)	5.0	15.0	7.5	12.5	17.5	25.0	35.0	45.0
Sample Type	Ring	Ring	Ring	Ring	Ring	Ring	Ring	Ring
Soil Identification	Brown silty clay (CL-ML), loose	Brown lean clay (CL)	Brown silt (ML)	Brown silty clay with sand (CL-ML)s	Brown lean clay (CL)	Dark brown lean clay (CL)	Brown clayey sand (SC)	Brown sandy lean clay s(CL)
Pocket Penetrometer (tons/ft^2)	N/A	2.25	>4.50	2.00	1.00	1.50	2.25	3.50
Weight Soil + Rings / Tube (g)	784.5	782.3	720.0	742.0	1154.0	1143.6	1206.0	995.8
Weight of Rings / Tube (g)	222.0	177.6	177.6	177.6	266.4	266.4	266.4	222.0
Average Length (in.)	5.00	4.00	4.00	4.00	6.00	6.00	6.00	5.00
Average Diameter (in.)	2.415	2.415	2.415	2.415	2.415	2.415	2.415	2.415
Wet. Wt. of Soil + Cont. (g)	237.4	240.5	253.9	812.2	209.1	225.8	1041.6	1019.5
Dry Wt. of Soil + Cont. (g)	211.3	208.3	237.2	716.3	176.3	190.4	903.5	858.9
Weight of Container (g)	38.4	76.8	74.6	251.1	58.8	56.6	108.5	107.6
Container No.								
Wet Density	93.6	125.7	112.8	117.3	123.0	121.6	130.2	128.7
Moisture Content (%)	**15.1**	**24.5**	**10.3**	**20.6**	**27.9**	**26.5**	**17.4**	**21.4**
Dry Density (pcf)	**81.3**	**101.0**	**102.3**	**97.3**	**96.2**	**96.1**	**111.0**	**106.0**
Degree of Saturation (%)	38.0	98.8	42.8	76.0	100.2	94.9	90.4	97.9

Leighton

MOISTURE & DENSITY of SOILS

ASTM D 2216 & ASTM D 2937

Project Name: Red Tail Ventura
Project No.: 13582.001
Tested By: S. Felter Date: 07/22/22

Boring No.	LB-3	LB-3	LB-4	LB-4	LB-5	LB-5		
Sample No.	R-1	R-2	R-1	R-2	R-1	R-2		
Depth (ft.)	5.0	15.0	5.0	15.0	5.0	15.0		
Sample Type	Ring	Ring	Ring	Ring	Ring	Ring		
Soil Identification	Grayish brown sandy lean clay s(CL), loose	Brown lean clay (CL)	Brown lean clay (CL)	Brown sandy lean clay s(CL)	Dark brown lean clay (CL)	Brown lean clay (CL)		
Pocket Penetrometer (tons/ft^2)	N/A	0.75	>4.50	1.50	>4.50	2.00		
Weight Soil + Rings / Tube (g)	873.2	1142.4	1100.1	582.9	757.6	1015.1		
Weight of Rings / Tube (g)	222.0	266.4	266.4	133.2	177.6	222.0		
Average Length (in.)	5.00	6.00	6.00	3.00	4.00	5.00		
Average Diameter (in.)	2.415	2.415	2.415	2.415	2.415	2.415		
Wet. Wt. of Soil + Cont. (g)	256.4	231.3	246.4	232.7	270.5	245.5		
Dry Wt. of Soil + Cont. (g)	234.5	197.8	214.3	199.6	245.8	207.9		
Weight of Container (g)	66.9	70.3	51.2	56.8	74.3	59.1		
Container No.								
Wet Density	108.3	121.4	115.6	124.7	120.6	131.9		
Moisture Content (%)	**13.1**	**26.3**	**19.7**	**23.2**	**14.4**	**25.3**		
Dry Density (pcf)	**95.8**	**96.2**	**96.6**	**101.2**	**105.4**	**105.3**		
Degree of Saturation (%)	46.4	94.2	71.3	94.0	64.9	113.6		

Leighton

MOISTURE & DENSITY of SOILS

ASTM D 2216 & ASTM D 2937

Project Name: Red Tail Ventura

Project No.: 13582.001

Tested By: S. Felter Date: 07/22/22

Leighton

MOISTURE CONTENT

ASTM D 2216

Project Name: Red Tail Ventura
Project No.: 13582.001

Tested By: S. Felter
Date: 07/22/22
Checked By: J. Ward
Date: 08/07/22

Boring No.	LB-2				
Sample No.	S-3				
Depth (ft)	15.0				
Sample Type	SPT				
Sample Description	Brown lean clay (CL)				
Wt. wet soil + container (g)	368.6				
Wt. dry soil + container (g)	287.4				
Weight of container (g)	38.7				
Moisture Content (%)	**32.6**				

Boring No.					
Sample No.					
Depth (ft)					
Sample Type					
Sample Description					
Wt. wet soil + container (g)					
Wt. dry soil + container (g)					
Weight of container (g)					
Moisture Content (%)					

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	Red Tail Ventura	Tested By:	J. Gonzalez	Date:	08/02/22
Project No.:	13582.001	Checked By:	A. Santos	Date:	08/03/22
Boring No.:	LB-2	Depth (ft.):	0-10		
Sample No.:	B-1				
Soil Identification:	Olive lean clay (CL)				

Preparation Method: [X] Moist [ ] Dry — [X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft³) 0.03330 — *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3619	3735	3764	3737		
Weight of Mold (g)	1826	1826	1826	1826		
Net Weight of Soil (g)	1793	1909	1938	1911		
Wet Weight of Soil + Cont. (g)	486.5	452.3	445.6	456.9		
Dry Weight of Soil + Cont. (g)	448.2	408.0	390.7	394.1		
Weight of Container (g)	39.4	39.6	39.5	38.0		
Moisture Content (%)	9.37	12.02	15.63	17.64		
Wet Density (pcf)	118.7	126.4	128.3	126.5		
Dry Density (pcf)	108.5	112.8	111.0	107.5		

Maximum Dry Density (pcf) **113.0** **Optimum Moisture Content (%)** **12.9**

PROCEDURE USED

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

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

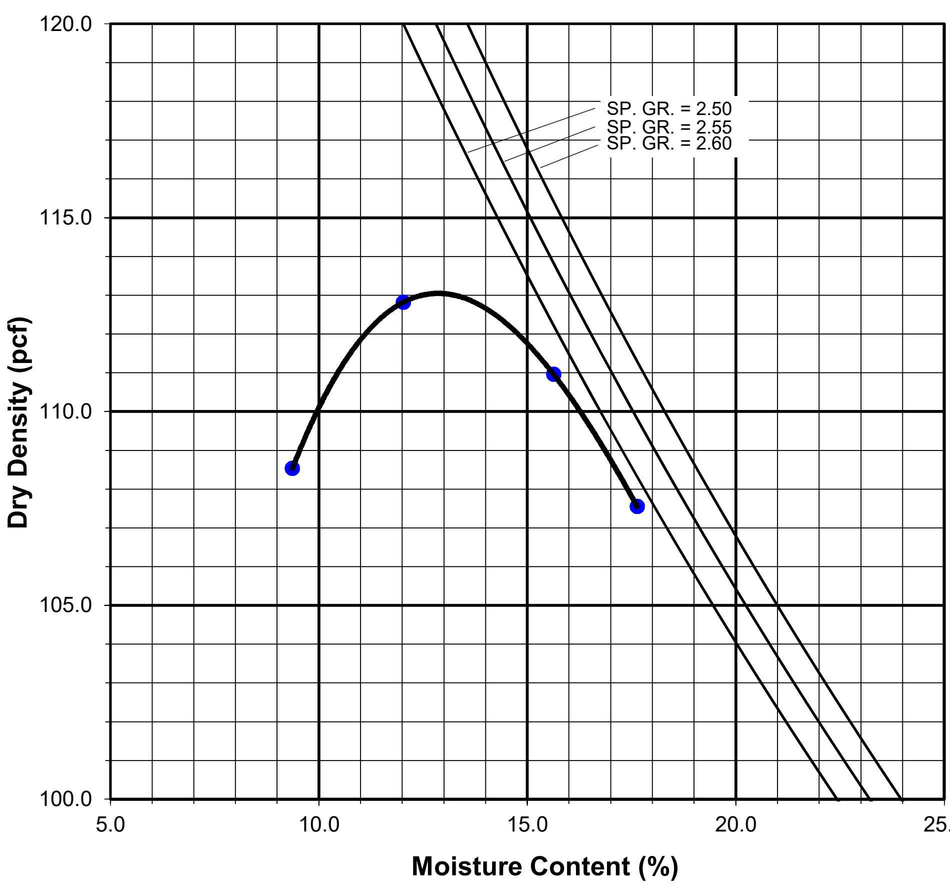
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



Boring No.	LB-2	LB-2	LB-2	LB-2	LB-2			
Sample No.	R-2	S-5	R-5	S-6	R-6			
Depth (ft.)	12.5	30.0	35.0	40.0	45.0			
Sample Type	Ring	SPT	Ring	SPT	Ring			
Soil Identification	Brown silty clay with sand (CL-ML)s	Brown lean clay with sand (CL)s	Brown clayey sand (SC)	Brown lean clay with sand (CL)s	Brown sandy lean clay s(CL)			
Moisture Correction								
Wet Weight of Soil + Container (g)	0.0	0.0	0.0	0.0	0.0			
Dry Weight of Soil + Container (g)	0.0	0.0	0.0	0.0	0.0			
Weight of Container (g)	1.0	1.0	1.0	1.0	1.0			
Moisture Content (%)	0.0	0.0	0.0	0.0	0.0			
Sample Dry Weight Determination								
Weight of Sample + Container (g)	716.3	749.1	903.5	967.8	858.9			
Weight of Container (g)	251.1	110.0	108.5	107.3	107.6			
Weight of Dry Sample (g)	465.2	639.1	795.0	860.5	751.3			
Container No.:								
After Wash								
Method (A or B)	A	A	A	A	A			
Dry Weight of Sample + Cont. (g)	385.7	270.7	537.5	295.5	335.0			
Weight of Container (g)	251.1	110.0	108.5	107.3	107.6			
Dry Weight of Sample (g)	134.6	160.7	429.0	188.2	227.4			
% Passing No. 200 Sieve	**71.1**	**74.9**	**46.0**	**78.1**	**69.7**			
% Retained No. 200 Sieve	28.9	25.1	54.0	21.9	30.3			

Leighton

PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name: Red Tail Ventura
Project No.: 13582.001
Tested By: S. Felter Date: 07/25/22

Leighton

ATTERBERG LIMITS
ASTM D 4318

Project Name:	Red Tail Ventura	Tested By:	S. Felter	Date:	07/25/22
Project No. :	13582.001	Input By:	J. Ward	Date:	08/07/22
Boring No.:	LB-2	Checked By:	J. Ward		
Sample No.:	S-3	Depth (ft.)	15.0		
Soil Identification:	Brown lean clay (CL)				

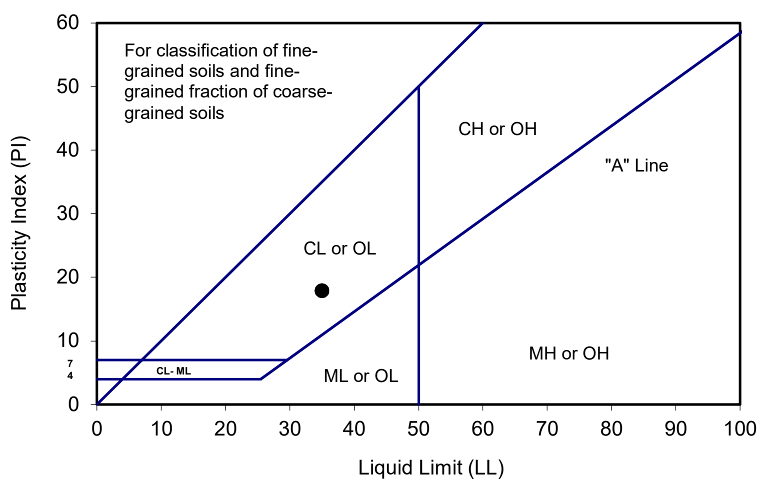
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			30	25	20	
Wet Wt. of Soil + Cont. (g)	10.08	10.25	20.84	20.19	20.31	
Dry Wt. of Soil + Cont. (g)	8.77	8.90	15.75	15.22	15.28	
Wt. of Container (g)	1.06	1.09	1.11	1.14	1.12	
Moisture Content (%) [Wn]	16.99	17.29	34.77	35.30	35.52	

