Creekside Assisted Living

Technical Appendices

Appendix E

Soils Report

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APPENDICES

1.0 INTRODUCTION

We recommend that all individuals utilizing this report read the preceding information sheet prepared by GBA (the Geoprofessional Business Association) and the Limitations, Section 7.0, located at the end of this report.

1.1 Purpose and Scope

This report presents the results of our geotechnical investigation for the site located on the southeast corner of Richmar Avenue and North Twin Oaks Valley Road in the City of San Marcos, California (Figure 1). The intent of this report is to provide specific geotechnical conclusions and recommendations for the currently proposed project.

1.2 Site Location and Description

The subject site is a rectangular shaped parcel consisting of approximately 3 acres (see Figure 2). In general, the site is bordered by North Twin Oaks Valley Road to the west, Richmar Avenue to the north, East Mission Road to the south, and a drainage wetland area to the east.

Currently the site is unoccupied and undeveloped, with a dirt path trending northwest to southeast throughout the site. Vegetation across the site consists of overgrown grasses, weeds and shrubs.

Site topography is nearly level with elevations gently sloping from the west to the east, ranging from approximately 570 to 590 feet above mean sea level (msl). A westerly descending fill slope is located along the western property line of the site and is approximately 20 feet in height over a horizontal distance of approximately 260 feet.

Site Latitude and Longitude
33.1434° N
117.1623° W

1.3 Proposed Development

We understand that the proposed residential development will primarily consist of 8 multi-family residential units. The proposed residential buildings are anticipated to be typical 2- to 3-story wood-frame structures with slab-on-grade foundations. Additionally, a 9 to 12 foot retaining wall is proposed along the eastern side of the site. Other improvements at the site will consist of associated roadways, utilities, landscape and hardscape. Import material up to 8 feet is anticipated to raise pads grades above the flood zone.

2.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

2.1 Site Investigation

Our exploration consisted of excavating five (5) 8-inch small diameter geotechnical borings (B-1 through B-5) to approximately 26.5 to 40 feet below the existing ground surface. Additionally, four (4) percolation tests were performed at the site as part of the subsurface exploration. All borings were drilled using a heavy-duty truck mounted hollow-stem auger drill rig. The four percolation test locations were also advanced with the hollow-stem auger drill rig to a depth of 5 feet below the existing ground surface. The percolation test well locations were presoaked overnight and the testing was performed the following day by the falling head method. During the exploration operations, a geologist from our firm prepared geologic logs and collected bulk and relatively undisturbed samples for laboratory testing and evaluation.

After logging, the borings were backfilled with bentonite. The boring logs are provided in Appendix B. Geotechnical boring and percolation test locations are depicted on Figure 2.

2.2 Laboratory Testing

Laboratory testing performed on soil samples representative of on-site soils obtained during the recent subsurface exploration included, moisture content, density determination, shear strength, grain size, expansion index, and a screening geochemical analysis for corrosion. A discussion of the laboratory tests performed and a summary of the laboratory test results are presented in Appendix C.

3.0 SUMMARY OF GEOTECHNICAL CONDITIONS

3.1 Geologic Setting

The project area is situated in the Peninsular Ranges Geomorphic Province. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California, and varies in width from approximately 30 to 100 miles (Norris and Webb, 1990). The province is characterized by mountainous terrain on the east composed mostly of Mesozoic igneous and metamorphic rocks, and relatively low-lying coastal terraces to the west underlain by late Cretaceous, Tertiary, and Quaternary age sedimentary rocks.

Gradual emergence of the region from the sea occurred in Pleistocene time, and numerous wave-cut platforms, most of which were covered by relatively thin marine and non-marine terrace deposits, formed as the sea receded from the land. Accelerated fluvial erosion during periods of heavy rainfall, coupled with the lowering of the base sea level during Quaternary times, resulted in the rolling hills, mesas, and deeply incised canyons which characterize the landforms we see in the general site area today.

3.2 Site-Specific Geology

Based on our subsurface exploration and review of pertinent geologic literature and maps, the geologic units underlying the site consist of undocumented artificial fill soils (Afu), Quaternary-aged Young and Old Alluvium (Qya and Qoa), and at depth undifferentiated Mesozoic-aged Metasedimentary/Metavolcanic (Mzu) basement rocks and Cretaceous Tonalite. Brief descriptions of the geologic units present on the site are presented in the following sections. The approximate aerial distributions of those units are shown on the Geotechnical Map (Figure 2).

3.2.1 Artificial Fill, Undocumented (Map Symbol – Afu)

The site generally consists of a previously placed fill area with approximately 1-2 feet thick across the site. Deeper fills associated with surrounding road improvements should be anticipated. The fill is characterized by moist and medium stiff to medium dense varying shades of brown to gray brown silty to sandy clays and clayey sands. Currently, there is not a geotechnical report discussing the placement and quality of the placed fill, therefore, at this time, the fill is considered to be undocumented. Fill was not encountered in our borings, but is associated with sewer and surrounding road improvements present on the site.

3.2.2 Quaternary - Aged Young Alluvium

Quaternary young alluvium is present beneath the undocumented fill in Boring B-3, a channelized deposit trending from the northern vicinity of the site to the southeastern vicinity of the site. The materials that comprise the young alluvial materials are predominantly brown to gray brown, moist to wet, medium stiff clays with varying amounts of silty and sandy constituents. We anticipate these materials will be 3 to 7 feet below existing grades.

3.2.3 Quaternary - Aged Old Alluvium (Map Symbol – Qoa)

Quaternary old alluvium is present beneath the undocumented fill and young alluvial deposits throughout the site. The materials that comprise the old alluvial materials vary in thickness and consistency from medium dense to very dense, moist to saturated silty and clayey sands to medium stiff to hard, moist to wet clays with varying silt and sandy constituents.

3.2.4 Cretaceous Tonalite (Kt)

Cretaceous-aged Tonalite was observed to be underlying the undocumented fill and alluvial deposits in the eastern portion of the site. As encountered, the Cretaceous-aged Tonalite deposits predominately consists of orange-brown and medium to dark grey to black, damp to moist, very-dense to hard, poorly-graded sandstones with interbedded quartz veins observed throughout.

3.2.5 Mesozoic-Aged Metasedimentary and Metavolcanic (Mzu)

Mesozoic-aged undifferentiated metasedimentary and metavolcanic geologic units were observed to underlie the majority of the site. When encountered, Mesozoic-aged undivided metasedimentary and metavolcanic geologic units primarily consisted of greenish-black, moist to wet, very dense to hard, silty to clayey sands with gravels.

3.3 Surface Water and Ground Water

No indication of surface water or evidence of surface ponding was encountered during our field exploration. Ground water was locally encountered in Borings B-1 through B-4 during our geotechnical investigation at the site at depths ranging from 15 to 30 feet below the ground surface. It should be noted that ground water levels may fluctuate with seasonal variations and irrigation and local perched ground water conditions may exist within cemented layers and sandy lenses within the quaternary alluvium deposits. Nevertheless, based on the above information, we do not anticipate ground water will be a constraint to the construction of the proposed improvements.

3.4 Engineering Characteristics of On-site Soils

Based on the results of our laboratory testing of representative on-site soils, and our professional experience on similar sites with similar soils conditions, the engineering characteristics of the on-site soils are discussed below.

3.4.1 Compressible Soils

The site is underlain by artificial fill and young alluvial soils which are considered compressible. Additionally, the upper portions of the old alluvium deposits are considered compressible. Portions of the compressible fill soils and alluvium deposits are expected to be removed during excavation operations for the proposed residential development at the project site. Recommendations for remedial grading of these soils are provided in the following sections of this report.

3.4.2 Expansion Potential

The majority of the onsite material is expected to have a low to medium expansion potential. However, higher expansive soils may be encountered during the grading of the site. It is recommended that highly expansive soils (EI>90), if encountered, are not used as engineered fill, and may require selective grading.

3.4.3 Soil Corrosivity

During our investigation, preliminary screenings of representative on-site soil samples were performed to evaluate their potential corrosive effect on concrete and ferrous metals. In summary, laboratory testing on the representative soil samples obtained during our subsurface exploration evaluated pH, minimum electrical resistivity, and chloride and soluble sulfate content. The samples tested had a measured pH of 7.53 and a measured minimum electrical resistivity of 1,300 ohm-cm. The test results also indicated that the samples had a chloride content of 24 parts per million (ppm), and a soluble sulfate content of less than 150 ppm.

3.4.4 Excavation Characteristics

It is anticipated the onsite soils can be excavated with conventional heavy-duty construction equipment. Localized cemented zones located within the old alluvial deposits, if encountered, may require heavy ripping or breaking. If oversize material (larger than 8 inches in maximum dimensions) is generated, it should be placed in non-structural areas or hauled off site. Localized interbedded gravels and cobbles may be encountered within the alluvial deposits. In addition, localized zones of friable sands may be encountered within the alluvial deposits. Beds of friable sands, gravel, and cobble may experience caving during unsupported excavation or drilling.

3.4.5 Percolation and Infiltration Rates

Percolation tests were performed in general accordance with the County of Riverside borehole percolation method and County of San Diego Regional Storm Water Standards. Based on our field percolation testing, the in-situ percolation rates and calculated infiltration rates at tested locations and depths are summarized in Table 1 below. It should be noted that we have used the following equation based upon the Porchet Method to convert measured percolation rates to infiltration rates in accordance with County of Riverside Standards (2011). In addition, we have included a recommended infiltration rate with a minimum factor of safety of 2 for the preliminary design of potential infiltration systems:

$$I_t = \frac{\Delta H * 60 * r}{\Delta t(r+2H_{AVG})}$$

Where:

I_t = calculated infiltration rate, inches/hour

ΔH = change in head over the time interval, inches

Δt = time interval, minutes

r = radius of test hole

H_{AVG} = average head over the time interval, inches

The field percolation test locations are shown on Figure 2 (Geotechnical Map). Field data and calculated percolation rates for each field percolation test location is presented in Appendix F.

Table 1
Percolation and Infiltration Rates

Test No.	Depth (ft)	Soil Type	Measured Percolation Rate (mins/in)	Calculated Infiltration Rate (inches/hr)	Recommended Infiltration Rate w/ FS of 2 (inches/hr)
P-1	4.17	Old Alluvium	NP	<0.01	<0.005
P-2	3.96	Old Alluvium	NP	<0.01	<0.005
P-3	3.75	Old Alluvium	NP	<0.01	<0.005
P-4	3.70	Old Alluvium	NP	<0.01	<0.005

NP – No percolation measured.

Based on the field percolation testing and the recommended calculated infiltration rates, the site is categorized as "No-Infiltration", as determined by the Storm Water Standards BMP Design Manual, San Diego Region, February 2016. The County of San Diego Infiltration Worksheet I-8, Categorization of Infiltration Feasibility Condition, has been completed and is presented in Appendix F. Note that the above percolation test results are representative of the tested locations and depths where they were performed. It should also be noted that percolation test field measurements are accurate to 0.01 feet. Varying subsurface conditions may exist outside of the test locations, which could alter the calculated percolation rate indicated below. In addition, it is important to note that percolation rates are not equal to infiltration rates. As a result, we have made a distinction between percolation rates where water movement is considered laterally and vertically versus infiltration rates where only the vertical direction is considered.

It is possible that the long term rate of transmissivity of permeable soil strata may be lower than the values obtained by testing. Infiltration may be influenced by a combination of factors including but not limited to: a highly variable vertical permeability and limited lateral extent of permeable soil strata; a reduction of permeability rates over time due to silting of the soil pore spaces; and other unknown factors. Accordingly, the possibility of future surface ponding of water, as well as, shallow groundwater impacts on subterranean structures such as basements, underground utilities, etc. should be anticipated as possible future conditions in all design aspects of the site.

4.0 SEISMIC AND GEOLOGIC HAZARDS

4.1 Local Faulting

Our review of available geologic literature (Appendix A) indicates that there are no known Active or Potentially Active faults transecting the site. The subject site is also not located within any State Mapped Earthquake Fault Zones or County of San Diego mapped fault zones. The nearest active fault is the Rose Canyon fault zone located approximately 12.6 miles west of the site (Blake, 2001).

4.2 Seismicity

The site is considered to lie within a seismically active region, as is all of Southern California. As previously mentioned above, the Rose Canyon fault zone located approximately 12.6 miles west of the site is considered the 'active' fault having the most significant effect at the site from a design standpoint.

4.3 Seismic Hazards

Severe ground shaking is most likely to occur during an earthquake on one of the regional active faults in Southern California. The effect of seismic shaking may be mitigated by adhering to the California Building Code or state-of-the-art seismic design parameters of the Structural Engineers Association of California.

4.3.1 Shallow Ground Rupture

No active faults are mapped crossing the site, and the site is not located within a mapped Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). Shallow ground rupture due to shaking from distant seismic events is not considered a significant hazard, although it is a possibility at any site.

4.3.2 Mapped Fault Zones

The site is not located within a State Mapped Earthquake Fault Zone (EFZ). As previously discussed, the subject site is not underlain by known active or potentially active faults.

4.3.3 Site Class

Utilizing 2016 California Building Code (CBC) procedures, we have characterized the site soil profile to be Site Class D based on our experience with similar sites in the project area and the results of our subsurface evaluation.

4.3.4 Building Code Mapped Spectral Acceleration Parameters

The effect of seismic shaking may be mitigated by adhering to the California Building Code and state-of-the-art seismic design practices of the Structural Engineers Association of California. Provided below in Table 2 are the spectral acceleration parameters for the project determined in accordance with the 2016 CBC (CBSC, 2016) and the USGS Worldwide Seismic Design Values tool (Version 3.1.0).

