APPENDIX I

REPORT OF GEOTECHNICAL RECONNAISSANCE

SPA05-001, GPA05-003, REZ05-005,
TM 5424, LOG NO. 05-02-009

for the

DRAFT SUBSEQUENT ENVIRONMENTAL IMPACT REPORT

August 2013

LIMITED GEOTECHNICAL INVESTIGATION, CAMPUS PARK WEST UNINCORPORATED SAN DIEGO COUNTY (NORTHEAST CORNER OF HWY 76 AND INTERSTATE 15), CALIFORNIA (GPA 05-003, SPA 05-001, REZ 05-005, TM 5424, ER 05-02-009

Prepared For:

PAPPAS INVESTEMENT
2020 L STREET, 5TH FLOOR
SACRAMENTO, CA 95814

Project No. 042410-003

September 13, 2012

Leighton and Associates, Inc.

A LEIGHTON GROUP COMPANY

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Project No. 042410-003

To: Pappas Investments
2020 L Street, 5th Floor
Sacramento, CA 95814

Attention: Mr. Thad Johnson

Subject: Limited Geotechnical Investigation
Campus Park West, Unincorporated San Diego County
(Northeast Corner of Hwy 76 and Interstate 15), California

In accordance with your request and authorization, we have conducted a limited geotechnical investigation for a proposed Campus Park West development located at the intersection of Pankey Road and Pala Road in the Pala area of San Diego County, California. Based on the results of our study, it is our professional opinion that the site is suitable for the proposed development provided that the recommendations presented herein are incorporated into the design, grading, and construction of the site. The accompanying report presents an initial summary of our investigation and provides preliminary geotechnical conclusions and recommendations relative to the proposed site development. Additional geotechnical studies will be required as plans become finalized.

If you have any questions regarding our report, please do not hesitate to contact this office. We appreciate this opportunity to be of service.

Respectfully submitted,

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1.0 INTRODUCTION

1.1 Purpose and Scope

This report presents the results of our limited geotechnical investigation for Campus Park West, a proposed residential and commercial development to be constructed on the Campus Park West property, Tract 5424, located at the intersection of Pankey Road and Pala Road (Highway SR-76) in the unincorporated Pala area of San Diego County, California (see Figure 1).

Our investigation included limited geotechnical exploration of the site, laboratory testing of selected soil samples, geotechnical analysis of the data collected, and preparation of this report. The purpose of our geotechnical investigation was to evaluate existing geotechnical conditions present at the site and to provide preliminary conclusions and geotechnical recommendations relative to the proposed development of the property. Additional geotechnical investigation will be required to finalize and refine the recommendations of this report.

1.1.1 Scope of Work

As part of our geotechnical investigation, we performed the following:

- Review of available pertinent, published and unpublished geotechnical literature maps, and aerial photographs (Appendix A).

- Review of the available previous geotechnical reports by Leighton and others (Appendix A) and conceptual site development plans (Project Design Consultants, 2008).

- Field reconnaissance of the existing onsite geotechnical conditions.

- Coordination with Underground Services Alert (USA) to locate potential underground utilities on or adjacent to the site.

- Subsurface exploration consisting of the excavation, logging, and sampling of four (4) small diameter exploratory borings utilized to develop the site design parameters. The borings were advanced utilizing a truck mounted drill rig to depths ranging from 40 to 51.5 feet. We also advanced six (6) Cone Penetrometer Tests (CPT's) to a maximum depth of 72 feet or refusal. The approximate boring and CPT's locations are shown on the Geotechnical Map (Plate 1). The logs of the borings and CPT's are presented in Appendix B.

- Laboratory testing of representative soil samples obtained from the subsurface exploration. Results of these tests are presented in Appendix C, with the exception of moisture/density determinations, which are provided on the boring logs (included in Appendix B).

- Compilation and analysis of the geotechnical data obtained from the field investigation and laboratory testing.

- Review local and regional seismicity, and provide seismic parameters for the site in accordance with 2010 California Building Code (CBC).

- Development of preliminary foundation design recommendations for the new Pankey Road Bridge.

- Preparation of this report presenting our findings, conclusions, and geotechnical recommendations with respect to the proposed design, site grading and general construction considerations.

1.2 Site Location and Description

The proposed project is generally located east of Interstate 15, east of Pankey Road and north of the Pala Road, as shown on Plate 1. The topography of the site area is variable, with gently rolling terrain in the northern portions of the site with elevations ranging from approximately 263 to 320 feet mean sea level (msl) to relatively flat areas in the southern portion of the site (i.e., parcels 3 and 4 that are south on Pala Road) with an elevation of approximately 266 feet msl.

Latitude: 33.3363 degrees
Longitude: -117.1513 degrees

1.3 Previous Studies

As background, Leighton has previously performed both a Mineral Resource Study and Phase 1 Environmental Site Assessment on the property. In addition, we are concurrently preparing a geotechnical investigation for the Campus Park Sewer Pump Station located north of Pala Road and east of Pankey Road.

Geotechnical assessments previously performed by others for the site and surrounding properties include the adjacent Campus Park site and Meadowbrook development and Palomar College extension project (Appendix A). These offsite studies included test pits, large-diameter borings, small diameter borings, and CPT's that were reviewed as part of our study.

1.4 Proposed Development

Based on our review of the conceptual site development plans (Project Design Consultants, 2008), we understand the proposed development will include construction of multi family residential buildings, commercial buildings, limited impact industrial/office buildings, streets, and associated improvements, such as, underground utilities, landscaping and pavement areas. In addition, a new bridge will also be constructed for the realignment of Pankey Road. We also understand that the proposed grading will include cuts and fills with 2 to 1 (horizontal : vertical) slopes.

2.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

Our subsurface investigation consisted of the excavation of four (4) small-diameter exploratory borings and six (6) CPT's. The purpose of these excavations was to evaluate the younger alluvial soils, depths to groundwater, collect soil samples for laboratory tests, and provide data to perform a liquefaction evaluation south of Pala Road. One boring and one CPT were advanced in the area of the southern bridge abutment for the new Pankey Road Bridge so that preliminary foundation recommendations for the design of the bridge could be developed. Exploration logs are presented in Appendix B. It should be noted that additional subsurface exploration will be required for the northern abutment of the bridge and other areas of the site. In addition, a future bridge widening is also proposed for the existing bridge on Pala Road (Highway 76) and additional future subsurface exploration will be required at that location.

The exploratory excavations were logged by a Geologist from our firm. Representative bulk and relatively undisturbed samples were obtained at frequent intervals for laboratory testing. The approximate locations of the borings and CPT's are shown on the Geotechnical Map (Plate 1). Subsequent to logging and sampling, the borings and CPT's were backfilled.

Laboratory testing was performed on representative samples to evaluate moisture and density, strength, grain size analysis, and geochemistry of the subsurface soils. A discussion of the laboratory tests performed and a summary of the laboratory test results are presented in Appendix C. In-situ moisture and density test results are provided on the boring logs (Appendix B).

3.0 SUMMARY OF GEOTECHNICAL CONDITIONS

3.1 Regional Geology

The site is located within the coastal subprovince of the Peninsular Ranges Geomorphic Province, near the western edge of the southern California batholith. The topography at the edge of the batholith changes from the rugged landforms developed on the batholith to the more subdued landforms, similar to those of the softer sedimentary formations of the coastal plain. Weathering, erosion and regional tectonic uplift created the valleys and ridges of the area.

3.2 Site-Specific Geology

Based on our geologic mapping, subsurface exploration, and review of our referenced geologic maps (Appendix A), the site is mantled by localized areas of undocumented fill, topsoil/colluvium, alluvium and terrace deposits. These units are underlain at depth by what has been mapped as Cretaceous-aged Granitic Rock. The geologic conditions across the site are presented on the Geotechnical Map (Plate 1).

3.2.1 Undocumented Fill Soils (Map Symbol Afu)

Several localized areas of undocumented fill are present on site and will require complete removal during site grading. The primary fill is associated with Pankey Road but other localized areas may also be present.

3.2.2 Topsoil/Colluvium (unmapped)

The rolling hills of the northern portion of the site are mantled by what is believed to be a thin veneer of weathered topsoil and colluvium. Although not investigated as part of this study, the soils are anticipated to consist of loose to medium dense silty sands and clays. These soils will require complete removal in areas of proposed development as they are unsuitable to support structural loads. Previous studies on adjacent parcels have shown these soils to be on the order of 1 to 5 feet in thickness although locally thicker deposits may be present.

3.2.3 Quaternary-aged Alluvium (Map Symbol-Qal)

Quaternary alluvial deposits are mapped across the southern portion of the site and in the low-lying drainage of the northern portions of the property. In the northern portions, the alluvium consists of poorly consolidated (loose) clays, silts, and sands that have accumulated in the lower-most drainage. In the southern portion, the alluvial material encountered predominately consisted of loose to medium dense, fine to coarse sand, silty fine to medium sand, and sandy silts. Alluvial soils south of Pala Road (Highway 76) are on the order of 70 feet in thickness.

3.2.4 Terrace Deposits (Map Symbol Qt)

The gently rolling terrain of the northern portions of the site consist predominately of Terrace Deposits. These sediments are differentiated from the younger alluvial deposits due to a greater degree of consolidation and typically the presence of more silts and clays. However, the upper portions are locally highly weathered and porous. With the exception of the upper weathered profile, these soils are anticipated to be suitable to support structural loads. Removals in the areas mapped as terrace deposits are estimated to be on the order of 2 to 10 feet. Additional testing should be performed to verify removal depths.

3.2.5 Granitic Rocks (Map Symbol Kgr)

Based on the regional geologic map, the entire site is underlain at depth by granitic rock. This unit is not anticipated to be encountered as part of the site development process. The unit consists of granodiorites, tonalities and gabbro.

3.3 Geologic Structure

Based on our subsurface explorations, review of the other geotechnical reports, and regional geologic maps, the alluvium and Terrace Deposits are generally massive with localized cross bedding and generally flat bedding where present.

3.4 Surface Water and Groundwater

Some minor surface water was seasonally observed during our field reconnaissance in the creek below the Pankey Road Bridge. In general, the surface water may drain as sheet flow from higher to lower portions of the site during rainy periods.

Groundwater was encountered within all four of the exploration borings with the shallowest at an approximate depth of 13 feet below the existing ground surface (bgs) in Boring B-3 (i.e., an elevation of 245 feet msl), and the deepest at an approximate depth of 18 feet bgs in Boring B-2, which is roughly an elevation of 248 feet msl. Should deep cuts or deep foundations be utilized, groundwater will likely be encountered. Note that groundwater levels may fluctuate seasonally and rise during rainy periods. We noted that the depth to groundwater levels decrease to the north in alluvial areas and will likely be encountered during remedial grading. Note that currently no test pits or borings have been excavated in the alluvial soils in the northern portion of the site to determine the depth to groundwater. However, test pits excavated several years ago east of the northern portion of the site, encountered groundwater as shallow as 3 feet below existing site grading. Therefore, the depth to ground water in the northern alluvial areas is assumed to range somewhere between 13 feet (i.e., Boring B-3) and 3 feet depth as noted in the older offsite test pits.

3.5 Landslides

Based on review of aerial photographs and site visits, there were no topographic features identified indicating ancient landslides on or adjacent to the site. In addition, there are no mapped landslides on the site. The potential for significant landslides or large-scale slope instability at the site is considered low.

3.6 Flood Hazard

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA Map 06073C0485G, 2007); the site is not located within a flood zone, with the exception of the parcel in the southeastern corner of Pankey Road and Pala Road. Based on review of dam inundation and topographic maps per SANGIS, the site is not located downstream from dam inundation areas.

3.7 Geochemical Considerations/Soil Corrosivity

The National Association of Corrosion Engineers (NACE) defines corrosion as “a deterioration of a substance or its properties because of a reaction with its environment.” From a geotechnical viewpoint, the “environment” is the prevailing foundation soils and the “substances” are reinforced concrete foundations or various types of metallic buried elements such as piles, pipes, etc. that are in contact with or within close vicinity of the soils.