Liquid Limit	**35**
Plastic Limit	**17**
Plasticity Index	**18**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 10.95

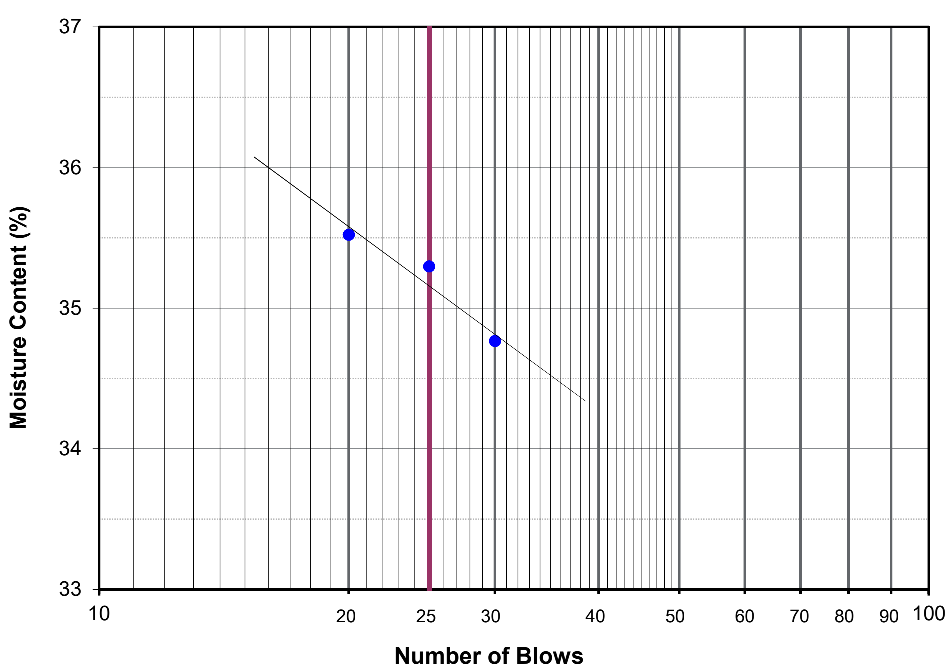
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

- [ ] Wet Preparation
  Multipoint - Wet
- [X] Dry Preparation
  Multipoint - Dry
- [X] Procedure A
  Multipoint Test
- [ ] Procedure B
  One-point Test



Atlantic Consultants

112 Bunker Court
Folsom, CA 95630
(ph) 916.849.6420 (fax) 916.983.1838
Kerri@AtlanticCorrosionEngineers.com
corrprincess@ardennet.com
www.AtlanticCorrosionEngineers.com

May 31, 2008

Gorian and Associates, Inc.
Attention: Chip DeVault
3595 Old Conejo Road
Thousand Oaks, CA 91320

Atlantic Job No.: 2008-031

Subject: Soil Chemistry Analysis for Gorian Job # 2503-0-0-100
1 Sample: FPA Land Development, Victoria Corp. Center (C-1)

Sample Number	As Rec'd Resistivity (ohm-cm)	[1]Minimum Resistivity (ohm-cm)	[2]pH	[3]Sulfate %	[3]Chloride %	[4]Ammonia %	[5]Keldahl Nitrogen %	(As Rec'd) Description
C-1	4,320,000	520	6.89	0.1460	0.0119	0.0020	0.1900	Medium Brown Clay, dry

NOTE: SAMPLES WERE ANALYZED IN ACCORDANCE WITH THE FOLLOWING METHODS.
1. MINIMUM RESISTIVITY DETERMINED BY SOIL BOX METHOD, (PER ASTM G-57)
2. PH MEASURED BY POTENTIOMETRIC METHOD USING STANDARD ELECTRODES. (PER CAL TRANS. #643)
3. CHLORIDE AND SULFATE WERE ANALYZED IN ACCORDANCE WITH EPA METHODS FOR CHEMICAL ANALYSIS FOR WATER AND WASTE, NO. 300 EPA-600/4-79-020. CONCENTRATION BY WEIGHT OF DRY SOIL.
4. AMMONIA WAS ANALYZED IN ACCORDANCE WITH EPA METHOD 350.2
5. KELDAHL NITROGEN WAS ANALYZED IN ACCORDANCE WITH EPA METHOD 351.2

CONCLUSIONS:

Material	Corrosion Class
Concrete	Moderate for sulfate exposure, negligible for chloride exposure. pH is neutral to slightly basic. (UBC Table 19-A-4)
Steel Cast/Ductile Iron Mortar Coated Steel	Corrosive
Copper Piping	Corrosive due to low resistivity, and the presence of nitrogen and ammonia exposure.

The test results and recommendations are based on the sample submitted, which may not be representative of overall site conditions. Additional sampling may be required to more fully characterize soil conditions

Sincerely,

Kerri M Howell

Kerri M. Howell, P.E. President

APPENDIX B

LABORATORY TESTING

General

Laboratory test results on selected relatively undisturbed and bulk samples are presented below. Tests were performed to evaluate the physical and engineering properties of the encountered earth materials, including field moisture and density, compaction characteristics, expansion potential, shear strength, consolidation potential, grain size analyses, and hydrometer analyses.

Field Density and Moisture Tests

In situ dry density and moisture content were determined from the relatively undisturbed samples obtained during drilling operations. The test results and a detailed description of the soils encountered are shown on the attached Logs of Subsurface Data in Appendix A.

Maximum Density-Optimum Moisture

Maximum density/optimum moisture tests (compaction characteristics) were performed on selected samples of the encountered materials. The tests were performed per ASTM D 1557 test method. The results are as follows:

Sample Identification	Visual Soil Classification	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-1 @ 0 - 1'	Alluvium – grayish brown sandy silty clay	119	12.0
B-2 @ 0 - 1'	Alluvium – grayish brown sandy silty clay	113	13.0

Soil Expansion Index Tests

A sample of the encountered soil was tested for expansiveness using the Expansion Index Test method (UBC 29-2). The results are as follows:

Sample Identification	Visual Soil Classification	Expansion Index	Expansion Index Range
B-1 @ 0 - 1'	Alluvium – grayish brown sandy silty clay	90	51 – 90
B-2 @ 0 - 1'	Alluvium – grayish brown sandy silty clay	75	51 – 90

Load Consolidation Tests

Load consolidation tests were conducted on relatively undisturbed soil samples. Test loads were added in increments to a maximum pressure of 8,000 psf or 9400 psf. Water was added at a normal pressure of 1000, 1175 and and 2000 psf to study the effect of moisture infiltration on potential consolidation behavior. The results are attached as graphic summaries on Figure B.1 through B.4.

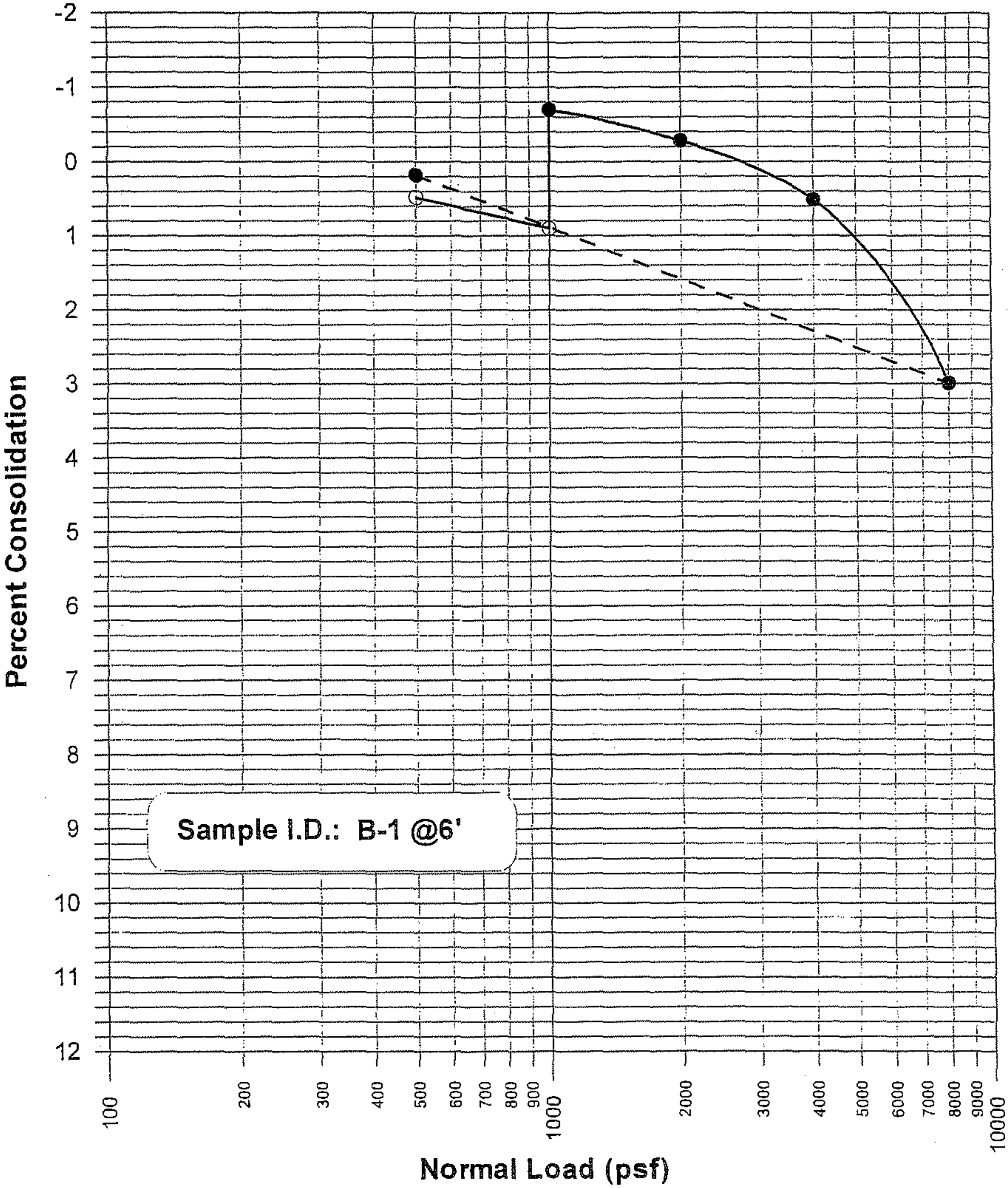
Direct Shear Tests

A direct shear test was performed on relatively undisturbed samples of the earth materials encountered during our exploratory program. The sample sets were saturated before being sheared under normal pressures ranging from 900 to 3,600 psf at a rate of 0.05 inches per minute. The ultimate shear strength results are attached as a graphic summaries on Figure B.5 and B.6.

Particle Size Analyses

Particle size analyses were performed on selected SPT samples of materials encountered in the boring B-3. The tests were performed to evaluate the percentage of fines (passing sieve # 200) and the percentage of clay, silt and sand (hydrometer test). Test results are indicated on the Logs of Subsurface Data in Appendix A.