Table 2
2016 CBC Mapped Spectral Acceleration Parameters

Site Class	D		
Site Coefficients	F_a	=	1.093
	F_v	=	1.604
Mapped MCE Spectral Accelerations	S_S	=	1.018g
	S_1	=	0.398g
Site Modified MCE Spectral Accelerations	S_{MS}	=	1.113g
	S_{M1}	=	0.639g
Design Spectral Accelerations	S_{DS}	=	0.742g
	S_{D1}	=	0.426g

Utilizing ASCE Standard 7-10, in accordance with Section 11.8.3, the following additional parameters for the peak horizontal ground acceleration are associated with the Geometric Mean Maximum Considered Earthquake (MCE_G). The mapped MCE_G peak ground acceleration (PGA) is 0.381g for the site. For a Site Class D, the F_{PGA} is 1.119 and the mapped peak ground acceleration adjusted for Site Class effects (PGA_M) is 0.426g for the site.

4.4 Secondary Seismic Hazards

In general, secondary seismic hazards can include soil liquefaction, seismically-induced settlement, lateral displacement, surface manifestations of liquefaction, landsliding, seiches, and tsunamis. The potential for secondary seismic hazards at the subject site is discussed below.

4.4.1 Liquefaction and Dynamic Settlement

Liquefaction and dynamic settlement of soils can be caused by strong vibratory motion due to earthquakes. Both research and historical data indicate that loose, saturated, granular soils are susceptible to liquefaction and dynamic settlement. Liquefaction is typified by a loss of shear strength in the affected soil layer, thereby causing the soil to behave as a viscous liquid. This effect may be manifested by excessive settlements and sand boils at the ground surface.

Based on our analysis, much of the alluvial soils encountered are considered too clay rich to experience liquefaction. In addition, the relatively dense nature of the underlying Old Alluvial deposits are considered too dense to exhibit the effects prone to a liquefiable event and thus the potential for adverse effects produced by liquefaction is considered low.

4.4.2 Lateral Spread

Empirical relationships have been derived (Youd et al., 1999) to estimate the magnitude of lateral spread due to liquefaction. These relationships include parameters such as earthquake magnitude, distance of the earthquake from the site, slope height and angle, the thickness of liquefiable soil, and gradation characteristics of the soil. Based on the low susceptibility to liquefaction and the formational material unit underlying the site, the possibility of earthquake-induced lateral spread is considered to be low for the site.

4.4.3 Tsunamis and Seiches

Based on the distance between the site and large, open bodies of water, and the elevation of the site with respect to sea level, the possibility of seiches and/or tsunamis is considered to be nil.

4.5 Landslides

Our investigation was limited primarily to the existing flat, undeveloped areas. No ancient landslides or other slope instability problems have been mapped on the subject site. In addition, no evidence of landsliding was encountered during our site investigation. Based on our review of geotechnical literature, site topography, and our observations, landsliding is not a constraint to the currently proposed development.

4.6 Flood Hazard

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2012); the site is located within a floodplain. Therefore, the potential for flooding of the site is considered moderate to high at current site grades.

5.0 CONCLUSIONS

Based on the results of our geotechnical investigation of the site, it is our opinion that the proposed improvements are feasible from a geotechnical standpoint, provided the following conclusions and recommendations are incorporated into the project plans and specifications.

- As the site is located in the seismically active southern California area, all structures should be designed to tolerate the dynamic loading resulting from seismic ground motions.
- The site is not transected by Potentially Active or Active faults.
- The existing onsite soils are generally suitable for use as engineered fill, provided they are free of organic material, debris, and rock fragments larger than 8 inches in maximum dimension. Onsite clay soils have a medium expansion potential, and if reused, will require moisture conditioning to be suitable for use as engineered fill in select areas.
- Import soil is anticipated to obtain site proposed grades. Recommendations are based on import material possessing an expansion index less than 50.
- Based on the results of our subsurface exploration, we anticipate that the onsite materials should be generally excavatable with conventional heavy-duty earthwork equipment. Localized cemented zones within the old alluvial deposits may be difficult to excavate and may require heavy ripping which can produce oversized rock fragments.
- Based on our experience with similar sites and the results of our investigations of the site, excavations within the alluvial and old alluvial deposits may encounter zones of poorly graded cohesionless sands that may cave or slough during site excavation and drilling. Therefore, measures to shore excavations should consider the presence of friable soil layers that will likely tend to cave during excavation.
- The static ground water table should not be encountered during remedial grading activities. Although not encountered during our exploration, localized seepage along cemented zones and sand lenses within the alluvial deposits may occur.
- Based on the results of our geotechnical evaluation, it is our opinion that the proposed site improvements can be supported on conventional reinforced concrete foundations.

- Although Leighton does not practice corrosion engineering, laboratory test results indicate the soils present on the site have a negligible potential for sulfate attack on normal concrete. In addition, the onsite soils are considered to be corrosive to buried uncoated ferrous metals. We recommend that a corrosion engineer be retained to design corrosion protection systems and to evaluate the appropriate concrete properties for the project.
- The new compacted artificial fill consisting of mixture of soils ranging from silty sands to sandy clays will have permeable and impermeable layers that can transmit and perched ground water in unpredictable ways. Low Impact Development (LID) measures may impact down gradient improvements and the use of some LID measures may not be appropriate for this project. It is likely that as a No-Infiltration site, impermeable membrane liners may be needed to prevent lateral migration of storm water. Any proposed bioretention stormwater systems design should be reviewed by geotechnical consultant and will likely require a 30 mil HDPE liner to prevent lateral migration of storm water.

6.0 RECOMMENDATIONS

6.1 Earthwork

We anticipate that earthwork at the site will consist of site preparation and remedial grading. We recommend that earthwork on the site be performed in accordance with the following recommendations and the General Earthwork and Grading Specifications for Rough Grading included in Appendix D. In case of conflict, the following recommendations supersede those in Appendix D.

6.1.1 Site Preparation

Prior to grading, all areas to receive structural fill, engineered structures, and pavements should be cleared of surface and subsurface obstructions, including any existing debris and undocumented fill, young alluvium, old slabs, loose, compressible, or unsuitable soils, and stripped of vegetation. Removed vegetation and debris should be properly disposed off-site. All areas to receive fill and/or other surface improvements should be scarified to a minimum depth of 8 inches, brought to optimum or above-optimum moisture conditions, and recompacted to at least 90 percent relative compaction based on ASTM Test Method D1557.

6.1.2 Excavations and Oversize Material

Excavations of the onsite materials may generally be accomplished with conventional heavy-duty earthwork equipment. However, local heavy ripping or breaking may be required if cemented zones within the old alluvial deposits is encountered. Excavation for utilities may also be difficult in some areas.

Due to the high-density characteristics of the old alluvial deposits, temporary shallow excavations less than 5 feet in depth with vertical sides should remain stable for the period required to construct utilities, provided the trenches are free of adverse geologic conditions. Overlying artificial fill soils and beds of friable sands within the young alluvium deposits present at the site may cave during trenching operations. In accordance with OSHA requirements, excavations deeper than 5 feet should be shored or be laid back in accordance with Section 6.2 if workers are to enter such excavations.

6.1.3 Removal of Compressible Soils

Potentially compressible undocumented fill, young alluvium, and the upper portions of the old alluvial deposits at the site may settle as a result of wetting or settle under the surcharge of engineered fill and/or structural loads supported on shallow foundations.

All undocumented fill soils and young alluvium at the site should be completely removed. In addition, all old alluvial deposits encountered within 3 feet from the bottom of the site settlement-sensitive improvements and foundations (i.e. residential structures and retaining walls) should be removed. Horizontally, the lateral limits of the removal excavations should extend at least 5 feet beyond the foundation limits of the site sensitive improvements. The bottom of all removals should be evaluated by a Certified Engineering Geologist to confirm conditions are as anticipated.

In general, the soil that is removed may be reused and placed as engineered fill provided the material is free of oversized rock, organic materials, and deleterious debris, and moisture conditioned to above optimum moisture content. Onsite soil with an expansion index greater than 50 should not be used within 5 feet of finish grade in the building pad. The actual depth and extent of the required removals should be confirmed during grading operations by the geotechnical consultant.

6.1.4 Engineered Fill

The onsite soils are generally suitable for use as compacted fill provided they are free of organic material, debris, and rock fragments larger than 6 inches in maximum dimension. All fill soils should be brought to at least 2 percent above optimum moisture conditions (i.e., depending on the soil types) and compacted in uniform lifts to at least 90 percent relative compaction based on laboratory standard ASTM Test Method D1557, 95 percent for wall backfill soils or if used for structural purposes (such as to support a footing, wall, etc.). We anticipate the majority of wall backfill will be compacted to 95% due to close proximity of the proposed buildings. The optimum lift thickness required to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in lifts not exceeding 8 inches in thickness.

Placement and compaction of fill should be performed in general accordance with the current City of San Marcos grading ordinances, sound construction practice, and the General Earthwork and Grading Specifications for Rough Grading presented in Appendix D.

6.1.5 Earthwork Shrinkage/Bulking

The volume change of excavated onsite materials upon recompaction as fill is expected to vary with material and location. Typically, the fill soils and alluvial deposits vary significantly in natural and compacted density, and therefore, accurate earthwork shrinkage/bulking estimates cannot be determined. However, based on the results of our geotechnical analysis and our experience, a 5 percent shrinkage factor is considered appropriate for the artificial fill, young alluvium, and a 3 to 5 percent bulking factor is considered appropriate for the old alluvial deposits.

6.1.6 Trench Backfill

Pipe bedding should consist of sand with a sand equivalent (SE) of not less than 30. Bedding should be extended the full width of the trench for the entire pipe zone, which is the zone from the bottom of the trench, to one foot above the top of the pipe. The sand should be brought up evenly on each side of the pipe to avoid unbalanced loads. Onsite materials will probably not meet bedding requirements. Except for predominantly clayey soils, the onsite soils may be used as trench backfill above the pipe zone (i.e. in the trench zone) provided they are free of organic matter and have a maximum particle size of three inches. Compaction by jetting or flooding is not recommended.

6.1.7 Expansive Soils and Selective Grading

Based on our laboratory testing and observations, we anticipate the onsite soil materials possess a low to medium expansion potential (Appendix C). Although not anticipated, should an abundance of highly expansive materials be encountered, selective grading may need to be performed. In addition, to accommodate conventional foundation design, the upper 5 feet of materials within the building pad and 5 feet outside the limits of the building foundation should have a very low to low expansion potential. (EI<50).

6.1.8 Import Soils

Import soils is anticipated at the site to bring the site up to the proposed grades above floodway, these soils should be granular in nature, and have an expansion index less than 50 (per ASTM Test Method D4829) and have a low corrosion impact to the proposed improvements. Beneath pavements, subgrade materials should possess an R-value of 20, or greater. Import soils and/or the borrow site location should be evaluated by the geotechnical consultant prior to import.

6.2 Temporary Excavations

Sloping excavations may be utilized when adequate space allows. Based on the results of our update evaluation, we provide the following recommendations for sloped excavations in fill soils or competent old alluvial deposits materials without seepage conditions.

Table 3
Maximum Slope Ratios

Excavation Depth (feet)	Maximum Slope Ratio In Fill Soils and Young Alluvium	Maximum Slope Ratio In Old Alluvial Deposits
0 to 5	1:1 (Horizontal to Vertical)	Vertical
5 to 20	1:1 (Horizontal to Vertical)	1:1 (Horizontal to Vertical)

The above values are based on the assumption that no surcharge loading or equipment will be placed within 10 feet of the top of slope. Care should be taken during excavation adjacent to the existing structures so that undermining does not occur. A "competent person" should observe the slope on a daily basis for signs of instability.

6.3 Foundation and Slab Considerations

At the time of drafting this report, building loads were not known. However, based on our understanding of the project, the proposed multi-family residential buildings may be constructed with conventional foundations or post-tensioned foundations. Foundations and slabs should be designed in accordance with structural considerations and the following recommendations. These recommendations assume that the import soils encountered within 5 feet of pad grade have a low potential for expansion (EI<50). If more expansive materials are encountered and selective grading cannot be accomplished, revised foundation recommendations may be necessary. The foundation recommendations below assume that the all building foundations will be underlain by properly compacted fill.

6.3.1 Conventional Foundations

Foundations and slabs should be designed in accordance with structural considerations and the following recommendations. These recommendations assume that the soils encountered within 5 feet of pad grade have a low potential for expansion and a differential fill thickness of less than 10 feet. Additional expansion testing should be performed as part of the fine grading operations. If medium or highly expansive soils are encountered and selective grading cannot be accomplished, additional foundation design may be necessary.

6.3.2 Preliminary Foundation and Slab Design

The proposed buildings may be supported by conventional, continuous or isolated spread footings. Footings should extend a minimum of 24 inches beneath the lowest adjacent soil grade. At these depths, footings may be designed for a maximum allowable bearing pressure of 3,000 pounds per square foot (psf) if founded in dense compacted fill soils. The allowable bearing pressures may also be increased by one-third when considering loads of short duration such as wind or seismic forces. The minimum recommended width of footings is 18 inches for continuous footings and 24 inches for square or round footings. Footings should be designed in accordance with the structural engineer's requirements.

Slabs on grade should be reinforced with reinforcing bars placed at slab mid-height. Slabs should have crack joints at spacings designed by the structural engineer. Columns, if any, should be structurally isolated from slabs. Slabs should be a minimum of 5 inches thick and reinforced with No. 3 rebars at 18 inches on center on center (each way). The slab should be underlain by 2-inch layer of clean sand (S.E. greater than 30). A moisture barrier (10-mil non-recycled plastic sheeting) should be placed below the sand layer if reduction of moisture vapor up through the concrete slab is desired (such as below equipment, living/office areas, etc.), which is in turn underlain by an additional 2-inches of clean sand. If applicable, slabs should also be designed for the anticipated traffic loading using a modulus of subgrade reaction of 140 pounds per cubic inch. All waterproofing measures should be designed by the project architect.