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates and/or pH values of less than 5.5. Typically, a Type II or VI concrete mix-design is provided when the soluble sulfate content of the soil exceeds 0.1 percent by weight or 1,000 ppm. The results of our laboratory tests on representative soils from the site indicated a soluble sulfate content of less than 0.0375 percent (375 ppm) and a pH range of 7.67 to 7.84, which suggests that the concrete should be designed in accordance with the negligible category.

A minimum resistivity value less than approximately 5,000 ohm-cm typically indicates a corrosive environment to buried, uncoated metallic conduits. The test results indicate minimum resistivity ranging from 611 to 4,828 ohm-cm, which indicates a very corrosion potential to buried uncoated metal pipes and conduits. Chloride testing (i.e., Content concentrations ranging from 12 to 73 ppm) indicates a low degree of corrosion potential. The test results are provided in Appendix C.

For appropriate evaluation and mitigation design for other substances with potential influence from corrosive soils, a corrosion engineer may be consulted. These other substances include (but are not necessarily limited to) buried copper tubing, aluminum elements in close vicinity of soils, or stucco finish, if any, that can be potentially influenced.

4.0 FAULTING AND SEISMICITY

4.1 Faulting

Our discussion of faults on the site is prefaced with a discussion of California legislation and state policies concerning the classification and land-use criteria associated with faults. By definition of the California Mining and Geology Board, an active fault is a fault which has had surface displacement within Holocene time (about the last 11,000 years). The State Geologist has defined a potentially active fault as any fault considered to have been active during Quaternary time (last 1,600,000 years) but that has not been proven to be active or inactive. This definition is used in delineating Fault-Rupture Hazard Zones as mandated by the Alquist-Priolo Earthquake Fault Zoning Act of 1972 and as most recently revised in 1997. The intent of this act is to assure that unwise urban development does not occur across the traces of active faults. Based on our review of the Fault-Rupture Hazard Zones, the site is not located within any Fault-Rupture Hazard Zone as created by the Alquist-Priolo Act (Hart, 1997).

The site, like the rest of Southern California, is seismically active as a result of being located near the active margin between the North American and Pacific tectonic plates. The principal source of seismic activity is movement along the northwest-trending regional fault zones such as the San Andreas, San Jacinto and Elsinore Faults Zones, as well as along less active faults such as the Newport-Inglewood (Offshore).

Our review of geologic literature pertaining to the site and general vicinity indicates that there are no known major or active faults on or in the immediate vicinity of the site. Evidence for faulting was not encountered during our field investigation. The nearest active regional fault is the Elsinore Temecula Fault approximately 7.8 miles (12.5 kilometers) to the northeast of the site, the Elsinore Julian Fault approximately 8.6 miles to the southeast, and the Newport-Inglewood Fault Zone located approximately 20.4 miles to the west (Blake, 2000).

4.2 Seismicity

The site can be considered to lie within a seismically active region, as can all of Southern California. Table 1 indicates potential seismic events that could be produced by the maximum moment magnitude earthquake. A maximum moment magnitude earthquake is the maximum expectable earthquake given the known tectonic framework. Site-specific seismic parameters for the site are included in Table 1 are the distances to the causative faults, earthquake magnitudes, and postulated ground accelerations as generated by the deterministic fault modeling software EQFAULT (Blake, 2000).

Table 1
Seismic Parameters for Active Faults (Blake, 2000)

Potential Causative Fault	Distance from Fault to Site (miles)	Maximum Moment Magnitude (Mw)	Peak Ground Motion at Mean Attenuation Relationship (g)
Elsinore-Temecula	7.8	6.8	0.31
Elsinore-Julian	8.7	7.1	0.34
Newport-Inglewood	20.4	7.1	0.16
Rose Canyon	21.6	7.2	0.16

As indicated in Table 1, the Elsinore-Julian Fault is the ‘active’ fault considered having the most significant effect at the site from a design standpoint.

Utilizing 2010 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.

The following table presents the spectral acceleration parameters for the project determined in accordance with the 2010 CBC (CBSC, 2010a) and the USGS Ground Motion Parameter Calculator (Version 5.1.0).

Table 2 Mapped Spectral Acceleration Parameters	
Site Class	D
Site Coefficients	F_a = 1.0 F_v = 1.5
Mapped MCE Spectral Accelerations	S_S = 1.491g S_1 = 0.574g
Site Modified MCE Spectral Accelerations	S_{MS} = 1.491g S_{M1} = 0.861g
Design Spectral Accelerations	S_{DS} = 0.994g S_{D1} = 0.574g

The peak horizontal ground acceleration associated with the Maximum Considered Earthquake Ground Motion is 0.6g. The peak horizontal ground acceleration associated with the Design Earthquake Ground Motion is 0.4g.

Secondary effects that can be associated with severe ground shaking following a relatively large earthquake include shallow ground rupture, soil liquefaction, and dynamic settlement. These secondary effects of seismic shaking are discussed in the following sections.

4.2.1 Shallow Ground Rupture

Ground rupture because of active faulting is not likely to occur on site due to the absence of known active faults. Cracking due to shaking from distant seismic events is not considered a significant hazard, although it is a possibility at any site.

4.2.2 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.

Design ground motion considered in our liquefaction triggering analyses was the design earthquake with moment magnitude 7.6 and peak ground acceleration (pga) of 0.4g. In the determination of the design magnitude, the USGS Earthquake Hazard Program, GMT, was used, which calculates the most probable modal magnitude based on a probabilistic Seismic Hazard Disaggregation of maximum magnitude earthquake at the site (see Appendix D).

The results of the liquefaction analyses indicate that potentially discontinuous layers of the alluvial materials in the southern portion of the site (i.e., improvements south of Pala Road and the new Pankey Road alignment and bridge site), as encountered in the borings and CPTs, are considered susceptible to liquefaction at the design earthquake ground motion. Summary plots of the analyses using the software LiquefyPro (Civil Tech, 2002) are provided in Appendix D.

Dynamic and post-liquefaction settlement was evaluated utilizing the Modified Robertson and Ishihara and Yoshimine methods. In addition, limited remedial grading (i.e., removal of up to 14 feet of existing alluvium) and the placement of 2 to 6 feet of additional fill above the existing grades were used in the analysis. Results of that analysis indicate total liquefaction-induced settlement on the order of 4 to 6 ½ inches can be anticipated as a result of the design earthquake event. Therefore, the near surface improvements, such as underground utlities and shallow foundations, may be subjected to differential settlements on the order of 2 to 6 inches. Plots of the liquefaction analysis are provided in Appendix D.

Considering the potential for liquefaction and the dynamic settlements at the site, remedial grading, ground improvement and/or deep foundations will need to be considered for the site improvements, buildings, and new bridge. Note that if only deep pile foundations are utilized, appropriate downdrag forces and flow failure loading from embankment instability need to be considered in the design. In general, the Ground improvement methods would consist of vibro-compaction of existing loose sand layers, and/or vibro-replacement/densification with stone columns techniques to mitigate the potential for liquefaction induced ground failures.

4.2.3 Lateral Spread

Lateral spreading can occur when saturated alluvial materials overlain by sloping ground liquefy and cause a reduction of lateral resisting force. Lateral spreading is manifested by lateral displacement and slumping of the embankment. Empirical relationships have been derived (Youd et. al., 1999) to estimate the magnitude of lateral spread due to the design seismic event. 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.

In the southern portion of the site, southwest of Lot 40 and 50 and CPT1 and CPT2, there is an existing natural slope approximately 10 feet in height. Depending on the final design and building locations, some mitigation may be required for lateral spreading.

4.2.4 Tsunamis and Seiches

Based on the distance between the site and large, open bodies of water, barriers between the site and the open ocean, the possibility of seiches and/or tsunamis is considered to be nil.

5.0 CONCLUSIONS

Based on the results of our geotechnical investigation of the site, it is our professional opinion that the proposed development of the site is feasible from a geotechnical standpoint, provided the following preliminary conclusions and recommendations are incorporated into the design, grading, and construction of the project. The following is a summary of the geotechnical factors that may affect development of the site.

- Based on our subsurface exploration and laboratory testing, the existing alluvial soils are considered potentially compressible. Removal and recompaction of alluvial soils above the groundwater table is recommended. Saturated alluvium that is left-in-place may consolidate/settle under proposed fill loads. Some of the settlement will occur during grading but there will be some ongoing settlement after grading is complete. We recommend that a series of settlement monuments be installed and a monitoring period established prior to construction of proposed improvements.

- The southern portion of the site (i.e., improvements south of Pala Road and the new Pankey Road alignment and bridge site) consists of sandy alluvial soils with shallow groundwater. The unsaturated portions of these alluvial soils are recommended for removal and recompaction during site grading. The remaining alluvial soils below the ground water are potentially liquefiable and will require mitigation, such as the use of vibro-compaction of existing loose sand layers and/or vibro-replacement/densification with stone columns mitigation techniques. The design of the mitigation will be dependent on final grading and building plans, results of additional geotechnical exploration and testing, and an evaluation of data by ground improvement contractors specializing in mitigation methods.

- The location of the new Pankey Road bridge is underlain by potentially compressible alluvial soils and shallow groundwater. In summary, additional geotechnical exploration and testing of the proposed foundation locations is recommended for the selection of an appropriate type of foundation. Currently, it is anticipated that the deep foundations (e.g., driven piles or CIDH piles) will be used to support the bridge abutments and pier supports. In addition, lateral loading of the abutments resulting from seismic instability may need to be mitigated with ground improvements.

- The upper portion of areas mapped as Terrace deposits are weathered, porous, and potentially compressible. These soils along with topsoil and colluvium are recommended for removal and recompaction during site grading. Based on our experience, review of logs from adjacent sites, and site observation, we anticipate removal depths to be on the order of 2 to 10 feet. Additional investigation is recommended to further evaluate site removals.

- Based on the subsurface explorations, groundwater was encountered in our exploratory borings at an approximately elevation of 248 feet msl (i.e., approximately 13 to 14 feet bgs) and may be shallower in the northern portion of the site. Dewatering may be required for construction of any deep excavations on this project. Design of the temporary dewatering system should be provided by a specialty contractor, which is reviewed and stamped by a California Registered Engineer.

- Based on laboratory testing and visual classification, the near surface fill soils on the site generally possess a low expansion potential. Localized deposits of highly expansive soils may also be present. If highly expansive soils are placed at grade additional recommendations will be required.

- Laboratory test results indicate the soils present on the site have a negligible potential for sulfate attack on concrete. However, onsite soils are considered to have a high potential for corrosion on buried uncoated metal pipes and conduits from minimum resistivity testing.

- The existing onsite soils appear to be suitable material for use as fill provided they are relatively free of organic material, debris, and rock fragments larger than 8 inches in maximum dimension. Oversize material if encountered should be placed in nonstructural areas or disposed of offsite.

- Active or potentially active faults are not known to exist on or in the immediate vicinity of the site.

- After completion of site grading and appropriate settlement monitoring period, we anticipate that the proposed buildings can be designed with conventional foundations. Some preliminary foundation design considerations are included herein but will be dependent on the building type, location, and future geotechnical studies.

6.0 RECOMMENDATIONS

6.1 Earthwork

We anticipate that earthwork at the site will consist of site preparation, remedial grading and placement of compacted fill. 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 E. In case of conflict, the following recommendations shall supersede those in Appendix E.

6.1.1 Site Preparation

Prior to grading of areas to receive structural fill or engineered structures and improvements, the areas should be cleared of surface vegetation, any existing debris, and removal of potentially compressible material. Vegetation and debris should be removed and properly disposed of offsite. Holes resulting from the removal of buried obstructions, which extend below finished site grades, should be replaced with suitable compacted fill material. Areas to receive fill and/or other surface improvements should be scarified to a minimum depth 8 inches, brought to above-optimum moisture condition, and recompacted to at least 90 percent relative compaction (based on American Standard of Testing and Materials [ASTM] Test Method D1557).