LOAD CONSOLIDATION TEST



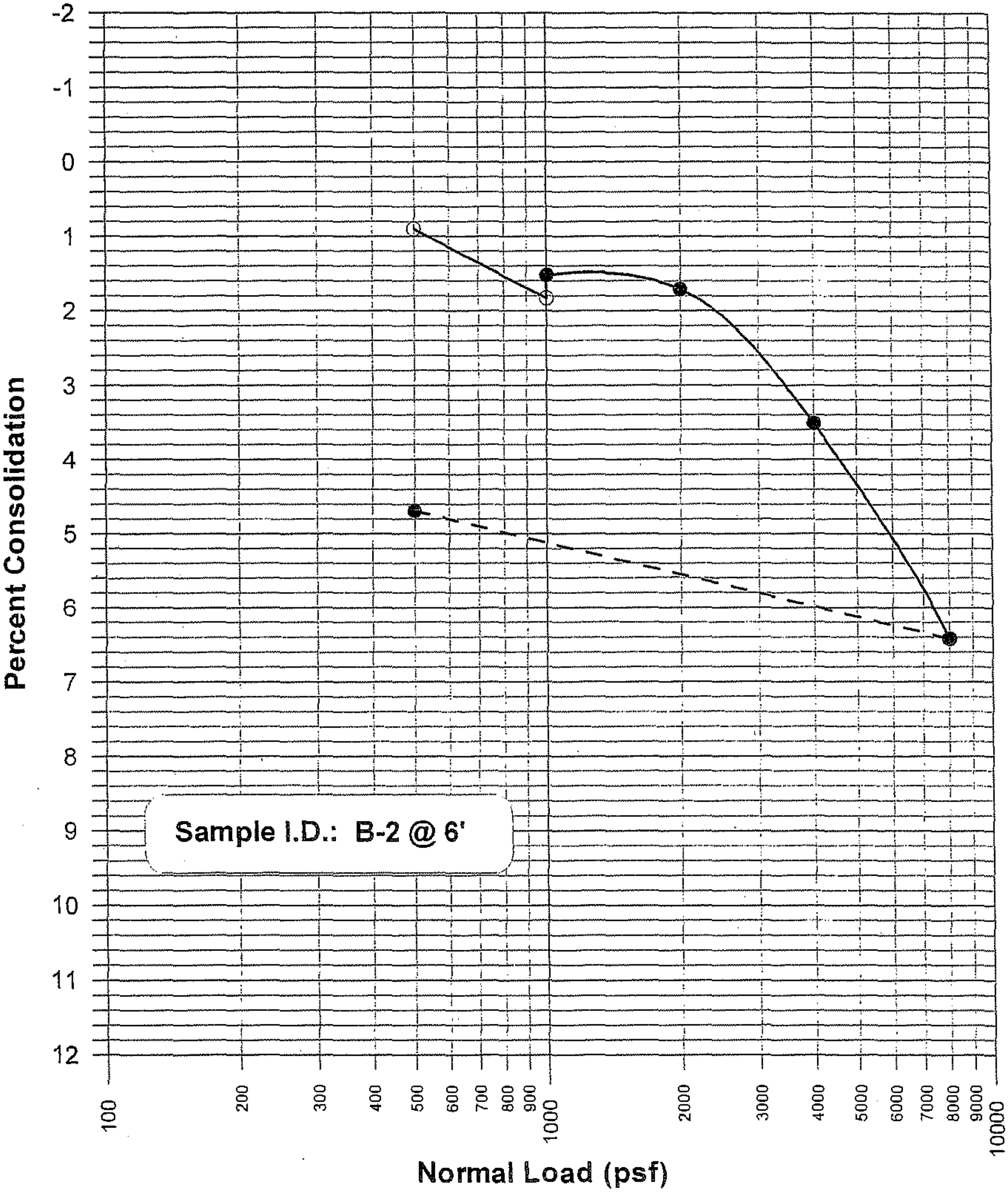
○ Field Moisture

● Effect of Adding Moisture

– – – – Rebound

Figure B.1
Work Order: 2503-0-0-10
GORIAN AND ASSOCIATES, INC.

LOAD CONSOLIDATION TEST



○ Field Moisture

● Effect of Adding Moisture

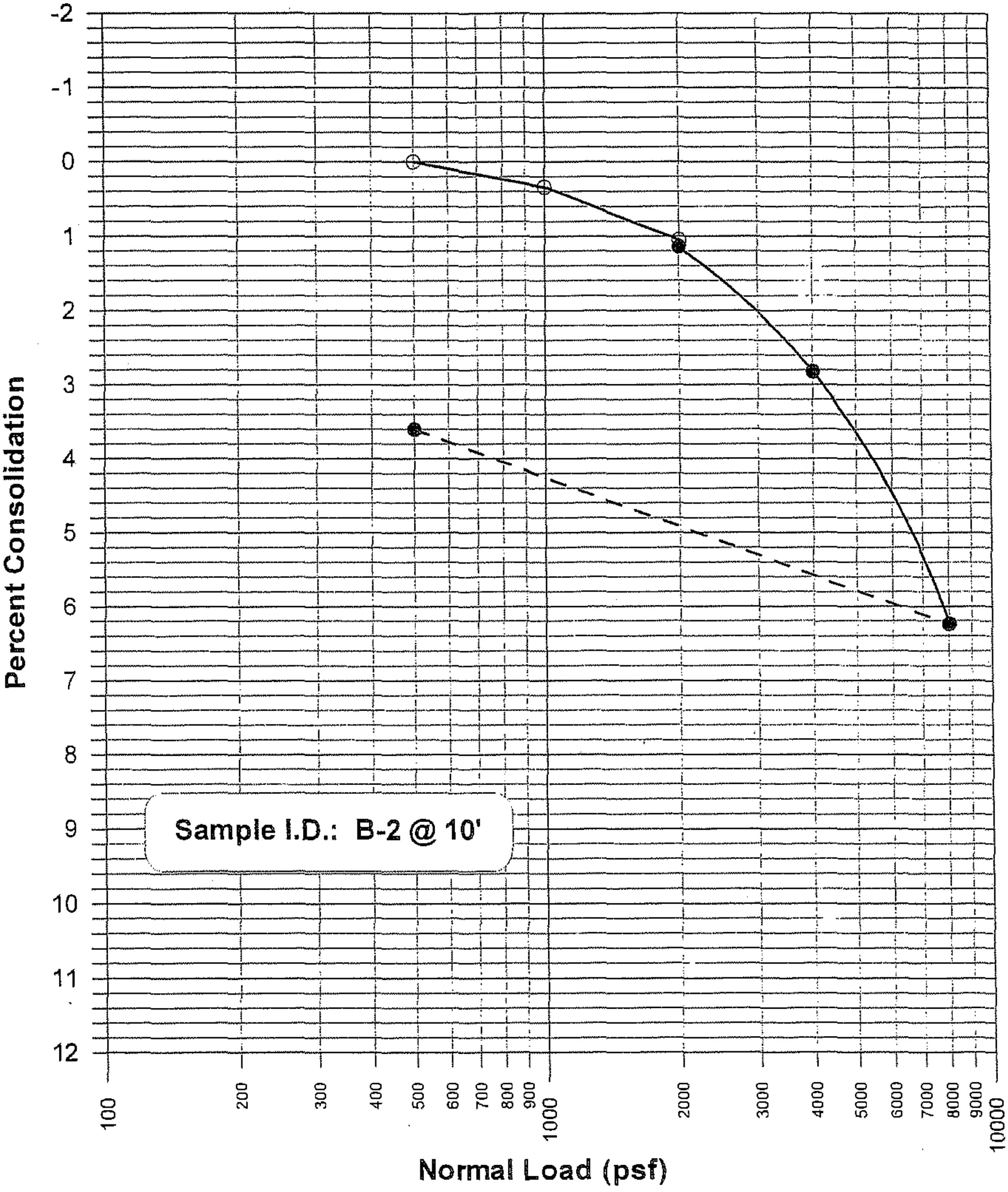
– – – – – Rebound

Figure B.2

Work Order: 2503-0-0-10

GORIAN AND ASSOCIATES, INC.

LOAD CONSOLIDATION TEST



○ Field Moisture

● Effect of Adding Moisture

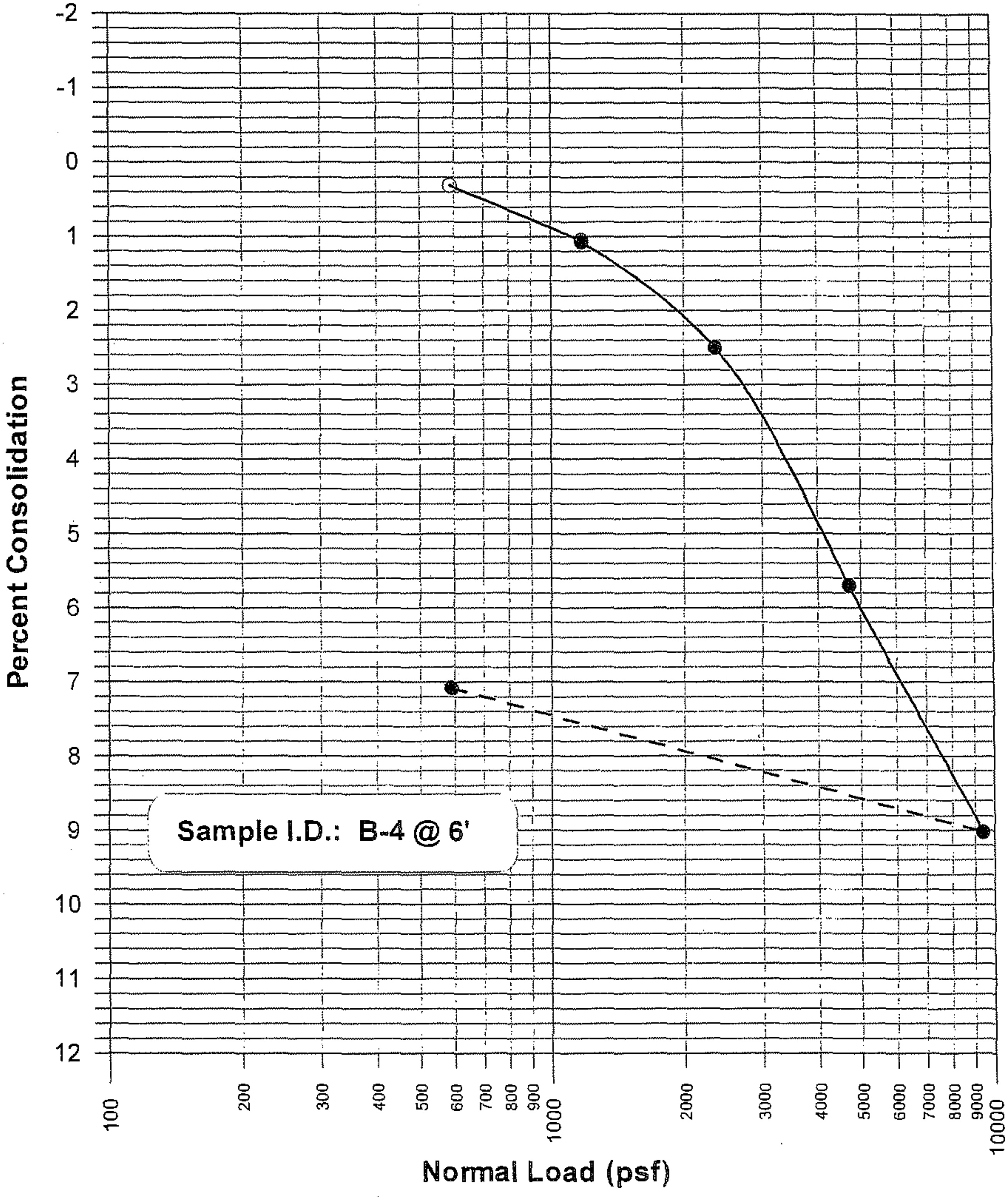
– – – – – Rebound

Figure B.3

Work Order: 2503-0-0-10

GORIAN AND ASSOCIATES, INC.