The slab subgrade soils underlying the foundation systems should be presoaked in accordance with the recommendations presented in Table 4 prior to placement of the moisture barrier and slab concrete. The subgrade soil moisture content should be checked by a representative of Leighton prior to slab construction.

6.3.3 Foundation Setback

We recommend a minimum horizontal setback distance from the face of slopes for all structural foundations, footings, and other settlement-sensitive structures as indicated on the Table 4 below. This distance is measured from the outside bottom edge of the footing, horizontally to the slope face, and is based on the slope height. However, the foundation setback distance may be revised by the geotechnical consultant on a case-by-case basis if the geotechnical conditions are different than anticipated.

Table 4 Minimum Foundation Setback from Slope Faces	
Slope Height	Setback
less than 5 feet	5 feet
5 to 15 feet	7 feet
15 to 30 feet	10 feet

Please note that the soils within the structural setback area possess poor lateral stability, and improvements (such as retaining walls, sidewalks, fences, pavements, etc.) constructed within this setback area may be subject to lateral movement and/or differential settlement. Potential distress to such improvements may be mitigated by providing a deepened footing or a grade beam foundation system to support the improvement. Based on USGS topographic maps, the buildings located in the northwestern portion of the site are located on an existing slope. These buildings will likely require retaining walls and deepened foundations.

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

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

6.3.4 Settlement

Fill depths between 5 and 15 feet are anticipated beneath the proposed building foundations following final grading. For conventional footings, the recommended allowable-bearing capacity is based on a maximum total and differential static settlement of 3/4 inch and 1/2 inch, respectively. Since settlements are a function of footing size and contact bearing pressures, some differential settlement can be expected where a large differential loading condition exists. However, for most cases, differential settlements are considered unlikely to exceed 1/2 inch.

6.3.5 Moisture Conditioning

The slab subgrade soils underlying the foundation systems should be presoaked in accordance with the recommendations presented in Table 5 prior to placement of the moisture barrier and slab concrete. The subgrade soil moisture content should be checked by a representative of Leighton prior to slab construction.

Presoaking or moisture conditioning may be achieved in a number of ways. But based on our professional experience, we have found that minimizing the moisture loss on pads that have been completed (by periodic wetting to keep the upper portion of the pad from drying out) and/or berming the lot and flooding for a short period of time (days to a few weeks) are some of the more efficient ways to meet the presoaking recommendations. If flooding is performed, a couple of days to let the upper portion of the pad dry out and form a crust so equipment can be utilized should be anticipated.

Table 5 Presoaking Recommendations Based on Finish Grade Soil Expansion Potential	
Expansion Potential	Presoaking Recommendations
Very Low	Near-optimum moisture content to a minimum depth of 6 inches
Low	120 percent of the optimum moisture content to a minimum depth of 12 inches below slab subgrade
Medium	130 percent of the optimum moisture content to a minimum depth of 18 inches below slab subgrade
High	130 percent of the optimum moisture content to a minimum depth of 24 inches below slab subgrade

6.3.6 Post-Tension Foundation Recommendations

As an alternative to the conventional foundations for the buildings, post-tensioned foundations may be used. We recommend that post-tensioned foundations be designed using the geotechnical parameters presented in the table below and criteria of the 2016 California Building Code and the Third Edition of Post-Tension Institute Manual. A post-tensioned foundation system designed and constructed in accordance with these recommendations is expected to be structurally adequate for the support of the buildings planned at the site provided our recommendations for surface drainage and landscaping are carried out and maintained through the design life of the project. Based on an evaluation of the depths of fill beneath the building pads, the attached Table 6 presents the recommended post-tension foundation category for residential buildings for this site.

Table 6
Post-Tensioned Foundation Design Recommendations

Design Criteria		Category I Very Low to Low Expansion Potential (EI 0 to 50)	Category II Medium Expansion Potential (EI 50 to 90)	Category III High Expansion Potential (EI 90 to 130)
Edge Moisture Variation, e_m	Center Lift:	9.0 feet	8.3 feet	7.0 feet
	Edge Lift:	4.8 feet	4.2 feet	3.7 feet
Differential Swell, y_m	Center Lift:	0.46 inches	0.75 inches	1.09 inches
	Edge Lift:	0.78 inches	1.32 inches	1.99 inches
Perimeter Footing Depth:		18 inches	24 inches	30 inches
Allowable Bearing Capacity		2,000 psf		

The post-tensioned (PT) foundation and slab should also be designed in accordance with structural considerations. For a ribbed PT foundation, the concrete slab section should be at least 5 inches thick. Continuous footings (ribs or thickened edges) with a minimum width of 12 inches and a minimum depth of 12 inches below lowest adjacent soil grade may be designed for a maximum allowable bearing pressure of 2,000 pounds per square foot. For a uniform thickness "mat" PT foundation, the perimeter cut off wall should be at least 8 inches below the lowest adjacent grade. However, note that where a foundation footing or perimeter cut off wall is within 3 feet (horizontally) of adjacent drainage swales, the adjacent footing should be embedded a minimum depth of 12 inches below the swale flow line. The allowable bearing capacity may be increased by one-third for short-term loading. The slab subgrade soils should be presoaked in accordance with the recommendation presented in Table 6 above prior to placement of the moisture barrier.

The slab should be underlain by a moisture barrier as discussed in Section 6.3.2 above. Note that moisture barriers can retard, but not eliminate moisture vapor movement from the underlying soils up through the slabs. We recommend that the floor covering installer test the moisture vapor flux rate prior to attempting applications of the flooring. "Breathable" floor coverings should be considered if the vapor flux rates are high. A slip-sheet or equivalent should be utilized above the concrete slab if crack-sensitive floor coverings (such as ceramic tiles, etc.) are to be placed directly on the concrete slab. Additional guidance is provided in ACI Publications 302.1R-04 Guide for Concrete Floor and Slab Construction and 302.2R-06 Guide for Concrete Slabs that Receive Moisture-Sensitive Floor Materials.

6.4 Lateral Earth Pressures and Retaining Wall Design

Should retaining walls be added to the project, Table 7 presents the lateral earth pressure values for level or sloping backfill for walls backfilled with and bearing against fully drained soils of very low to low expansion potential (less than 50 per ASTM D4829).

Table 7
Static Equivalent Fluid Weight (pcf)

Conditions	Level	2:1 Slope
Active	35	55
At-Rest	55	65
Passive	350 (Maximum of 3 ksf)	150 (sloping down)

Walls up to 10 feet in height should be designed for the applicable equivalent fluid unit weight values provided above. If conditions other than those covered herein are anticipated, the equivalent fluid unit weight values should be provided on an individual case-by-case basis by the geotechnical engineer. A surcharge load for a restrained or unrestrained wall resulting from automobile traffic may be assumed to be equivalent to a uniform lateral pressure of 75 psf which is in addition to the equivalent fluid pressure given above. For other uniform surcharge loads, a uniform pressure equal to 0.35q should be applied to the wall. The wall pressures assume walls are backfilled with free draining materials and water is not allowed to accumulate behind walls. A typical drainage design is contained in Appendix D. Wall backfill should be compacted by mechanical methods to at least 90 percent relative compaction (based on ASTM D1557). If foundations are planned over the backfill, the backfill should be compacted to 95 percent. Wall footings should be designed in accordance with the foundation design recommendations and reinforced in accordance with structural considerations. For all retaining walls, we recommend a minimum horizontal distance from the outside base of the footing to daylight as outlined in Section 6.3.3.

Lateral soil resistance developed against lateral structural movement can be obtained from the passive pressure value provided above. Further, for sliding resistance, the friction coefficient of 0.35 may be used at the concrete and soil interface. These values may be increased by one-third when considering loads of short duration including wind or seismic loads. The total resistance may be taken as the sum of the frictional and passive resistance provided that the passive portion does not exceed two-thirds of the total resistance.

To account for potential redistribution of forces during a seismic event, retaining walls providing lateral support where exterior grades on opposites sides differ by more than 6 feet fall under the requirements of 2016 CBC Section 1803.5.12 and/or ASCE 7-10 Section 15.6.1 and should also be analyzed for seismic loading. For that analysis, an additional uniform lateral seismic force of 8H should be considered for the design of the retaining walls with level backfill, where H is the height of the wall. This value should be increased by 150% for restrained walls.

Based on the geotechnical conditions of the site and anticipate import, the recommended soil parameters presented on Table 8 should be utilized in the design of the proposed MSE retaining walls. Temporary sloping should be performed in accordance with current OSHA requirements.

Table 8 Retaining Wall Soil Parameters			
Soil Parameter	Reinforced Zone	Retained Zone	Foundation Zone
Internal Friction Angle (degrees)	28	28	28
Cohesion (psf)	0	0	0
Total Unit Weight (pcf)	125	125	125

Additional details relevant to the design of the MSE wall are presented on Detail G - Segmental Retaining Walls in Appendix D - General Earthwork and Grading Specifications. In addition, we recommend that water should be prevented from infiltrating into the reinforced soil zone. All drains and swales should outlet to suitable locations as determined by the project civil engineer. In general, the project civil engineer should verify that the subdrain is connected to the proper drainage facility.

Note that we also recommend a 7 foot minimum horizontal setback distance from the face of slopes for all retaining wall footings. This distance is measured from the outside bottom edge of the footing, horizontally to the slope face and is based on the slope height and type of soil. Appropriate surcharge pressures should also be applied for walls influenced within the retained or reinforced zones by improvements or vehicular traffic. The wall design engineer should also select grid design strength based on deflections tolerable to the proposed improvements. Settlement sensitive structures should not be located within the reinforced zone or active backfill prism.

6.5 Geochemical Considerations

Concrete in direct contact with soil or water that contains a high concentration of soluble sulfates can be subject to chemical deterioration commonly known as "sulfate attack." Soluble sulfate results (Appendix C) indicated a negligible soluble sulfate content. We recommend that concrete in contact with earth materials be designed in accordance with Section 4 of ACI 318-11 (ACI, 2011).

Based on the results of preliminary screening laboratory testing, the site soils have a generally very high corrosion potential to buried uncoated metal conduits. We recommend measures to mitigate corrosion be implemented during design and construction.

6.6 Concrete Flatwork

Concrete sidewalks and other flatwork (including construction joints) should be designed by the project civil engineer and should have a minimum thickness of 4 inches. For all concrete flatwork, the upper 12 inches of subgrade soils should be moisture conditioned to at least 2 percent above optimum moisture content and compacted to at least 90 percent relative compaction based on ASTM Test Method D1557 prior to the concrete placement.

6.7 Preliminary Pavement Design

The appropriate pavement section will depend on the type of subgrade soil, shear strength, traffic load, and planned pavement life. Pavement sections for the city streets should be designed in accordance with the City of San Marcos requirements.

For planning purposes only, preliminary pavement sections were developed based on our laboratory testing (i.e., assumed minimum R-value of 19) and potential Traffic Indices (TI) of 4.5, 5, and 6. As required by the City of San Marcos, final pavement designs should be completed after grading operations, but prior to street section construction where R-value confirmation tests can be performed on actual subgrade materials.

Table 9
Preliminary Pavement Sections

Traffic Index	Preliminary Pavement
4.5	4 inches AC over 4 inches Aggregate Base
5	4 inches AC over 5 inches Aggregate Base
6	4 inches AC over 9 inches Aggregate Base

Prior to placement of the aggregate base, the upper 12 inches of subgrade soils should be scarified, moisture-conditioned to at least optimum moisture content and compacted to a minimum 95 percent relative compaction based on American Standard of Testing and Materials (ASTM) Test Method D1557.

Class 2 Aggregate Base or Crushed Aggregate Base should then be placed and compacted at a minimum 95 percent relative compaction in accordance with ASTM Test Method D1557. The aggregate base material (AB) should be a maximum of 6 inches thick below the curb and gutter and extend a minimum of 6 inches behind the back of the curb. The AB should conform to and placed in accordance with the approved grading plans, the City of San Marcos, and latest revision of the Standard Specifications Public Works Construction (Greenbook).

The Asphalt Concrete (AC) material should conform to Caltrans Standard Specifications, Sections 39 and 92, with a Performance Grade (PG) of 64-10, and the City of San Marcos requirements. The placement of the AC should be in accordance with the approved grading plans, Section 203-6 of the "Greenbook" Standard Specifications for Public Works Construction, and the City of San Marcos requirements. AC sections greater than 3-inches thick should be placed in two lifts. The 1st lift should be a 2-inch minimum base course consisting of a 3/4-inch maximum coarse aggregate. The 2nd lift should be a 2-inch minimum surface capping course consisting of a 1/2-inch maximum coarse aggregate. No single lift shall be greater than 3 inches.

If pavement areas are adjacent to heavily watered landscaping areas, we recommend some measures of moisture control be taken to prevent the subgrade soils from becoming saturated. It is recommended that the concrete curbing, separating the landscaping area from the pavement, extend below the aggregate base to help seal the ends of the sections where heavy landscape watering may have access to the aggregate base. Concrete swales should be designed if asphalt pavement is used for drainage of surface waters.

6.8 Control of Ground Water and Surface Waters

Regarding Low Impact Development (LID) measures, we are of the opinion that infiltration basins, and other onsite storm water retention and infiltration systems can potentially create adverse perched ground water conditions when not installed using proper design recommendations (such as the use of liners) and infiltration design parameters. Due to the dense nature of the alluvial deposits and resulting very low infiltration rate, we do not recommend the use of infiltration type LID devices at the site.

6.9 Construction Observation

The recommendations provided in this report are based on preliminary design information and subsurface conditions disclosed by widely spaced excavations. The interpolated subsurface conditions should be checked by Leighton in the field during construction. Construction observation of all onsite excavations and field density testing of all compacted fill should be performed by a representative of this office. We recommend that all excavations be mapped by the geotechnical consultant during grading to determine if any potentially adverse geologic conditions exist at the site.