Site removals are anticipated to include alluvial soils to within 2 feet of the static groundwater table, topsoil, colluvium, undocumented fill, and weathered upper profile of the Terrace Deposits. In the southern portion of the site, the remaining alluvial soils below the groundwater will require mitigation (i.e., ground improvement for liquefaction).

6.1.2 Excavations and Oversize Material

Excavations of the onsite alluvium and sedimentary materials may generally be accomplished with conventional heavy-duty earthwork equipment. Any oversized rock that is encountered should be placed as fill in accordance with the recommendations presented Appendix E, or hauled off site for disposal.

6.1.3 Fill Placement

The onsite soils are generally suitable for reuse as compacted fill, provided they are free of organic materials and debris. Areas to receive structural fill and/or other surface improvements should be scarified to a minimum depth of 8 inches; brought to at least 2 percent above optimum moisture content; and recompacted to at least 90 percent relative compaction (based on ASTM Test Method D1557). The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches in thickness. Placement and compaction of fill should be performed in general accordance with the current County of San Diego grading ordinances under the observation and testing of the geotechnical consultant, sound construction practices, and the General Earthwork and Grading Specifications for Rough Grading presented in Appendix E.

Proposed fills placed on slopes steeper than 5 to 1 (horizontal to vertical) and repairs of the existing fill slopes should be keyed and benched into dense formational or competent fill soils (see Appendix E for benching details). Fills placed within 5 feet of finish pad grades should consist of granular soils of very low to medium expansion potential and contain no materials over 8 inches in maximum dimension. Oversize material, if encountered, may be incorporated into structural fills if placed in accordance with the recommendation of Appendix E.

Import soils, if necessary, should consist of granular soils of very low to low expansion potential (expansion index 50) and contain no materials over 8 inches in maximum dimension.

6.1.4 Expansive Soils and Selective Grading

It is not anticipated that extensive amounts of highly expansive soils will be encountered during site grading. We anticipate that the grading of the onsite soil will generate material that has a low to medium potential for expansion. However, expansion testing should be performed on the finish grade soils to verify their expansion potential. If highly expansive soils are present within 5 feet of finish grade, special foundation and slab considerations will be required.

6.2 Temporary Excavations

Sloped excavations may be utilized when adequate space allows. Based on findings, we provide the following recommendations for sloped excavations in fill soils or competent formational materials without seepage conditions.

Table 3
Temporary Excavation Recommendations

Excavation Depth Below Adjacent Surface (feet)	Maximum Slope Ratio In Alluvium and Fill Soils	Maximum Slope Ratio In Competent Formational Material
0 to 5	¾:1 (H : V)	Vertical
5 to 20	1:1	1:1

Excavations greater than 20 feet in height will require an alternative sloping plan or shoring plan prepared by a California registered civil engineer. 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. All excavations should comply with OSHA requirements. Care should be taken during excavation adjacent to the existing structures so that undermining does not occur. The contractor's "competent person" should review all excavations on a daily basis for signs of instability.

6.3 Surface Drainage and Erosion

Surface drainage should be controlled at all times. The proposed structure should have an appropriate drainage system to collect roof runoff. Positive surface drainage should be provided to direct surface water away from the structure toward the street or suitable drainage facilities. Planters should be designed with provisions for drainage to the storm drain. Ponding of water should be avoided adjacent to the structure.

Regarding Low Impact Development (LID) measures, all proposed infiltration basins, and other onsite retention systems are located within existing fill areas.

Additional testing of infiltration characteristics of existing and proposed fill soils will be required in the future per County of San Diego Guidelines.

6.4 Foundation and Slab Considerations

Building foundations and slabs should be designed in accordance with structural considerations and the following recommendations. These recommendations assume that remedial grading, ground improvements (i.e., the southern portion of the site), the soils encountered within 5 feet of pad grade have a low to medium potential for expansion and a differential fill thickness of less than 20 feet. Note that additional expansion testing should be performed as part of the fine grading operations. If highly expansive soils are encountered and selective grading cannot be accomplished, additional foundation design may be necessary. Final foundation recommendations will be provided at the completion of site grading and may be revised when building types are finalized.

6.4.1 Preliminary Foundation and Slab Design

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 2,500 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.

We recommend a minimum horizontal setback distance from the face of slopes for all structural footings and settlement-sensitive structures. This distance is measured from the outside edge of the footing, horizontally to the slope face (or to the face of a retaining wall) and should be a minimum of H/2, where H is the slope height (in feet). The setback should not be less than 10 feet and need not be greater than 20 feet. Please note that the soils within the structural setback area, other than those addressed within this report, 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.

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). If applicable, slabs should also be designed for the anticipated traffic loading using a modulus of subgrade reaction of 130 pounds per cubic inch.

In accordance with the current guidelines of the 2010 CALGreen Code, Section 4.505.2, post-tensioned and conventional slabs should be underlain by a vapor barrier which is in turn underlain by 4 inches of 1/2 inch gravel. The slab subgrade soils should be presoaked prior to the placement of gravel. ACI 302.2R-06 guidance recommends use of a vapor barrier with a perm rating of 0.01 or less where moisture-sensitive floor coverings are provided. The vapor barrier should possess adequate puncture resistance such that these properties are preserved when subjected to construction traffic. All waterproofing measures should be designed by the project architect.

Placement of concrete in direct contact with the vapor barrier requires additional design and construction considerations on the part of the structural engineer, architect and contractor. 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. Only an experienced concrete contractor familiar with proper construction techniques needed for constructing slabs directly on the vapor retarder/barrier should perform the work.

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.

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 has 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 4
Presoaking Recommendations Based on Finish Grade Soil Expansion Potential

Expansion Potential	Presoaking Recommendations
Very Low to 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	140 percent of the optimum moisture content to a minimum depth of 24 inches below slab subgrade

6.4.2 Settlement

Fill depths between 5 and 30 feet are anticipated beneath the proposed building footings following final grading. Based on this configuration, the maximum total settlement is estimated at approximately 1 inch with differential settlement anticipated to be approximately ¾ to 1 inch over a horizontal distance of 100 feet.

6.4.3 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 table below and criteria of the 2010 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 5 presents the recommended post-tension foundation category for residential buildings on subject site.

Table 5

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 51 to 90)	Category III High Expansion Potential (EI 91 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:	0.65 inches	1.09 inches	1.65 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 slabs 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 4 above prior to placement of the moisture barrier.

The slab should be underlain by a moisture barrier as discussed in Section 6.41 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.

6.5 Retaining Wall Design and Lateral Earth Pressure

Several relatively small retaining walls are proposed at the southern site entry. For design purposes, the following lateral earth pressure values for level or sloping backfill are recommended for retaining walls backfilled with onsite soils of low to medium expansion potential (expansion potential less than 70 per ASTM Test Method D4829).

Table 6
Static Equivalent Fluid Weight (pcf)

Conditions	Level	2:1 Slope
Active	35	55
At-Rest	55	75
Passive	350 (Maximum of 3 ksf)	150 (Sloping Down)

Unrestrained (yielding) cantilever walls up to 10 feet in height should be designed for an active equivalent pressure value provided in table above. If conditions other than those covered herein are anticipated, the equivalent fluid pressure values should be provided on an individual case basis by the geotechnical engineer.

The wall pressures assume walls are backfilled with free draining materials and water is not allowed to accumulate behind walls. A typical wall drainage design is provided in Appendix E. Importing or selective grading may be necessary to obtain retaining wall backfill material.

Wall backfill should be brought to at least 2 percent above the optimum moisture content and compacted by mechanical methods to at least 90 percent relative compaction (based on ASTM D1557). Wall footings should be designed in accordance with the foundation design recommendations and reinforced in accordance with structural considerations. The bearing pressure for retaining walls should be limited to 2,000 psf for footing founded in compacted fill. Footing embedment depth should be at least 24 inches below the lowest adjacent grade. For all retaining walls, we recommend a minimum horizontal distance from the outside base of the footing to daylight of 10 feet.

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 0.33 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.

6.6 Preliminary Pavement Design

The appropriate pavement section will depend on the type of subgrade soil, shear strength, traffic load, and planned pavement life. Since an evaluation of the actual subgrade soils cannot be made at this time, we have used an assumed R-value of 20 and Traffic Indices (TI) of 4.5, 5 and 6 for various pavements, such as, parking lots, driveways and streets. The range of onsite pavement sections presented on Table 6 is to be used for preliminary planning purposes only. Final pavement designs should be completed after R-value tests have been performed on actual subgrade materials.

Table 7 Preliminary Pavement Section	
Traffic Index	Pavement Designs
4.5	3 inches AC over 6 inches Class 2 Aggregate Base
5	3 inches AC over 8 inches Class 2 Aggregate Base
6	4 inches AC over 9 inches Class 2 Aggregate Base

Asphalt Concrete (AC) and Class 2 aggregate base should conform to and be placed in accordance with the latest revision of California Department of Transportation Standard Specifications. Prior to placing the pavement section, the subgrade soils should have a relative compaction of at least 95 percent to a minimum depth of 12 inches (based on ASTM Test Method D1557). Aggregate Base should be compacted to a minimum of 95 percent relative compaction (based on ASTM Test Method D1557) prior to placement of the AC.

For areas subject to unusually heavy truck loading (i.e., trash trucks, delivery trucks, etc.), we recommend minimum 7 inch full depth of Portland Cement Concrete (PCC) section with appropriate steel reinforcement and crack-control joints as designed by the project structural engineer.

All concrete pavement sections, including concrete curbs and gutters, should be underlain by at least 6 inches of aggregate base (AB) compacted to 95 percent relative compaction.

6.7 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 3 to 6 percent above optimum moisture content depending on the soil type and compacted to at least 90 percent relative compaction based on ASTM Test Method D1557 prior to the concrete placement.

6.8 Slope Maintenance Guidelines

It is the responsibility of the owner or owner's association to maintain the slopes, including adequate planting, proper irrigation and maintenance, and repair of faulty irrigation systems. To reduce the potential for erosion and slumping of graded slopes, all slopes should be planted with ground cover, shrubs, and plants that develop dense, deep root structures and require minimal irrigation. Slope planting should be carried out as soon as practical upon completion of grading. Surface-water runoff and standing water at the top-of-slopes should be avoided. Oversteepening of slopes should also be avoided during construction activities and landscaping. Maintenance of proper drainage, undertaking of improvements in accordance with sound engineering practices, and proper maintenance of vegetation, including regular slope irrigation, should be performed. Slope irrigation sprinklers should be adjusted to provide maximum uniform coverage with minimal of water usage and overlap. Overwatering and consequent runoff and ground saturation should be avoided. If automatic sprinklers systems are installed, their use must be adjusted to account for rainfall conditions.

Trenches excavated on a slope face for any purpose should be properly backfilled and compacted in order to obtain a minimum of 90 percent relative compaction, in accordance with ASTM Test Method D1557. Observation/testing by the geotechnical consultant during trench backfill are recommended. A rodent-control program should be established and maintained. Prior to planting, recently graded slopes should be temporarily protected against erosion resulting from rainfall, by the implementing slope protection measures such as polymer covering, jute mesh, etc.

6.9 Landscaping and Post-Construction

Landscaping and post-construction practices carried out by the owner and their representatives exert significant influences on the integrity of structures founded on expansive soils. Improper landscaping and post-construction practices, which are beyond the control of the geotechnical engineer, are frequently the primary cause of distress to these structures. Recommendations for proper landscaping and post-construction practices are provided in the following paragraphs within this section. Adhering to these recommendations will help in minimizing distress due to expansive soils, and help ensure that such effects are limited to cosmetic damages, without compromising the overall integrity of structures.

Initial landscaping should be done on all sides adjacent to the foundation of a structure or associated improvements, and adequate measures should be taken to ensure drainage of water away from the foundation or improvement. If larger, shade providing trees are desired, such trees should be planted away from structures or improvements (at a minimum distance equal to half the mature height of the tree) in order to prevent penetration of the tree roots beneath the foundation of the structure or improvement.