LOAD CONSOLIDATION TEST



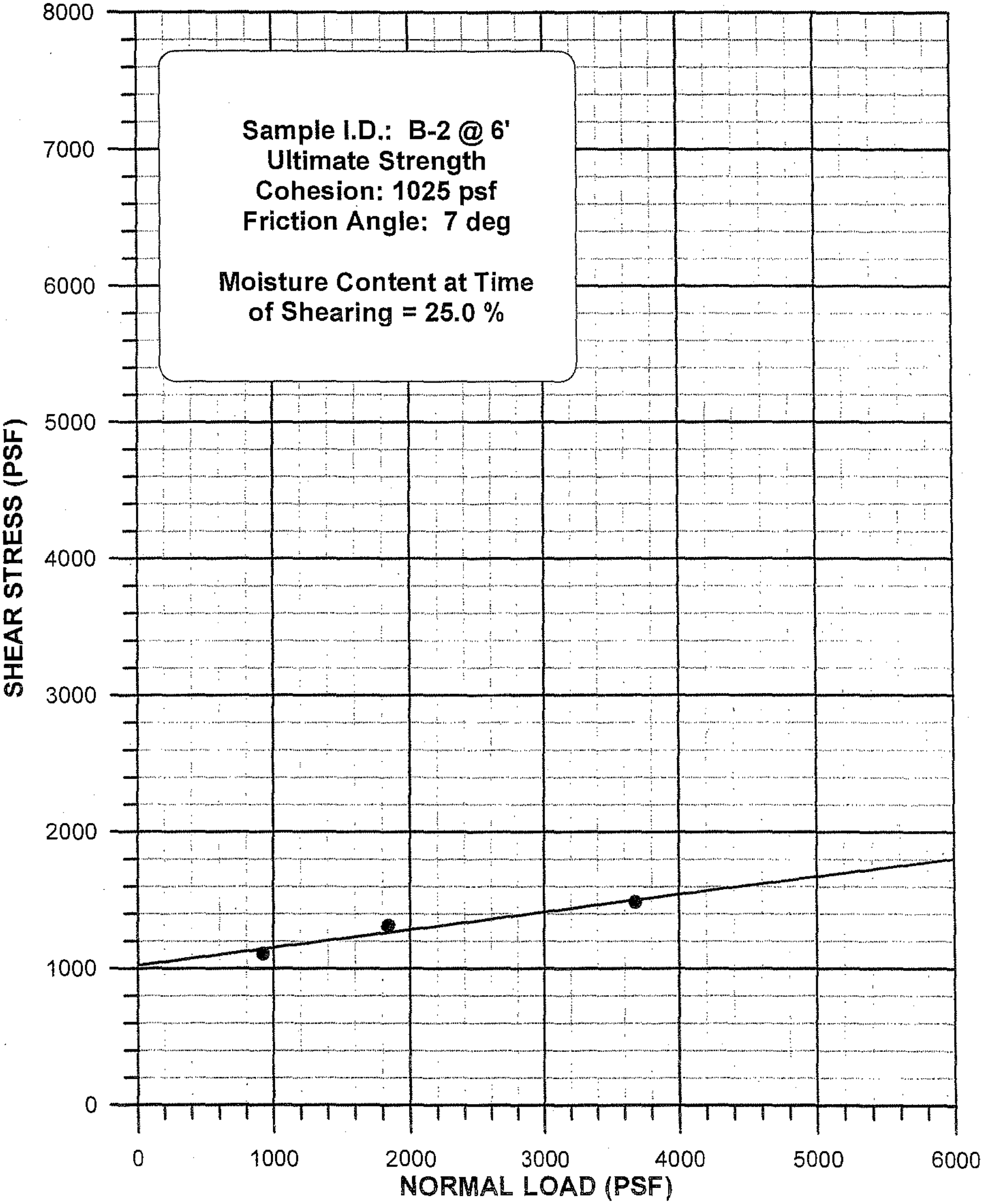
○ Field Moisture

● Effect of Adding Moisture

– – – – – Rebound

Figure B.4
Work Order: 2503-0-0-10
GORIAN AND ASSOCIATES, INC.

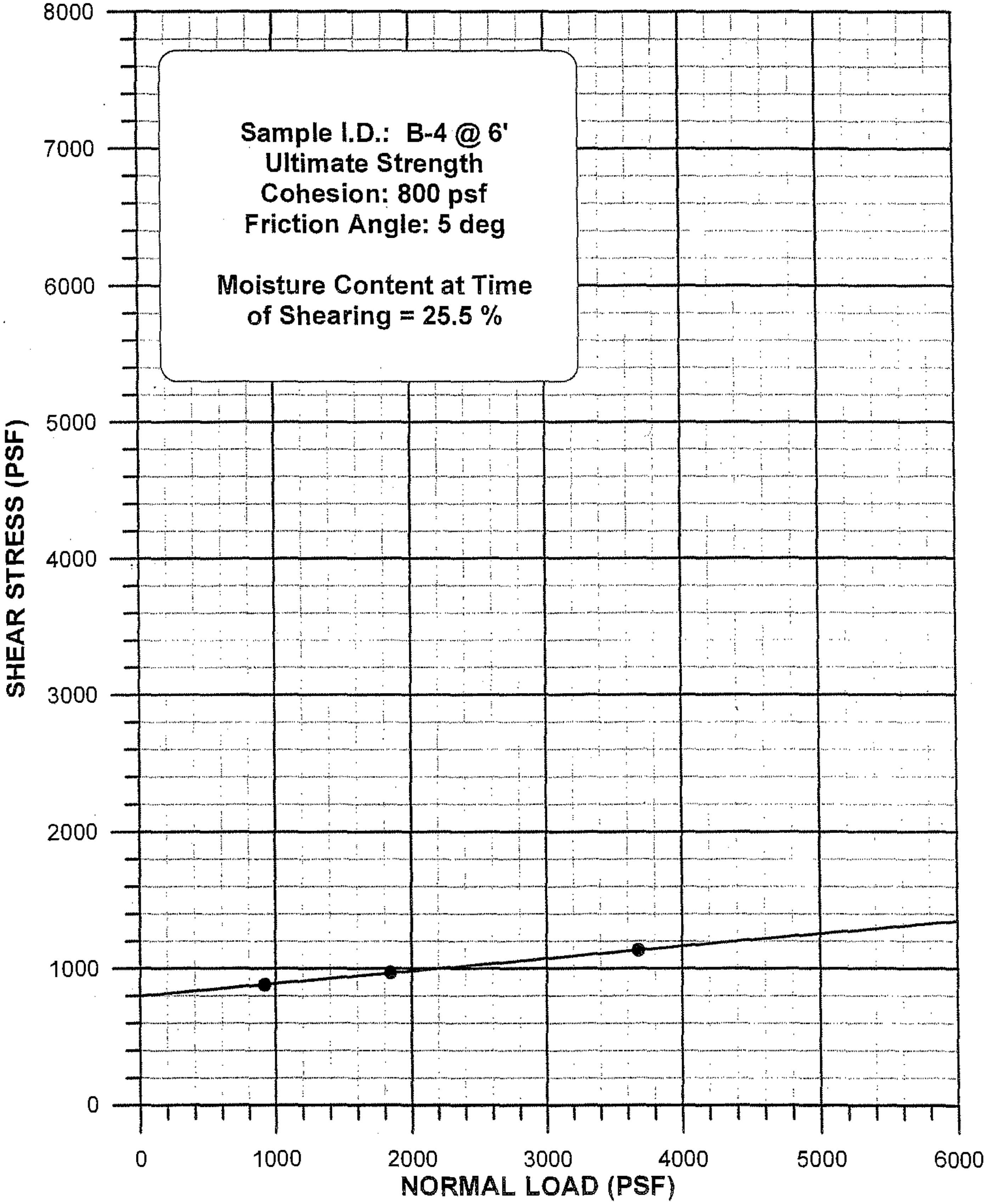
RESULTS OF DIRECT SHEAR TEST
UNDISTURBED, SATURATED SAMPLE



Explanation: B-9 @ 12' = Sample taken from Boring 9 at a depth of 12'

Figure B.5
Work Order: 2503-0-0-10
GORIAN AND ASSOCIATES, INC.

RESULTS OF DIRECT SHEAR TEST
UNDISTURBED, SATURATED SAMPLE



Explanation: B-9 @ 12' = Sample taken from Boring 9 at a depth of 12'

Figure B.6
Work Order: 2503-0-0-10
GORIAN AND ASSOCIATES, INC.

"We Test the Earth"

PACIFIC MATERIALS LABORATORY, INC.

June 6, 2008
Lab No. 33686-3
File No. 08-7492-3

Gorian & Associates, Inc.
3595 Old Conejo Road
Thousand Oaks, CA 91320

SUBJECT: **R-Value Testing**
Samples Delivered to Laboratory

Gentlemen:

Pursuant to your request, R-Value testing was performed on soil samples delivered to our laboratory. R-Value testing was performed in accordance with California Test 301-F criteria. The test results follow:

R-VALUE RESULTS

PROJECT: FPA Land Development
LOCATION: Parcel at Victoria and Olivas Park

Soil Description: Black Brown Clay

ITEM	1	2	3
Compaction Pressure - psi	75/100	100/125	125/150
Initial Moisture - %	25.0	25.0	25.0
Moisture at Compaction - %	27.6	26.6	25.5
Density - pcf	93.7	95.5	97.1
R-Value	6	8	9
Exudation Pressure	289	370	430
Expansion Pressure thickness ft.	0.17	0.63	0.80

Assigned R-Value: 6*

Footnote:
*Verify R-value based upon expansion thickness *(see California Test 301-F procedures).*

Thank you for allowing *Pacific Materials Laboratory, Inc.* to be of service. If we may be of further service regarding this or other geotechnical issues, please do not hesitate to call (805) 482-9801, fax (805) 445-6551 or write.

Respectfully Submitted,
PACIFIC MATERIALS LABORATORY, INC.

REGISTERED PROFESSIONAL ENGINEER
DOUGLAS C PAPAY
GE 664
Exp 3/31/09
GEOTECHNICAL
STATE OF CALIFORNIA

Douglas C. Papay, GE 664
President

DCP:ma
cc: Addressee (3)

150 Wood Road, Suite B • Camarillo, CA 93010 • Office (805) 482-9801~Fax (805) 445-6551 • Email: pacificmaterialslab@msn.com

APPENDIX D

Liquefaction Analysis

Leighton

SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA

PROJECT INFORMATION

Project Name	Redtail
Project No.	13582.001
Project Location	Ventua
Analyzed By	RPH
Reviewed By	

SEISMIC DESIGN PARAMETERS

Earthquake Moment Magnitude, M_w	7.53
Peak Ground Acceleration, A_{max}	0.94 g
Factor of Safety Against Liquefaction, FS	1.20

BORING DATA AND SITE CONDITIONS

Boring No.	B-1
Ground Surface Elevation	100.00 feet
Proposed Grade Elevation	100.00 feet
GWL Depth Measured During Test	50.00 feet
GWL Depth Used in Design	15.00 feet
Borehole Diameter	8.00 inches
Hammer Weight	140.00 pounds
Hammer Drop	30.00 inches
Hammer Energy Efficiency Ratio, ER	80.00 %
Hammer Distance to Ground Surface	5.00 feet
Topographic Site Condition:	TSC1 (Level Ground with No Nearby Free Face)
- Ground Slope, S	0.00 %
- Free Face (L/H) Ratio	N/A H = feet

SUMMARY OF RESULTS

Severity of Liquefaction:

Total Thickness of Liquefiable Soils: 10.00 feet (cumulative total thickness in the upper 65 feet)

Liquefaction Potential Index (LPI): 9.76 *** (High risk, with moderate liquefaction effects)

Seismic Ground Settlements:	**Analysis Method**	**Upper 30 feet**	**Upper 50 feet**	**Upper 65 feet**	
Seismic Compression Settlement:	Pradel (1998)	0.00 inches	0.00 inches	0.00 inches	(Dry/Unsaturated Soils)
Liquefaction-Induced Settlement:	Ishihara and Yoshimine (1992)	0.00 inches	1.48 inches	1.48 inches	(Saturated Soils)
Total Seismic Settlement:		0.00 inches	1.48 inches	1.48 inches	

Seismic Lateral Displacements:	**Analysis Method**	**Upper 30 feet**	**Upper 50 feet**	**Upper 65 feet**	
Cyclic Lateral Displacement:	Tokimatsu and Asaka (1998)	0.00 inches	0.40 inches	0.40 inches	(During Ground Shaking)
Lateral Spreading Displacement:	Zhang et al. (2004)	0.00 inches	0.00 inches	0.00 inches	(After Ground Shaking)

NOTES AND REFERENCES

\+ This method of analysis is based on observed seismic performance of level ground sites using correlation with normalized and fines-corrected SPT blow count, $(N_1)_{60cs} = f\{(N_1)_{60}, FC\}$ where $(N_1)_{60} = N_{field}\, C_N\, C_E\, C_B\, C_R\, C_S$

++ Liquefaction susceptibility screening is performed to identify soil layers assessed to be non-liquefiable based on laboratory test results using the criteria proposed by Cetin and Seed (2003), Bray and Sancio (2006), or Idriss and Boulanger (2008).

* FS_{liq} = Factor of Safety against liquefaction = (CRR/CSR), where $CRR = CRR_{7.5}\, MSF\, K_\sigma\, K_\alpha$, MSF = Magnitude Scaling Factor, $K_\sigma = f[(N_1)_{60}, \sigma'_{vo}]$, K_α =1.0, (level ground), CSR = Cyclic Stress Ratio = $0.65\, A_{max}\, (\sigma_{vo}/\sigma'_{vo})\, r_d$, and $CRR_{7.5}$ = Cyclic Resistance Ratio is a function of $(N_1)_{60cs}$ and corrected for an earthquake magnitude M_w of 7.5.

** Residual strength values of liquefied soils are based on correlation with post-earthquake, normalized and fines-corrected SPT blow count derived by Idriss and Boulanger (2008).

*** Based on Iwasaki et al. (1978) and Toprak and Holzer (2003)

+ Reference: Boulanger, R.W. and Idriss, I.M. (2014), "CPT and SPT Based Liquefaction Triggering Procedures," University of California Davis, Center for Geotechnical Modeling Report No. UCD/CGM-14/01, 1-134.