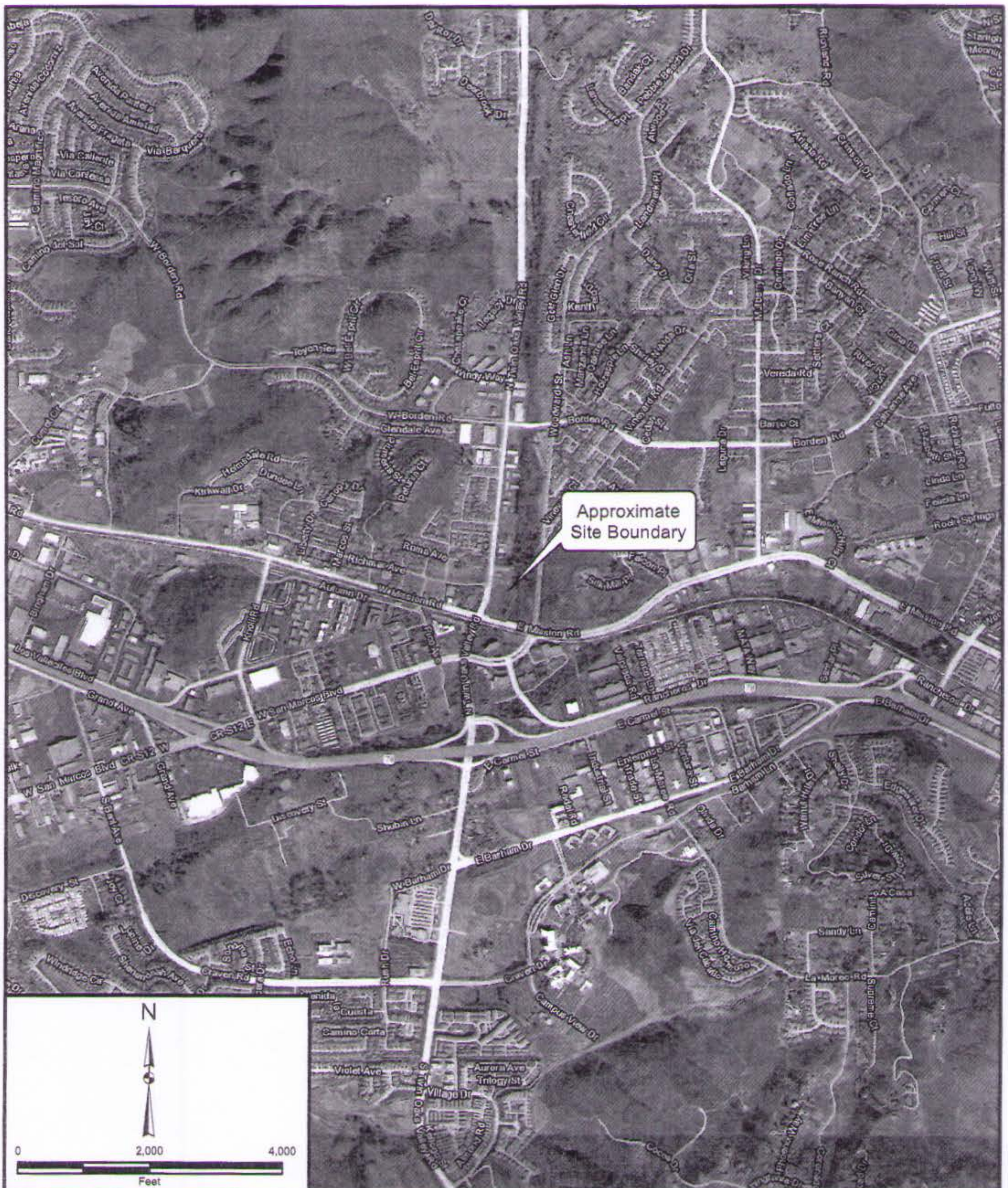
6.10 Plan Review

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

7.0 LIMITATIONS

The conclusions and recommendations presented in this report are based in part upon data that were obtained from a limited number of observations, site visits, excavations, samples, and tests. Such information is by necessity incomplete. The nature of many sites is such that differing geotechnical or geological conditions can occur within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report can be relied upon only if Leighton has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site.

Figures



Project: 11777.001	Eng/Geol: WDO/MDJ
Scale: 1 " = 2,000 '	Date: October 2017

Base Map: ESRI ArcGIS Online 2017
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

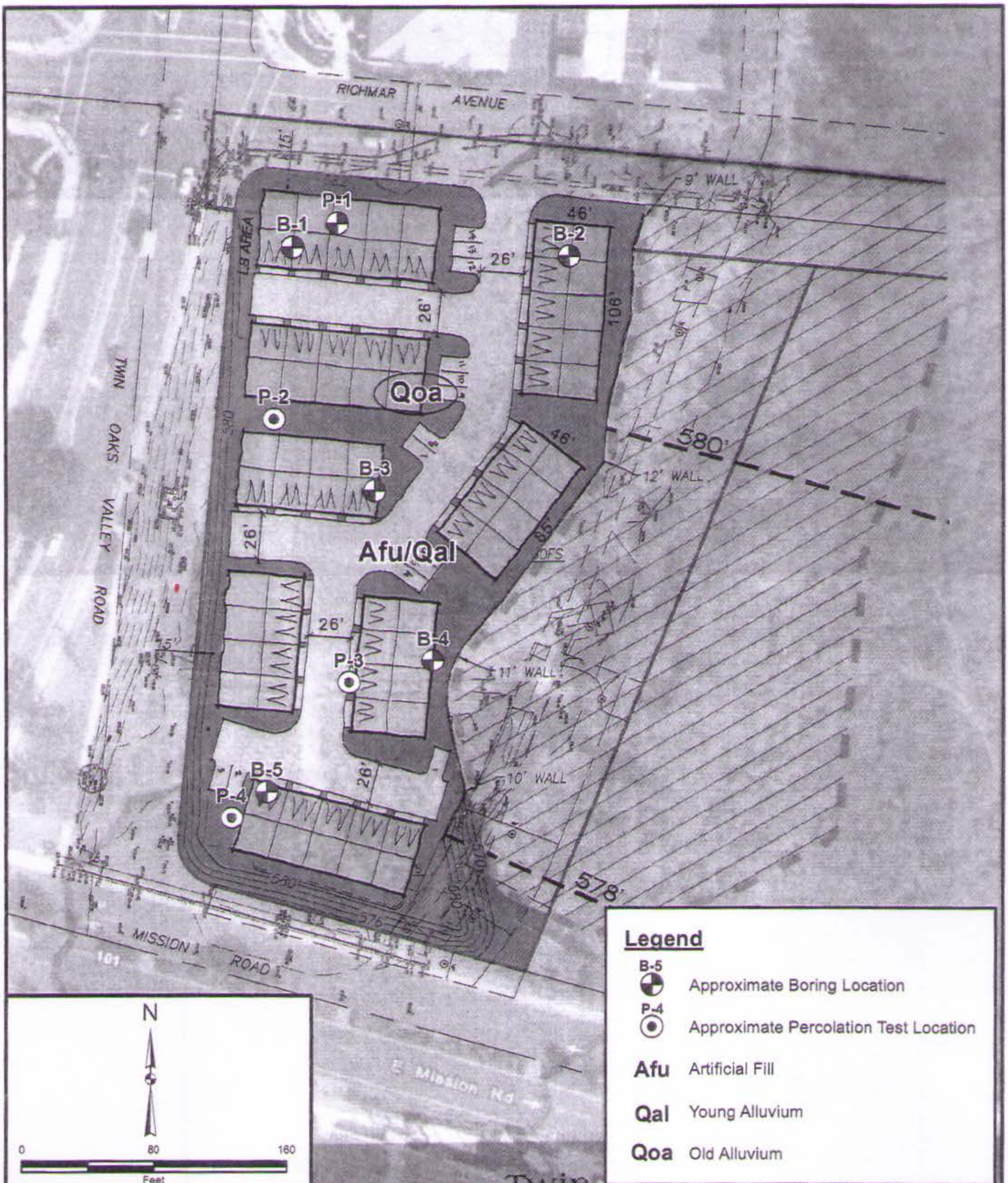
SITE LOCATION MAP

Warmington Residential
San Marcos, California

Figure 1

Leighton

Map Saved as P:\Drafting\11777\001\Maps\11777-001_F01_SLM_2017-10-19.mxd on 10/20/2017 10:41:41 AM



Project: 11777.001	Eng/Geol: WDO/MDJ
Scale: 1 " = 80 '	Date: October 2017

Base Map: ESRI ArcGIS Online 2017
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

GEOTECHNICAL MAP

Warmington Residential
San Marcos, California

Figure 2

Leighton

Map Saved as P:\Drafting\11777\001\Maps\11777-001_F02_GM_2017-10-19.mxd on 10/20/2017 10:41:54 AM

Appendix A
References

APPENDIX A

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Appendix B
Boring Logs and Percolation Tests

GEOTECHNICAL BORING LOG KEY

Date ______ Sheet 1 of 1
Project KEY TO BORING LOG GRAPHICS Project No. ______
Drilling Co. ______ Type of Rig ______
Hole Diameter ______ Drive Weight ______ Drop "
Elevation Top of Elevation ' Location ______

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION Logged By ______ Sampled By ______	Type of Tests
	0								Asphaltic concrete.	
									Portland cement concrete.	
								CL	Inorganic clay of low to medium plasticity; gravelly clay; sandy clay; silty clay; lean clay	
								CH	Inorganic clay; high plasticity, fat clays.	
								OL	Organic clay; medium to plasticity, organic silts.	
	5							ML	Inorganic silt; clayey silt with low plasticity.	
								MH	Inorganic silt; diatomaceous fine sandy or silty soils; elastic silt.	
								ML-CL	Clayey silt to silty clay.	
								GW	Well-graded gravel; gravel-sand mixture, little or no fines.	
								GP	Poorly graded gravel; gravel-sand mixture, little or no fines.	
	10							GM	Silty gravel; gravel-sand-silt mixtures.	
								GC	Clayey gravel; gravel-sand-clay mixtures.	
								SW	Well-graded sand; gravelly sand, little or no fines.	
								SP	Poorly graded sand; gravelly sand, little or no fines.	
								SM	Silty sand; poorly graded sand-silt mixtures.	
	15							SC	Clayey sand; sand-clay mixtures.	
									Bedrock.	
									Ground water encountered at time of drilling.	
				B-1					Bulk Sample 1.	
	20			B-1					Bulk Sample 2.	
				C-1					Core Sample.	
				G-1					Grab Sample.	
				R-1					Modified California Sampler (3" O.D., 2.5 I.D.).	
				SH-1					Shelby Tube Sampler (3" O.D.).	
	25			S-1					Standard Penetration Test SPT (Sampler (2" O.D., 1.4" I.D.).	
				PUSH					Sampler Penetrates without Hammer Blow.	
									Bulk Sample 2.	
	30									

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
SH SHELBY TUBE

TYPE OF TESTS:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
SA SIEVE ANALYSIS
AT ATTERBURG LIMITS
EI EXPANSION INDEX
RV R-VALUE

LEIGHTON

GEOTECHNICAL BORING LOG B-1

Project No.	11777.001	**Date Drilled**	9-6-17
Project	Warmington San Marcos	**Logged By**	CDL
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	CME-95 - 140lb - Autohammer - 30" Drop	**Ground Elevation**	580' msl
Location	See Figure 2	**Sampled By**	CDL

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.	
580	0			B-1 0-5'				ML	QUATERNARY OLD ALLUVIUM (Qoa) @ 0': Sandy SILT, brownish yellow, dry, fine SAND	
								CL/ML		
575	5			R-1	23 40 50/3"	128	15		@ 5': Clayey SILT, hard, mottled brown and dark brown, moist, some fine SAND, manganese nodules/staining	
570	10			S-1	11 16 28				@ 10': Clayey SILT, hard, mottled brown and dark brown, moist, some coarse SAND, manganese nodules/staining	
565	15				14 23 37	114	14	SM	@ 15': Silty SAND, dense, light olive-brown, moist, fine SAND, mild oxidation, infilled root casts	
560	20			S-2	7 12 13			SP	@ 20': Poorly-graded SAND, dense, light olive-brown, wet, fine to coarse SAND in poorly graded thin beds	
555	25			R-3	7 16 25	101	23	SC CL	@ 25': Clayey SAND, medium dense, dark olive-brown, moist, coarse SAND @ 26': Silty CLAY, hard, dark olive-brown, moist, trace fine SAND	
									Total Depth = 26.5 Feet **Perched groundwater encountered at 20-25 feet** **Backfilled on 9/6/17**	
550	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

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

GEOTECHNICAL BORING LOG B-2

Project No.	11777.001	**Date Drilled**	9-6-17
Project	Warmington San Marcos	**Logged By**	CDL
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	CME-95 - 140lb - Autohammer - 30" Drop	**Ground Elevation**	580' msl
Location	See Figure 2	**Sampled By**	CDL

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
580	0			B-1 0-5'				CL	QUATERNARY OLD ALLUVIUM (Qoa) @ 0': Silty CLAY, dark reddish brown, dry	
								CH		
575	5			R-1	10 12 17	106	21		@ 5': Fat CLAY, hard, mottled dark reddish brown, moist, poorly developed paleosol, irregular ped facies	
570	10			S-1	8 10 14			SM	@ 10': Silty SAND, medium dense, olive-brown, moist, fine SAND, some CLAY, mild oxidization	-200
565	15			R-2	8 15 26	105	21	CL	@ 15': CLAY with fine SAND, hard, mottled dark reddish brown and olive-brown, moist, trace fine micaceous SAND, moderately developed paleosol, charcoal fragments	
560	20			S-2	5 7 10			ML CL	@ 20': Clayey SILT, very stiff, mottled dark reddish brown and olive-brown, moist, some fine mica SAND @ 20.8': Silty CLAY, very stiff, mottled dark reddish brown and olive-brown, moist, no SAND	
555	25			R-3	18 50/5"	115	17	ML/CL	BEDROCK (RESIDUAL SOIL/META SEDIMENTARY (Mzu) @ 25': Silty to clayey SANDSTONE, very dense, greenish black, moist, fine to coarse SAND with depth SILT to CLAY with depth root casts healed with reddish brown matrix	
550	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