Locating planters adjacent to buildings or structures should be avoided as much as possible. If planters are utilized in these locations, they should be properly designed so as to prevent fluctuations in the moisture content of the subgrade soils. Planting areas at grade should be provided with appropriate positive drainage. Wherever possible, exposed soil areas should be above paved grades. Planters should not be depressed below adjacent paved grades unless provisions for drainage, such as catch basins and drains, are made. Adequate drainage gradients, devices, and curbing should be provided to prevent runoff from adjacent pavement or walks into planting areas.

Watering should be done in a uniform, systematic manner as equally as possible on all sides of the foundation, to keep the soil moist. Irrigation methods should promote uniformity of moisture in planters and beneath adjacent concrete flatwork. Overwatering and underwatering of landscape areas must be avoided. Areas of soil that do not have ground cover may require more moisture, as they are more susceptible to evaporation. Ponding or trapping of water in localized areas adjacent to the foundations can cause differential moisture levels in subsurface soils and, therefore, should not be allowed. Trees located within a distance of 20 feet of foundations would require more water in periods of extreme drought, and in some cases, a root injection system may be required to maintain moisture equilibrium. During extreme hot and dry periods, close observations should be carried out around foundations to ensure that adequate watering is being undertaken to prevent soil from separating or pulling back from the foundation.

6.10 Construction Observation and Testing and Plan Review

The geotechnical consultant should perform construction observation and testing during the fine, and post grading operations, future excavations and foundation or retaining wall construction at the site. Additionally, footing excavations should be observed and moisture determination tests of the slab subgrade soils should be performed by the geotechnical consultant prior to the pouring of concrete. Foundation design plans should also be reviewed by the geotechnical consultant prior to excavations.

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 and PLATES

Project: 042410-003	Eng/Geol: WDO/MRS
Scale: 1 " = 2,000 '	Date: September, 2012

Base Map: ESRI Resource Center, 2010
Thematic Info: Leighton
Author: pbillock (mmurphy)

SITE LOCATION MAP
Pappas Investments
Campus Park West
San Diego County, California

Figure 1

Leighton

Map Saved as P:\drafting\042410\003\GIS\of_2012-08-24\Figure1.mxd on 8/30/2012 4:19:24 PM

REFERENCE: PDC ENGINEERING PRELIMINARY GRADING PLANS, CAMPUS PARK, 2008.

PLATE 1	GEOTECHNICAL MAP PAPPAS INVESTMENTS CAMPUS PARK WEST SAN DIEGO COUNTY, CALIFORNIA	
Leighton	Proj: 042410-003	Eng/Geol: WDO/MRS
	Scale: 1"=200'	Date: 09/2012
Drafted By: MAM	Checked By:	

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

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APPENDIX A (Continued)

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Shepardson Engineering Associates Inc., 2006, Update Geotechnical Assessment. Proposed Passerelle Subdivision, Pala Rd, TM 5338, San Diego County, California, S.E.A. 201171-04, dated October 4, 2006.

Shepardson Engineering Associates Inc., 2006, Update Geotechnical Assessment. Proposed Campus Park, Pala Rd, TM 5338, San Diego County, California, S.E.A. 201171-04, dated April 11, 2007.

APPENDIX B

BORING and CPT LOGS

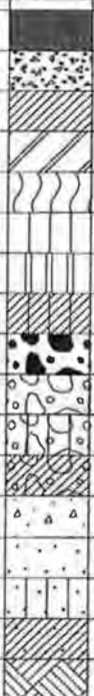
GEOTECHNICAL BORING LOG KEY

Project No.		**Date Drilled**	
Project	KEY TO BORING LOG GRAPHICS	**Logged By**	
Drilling Co.		**Hole Diameter**	
Drilling Method		**Ground Elevation**	
Location		**Sampled By**	

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

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.



Depth Feet	Soil Class. (U.S.C.S.)	Soil Description
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

Depth Feet	Sample No.	Description
▽		Ground water encountered at time of drilling
	B-1	Bulk Sample
20	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.)
	S-1	Standard Penetration Test SPT (Sampler (2" O.D., 1.4" I.D.)
25	PUSH	Sampler Penetrates without Hammer Blow
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
- TR THERMAL RESISTIVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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

GEOTECHNICALBORING LOG B-1a

Project No.	042343-002	**Date Drilled**	8-10-12
Project	Campus Park - Sewer Lift Station	**Logged By**	BSS
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	264'
Location	See Boring Location Map	**Sampled By**	BSS

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				SM	**ALLUVIUM (QAL):** SILTY SAND, loose, grayish brown, moist, fine sand, micaceous	
260	5			R-1	2 2 3	77	14		loose, dark grayish brown, moist, very fine to fine sand, micaceous	
255	10			R-2	2 2 5	89	26		loose, dark gray, moist to wet, fine sand, very micaceous, more silt	
250	15			R-3	2 3 5	86	17	SC-SM	SILTY, CLAYEY SAND, loose, grayish brown, moist to wet, very fine sand, micaceous, some oxidation, few dark gray clay seams	
245	20			S-4	7 9 10			SW-SM	Well-graded SAND, medium dense, grayish brown, moist to wet, fine to medium sand, some silt (6% fines)	SA
240	25			S-5	2 3 5			SM	SILTY SAND, medium dense, dark gray, wet, fine sand, very micaceous, few clay (21% fines)	SA
235	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. ***

GEOTECHNICALBORING LOG B-1a

Project No.	042343-002	**Date Drilled**	8-10-12
Project	Campus Park - Sewer Lift Station	**Logged By**	BSS
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	264'
Location	See Boring Location Map	**Sampled By**	BSS

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
	30			S-6	4 3 3				loose, dark gray, wet, very fine sand, micaceous	
								SW	Well-graded SAND, loose, gray, wet, fine to coarse sand with fine gravel	
230	35			S-7	2 2 4			SM	SILTY SAND, medium dense, dark gray to brown, wet, very fine to fine sand, some mica (41% fines)	SA
225	40			S-8	1 2 3			CL	SANDY Lean CLAY, medium stiff, brown, moist to wet, fine sand	
220	45			S-9	3 4 7			SM	SILTY SAND, medium dense, dark olive brown, wet, some mica (36% fines)	SA
215	50			S-10	1 2 3				loose, dark brown, wet, fine sand, some coarse sand, more clay	
210	55								Drilled to 50' Sampled to 51.5' Groundwater at 16' Backfilled with with grout and bentonite Hole caved to 15.7'	
205	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

GEOTECHNICALBORING LOG B-1

Project No.	042410-003	**Date Drilled**	8-9-12
Project	Pappas Campus Park West	**Logged By**	BSS
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Automatic - 30" Drop	**Ground Elevation**	266'
Location	See Boring Location Map	**Sampled By**	BSS

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.

Graphic Log: N S

Elevation Feet	Depth Feet	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0						SM	**ALLUVIUM (QAL):** SILTY SAND, loose, grayish brown, dry, very fine sand	
265									
	5		R-1	2 4 4	80	12		loose, dark grayish brown, dry to moist, very fine sand, pirous, micaeous, some roots	
260									
	10		R-2	4 6 7	94	6	SP-SM	Poorly graded SAND with SILT, medium dense, grayish brown, moist, fine sand, some oxidation	
255									
	15		R-3	7 12 15	102	2	SW	Well-graded SAND, medium dense, light gray, moist, fine to coarse sand with fine gravel, subrounded	
250									
	20		R-4	11 12 15	123	13		medium dense, light gray, moist to wet, fine to coarse sand, micaeous, trace silt	
245									
	25		S-5	4 6 11				medium dense, dark gray, wet, fine to coarse sand with fine gravel, micaous	
240									
	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. ***

GEOTECHNICALBORING LOG B-1

Project No.	042410-003	**Date Drilled**	8-9-12
Project	Pappas Campus Park West	**Logged By**	BSS
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Automatic - 30" Drop	**Ground Elevation**	266'
Location	See Boring Location Map	**Sampled By**	BSS

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
235	30			S-6	4 4 5			SW-SM	Well-graded SAND with SILT, medium dense, dark gray, wet, fine to coarse sand with fine gravel, some mica	
230	35			S-7	3 4 11				medium dense, gray, wet, fine to coarse sand, some gravel	
								SM	SILTY SAND, medium dense, dark grayish brown, moist to wet, very fine sand, micaceous	
225	40			S-8	3 5 6				medium dense, dark grayish brown, wet, very fine to fine sand, some mica	
220	45			S-9	8 9 15				medium dense, dark grayish brown, wet, fine to medium sand, some mica, rare gravel	
215	50			S-10	2 3 4				loose, grayish brown, wet, fine to medium sand, some mica	
									Drilled to 50' Sampled to 51.5' Groundwater at 17.5' Backfilled with grout and bentonite (8/9/12)	
210	55									
	60									

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

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

GEOTECHNICALBORING LOG B-2

Project No.	042410-003	**Date Drilled**	8-9-12
Project	Pappas Campus Park West	**Logged By**	BSS
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Automatic - 30" Drop	**Ground Elevation**	266'
Location	See Boring Location Map	**Sampled By**	BSS

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								Surface covered with base	
265				B-1				SM	**ALLUVIUM (QAL):** SILTY SAND, loose, dark brown, dry to moist, very fine sand	
260	5			R-1	5 4 5	96	9		loose, brown, moist, very fine sand, porous, some mica	
255	10			R-2	5 9 9	99	3	SP	Poorly graded SAND, medium dense, light brownish gray, moist, fine sand, some silt, mica, oxidation	
250	15			R-3	7 8 14	100	9	SW-SM	Well-graded SAND with SILT, medium dense, gray, moist to wet, fine to coarse sand, some mica	
245	20			R-4	6 8 8	105	20		medium dense, dark gray, wet, fine to coarse sand with fine gravel	
240	25			S-5	4 7 14			SW	Well-graded SAND with GRAVEL, medium dense, dark gray, wet, fine to coarse sand with fine gravel	
	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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GEOTECHNICALBORING LOG B-2

Project No.	042410-003	**Date Drilled**	8-9-12
Project	Pappas Campus Park West	**Logged By**	BSS
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Automatic - 30" Drop	**Ground Elevation**	266'
Location	See Boring Location Map	**Sampled By**	BSS

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
235	30			S-6	3 11 14				Well-graded SAND with GRAVEL, medium dense, dark gray, wet, fine to coarse sand with fine gravel dense, dark gray, wet, fine to coarse sand, micaceous, some silt	
230	35			S-7	4 9 14			SW-SM	Well-graded SAND with SILT, medium dense, grayish brown, wet, fine to coarse sand, some mica	
225	40			S-8	3 6 11			SP-SM	Poorly graded SAND with SILT, medium stiff, dark gray, wet, fine sand, some mica, few gravel	
220	45			S-9	3 8 11			SM	SILTY SAND, medium dense, dark gray, wet, fine to medium sand, some mica	
215	50								hole caved, couldn't sample	
									Drilled to 50' Sampled to 51.5' Groundwater at 18.5' Backfilled with grout and bentonite (8/9/12)	
210	55									
	60									

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

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

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

GEOTECHNICALBORING LOG B-3

Project No.	042410-003	Date Drilled	8-10-12
Project	Pappas Campus Park West	Logged By	BSS
Drilling Co.	Baja Exploration	Hole Diameter	8"
Drilling Method	Hollow Stem Auger - 140lb - Automatic - 30" Drop	Ground Elevation	258'
Location	See Boring Location Map	Sampled By	BSS

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			B-1				SM	**ALLUVIUM (QAL):** SILTY SAND, loose, brown, dry to moist, very fine sand	
255									medium dense, moist	
	5			R-1	11 23 31				dense, dark reddish brown, moist, very fine sand, few mica	
250										
	10			R-2	7 10 16			SC	CLAYEY SAND, medium dense, dark reddish brown, moist, very fine sand, rare mica	
245										
	15			R-3	3 9 16				medium dense, dark reddish brown, moist to wet, very fine sand, few mica	
240										
	20			R-4	11 18 27			ML	SILT with SAND, stiff, grayish brown, moist to wet, very fine sand, few clay seams	
235										
	25			S-5	2 3 6				medium stiff, olive brown, moist to wet, very fine sand, some mica, trac clay	
230										
	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