INPUT SOIL PROFILE DATA

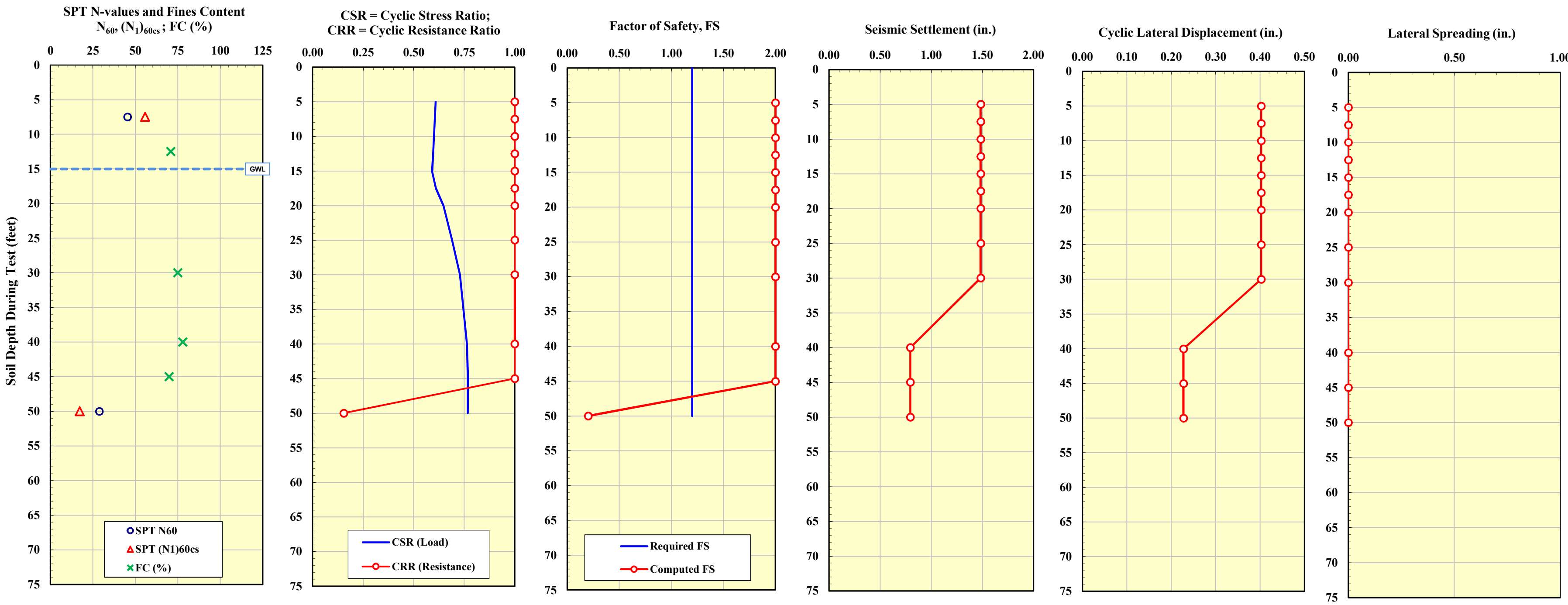
Depth to Top of Soil Layer (feet)	Depth to Bottom of Soil Layer (feet)	Material Type USCS Group Symbol (ASTM D2487)	Liquefaction Susceptibility Screening ++ Susceptible Soil? (Y/N)	Total Soil Unit Weight γ_t (pcf)	Type of Soil Sampler	Field SPT Blow Count N_{field} (blows/ft)	Fines Content FC (%)
0.00	5.00	SM	N	120.00	MCal	9.00	
5.00	7.50	SM	Y	120.00	SPT1	37.00	
7.50	10.00	CL	N	120.00	MCal	17.00	
10.00	12.50	CL	N	120.00	SPT1	11.00	71.00
12.50	15.00	CL	N	125.00	MCal	1.00	
15.00	17.50	CL	N	125.00	SPT1	13.00	
17.50	20.00	CL	N	125.00	MCal	2.00	
20.00	25.00	CL	N	125.00	SPT1	18.00	
25.00	30.00	CL	N	130.00	MCal	8.00	75.00
30.00	35.00	SC	Y	130.00	SPT1	20.00	46.00
35.00	40.00	CL	N	130.00	MCal	7.00	78.00
40.00	45.00	CL	N	130.00	SPT1	25.00	70.00
45.00	50.00	SC	Y	130.00	MCal	29.00	

LIQUEFACTION TRIGGERING ANALYSIS BASED ON R.W. BOULANGER AND I.M. IDRISS (2014) METHOD +

Total Vert. Stress (Design) σ_{vo} (psf)	Effective Vert. Stress (Design) σ'_{vo} (psf)	SPT Corr. for Vert. Stress C_N	SPT Corr. for Hammer Energy C_E	SPT Corr. for Borehole Size C_B	SPT Corr. for Rod Length C_R	SPT Corr. for Sampling Method C_S	Corrected SPT Blow Count N_{60}	Normalized SPT Blow Count $(N_1)_{60}$	Fines Corrected SPT Blow Count $(N_1)_{60cs}$	Shear Stress Reduction Coefficient r_d	Correction for High Overburden Stress K_σ	Cyclic Stress Ratio CSR	Cyclic Resistance Ratio CRR	Factor of Safety * FS_{liq}	Liquefaction Analysis Results	Residual Shear Strength ** S_r (psf)	Seismic Porewater Pressure Ratio r_u (%)	Cumulative Seismic Settlement (inches)	Cumulative Cyclic Lateral Displacement (inches)	Cumulative Lateral Spreading Displacement (inches)
300.00	300.00									1.000		0.609						1.48	0.40	0.00
750.00	750.00	1.229	1.333	1.150	0.800	1.000	45.4	55.8	55.8	0.992	1.100	0.604						1.48	0.40	0.00
1,050.00	1,050.00									0.985		0.600						1.48	0.40	0.00
1,350.00	1,350.00									0.978		0.596						1.48	0.40	0.00
1,656.25	1,656.25									0.970		0.591						1.48	0.40	0.00
1,968.75	1,890.75									0.962		0.610						1.48	0.40	0.00
2,281.25	2,047.25									0.954		0.647						1.48	0.40	0.00
2,750.00	2,282.00									0.940		0.690						1.48	0.40	0.00
3,387.50	2,607.50									0.920		0.728						1.48	0.40	0.00
4,037.50	2,945.50	0.746	1.333	1.150	1.000	1.000	30.7	22.9	28.5	0.899	0.895	0.751	0.359	0.48	LIQUEFY	961.09	100.00	1.48	0.40	0.00
4,687.50	3,283.50									0.878		0.763						0.80	0.23	0.00
5,337.50	3,621.50									0.856		0.768						0.80	0.23	0.00
5,987.50	3,959.50	0.601	1.333	1.150	1.000	0.650	28.9	17.4	17.4	0.833	0.867	0.767	0.153	0.20	LIQUEFY	951.13	100.00	0.80	0.23	0.00

SIMPLIFIED LIQUEFACTION HAZARDS ASSESSMENT USING STANDARD PENETRATION TEST (SPT) DATA

PROJECT INFORMATION	
Project Name	Redtail
Project No.	13582.001
Project Location	Ventua
Analyzed By	RPH
Reviewed By	



Analysis Methods Used ==>>

Liquefaction Triggering:		Seismic Settlements:	Cyclic Lateral Displacements:	Lateral Spreading:
Boulanger-Idriss (2014)	Above GWL:	Pradel (1998)	Pradel (1998)	Zhang et al. (2004)
	Below GWL:	Ishihara and Yoshimine (1992)	Tokimatsu and Asaka (1998)	

REFERENCES:

1. Boulanger, R.W. and Idriss, I.M. (2014), "CPT and SPT Based Liquefaction Triggering Procedures," University of California Davis, Center for Geotechnical Modeling Report No. UCD/CGM-14/01, 1-134.
2. Bray, J.D., and Sancio, R.B. (2006). "Assessment of the liquefaction susceptibility of fine-grained soils," Journal of Geotech. and Geoenv. Engineering, ASCE 132 (9), 1165-1177.
3. Cetin, K.O. and Seed, R.B., et al. (2004), "Standard penetration test-based probabilistic and deterministic assessment of seismic soil liquefaction potential," Journal of Geotech. and Geoenv. Engineering, ASCE 130 (12), 1314-1340.
4. Idriss, I.M. and Boulanger, R.W. (2008), "Soil Liquefaction During Earthquakes", Earthquake Engineering Research Institute (EERI), Monograph MNO-12.
5. Ishihara, K. and Yoshimine, M. (1992), "Evaluation of settlements in sand deposits following liquefaction during earthquakes," Soils and Foundations, Japanese Geotechnical Society, 32 (1), 173-188.
6. Iwasaki, T., et al. (1978), "A practical method for assessing soil liquefaction potential based on case studies at various sites in Japan," Proceedings Of 3rd International Conference of Microzonation, San Francisco, 885-896.
7. Olson, S.M. and Johnson, C.I. (2008), "Analyzing Liquefaction-Induced Lateral Spreads Using Strength Ratios," Journal of Geotech. and Geoenv. Engineering, ASCE 134 (8), 1035-1049.
8. Pradel, D. (1998), "Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils," Journal of Geotechnical Engineering, ASCE 124 (4), pp. 364-368.
9. Seed, R.B. and Harder, L.F. (1990), "SPT-based analysis of cyclic pore pressure generation and undrained residual strength, Proceedings Of Seed Memorial Symposium, Vancouver, B.C., 351-376.
10. Tokimatsu, K. and Seed, H.B. (1987), "Evaluation of settlements in sands due to earthquake shaking," Journal of Geotechnical Engineering, ASCE 113 (GT8), 861-878.
11. Tokimatsu, K. and Asaka, Y. (1998), "Effects of liquefaction-induced ground displacementson pile performance in the 1995 Hyogoken-Nambu Earthquake," Soils and Foundations, Special Issue, Japan Geotechnical Society, 163-177.
12. Toprak, S. and Holzer, T.L. (2003), "Liquefaction Potential Index: Field Assessment," Journal of Geotechnical and Geoenviromental Engineering, ASCE 129 (4), 315-322.
13. Youd, T.L, Idriss, I.M., et al. (2001), "Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF Workshops", Journal of Geotech. and Geoenv. Engineering, ASCE 127 (10), 817-833.
14. Zhang, G, Robertson, P.K. and Brachman, R.W.I. (2004), "Estimating liquefaction-induced lateral displacement using the standard penetration test or cone penetration test," Journal of Geotech. and Geoenv. Engineering, ASCE 130 (8), 861-871.

APPENDIX E

Infiltration Testing

Leighton

Results of Falling Head Infiltration Test

Project: 13582.001

Exploration #/Location:	B-6
Depth Boring drilled, bgs (ft):	5
Tested by:	RM
USCS Soil Type in test zone:	SM
Weather (start to finish):	Clear
Water Source/pH:	
Measured boring diameter:	8 in.
Depth to GW or aquitard, bgs:	100 ft
Well Prep:	

4 in. Well Radius

Initial estimated Depth to Water Surface (in.):	46
Average depth of water in well, "h" (in.):	13
approx. h/r:	3.1
Tu (Fig. 8) (ft):	96.2
Tu>3h?:	yes, OK

Cross-sectional area for flow calcs based on Δh

Well pack sand porosity:	0.4
Casing outer diameter, in.:	2.3
Casing inner diameter, in.:	2.1
Cross-sectional area, in.^2:	21.9

Use of Barrels:	No
Use of Flow Meter:	No
Test Type:	Falling Head

	ft	in.	Total (in.)
Depth to bottom of well measured from top of a	4.88 ft		58.5
Casing stickup measured above top of auger (	0. ft		0
Depth to top of sand from top of casing	3.67 ft		

Depth of well bottom below top of casing (in): 59

Field Data | **Calculations**

Date	Time	Depth to WL in Boring (measured from top of casing) ft	in.	Water Temp (deg F)	Refilled? (or Comments)	Δt (min)	Total Elapsed Time (min)	Depth to WL in well (in.)	h, Height of Water in Well (in.)	Δh (in.)	Avg. h	Vol Change (in.^3) from supply	from Δh	Total	Flow (in^3/ min)	q, Flow (in^3/ hr)	Average Infiltration Surface Area, (in^2)	V (Fig 9)	K20, Coef. Of Perme-ability at 20 deg C (in./hr)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date	Start time:																			
7/14/2022	10:43																			
7/14/22	10:43		44				0	44.0	14.5											
7/14/22	10:53		48.38			10	10	48.4	10.1	-4.375	12	0	96	96	10	575	360	0.9	0.9	1.5
7/14/22	10:58		44				15	44.0	14.5											
7/14/22	11:08		48			10	25	48.0	10.5	-4	13	0	88	88	9	526	364	0.9	0.8	1.3
7/14/22	11:11		44				28	44.0	14.5											
7/14/22	11:21		48			10	38	48.0	10.5	-4	13	0	88	88	9	526	364	0.9	0.8	1.3
7/14/22	11:24		44				41	44.0	14.5											
7/14/22	11:34		47.63			10	51	47.6	10.9	-3.625	13	0	79	79	8	476	369	0.9	0.7	1.2
7/14/22	11:37		44				54	44.0	14.5											
7/14/22	11:47		47.5			10	64	47.5	11.0	-3.5	13	0	77	77	8	460	371	0.9	0.6	1.1
7/14/22	11:51		44				68	44.0	14.5											
7/14/22	12:01		47.38			10	78	47.4	11.1	-3.375	13	0	74	74	7	444	372	0.9	0.6	1.1
											Raw Rate for design, prior to application of adjustment factors:									

APPENDIX F

Utility Density Tests by Gorian and Associates, Inc.