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

GEOTECHNICAL BORING LOG B-2

Project No.	11777.001	**Date Drilled**	9-6-17
Project	Warmington San Marcos	**Logged By**	CDL
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	CME-95 - 140lb - Autohammer - 30" Drop	**Ground Elevation**	580' msl
Location	See Figure 2	**Sampled By**	CDL

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
550	30			S-3	7 10 17			SM	BEDROCK (RESIDUAL SOIL/META SEDIMENTARY (Mzu) continued) @ 30': Silty SANDSTONE, dense, greenish black, wet, fine SAND, liquefaction dilatancy from driving sample with upper 1 foot	
545	35			S-4	31 50/2"			GC	@ 35': Refusal on bedrock, sample @ 35.5': Clayey GRAVEL CONGLOMERATE, very dense, greenish black, wet, coarse SAND, some small angular gravel **Total Depth = 35.5 Feet** **Groundwater encountered at 30 feet time of drilling** **Backfilled on 9/6/17**	
540	40									
535	45									
530	50									
525	55									
520	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

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

GEOTECHNICAL BORING LOG B-3

Project No.	11777.001	**Date Drilled**	9-6-17
Project	Warmington San Marcos	**Logged By**	CDL
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	CME-95 - 140lb - Autohammer - 30" Drop	**Ground Elevation**	579' msl
Location	See Figure 2	**Sampled By**	CDL

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 0-5'				ML	QUATERNARY YOUNG ALLUVIUM (Qal) @ 0': Sandy SILT, yellowish red, moist, fine SAND	EI, CR
								SP		
575	5			S-1	6 7 10				@ 5': Poorly-graded SAND, medium dense, strong brown, damp, medium SAND	
								SC	QUATERNARY OLD ALLUVIUM (Qoa)	
570	10			R-1	14 39 50/5"	119	10		@ 10': Clayey SAND, very dense, dark brown, moist, coarse SAND, fine downwards, to medium to coarse SAND, fines reduce with depth (damaged rings)	
565	15			S-2	6 9 15				@ 15': Clayey SAND, dense, dark brown, moist, medium to coarse SAND, 1 foot interbed of CLAY, medium expansive, manganese, moderately developed paleosol	
560	20			R-2	7 12 25	113	17		@ 20': Clayey SAND, medium dense, dark brown, upper sample is wet, fine to coarse SAND, manganese development, trace well-rounded GRAVEL	
555	25			S-3	21 15 50/2"				@ 25': Clayey SAND, very dense, dark brown, wet, fine to coarse SAND, angular fine gravel grades with depth to silty SAND, very dense, reddish brown, moist, fine SAND	
550	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

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

GEOTECHNICAL BORING LOG B-3

Project No.	11777.001	**Date Drilled**	9-6-17
Project	Warmington San Marcos	**Logged By**	CDL
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	CME-95 - 140lb - Autohammer - 30" Drop	**Ground Elevation**	579' msl
Location	See Figure 2	**Sampled By**	CDL

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.	
	30			S-4	16 50/6"			SC	@ 30': Clayey SAND, very dense, yellowish red, wet, medium to coarse SAND	
									Total Depth = 30.5 Feet **Groundwater encountered at 20 feet** **Backfilled on 9/6/17**	
545										
	35									
540										
	40									
535										
	45									
530										
	50									
525										
	55									
520										
	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

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GEOTECHNICAL BORING LOG B-4

Project No.	11777.001	**Date Drilled**	9-6-17
Project	Warmington San Marcos	**Logged By**	CDL
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	CME-95 - 140lb - Autohammer - 30" Drop	**Ground Elevation**	576' msl
Location	See Figure 2	**Sampled By**	CDL

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
575	0							SM	QUATERNARY OLD ALLUVIUM (Qoa) @ 0': Silty SAND, yellowish red, damp, fine SAND	
570	5			R-1	12 12 16	128	10	SC/CL	@ 5': Clayey SAND to sandy CLAY, hard, mottled dark reddish brown and strong brown, moist, medium to fine SAND with depth, manganese development	DS
565	10			S-1	5 5 9				@ 10': Sandy clayey SILT, very stiff, mottled strong brown and olive-brown, moist, fine SAND with coarse SAND, grades with depth to sandy SILT, fully decomposed vertical rootlets	
560	15			R-2	7 14 21	104	22	SC	@ 15': Clayey SAND, medium dense, olive-brown, wet, fine to coarse SAND with depth, very thin braided channels, trace fine subround GRAVEL	
555	20			S-2	4 7 9			SW	@ 20': Well-graded SAND, medium dense, olive-brown, wet, fine to coarse, low sample recovery	
550	25			R-3	11 24 50/4"			ML	@ 25': Sandy SILT, hard, mottled to laminated olive-brown and brown, moist, very thin sand bed	
	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

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GEOTECHNICAL BORING LOG B-4

Project No.	11777.001	**Date Drilled**	9-6-17
Project	Warmington San Marcos	**Logged By**	CDL
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	CME-95 - 140lb - Autohammer - 30" Drop	**Ground Elevation**	576' msl
Location	See Figure 2	**Sampled By**	CDL

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
	30			S-3	15 35 47			GC	@ 30': Clayey GRAVEL, very dense, variegated (orange, reddish brown greenish black) moist, angular	
545										
									CRETACEOUS TONALITE (Kt)	
	35			S-4	50/2"				@ 35': Tonalite, poorly-graded SAND, very dense, damp, quartz vein, sample recovered by coring effect, weathered	
540										
									@ 40': Tonalite, poorly-graded SAND, orange-brown, some clay development	
	40			S-5	50/3"					
535									**Total Depth = 40 Feet** **Groundwater encountered at 15 feet at time of drilling** **Backfilled on 9/6/17**	
	45									
530										
	50									
525										
	55									
520										
	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

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GEOTECHNICAL BORING LOG B-5

Project No.	11777.001	**Date Drilled**	9-6-17
Project	Warmington San Marcos	**Logged By**	CDL
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	CME-95 - 140lb - Autohammer - 30" Drop	**Ground Elevation**	577' msl
Location	See Figure 2	**Sampled By**	CDL

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 0-5'				SM	QUATERNARY OLD ALLUVIUM (Qoa) @ 0': Silty SAND, yellowish red, damp, fine SAND	
575										
	5			S-1	14 27 38			ML	@ 5': Sandy SILT, hard, mottled reddish brown and strong brown, damp, fine SAND, manganese development	-200
570										
	10			R-1	9 15 23	110	18		@ 10': Sandy SILT, hard, mottled reddish brown and strong brown, damp, fine SAND, root clasts, moist	
565										
	15			S-2	8 10 14			SM	@ 15': Silty SAND, dense, brown, very moist, fine to medium SAND	
560										
	20			R-2	11 17 25	117	15	SW	@ 20': Well-graded SAND, medium dense, brown, wet, fine to coarse SAND, some CLAY	
555										
	25			S-3	4 9 12			SM	@ 25': Silty SAND, dense, mottled reddish brown and brown, very moist, fine SAND, micaceous	
550										
	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

GEOTECHNICAL BORING LOG B-5

Project No.	11777.001	**Date Drilled**	9-6-17
Project	Warmington San Marcos	**Logged By**	CDL
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	CME-95 - 140lb - Autohammer - 30" Drop	**Ground Elevation**	577' msl
Location	See Figure 2	**Sampled By**	CDL

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
	30			S-4	6 6 18			CL	BEDROCK (RESIDUAL SOIL, META SEDIMENTARY (Mzu) @ 30': CLAY, hard, greenish black, very moist, some fine SAND	
545									**Total Depth = 31 Feet** **No groundwater encountered at time of drilling** **Backfilled on 9/6/17**	
	35									
540										
	40									
535										
	45									
530										
	50									
525										
	55									
520										
	60									

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.

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

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FIELD PERCOLATION TEST DATA SHEET

Project Name:	Warminton	Project No.:	111777.001
Proj. Address:	Twin Oaks Road, San Marcos CA		

SOIL TYPE / TEST LOCATION / BOREHOLE

Soil Type: brown silty sand

Location: P-1

Hole Dia: 8"

Depth 4.17'

Tested by:SMM Pre-Saturation Date:9/6/2017 Test Date:9/7/2017

Notes: Measurements in 100ths of foot

Time of Day	Interval / Notes	Water Level	Time of Day	Interval / Notes	Water Level
9:15		2.62			
9:45	30 min	2.62			
10:15	31 min	2.63			
10:45	32 min	2.64			
11:15	33 min	2.64			
11:45	34 min	2.64			
12:15	35 min	2.65			
12:45	36 min	2.65			
1:15	37 min	2.65			
1:45	38 min	2.66			
2:15	39 min	2.66			
2:45	40 min	2.66			
3:15	41 min	2.66			

FOR OFFICE USE ONLY	DATE RECEIVED:	By:

Notes: 250.0 min/inch

Leighton

FIELD PERCOLATION TEST DATA SHEET

Project Name:	Warminton	Project No.:	111777.001
Proj. Address:	Twin Oaks Road, San Marcos CA		

SOIL TYPE / TEST LOCATION / BOREHOLE

Soil Type: brown silty sand

Location: P-2

Hole Dia: 8"

Depth 3.96'

Tested by:SMM Pre-Saturation Date:9/6/2017 Test Date:9/7/2017

Notes: Measurements in 100ths of foot

Time of Day	Interval / Notes	Water Level	Time of Day	Interval / Notes	Water Level
9:11		2.80			
9:41	30 min	2.80			
10:11	30 min	2.81			
10:41	30 min	2.82			
11:11	30 min	2.82			
11:41	30 min	2.83			
12:11	30 min	2.83			
12:41	30 min	2.83			
1:11	30 min	2.84			
1:41	30 min	2.84			
2:11	30 min	2.84			
2:41	30 min	2.85			
3:11	30 min	2.85			

FOR OFFICE USE ONLY	**DATE RECEIVED:**	**By:**

Notes: perc rate 500 min/inch

Leighton

FIELD PERCOLATION TEST DATA SHEET

Project Name:	Warminton	Project No.:	111777.001
Proj. Address:	Twin Oaks Road, San Marcos CA		

SOIL TYPE / TEST LOCATION / BOREHOLE

Soil Type: brown silty sand

Location: P-3

Hole Dia: 8"

Depth 3.75'

Tested by:SMM   Pre-Saturation Date:9/6/2017   Test Date:9/7/2017

Notes: Measurements in 100ths of foot

Time of Day	Interval / Notes	Water Level	Time of Day	Interval / Notes	Water Level
9:07		2.80			
9:37	30 min	2.81			
10:07	30 min	2.82			
10:37	30 min	2.82			
11:07	30 min	2.82			
11:37	30 min	2.82			
12:07	30 min	2.82			
12:37	30 min	2.82			
1:07	30 min	2.82			
1:37	30 min	2.82			
2:07	30 min	2.82			
2:37	30 min	2.82			
3:07	30 min	2.82			

FOR OFFICE USE ONLY	DATE RECEIVED:	By:

Notes: no perc

Leighton

FIELD PERCOLATION TEST DATA SHEET

Project Name:	Warminton	Project No.:	111777.001
Proj. Address:	Twin Oaks Road, San Marcos CA		

SOIL TYPE / TEST LOCATION / BOREHOLE

Soil Type: brown silty sand

Location: P-4

Hole Dia: 8"

Depth 3.7'

Tested by:SMM Pre-Saturation Date:9/6/2017 Test Date:9/7/2017

Notes: Measurements in 100ths of foot

Time of Day	Interval / Notes	Water Level	Time of Day	Interval / Notes	Water Level
9:02		2.75			
9:32	30 min	2.75			
10:02	30 min	2.78			
10:32	30 min	2.8			
10:32	add Water	2.75			
11:02	30 min	2.75			
11:32	30 min	2.76			
12:02	30 min	2.77			
12:32	30 min	2.77			
1:02	30 min	2.77			
1:32	30 min	2.77			
2:02	30 min	2.78			
2:32	30 min	2.78			
3:02	30 min	2.78			

FOR OFFICE USE ONLY	DATE RECEIVED:	By:

Notes: no perc

Appendix C
Laboratory Testing Procedures and Test Results

APPENDIX C

Laboratory Testing Procedures and Test Results

Direct Shear Test: A direct shear test were performed on a selected undisturbed sample which was soaked for a minimum of 24 hours under a surcharge equal to the applied normal force during testing. After transfer of the sample to the shear box and reloading of the sample, the pore pressures set up in the sample (due to the transfer) were allowed to dissipate for a period of approximately 1 hour prior to application of shearing force. The sample was tested under various normal loads utilizing a motor-driven, strain-controlled, direct-shear testing apparatus at a strain rate of less 0.05 inches per minute. The test result is presented on the attached figure.

Moisture and Density Determination Tests: Moisture content (ASTM Test Method D2937) and dry density determinations were performed on relatively undisturbed ring samples obtained from the test borings and/or trenches. The results of these tests are presented in the geotechnical boring logs (Appendix B).

Particle/Grain Size Analysis: Particle size analysis was performed by mechanical sieving and wash sieving methods according to ASTM D1140. Plots of sieve results are provided on the figures in this appendix.