GEOTECHNICALBORING LOG B-3

Project No.	042410-003	**Date Drilled**	8-10-12
Project	Pappas Campus Park West	**Logged By**	BSS
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Automatic - 30" Drop	**Ground Elevation**	258'
Location	See Boring Location Map	**Sampled By**	BSS

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
	30			S-6	2 4 6				SILT with SAND, stiff, grayish brown, moist to wet, very fine sand, few clay seams medium stiff, light olive brown, moist to wet, very fine to fine sand, micaceous	
225										
	35			S-7	4 5 8			CL-ML	SILTY CLAY, stiff, grayish brown, moist to wet, some mica, white mottled	
220										
	40			S-8	3 5 11				stiff, light reddish brown, moist to wet, very fine sand	
215									??Formation??	
	45			S-9	4 7 14			CL	SANDY Lean CLAY, stiff, dark reddish brown, moist to wet, fine sand	
210										
	50			S-10	9 12 19			SC	CLAYEY SAND, dense, dark reddish brown, wet, fine to coarse sand	
									Drilled to 50' Sampled to 51.5' Groundwater at 13.5' Backfilled with grout and bentonite (8/10/12)	
205										
	55									
200										
	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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GEOTECHNICALBORING LOG B-4

Project No.	042410-003	**Date Drilled**	8-10-12
Project	Pappas Campus Park West	**Logged By**	BSS
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Automatic - 30" Drop	**Ground Elevation**	262'
Location	See Boring Location Map	**Sampled By**	BSS

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0			B-1				SM SM	**ALLUVIUM (QAL):** SILTY SAND, loose, light brown, dry to moist, very fine sand, trace clay	
260										
	5			R-1	5 5 8	93	19		medium dense, grayish brown, moist, very fine sand, micaceous, some rootlets	
255										
	10			R-2	1 3 4	77	40	ML	SANDY SILT, soft, dark gray, moist to wet, very fine sand, micaceous, some rootlets	
250										
	15			R-3	2 3 4	96	28	CL-ML	SILTY CLAY, medium stiff, dark gray, moist to wet, very fine sand, some rootlets	
245										
	20			S-4	1 1 2				medium stiff, gray, wet, some mica	
240										
	25			S-5	2 2 3			SC	CLAYEY SAND, medium dense, grayish brown, moist to wet, fine sand, micaceous, few coarse grains	
235										
	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. ***

GEOTECHNICALBORING LOG B-4

Project No.	042410-003	**Date Drilled**	8-10-12
Project	Pappas Campus Park West	**Logged By**	BSS
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Automatic - 30" Drop	**Ground Elevation**	262'
Location	See Boring Location Map	**Sampled By**	BSS

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-6	2 2 4				CLAYEY SAND, medium dense, grayish brown, wet, fine to medium sand, some dark gray clay seams	
230										
	35			S-7	2 1 3			SM	SILTY SAND, medium dense, reddish brown, wet, fine to medium sand, some clay	
225										
									rods got jammed in the hole so couldn't sample or drill anymore.	
	40								Drilled to 40' Sampled to 35' Groundwater at 14.3' Backfilled with grout and bentonite (8/10/12)	
220										
	45									
215										
	50									
210										
	55									
205										
	60									

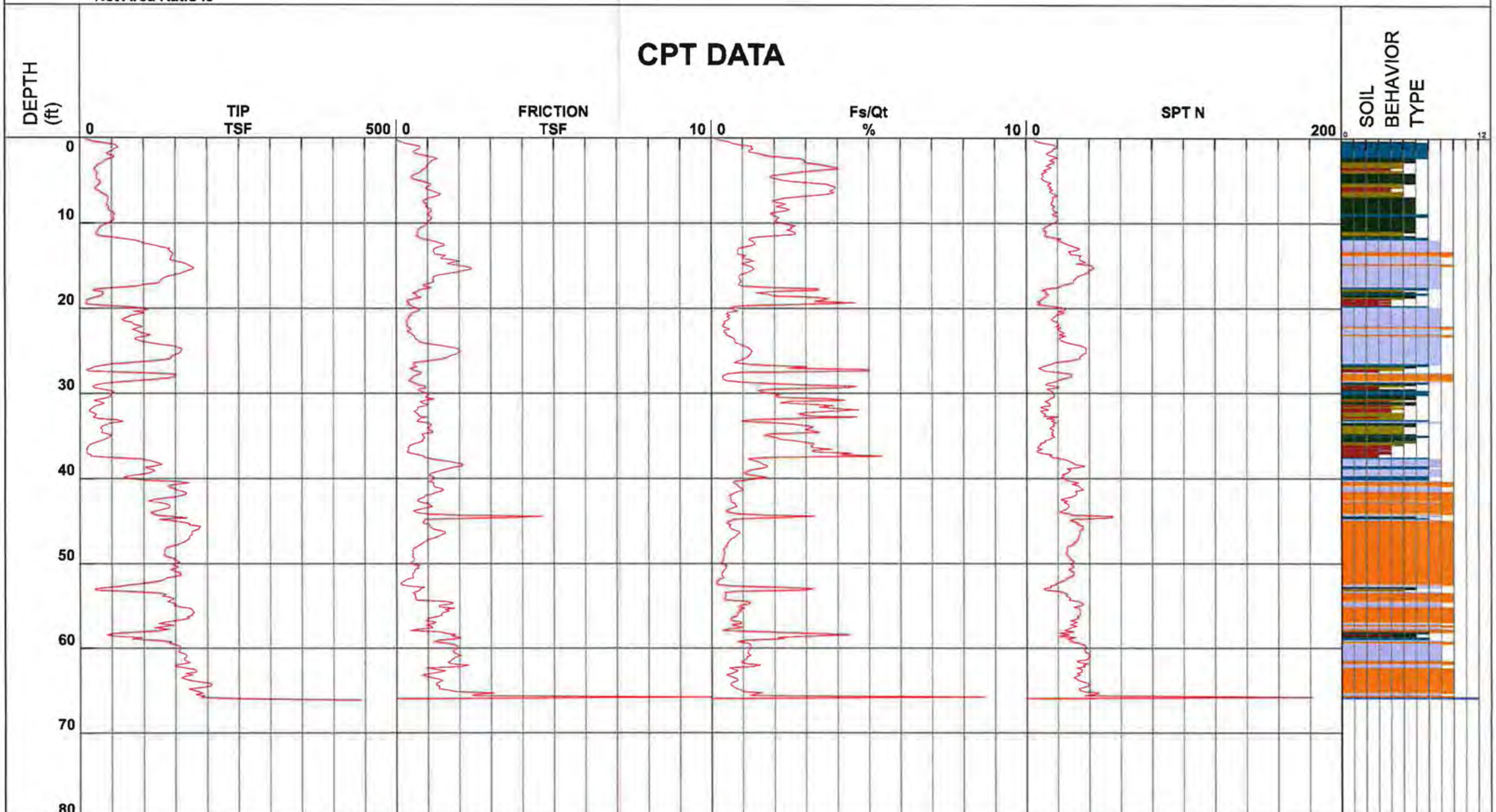
SAMPLE TYPES:
B BULK SAMPLE
C CORE SAMPLE
G GRAB SAMPLE
R RING SAMPLE
S SPLIT SPOON SAMPLE
T TUBE SAMPLE

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

Leighton & Associates

Project	Campus Park	Operator	RA-JC	Filename	SDF(003).cpt
Job Number	04241-003	Cone Number	DDG1185	GPS	
Hole Number	CPT-01	Date and Time	8/9/2012 11:26:38 AM	Maximum Depth	66.11 ft
EST GW Depth During Test		18.00 ft			

Net Area Ratio .8



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

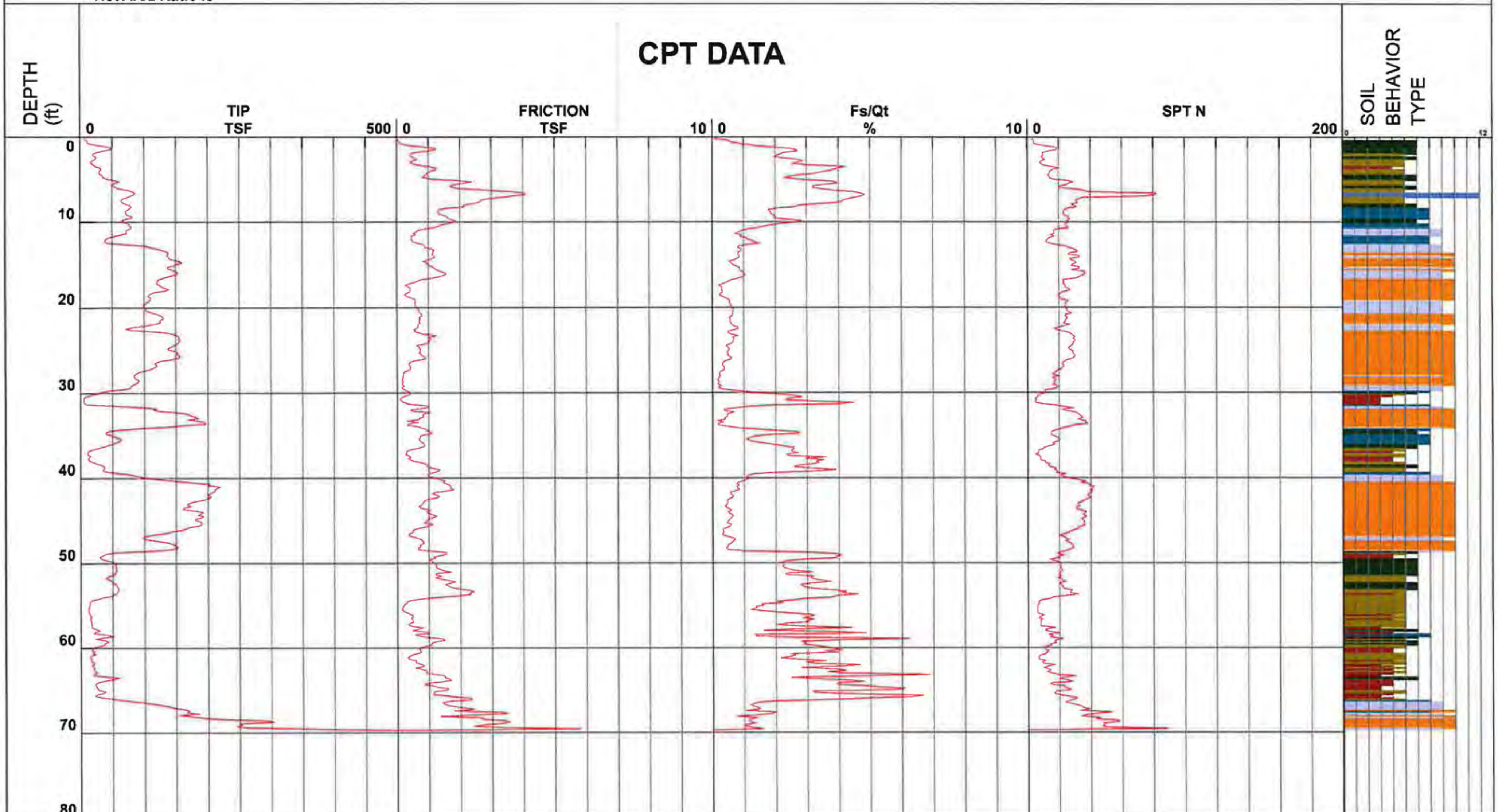
S*Soil behavior type and SPT based on data from UBC-1983

Leighton & Associates

Project	Campus Park	Operator	RA-JC	Filename	SDF(524).cpt
Job Number	04241-003	Cone Number	DDG1185	GPS	
Hole Number	CPT-02	Date and Time	8/9/2012 9:08:10 AM	Maximum Depth	69.88 ft
EST GW Depth During Test	18.00 ft				