FPA VENTURA OLIVAS, LLC
Victoria Corporate Center

W.O.: 2503-1-R-301

TABLE I
COMPACTION TEST SUMMARY

TEST NUMBER	DATE	DEPTH BELOW FINISHED SURFACE (FT.)	MOISTURE CONTENT (%)	UNIT DRY DENSITY (LBS/CU.FT.)	RELATIVE COMPACTION (%)	SOIL TYPE
SEWER TRENCH BACKFILL						
1	10/20/17	2.0	17.8	107.0	90	2
2	10/20/17	2.0	18.9	99.1	84*	2
2A	10/20/17	2.0	18.7	108.6	92	2
3	10/20/17	1.0	17.6	107.1	90	2
4	10/20/17	1.0	18.0	105.9	90	1
5	10/24/17	1.0	16.9	105.6	90	1
6	10/24/17	1.0	17.3	106.2	91	1
7	10/24/17	1.0	17.7	106.0	90	1
8	10/24/17	1.0	17.5	105.4	90	1
9	02/26/18	4.5	16.7	106.1	91	1
10	02/26/18	2.5	19.4	98.7	84*	1
10A	02/26/18	2.5	15.0	105.4	90	1
11	02/26/18	5.0	17.8	106.3	91	1
12	02/26/18	2.5	15.7	105.6	90	1
13	03/09/18	3.0	16.9	105.8	90	1
14	03/09/18	2.5	19.9	108.2	92	1
15	03/09/18	2.5	19.6	106.3	91	1
16	03/09/18	2.0	19.5	106.4	91	1
17	04/05/18	2.5	14.3	105.1	90	1
18	04/05/18	2.0	14.4	105.2	90	1
19	04/05/18	2.0	14.2	108.9	93	1

* - INDICATES TEST BELOW MINIMUM COMPACTION REQUIREMENT
A - INDICATES RETEST OF FAILED AREA AFTER BEING REWORKED

GORIAN AND ASSOCIATES, INC.

TABLE I
COMPACTION TEST SUMMARY

TEST NUMBER	DATE	DEPTH BELOW FINISHED SURFACE (FT.)	MOISTURE CONTENT (%)	UNIT DRY DENSITY (LBS/CU.FT.)	RELATIVE COMPACTION (%)	SOIL TYPE
STORM DRAIN TRENCH BACKFILL						
1	10/10/17	2.0	18.5	106.2	91	1
2	10/10/17	1.0	19.1	106.0	90	1
3	10/10/17	2.5	20.3	105.8	90	1
4	10/10/17	1.0	19.8	105.7	90	1
5	10/10/17	3.5	17.9	106.0	90	1
6	10/10/17	1.0	20.4	105.2	90	1
7	10/11/17	4.0	18.6	106.4	91	1
8	10/11/17	4.5	19.3	106.5	91	1
9	10/11/17	1.5	20.0	106.1	91	1
10	10/11/17	1.5	18.9	106.2	91	1
11	10/17/17	4.0	13.6	105.8	90	1
12	10/17/17	2.5	13.1	105.1	90	1
13	10/17/17	1.0	13.2	105.4	90	1
14	10/18/17	1.0	15.6	109.2	92	3
15	10/18/17	1.0	16.5	106.2	90	3
16	10/18/17	3.0	15.3	106.2	90	3
17	10/18/17	1.0	11.9	108.3	92	3
18	10/18/17	1.0	14.6	107.4	91	3
19	10/18/17	1.0	14.3	106.6	90	3
20	10/18/17	2.0	15.2	106.0	90	3
21	10/18/17	1.0	14.6	107.3	91	3
22	10/18/17	1.0	13.8	108.8	92	3
23	10/20/17	1.0	17.3	105.1	90	1
24	10/20/17	1.0	18.6	106.2	91	1
25	10/23/17	3.0	14.0	107.1	91	1
26	10/23/17	1.0	16.6	106.5	91	1
27	10/23/17	3.0	17.2	105.2	90	1
28	10/23/17	1.0	16.9	105.8	90	1
29	10/23/17	1.0	17.4	105.6	90	1
30	03/08/18	1.0	17.1	105.4	90	1
31	03/08/18	1.0	18.1	105.9	90	1
32	03/08/18	1.0	15.1	106.7	91	1
33	03/08/18	1.0	16.7	106.7	91	1
34	03/08/18	1.5	19.3	105.7	90	1
35	03/08/18	1.5	20.1	105.5	90	1
36	03/08/18	1.5	18.6	105.6	90	1

* - INDICATES TEST BELOW MINIMUM COMPACTION REQUIREMENT
A - INDICATES RETEST OF FAILED AREA AFTER BEING REWORKED

TABLE I
COMPACTION TEST SUMMARY

TEST NUMBER	DATE	DEPTH BELOW FINISHED SURFACE (FT.)	MOISTURE CONTENT (%)	UNIT DRY DENSITY (LBS/CU.FT.)	RELATIVE COMPACTION (%)	SOIL TYPE
WATER TRENCH BACKFILL						
1	02/16/18	1.5	20.8	103.2	88*	1
1A	02/16/18	1.5	19.1	105.1	90	1
2	02/26/18	2.0	18.2	106.5	91	1
3	02/26/18	2.0	15.5	105.9	90	1
4	02/26/18	2.0	14.4	111.1	94	2
5	04/02/18	1.0	18.3	104.9	90	1
6	04/03/18	1.0	19.1	105.2	90	1
7	04/03/18	1.0	19.3	105.0	90	1
8	04/03/18	1.0	18.5	104.9	90	1
9	04/04/18	3.0	20.5	100.4	86*	1
9A	04/19/18	3.0	16.4	105.7	90	1
10	04/05/18	2.0	17.9	105.2	90	1
11	04/05/18	2.0	16.9	105.4	90	1
12	04/05/18	2.0	15.6	106.7	91	1
13	04/06/18	2.0	14.7	101.9	87*	1
13A	04/09/18	2.0	13.5	105.3	90	1
14	04/09/18	2.0	15.2	107.4	92	1
15	04/09/18	1.5	14.1	106.2	91	1
GAS TRENCH BACKFILL						
1	05/30/18	1.5	14.3	110.6	93	2
2	05/30/18	1.5	15.9	111.2	94	2
3	05/30/18	1.5	14.7	109.5	92	2
4	06/13/18	1.0	15.9	106.2	91	1
5	06/13/18	1.0	16.3	107.0	91	1
EDISON TRENCH BACKFILL						
1	06/11/18	1.5	15.2	107.5	92	1
2	06/11/18	1.5	13.5	108.1	92	1
3	06/11/18	1.5	14.6	106.9	91	1
4	06/12/18	1.5	14.1	106.5	91	1
5	06/12/18	1.5	16.3	107.8	92	1

* - INDICATES TEST BELOW MINIMUM COMPACTION REQUIREMENT
A - INDICATES RETEST OF FAILED AREA AFTER BEING REWORKED

APPENDIX G

General Earthwork and Grading Specifications

APPENDIX G

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

TABLE OF CONTENTS

Section		Appendix G Page
1.0	GENERAL	1
1.1	Intent	1
1.2	The Geotechnical Consultant of Record	1
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	3
2.5	Evaluation/Acceptance of Fill Areas	4
3.0	FILL MATERIAL	4
3.1	General	4
3.2	Oversize	4
3.3	Import	4
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	5
4.6	Frequency of Compaction Testing	6
4.7	Compaction Test Locations	6
5.0	SUBDRAIN INSTALLATION	6
6.0	EXCAVATION	6
7.0	TRENCH BACKFILLS	7
7.1	Safety	7
7.2	Bedding and 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 Subdrain	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. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas

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

3.0 FILL MATERIAL

3.1 General

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

3.2 Oversize

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 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, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 EXCAVATION

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

7.0 TRENCH BACKFILLS

7.1 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.

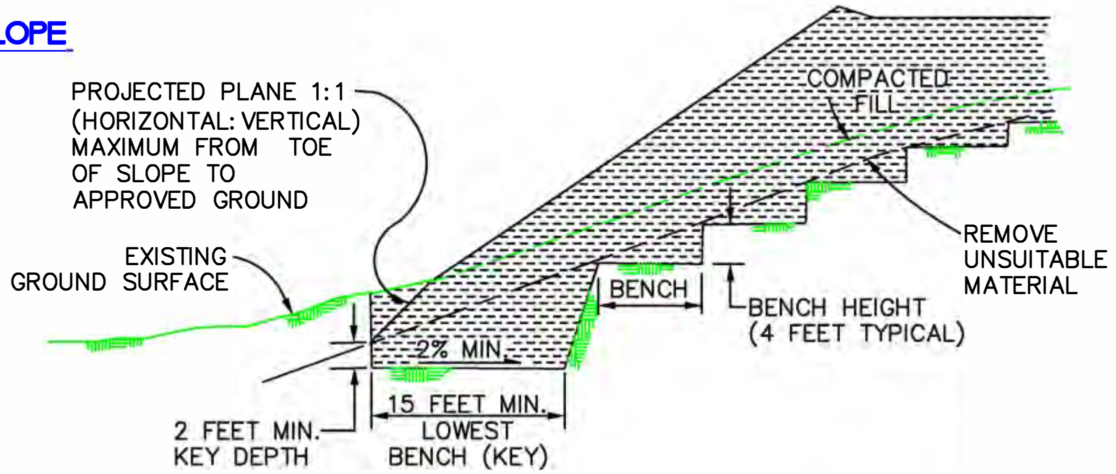
7.3 Lift Thickness

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.