Expansion Index Tests: The expansion potential of selected materials was evaluated by the Expansion Index Text, ASTM Test Method 4829. Specimens are molded under a given compactive energy to approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimens are loaded to an equivalent 144 psf surcharge and are inundated with water until volumetric equilibrium is reached. The results of these tests are presented in the table below:

Sample Location	Sample Description	Expansion Index	Expansion Potential
B-1 @ 0 to 5 feet	Clayey SAND (SC)	65	Medium

APPENDIX C (Continued)

Soluble Sulfates: The soluble sulfate content of a selected sample was determined by standard geochemical methods (Caltrans Test Method CT417). The test result is presented in the table below:

Sample Location	Sulfate Content (%)	Potential Degree of Sulfate Attack*
B-1 @ 1 foot to 5 feet	0.0150	Negligible

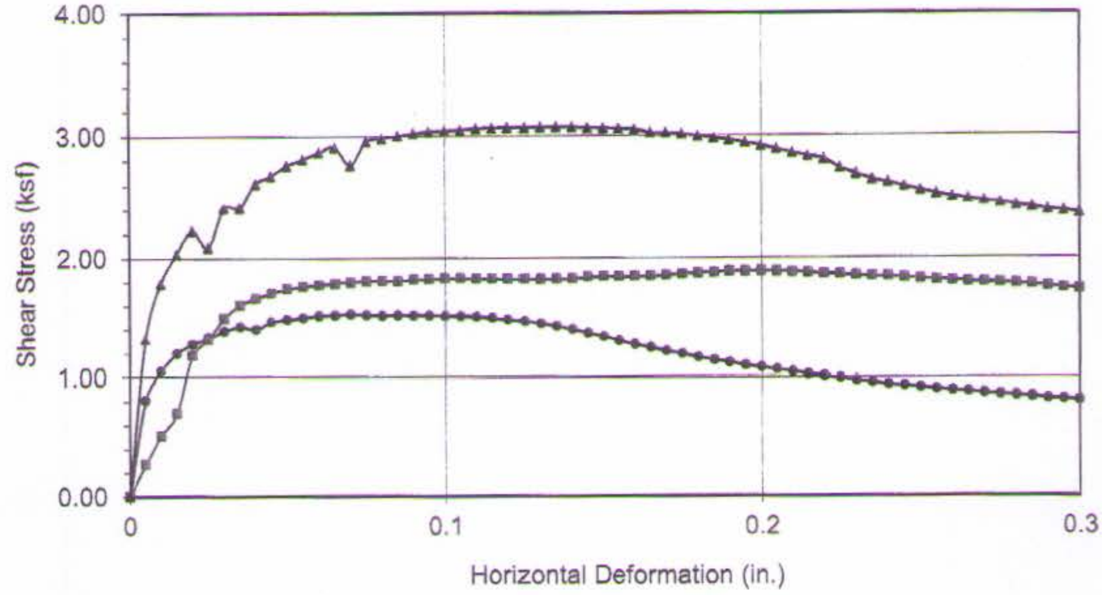
* Based on the 2008 edition of American Concrete Institute (ACI) Committee 318R, Table No. 4.2.1.

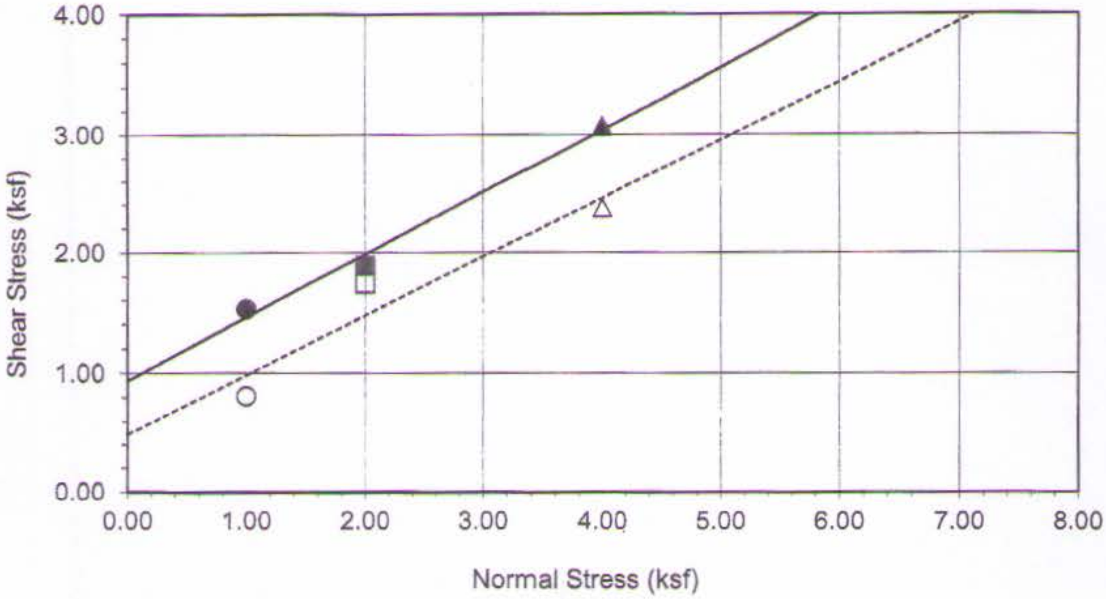
Chloride Content: Chloride content was tested in accordance with DOT Test Method No. 422. The results are presented below:

Sample Location	Chloride Content, ppm
B-1 @ 1 foot to 5 feet	24

Minimum Resistivity and pH Tests: Minimum resistivity and pH tests were performed in general accordance with California Test Method 643. The results are presented in the table below:

Sample Location	pH	Minimum Resistivity (ohms-cm)
B-1 @ 1 foot to 5 feet	7.53	1300





Boring No.	**B-4**
Sample No.	**R-1**
Depth (ft)	**5**
Sample Type:	Ring
Soil Identification: Lean Clay (CL), Reddish Brown.	

Strength Parameters

	C (psf)	ϕ (°)
Peak	938	28
Ultimate	490	26

Normal Stress (kip/ft²)	1.000	2.000	4.000
Peak Shear Stress (kip/ft²)	● 1.527	■ 1.891	▲ 3.069
Shear Stress @ End of Test (ksf)	○ 0.807	□ 1.737	△ 2.372
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	18.79	18.79	18.79
Dry Density (pcf)	111.0	112.1	108.1
Saturation (%)	97.9	100.7	90.7
Soil Height Before Shearing (in.)	1.0024	0.9868	0.9760
Final Moisture Content (%)	23.4	22.2	22.5

Leighton

DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 11777.001

Warmington/Due Dilligance

09-17

Appendix D
General Earthwork and Grading Specifications for Rough Grading

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. 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.

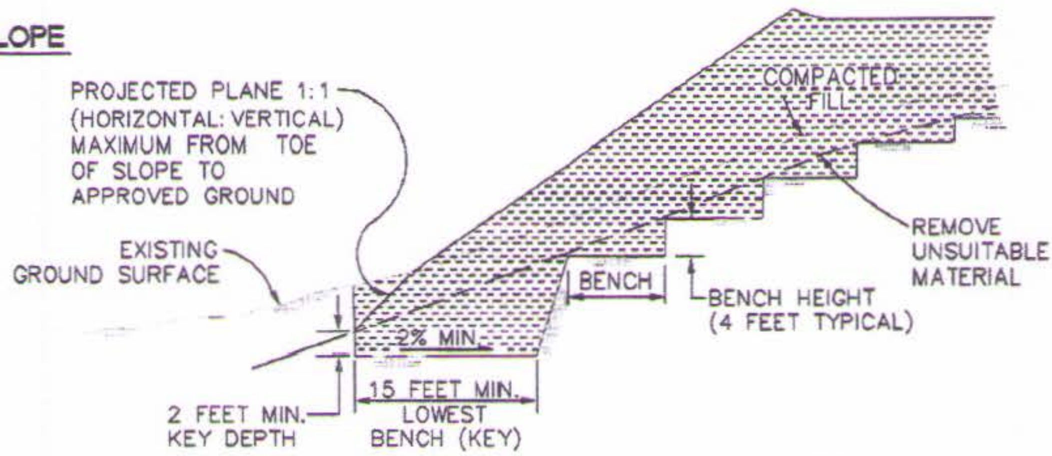
7.3 Lift Thickness

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.

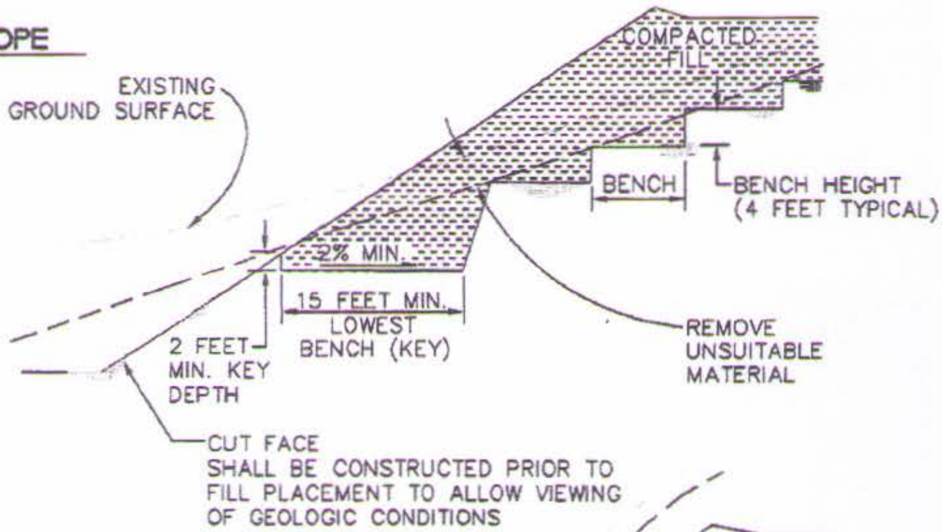
7.4 Observation and Testing

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

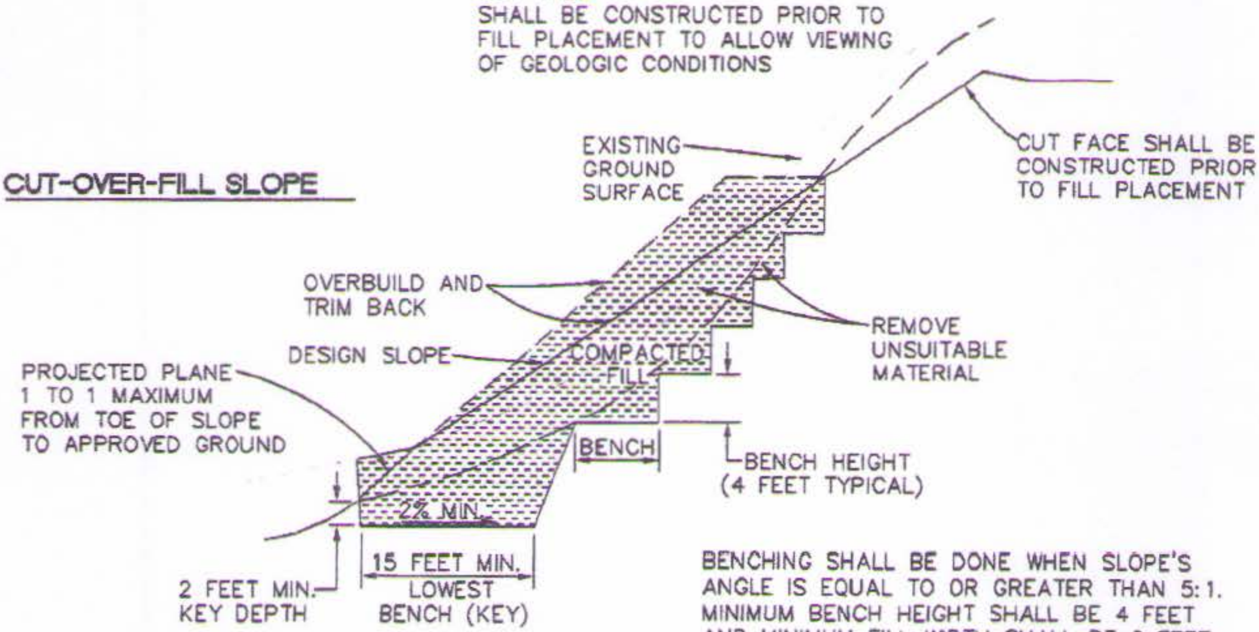
FILL SLOPE



FILL-OVER-CUT SLOPE



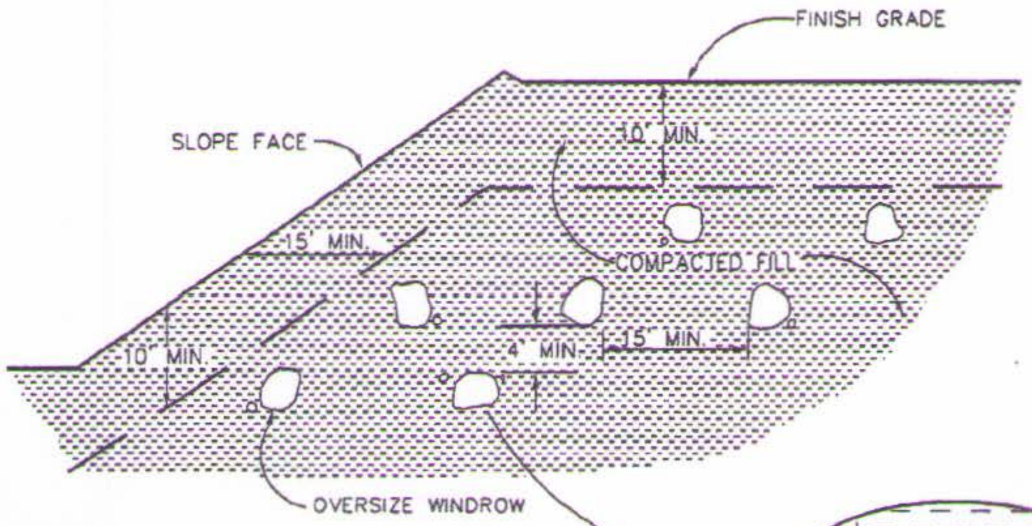
CUT-OVER-FILL SLOPE

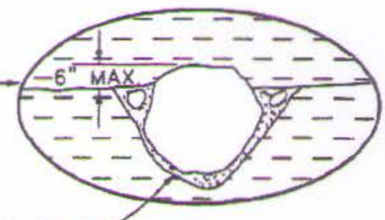


BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

KEYING AND BENCHING

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL A

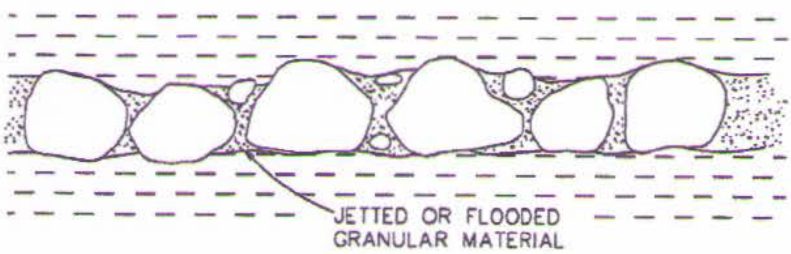




GRANULAR MATERIAL TO BE DENSIFIED IN PLACE BY FLOODING OR JETTING.