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

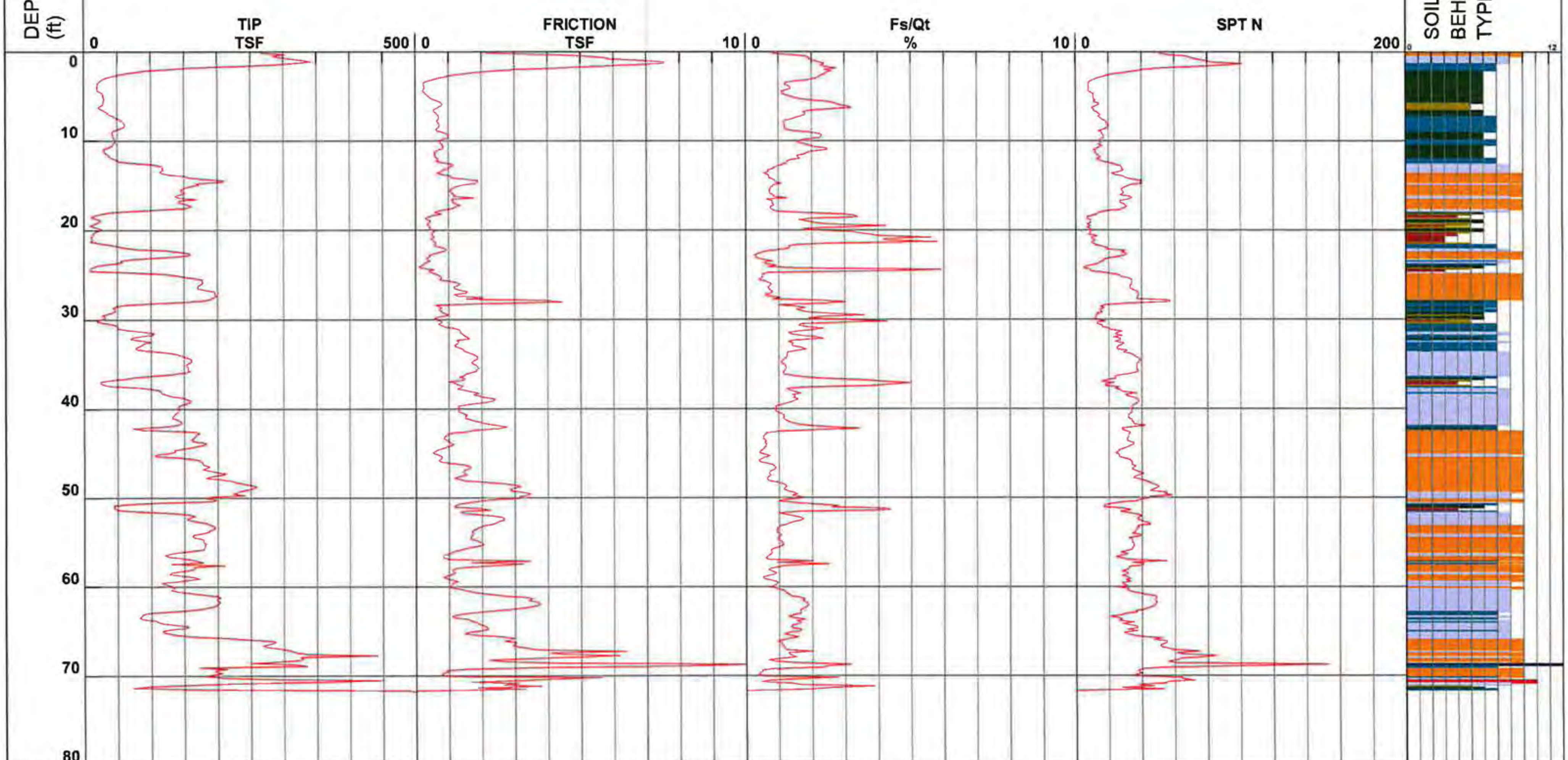
S*Soil behavior type and SPT based on data from UBC-1983

Leighton & Associates

Project	Campus Park	Operator	RA-JC	Filename	SDF(525).cpt
Job Number	04241-003	Cone Number	DDG1185	GPS	
Hole Number	CPT-03	Date and Time	8/9/2012 10:07:13 AM	Maximum Depth	71.85 ft
EST GW Depth During Test		18.00 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

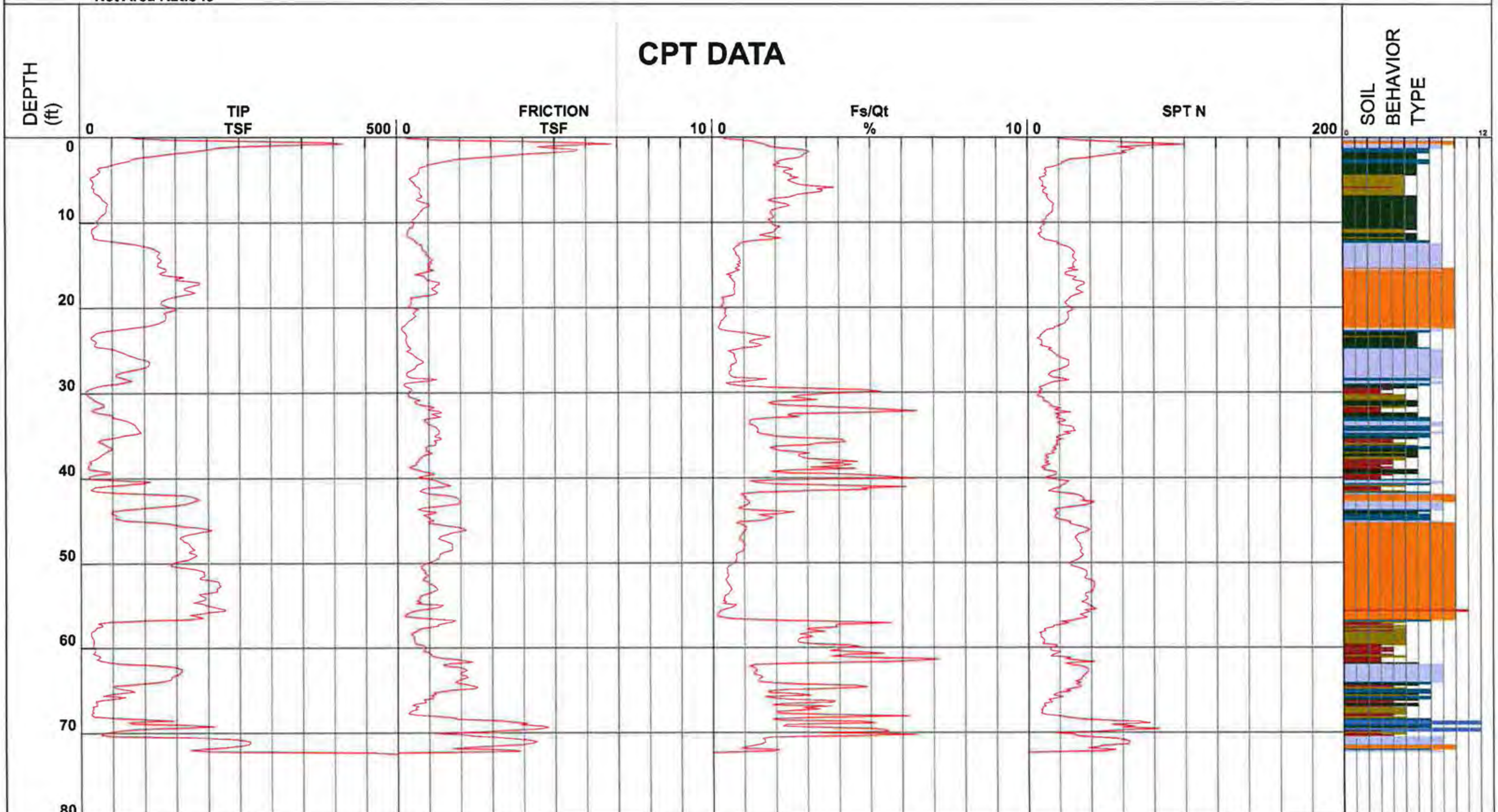
S*Soil behavior type and SPT based on data from UBC-1983

Leighton & Associates

Project	Campus Park	Operator	RA-JC	Filename	SDF(526).cpt
Job Number	04241-003	Cone Number	DDG1185	GPS	
Hole Number	CPT-04	Date and Time	8/9/2012 11:53:46 AM	Maximum Depth	72.51 ft
EST GW Depth During Test		0.00 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

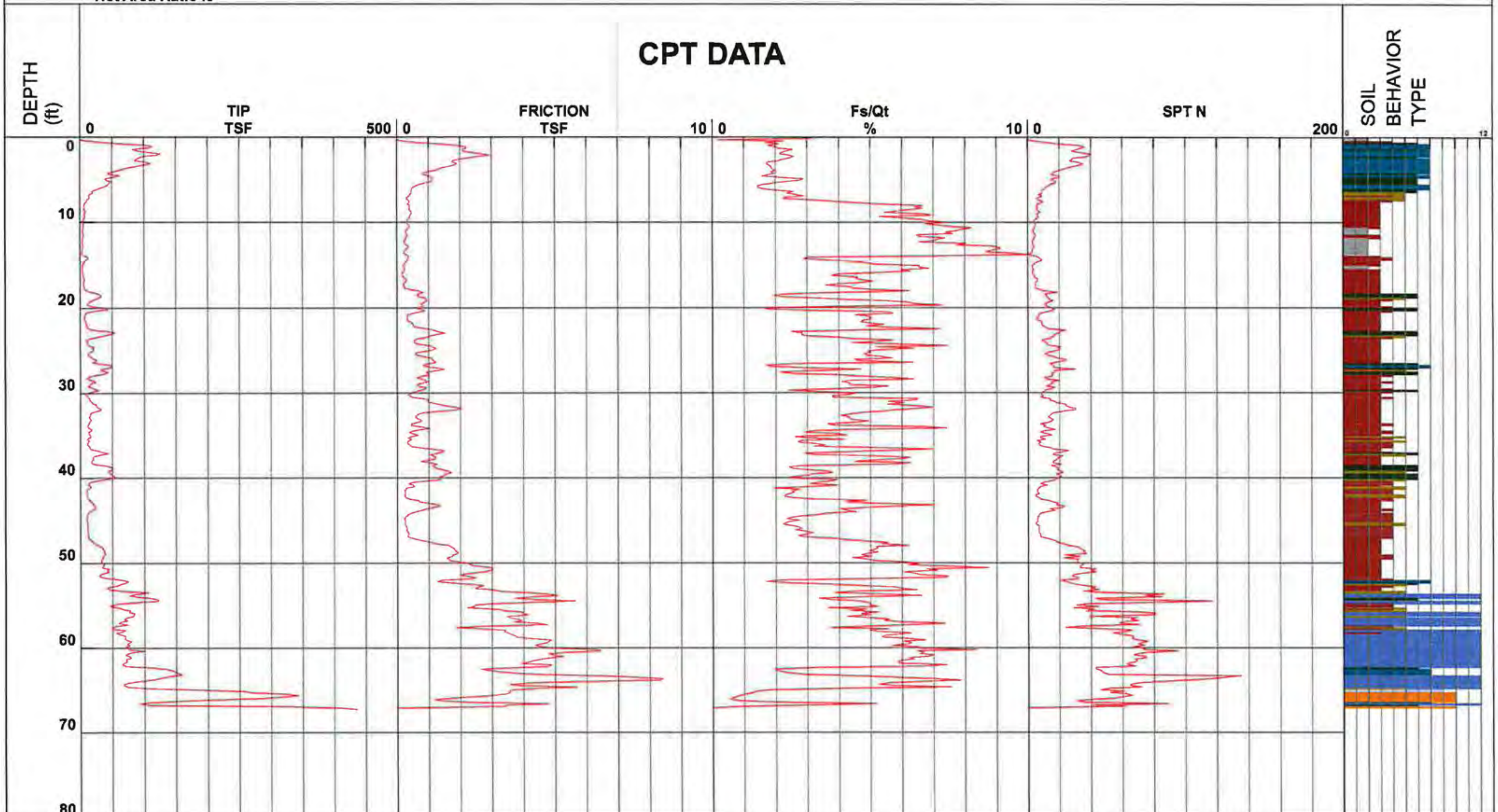
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Leighton & Associates