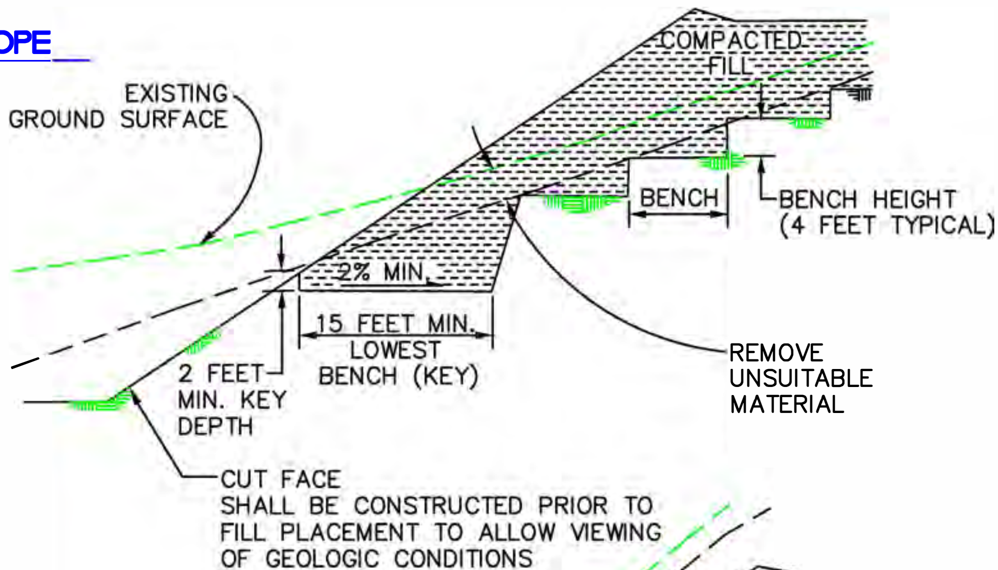
7.4 Observation and Testing

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

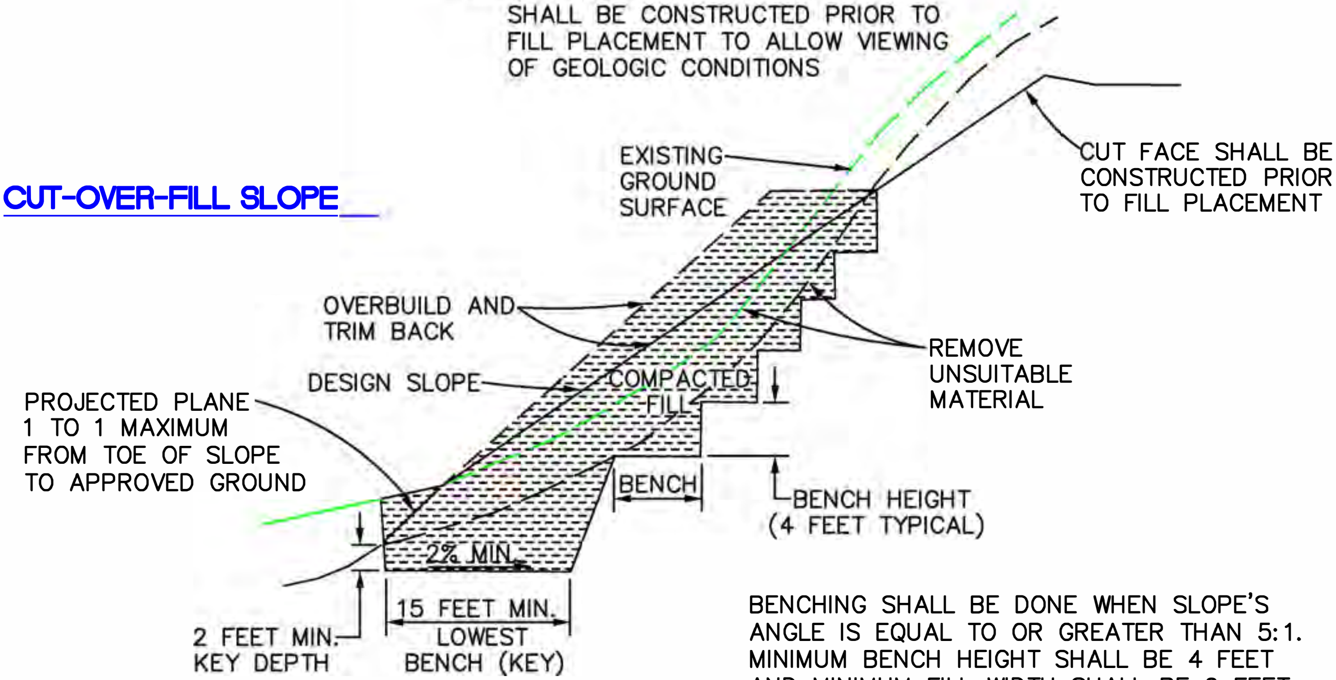
FILL SLOPE



FILL-OVER-CUT SLOPE

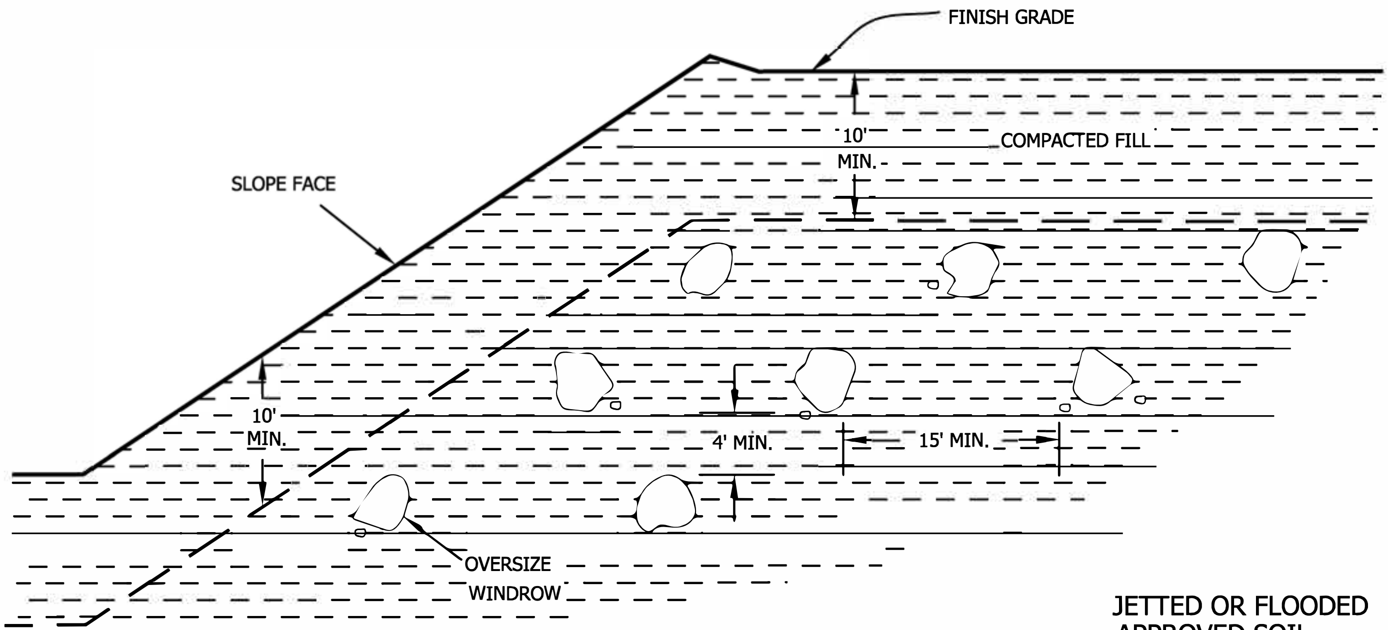


CUT-OVER-FILL SLOPE

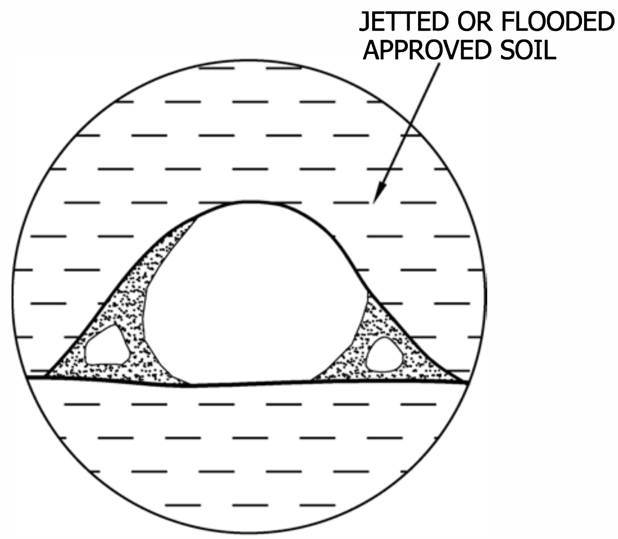


BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

KEYING AND BENCHING	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS A	Leighton

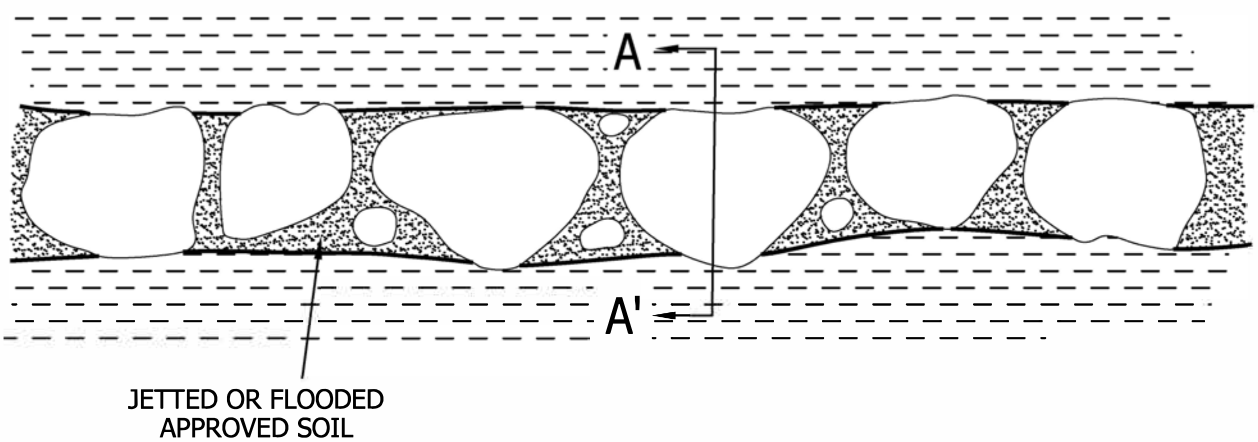


- Oversize rock is larger than 8 inches in largest dimension.
- Backfill with approved soil jetted or flooded in place to fill all the voids.
- Do not bury rock within 10 feet of finish grade.
- Windrow of buried rock shall be parallel to the finished slope face.



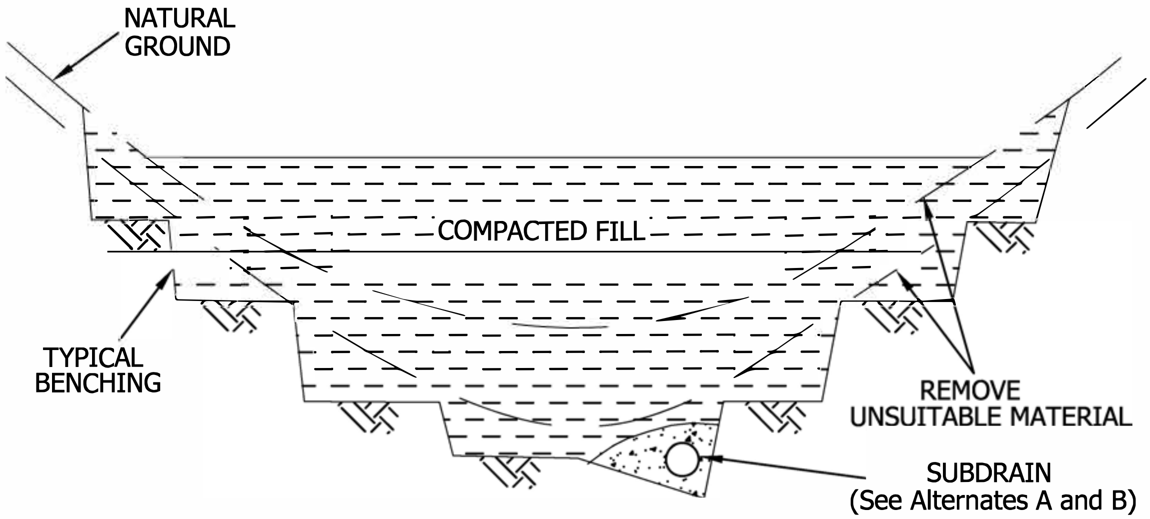
SECTION A-A'

PROFILE ALONG WINDROW

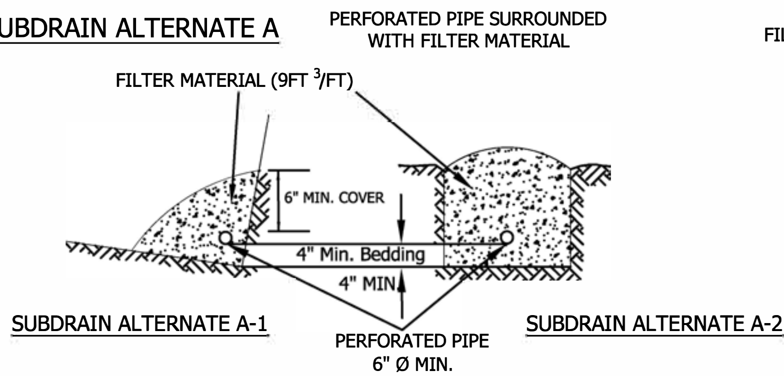


OVERSIZE ROCK DISPOSAL	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS B	Leighton

P:Drafting\templates\details\over-rock-disp.dwg (7/00)