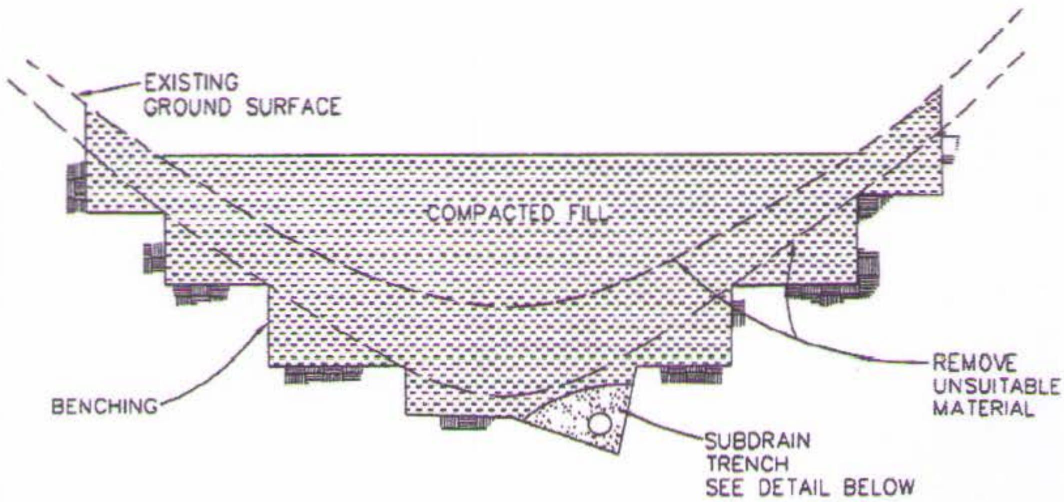
DETAIL

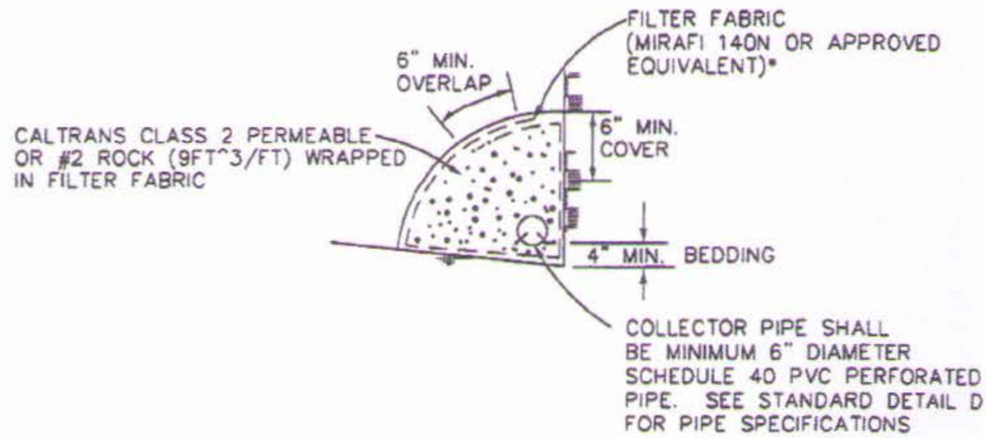
- OVERSIZE ROCK IS LARGER THAN 8 INCHES IN LARGEST DIMENSION.
- EXCAVATE A TRENCH IN THE COMPACTED FILL DEEP ENOUGH TO BURY ALL THE ROCK.
- BACKFILL WITH GRANULAR SOIL JETTED OR FLOODED IN PLACE TO FILL ALL THE VOIDS.
- DO NOT BURY ROCK WITHIN 10 FEET OF FINISH GRADE.
- WINDROW OF BURIED ROCK SHALL BE PARALLEL TO THE FINISHED SLOPE.



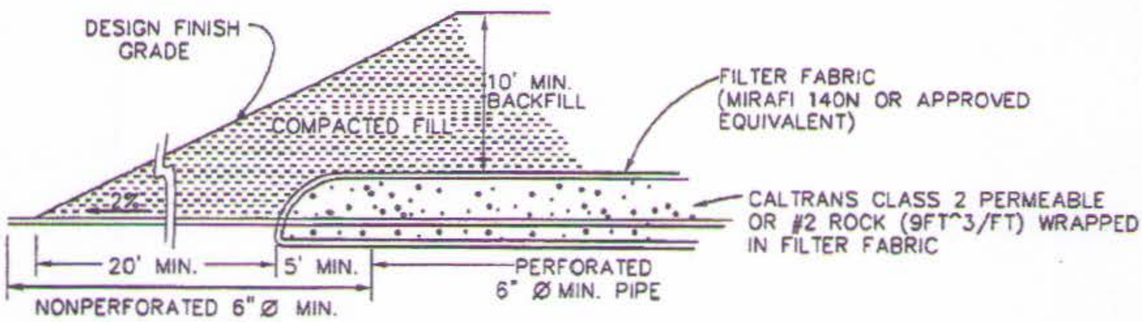
TYPICAL PROFILE ALONG WINDROW

OVERSIZE ROCK DISPOSAL	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL B



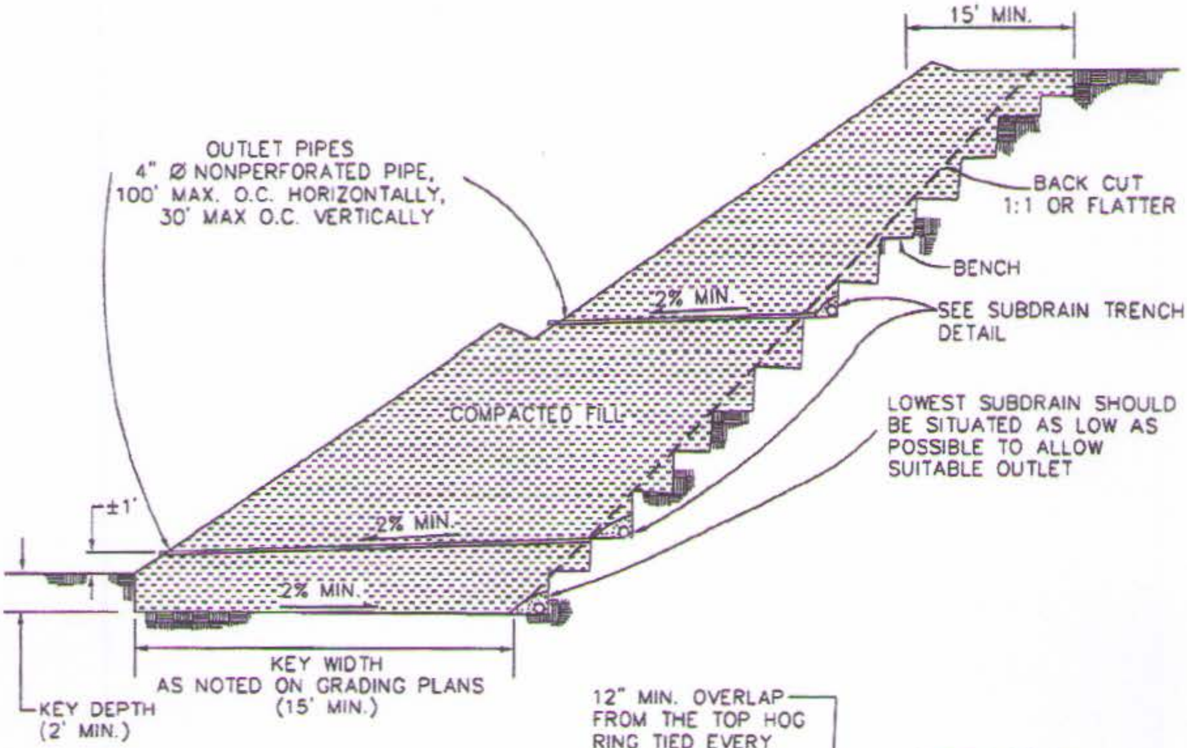


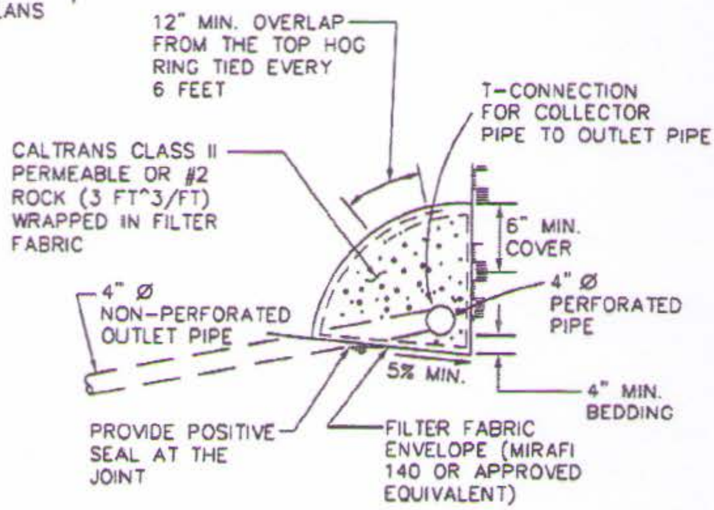
SUBDRAIN DETAIL



DETAIL OF CANYON SUBDRAIN OUTLET

CANYON SUBDRAINS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL C





SUBDRAIN TRENCH DETAIL

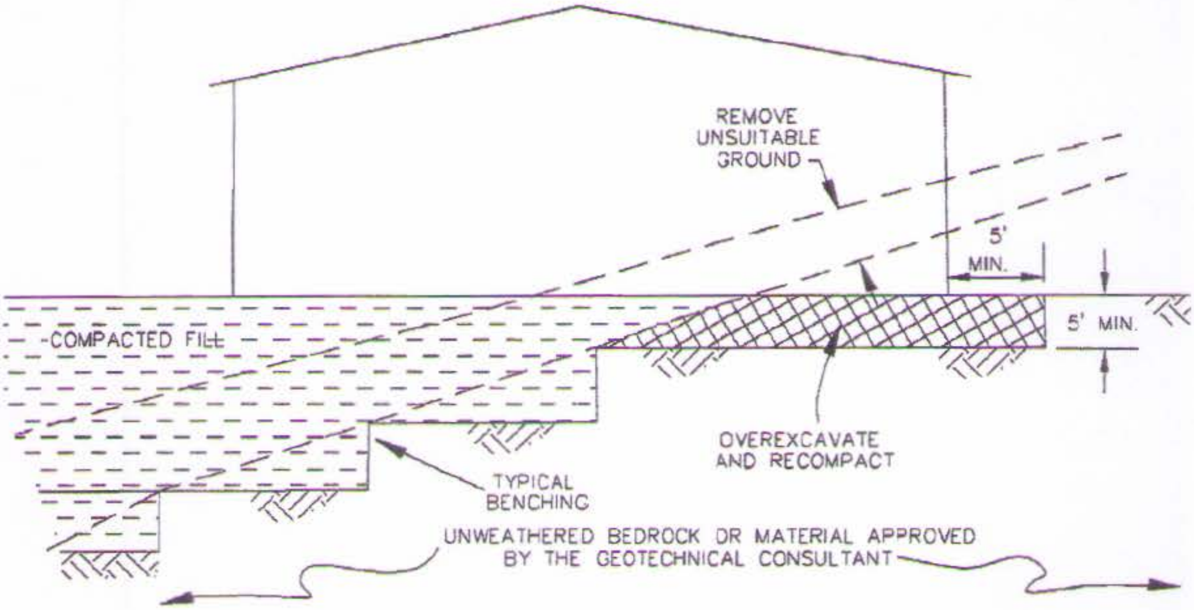
SUBDRAIN INSTALLATION – subdrain collector pipe shall be installed with perforation down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" to 1/2" if drill holes are used. All subdrain pipes shall have a gradient of at least 2% towards the outlet.

SUBDRAIN PIPE – Subdrain pipe shall be ASTM D2751, SDR 23.5 or ASTM D1527, Schedule 40, or ASTM D3034, SDR 23.5, Schedule 40 Polyvinyl Chloride Plastic (PVC) pipe.

All outlet pipe shall be placed in a trench no wider than twice the subdrain pipe.