Project	Campus Park	Operator	RA-JC	Filename	SDF(527).cpt
Job Number	04241-003	Cone Number	DDG1185	GPS	
Hole Number	CPT-05	Date and Time	8/9/2012 1:03:16 PM	Maximum Depth	67.26 ft
EST GW Depth During Test		18.00 ft			

Net Area Ratio .8



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

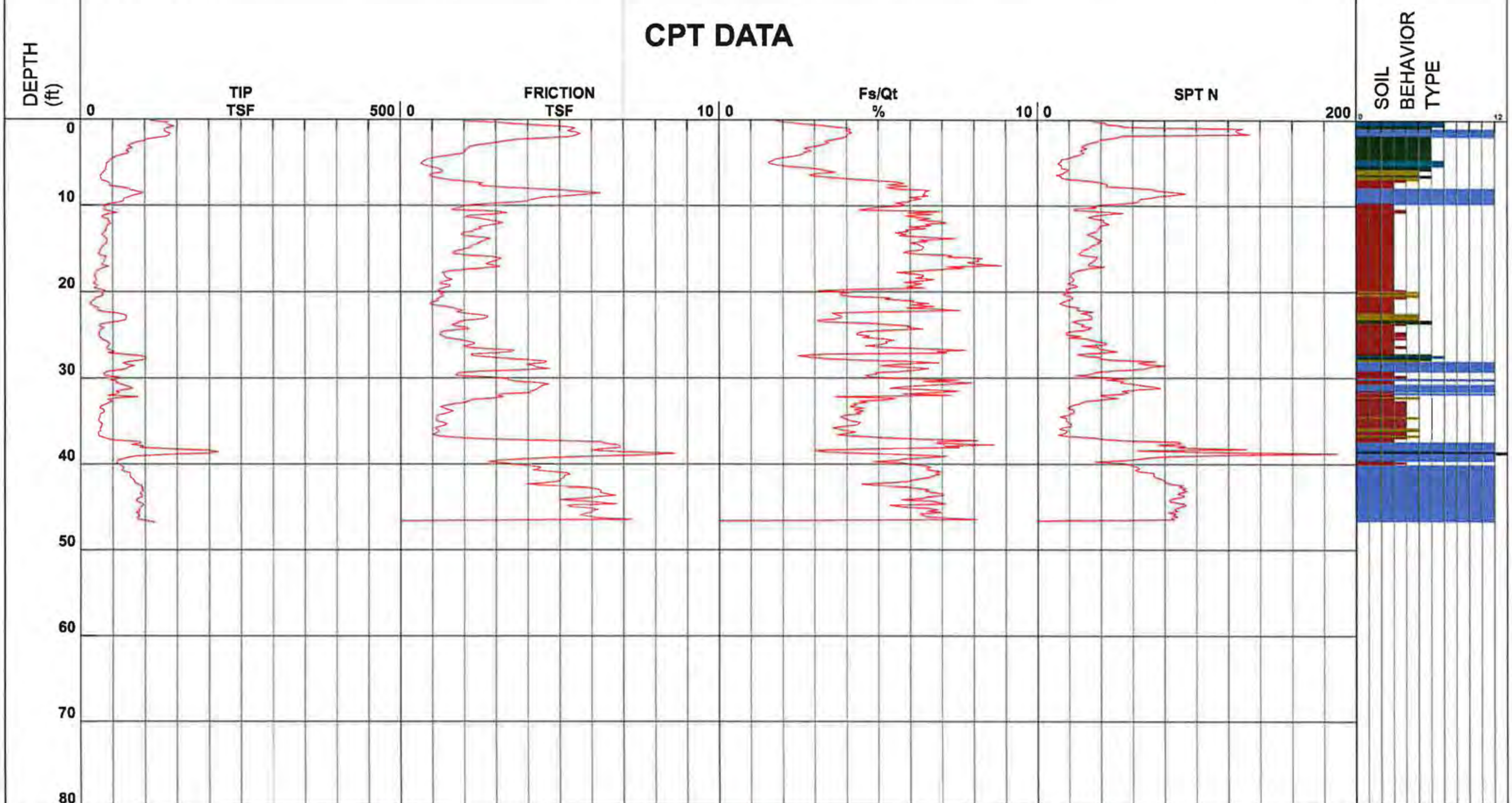
S*Soil behavior type and SPT based on data from UBC-1983

Leighton & Associates

Project	Campus Park	Operator	RA-JC	Filename	SDF(528).cpt
Job Number	04241-003	Cone Number	DDG1185	GPS	
Hole Number	CPT-06	Date and Time	8/9/2012 2:11:12 PM	Maximum Depth	46.75 ft
EST GW Depth During Test	18.00 ft				

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

APPENDIX C

LABORATORY DATA ANALYSIS

APPENDIX C

Laboratory Testing Procedures and Test Results

Particle Size Analysis (ASTM D 422): Particle size analysis was performed by mechanical sieving methods according to ASTM D 422. Plots of the sieve results are provided on the figure in this appendix.

Moisture and Density Determination Tests: Moisture content and dry density determinations were performed on relatively undisturbed samples obtained from the test borings. The results of these tests are presented in the boring logs. Where applicable, only moisture content was determined from "undisturbed" or disturbed samples.

Soluble Sulfates: The soluble sulfate contents of selected samples were determined by standard geochemical methods. The test results are presented in the table below:

Sample Location	Sulfate Content (ppm)	Potential Degree of Sulfate Attack
B-1 at 5 Feet	<150	Negligible
B-2 at 5 Feet	375	Negligible

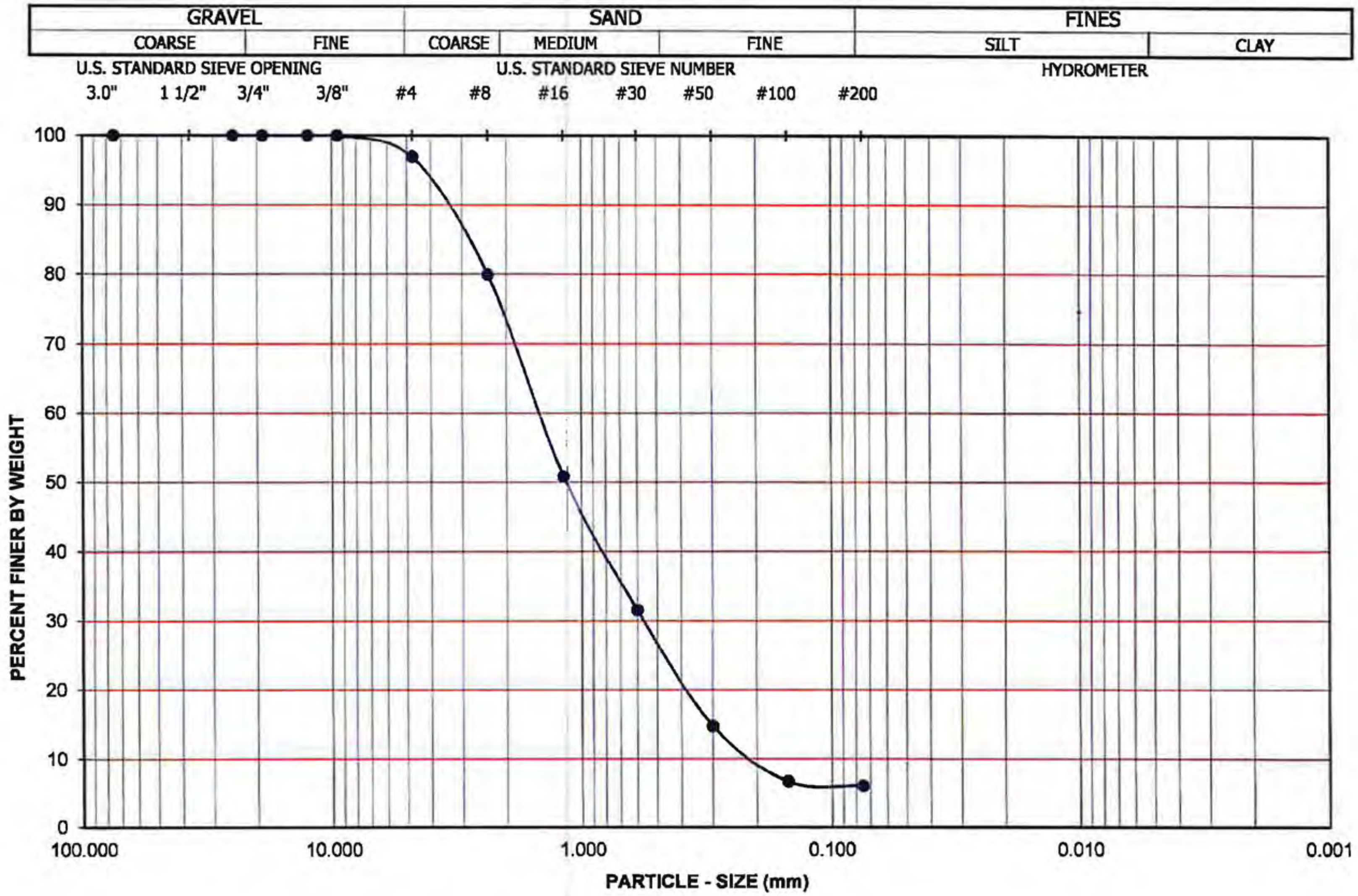
Chloride Content: Chloride content was tested in accordance with DOT Test Method No. 422. The results are presented below:

Sample Location	Chloride Content (ppm)	Degree of Corrosivity
B-1 at 5 Feet	12	Threshold
B-2 at 5 Feet	73	Threshold

APPENDIX C (Continued)

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)	Corrosion Potential
B-1 at 5 Feet	7.67	4828	Severe
B-2 at 5 Feet	7.84	611	Very Severe