SUBDRAIN ALTERNATE A

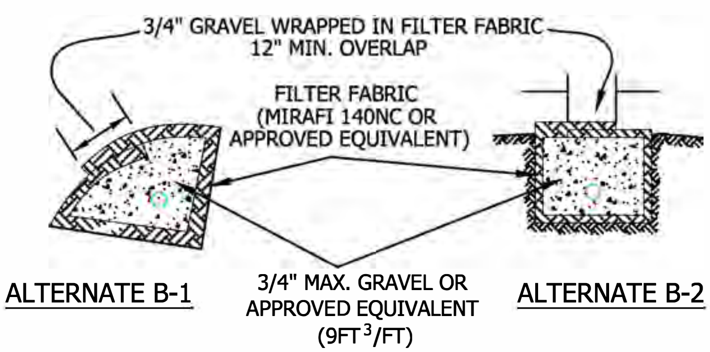


FILTER MATERIAL

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

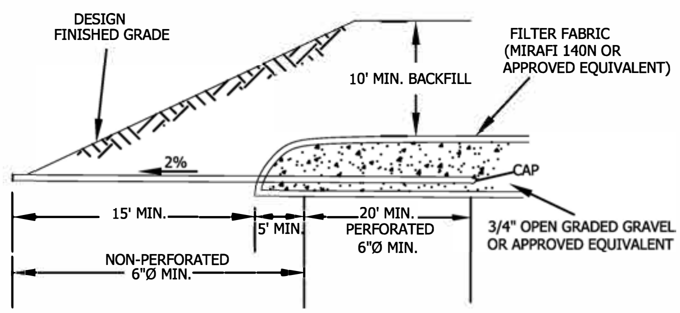
Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

SUBDRAIN ALTERNATE B



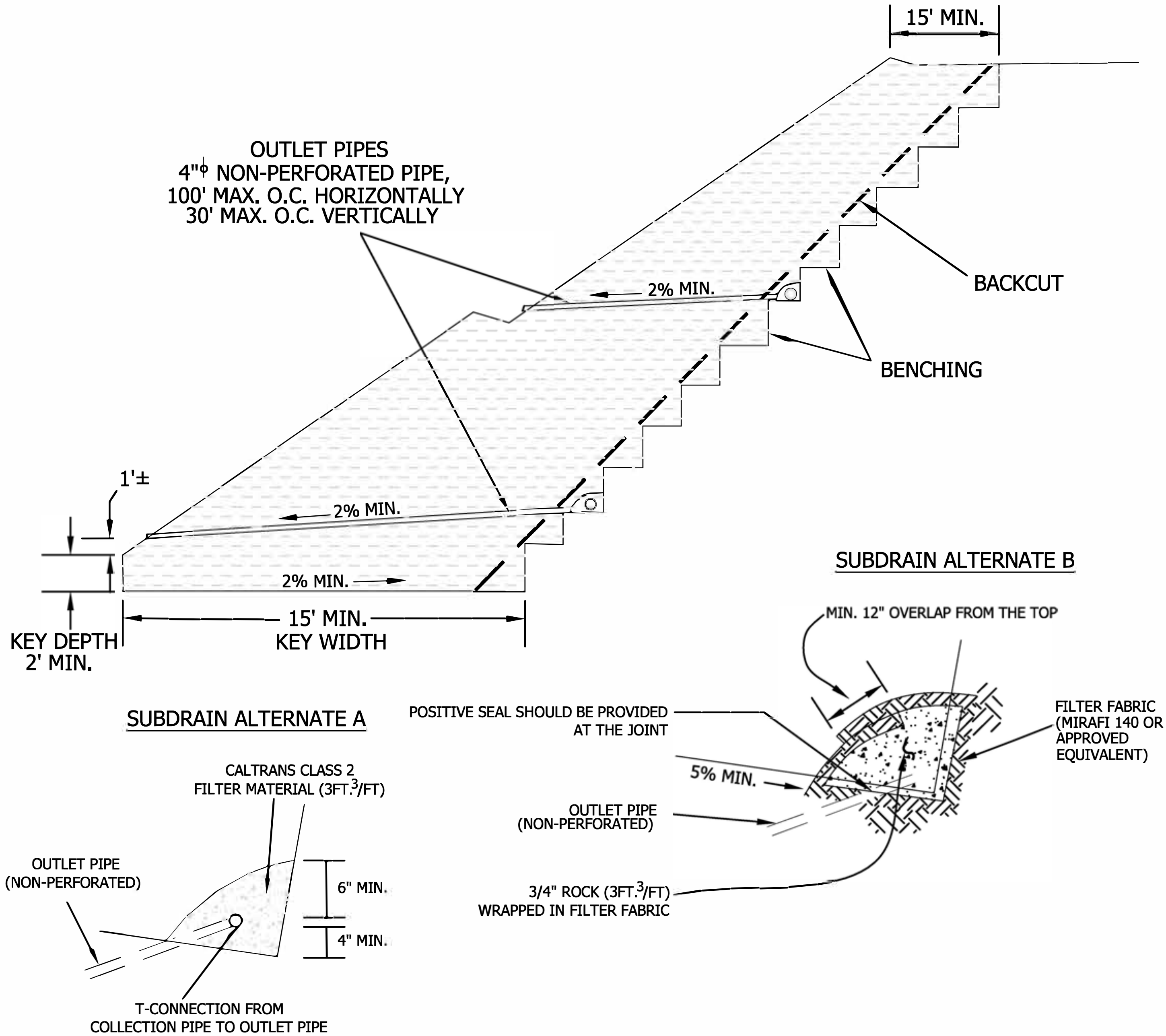
○ PERFORATED PIPE IS OPTIONAL PER GOVERNING AGENCY'S REQUIREMENTS

DETAIL OF CANYON SUBDRAIN TERMINAL



CANYON SUBDRAIN	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS C	Leighton

Drafting\templates\details\canyon_drains.dwg (7/00)

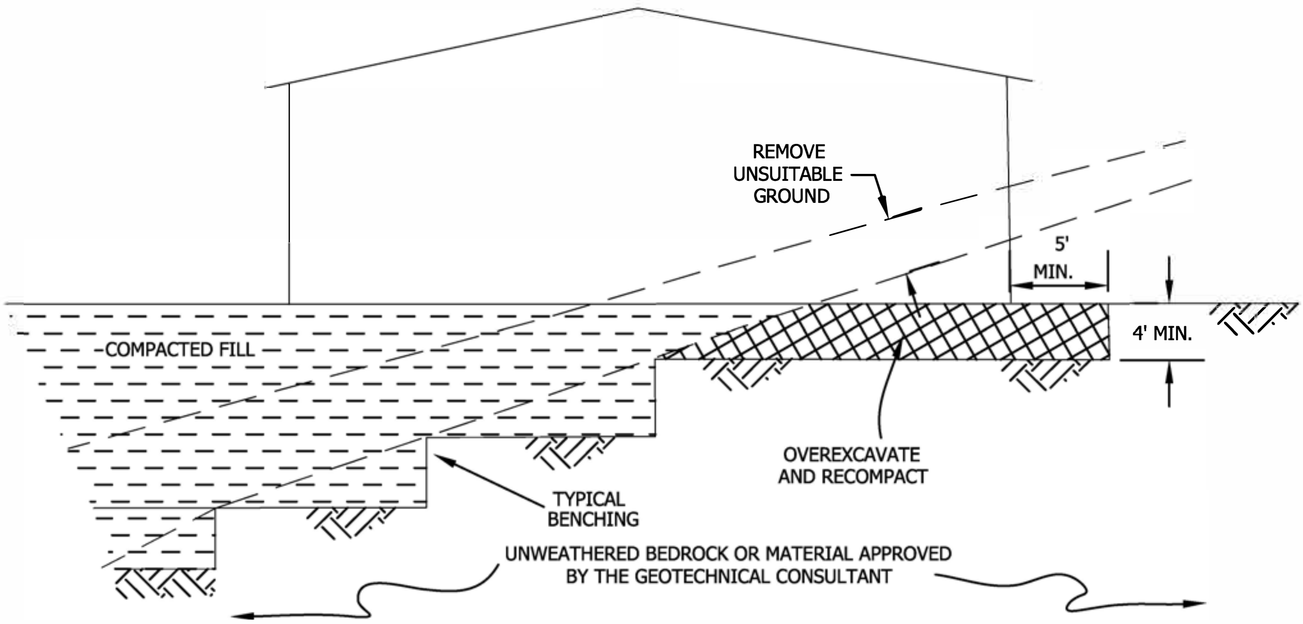


- SUBDRAIN INSTALLATION - Subdrain collector pipe shall be installed with perforations down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4"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

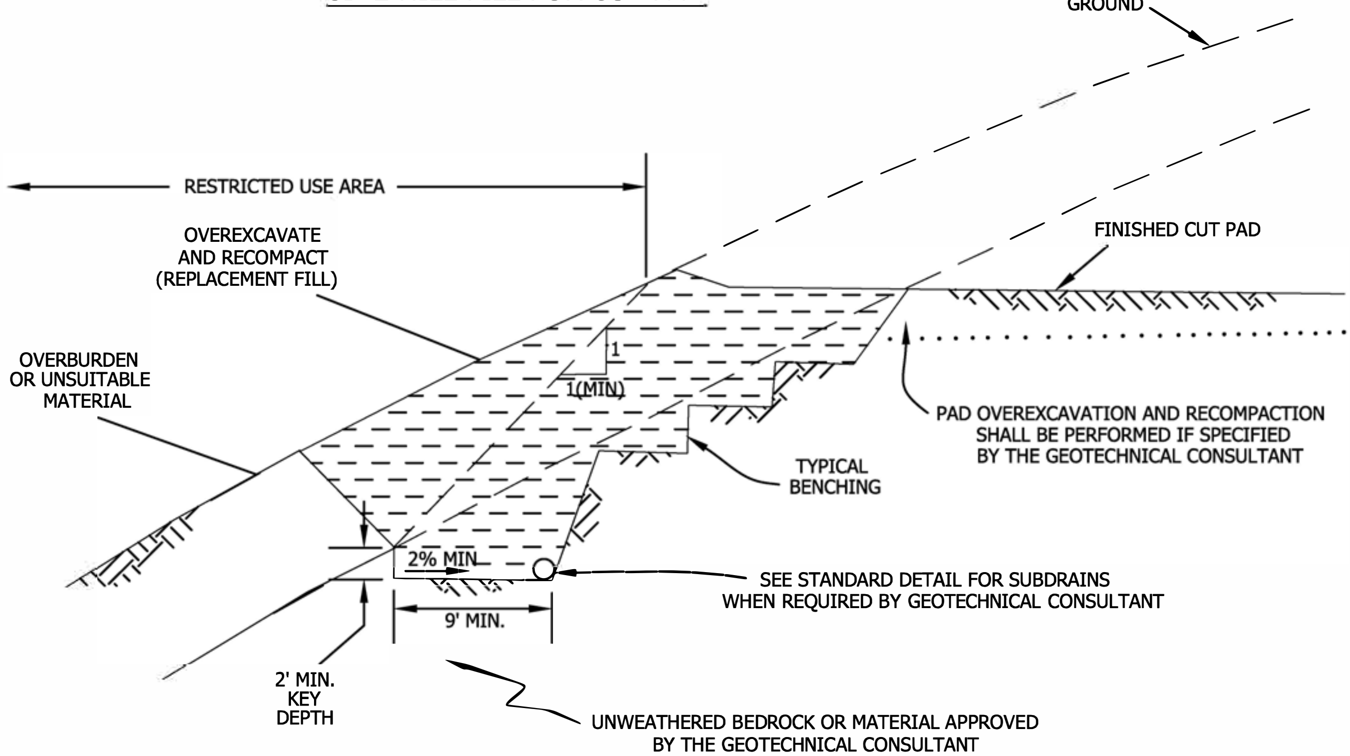
BUTTRESS OR REPLACEMENT FILL SUBDRAINS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS D	Leighton

P:Drafting\templates\details\buttress_repl_sub.dwg (7/00)

CUT-FILL TRANSITION LOT OVEREXCAVATION



SIDE HILL FILL FOR CUT PAD



TRANSITION LOT FILLS AND SIDE HILL FILLS

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAILS E

Leighton

P:Drafting\templates\details\transition_fills.dwg (7/00)

Appendix H

GBA Important Information About This Geotechnical Engineering Report

Important Information about This

Geotechnical-Engineering Report

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

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

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, 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 <u>not</u> 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 <u>not</u> be adequate to develop geotechnical design recommendations for the project.

Do <u>not</u> 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 <u>not</u> rely on an executive summary. Do <u>not</u> 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 <u>not</u> 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 <u>Not</u> 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 <u>not</u> 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 <u>not</u> building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org