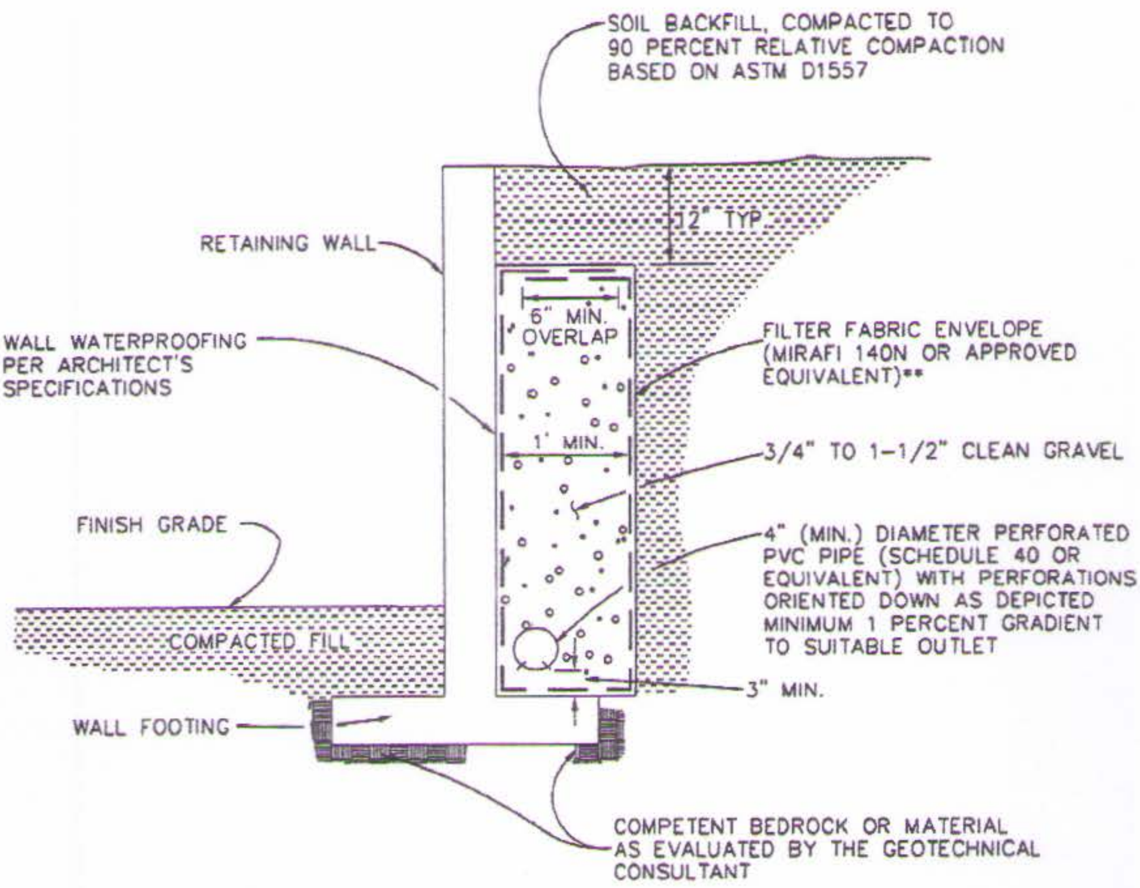
BUTTRESS OR REPLACEMENT FILL SUBDRAINS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL D

CUT–FILL TRANSITION LOT OVEREXCAVATION



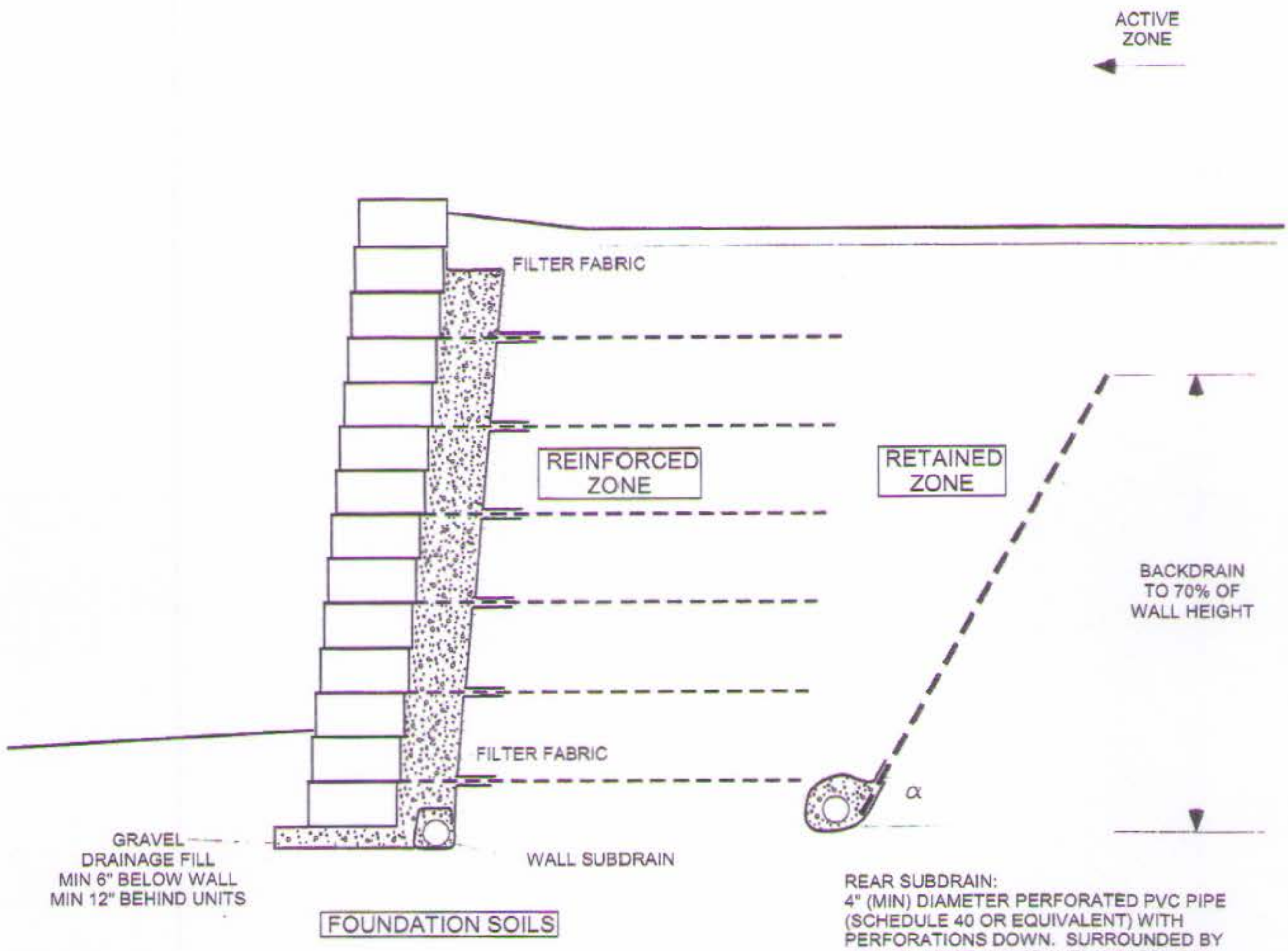
TRANSITION LOT FILLS

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAIL E



NOTE: UPON REVIEW BY THE GEOTECHNICAL CONSULTANT, COMPOSITE DRAINAGE PRODUCTS SUCH AS MIRADRAIN OR J-DRAIN MAY BE USED AS AN ALTERNATIVE TO GRAVEL OR CLASS 2 PERMEABLE MATERIAL. INSTALLATION SHOULD BE PERFORMED IN ACCORDANCE WITH MANUFACTURER'S SPECIFICATIONS.

RETAINING WALL DRAINAGE	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL F



REAR SUBDRAIN:
4" (MIN) DIAMETER PERFORATED PVC PIPE (SCHEDULE 40 OR EQUIVALENT) WITH PERFORATIONS DOWN. SURROUNDED BY 1 CU. FT/FT OF 3/4" GRAVEL WRAPPED IN FILTER FABRIC (MIRAFI 140N OR EQUIVALENT)

OUTLET SUBDRAINS EVERY 100 FEET, OR CLOSER, BY TIGHTLINE TO SUITABLE PROTECTED OUTLET

NOTES:

1) MATERIAL GRADATION AND PLASTICITY

REINFORCED ZONE:

SIEVE SIZE	% PASSING
1 INCH	100
NO. 4	20-100
NO. 40	0-60
NO. 200	0-35

FOR WALL HEIGHT < 10 FEET, PLASTICITY INDEX < 20
FOR WALL HEIGHT 10 TO 20 FEET, PLASTICITY INDEX < 10
FOR TIERED WALLS, USE COMBINED WALL HEIGHTS
WALL DESIGNER TO REQUEST SITE-SPECIFIC CRITERIA FOR WALL HEIGHT > 20 FEET

GRAVEL DRAINAGE FILL:

SIEVE SIZE	% PASSING
1 INCH	100
3/4 INCH	75-100
NO. 4	0-60
NO. 40	0-50
NO. 200	0-5

2) CONTRACTOR TO USE SOILS WITHIN THE RETAINED AND REINFORCED ZONES THAT MEET THE STRENGTH REQUIREMENTS OF WALL DESIGN.

3) GEOGRID REINFORCEMENT TO BE DESIGNED BY WALL DESIGNER CONSIDERING INTERNAL, EXTERNAL, AND COMPOUND STABILITY.

3) GEOGRID TO BE PRETENSIONED DURING INSTALLATION.

4) IMPROVEMENTS WITHIN THE ACTIVE ZONE ARE SUSCEPTIBLE TO POST-CONSTRUCTION SETTLEMENT. ANGLE $\alpha = 45+\phi/2$, WHERE ϕ IS THE FRICTION ANGLE OF THE MATERIAL IN THE RETAINED ZONE.

5) BACKDRAIN SHOULD CONSIST OF J-DRAIN 302 (OR EQUIVALENT) OR 6-INCH THICK DRAINAGE FILL WRAPPED IN FILTER FABRIC. PERCENT COVERAGE OF BACKDRAIN TO BE PER GEOTECHNICAL REVIEW.

SEGMENTAL RETAINING WALLS

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAIL G

Appendix E
GBA Insert

Important Information about This

Geotechnical-Engineering Report

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

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

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems 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 below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project 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.*

This Report May Not Be Reliable

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

- for a different client;
- for a different project;
- 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, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been 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 your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment 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 from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final,* because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed 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 full-time 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 on hand quickly 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 observation.

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 informational 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, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. 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. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any 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 yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

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, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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

Appendix F
County of San Diego Form I-8

Categorization of Infiltration Feasibility Condition

FORM I-8

Part 1 - Full Infiltration Feasibility Screening Criteria

Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
1	**Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X

Provide basis:

Based on our field percolation testing, the in-situ infiltration rates of the soils at the subject site are less than 0.01 inches per hour (Leighton, 2017). Specifically, the calculated infiltration rate via the Porchet Method and applied safety factor of 2 is less than 0.01 inches per hour across the site and therefore the site is considered appropriate for a "No-Infiltration" designation.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.

2	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.	X	

Provide basis:

If the infiltration rates were greater than 0.5 inches per hour, it may be possible that the risk of geotechnical hazards would not be increased provided mitigation is performed for any underground utilities/structures, slopes (i.e., setbacks) and undocumented fill depths greater than 5 feet within the vicinity of the proposed infiltration site.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.

FORM I-8 Page 2 of 4

Criteria	Screening Question	Yes	No
3	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	

Provide basis:

If the infiltration rates were greater than 0.5 inches per hour, it may be possible that the risk of groundwater contamination would not be increased provided there are no contaminated soil or groundwater sites within 250 feet of the proposed infiltration site. In addition, groundwater depths are anticipated to be greater than 50 feet bgs.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.

4	**Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	

Provide basis:

If the infiltration rates were greater than 0.5 inches per hour, it may be possible that potential water balance issues would not be affected provided there are no unlined site drainages/creeks/streams within 250 feet of the proposed infiltration site.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.

Part 1 Result*	If all answers to rows 1 - 4 are "**Yes**" a full infiltration design is potentially feasible. The feasibility screening category is **Full Infiltration** If any answer from row 1-4 is "**No**", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design. Proceed to Part 2	Go to Part 2

Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria

Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
5	**Do soil and geologic conditions allow for infiltration in any appreciable rate or volume?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X

Provide basis:

Based on our field percolation testing, the in-situ infiltration rates of the soils at the subject site are less than 0.01 inches per hour (Leighton, 2017). Specifically, the calculated infiltration rate via the Porchet Method and applied safety factor of 2 is less than 0.01 inches per hour across the site and therefore the site is considered appropriate for a "No-Infiltration" designation.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

6	**Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.	X	

Provide basis:

If partial infiltration conditions (greater than 0.01 inches per hour) existed across the site, it may be possible that the risk of geotechnical hazards will not be increased by partial infiltration provided mitigation is performed for any underground utilities/structures, slopes (i.e., setbacks) and undocumented fill depths greater than 5 feet within the vicinity of the proposed infiltration site. Mitigation includes subsurface vertical barriers and subdrains to limit perched ground water mounding conditions.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

<table>
<tr><th colspan="4">FORM I-8 Page 4 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>7</td><td>Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:

If partial infiltration conditions (greater than 0.01 inches per hour) existed across the site, it may be possible that the risk of groundwater contamination will not be increased by partial infiltration provided there are no contaminated soil or groundwater sites within 250 feet of the proposed infiltration site. In addition, groundwater depths are anticipated to be greater than 50 feet bgs.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</td></tr>
<tr><td>8</td><td>Can infiltration be allowed without violating downstream water rights? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:

If partial infiltration conditions (greater than 0.01 inches per hour) existed across the site, violation of downstream water rights is not anticipated based on the site location and that there are no unlined site drainages/creeks/streams within 250 feet of the proposed infiltration site.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</td></tr>
<tr><td>Part 2 Result*</td><td colspan="2">If all answers from row 5-8 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration.

If any answer from row 5-8 is no, then infiltration of any volume is considered to be infeasible within the drainage area. The feasibility screening category is No Infiltration.</td><td>No Infiltration Feasibility</td></tr>
</table>