Project Name: CAMPUS PARK SEWER LIFT STATION

Project No.: 042343-002

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

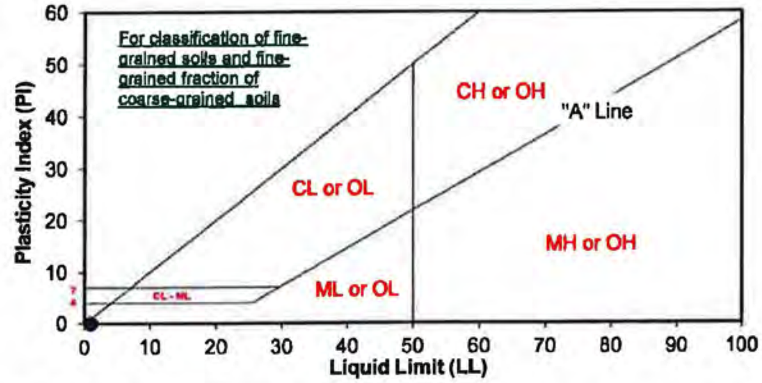
Exploration No.: B-1A   Sample No.: S-4

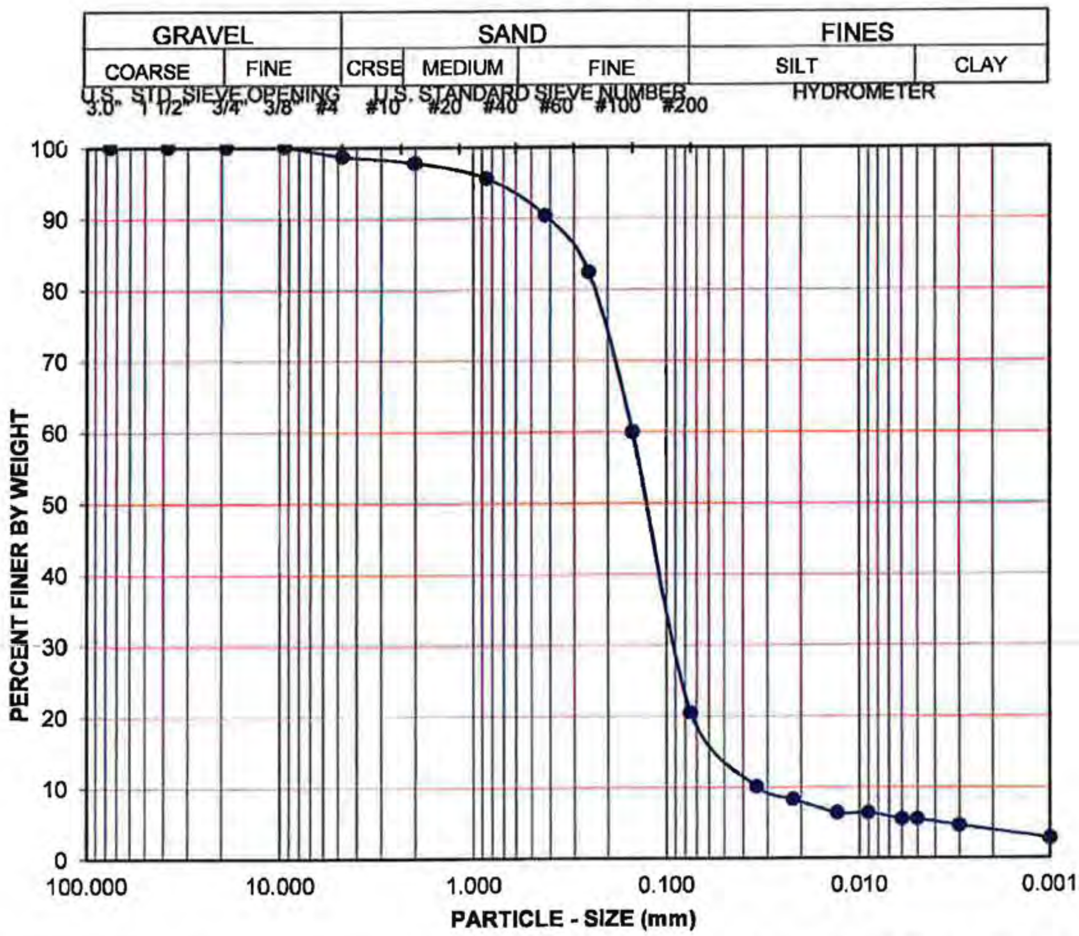
Depth (feet): 20.0   Soil Type : SW-SM

Soil Identification: WELL GRADED SAND WITH SILT (SW-SM), dark gray

GR:SA:FI : (%)   3 : 91 : 6

Aug-12





Boring No.	Sample No.	Depth (ft.)	Soil Type	GR:SA:FI (%)	LL,PL,PI
B-1A	S-5	25	SM	1:78:21	**,**,**

Sample Description:
SILTY SAND (SM), black

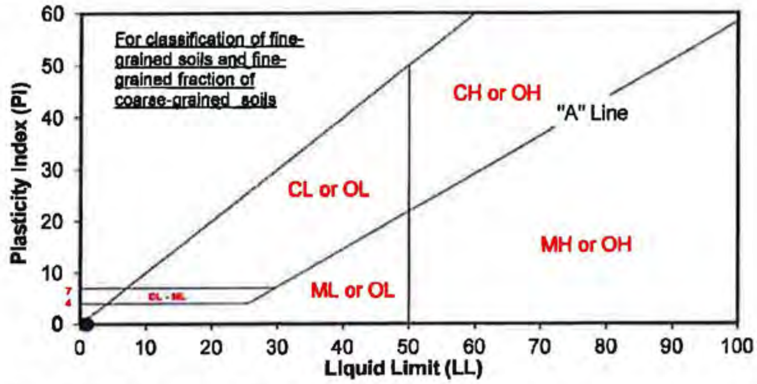
Leighton

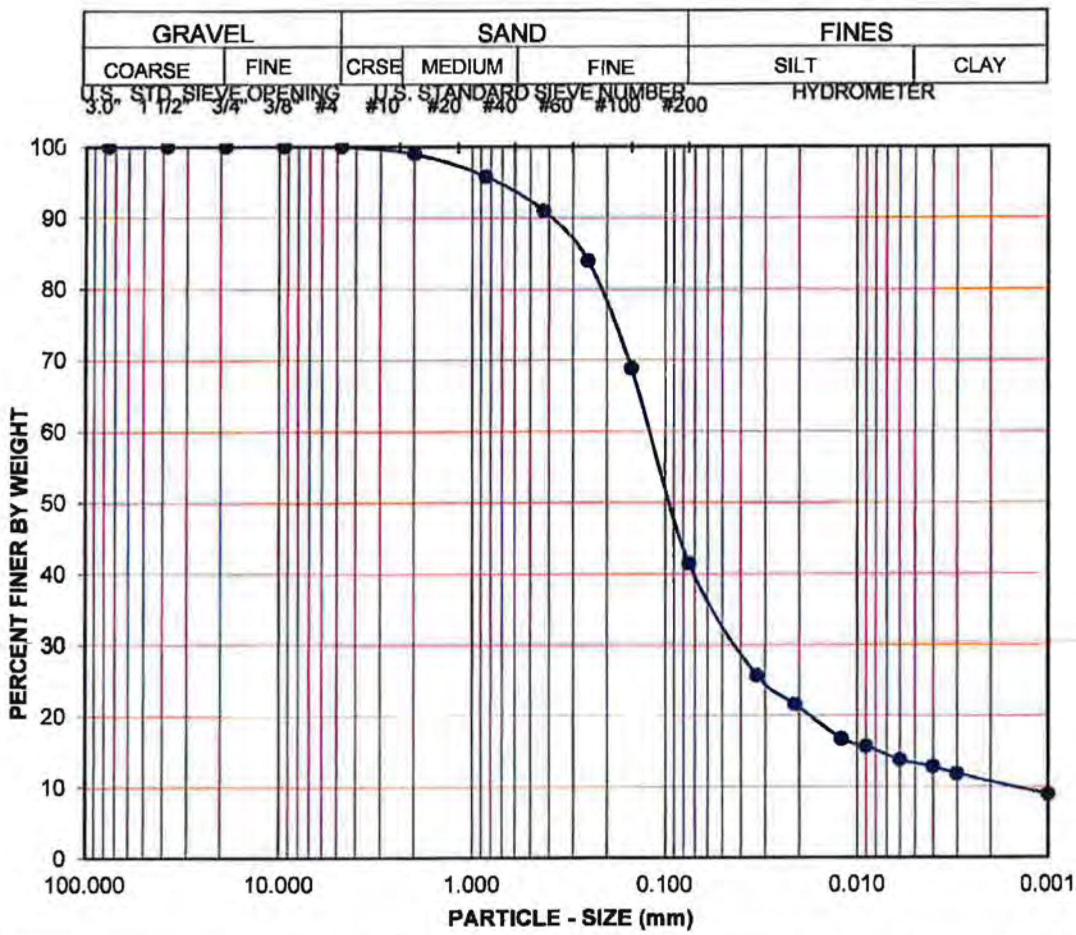
Project No.: 042343-002

CAMPUS PARK SEWER LIFT

ATTERBERG LIMITS, PARTICLE - SIZE CURVE
ASTM D 4318, D 422

Rev. 08-04





Boring No.	Sample No.	Depth (ft.)	Soil Type	GR:SA:FI (%)	LL,PL,PI
B-1A	S-7	35	SM	0:59:41	**,**,**

Sample Description:
SILTY SAND (SM), dark olive brown

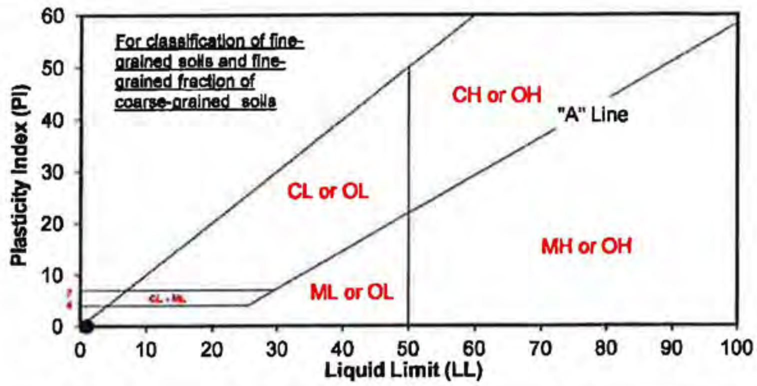
Project No.: 042343-002

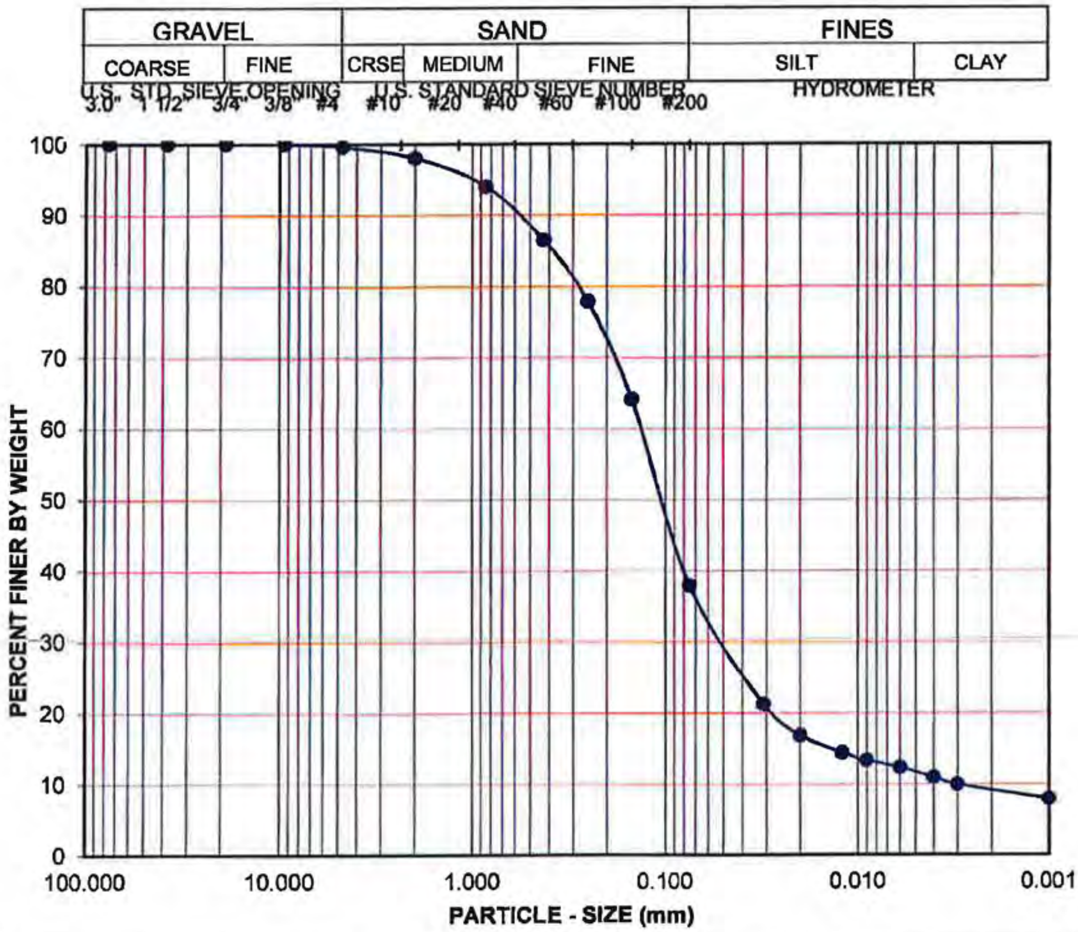
CAMPUS PARK SEWER LIFT

ATTERBERG LIMITS, PARTICLE - SIZE CURVE
ASTM D 4318, D 422

Leighton

Rev. 08-04





Boring No.	Sample No.	Depth (ft.)	Soil Type	GR:SA:FI (%)	LL,PL,PI
B-1A	S-9	45	SM	0:62:38	**,**,**

Sample Description:
SILTY SAND (SM), dark olive brown

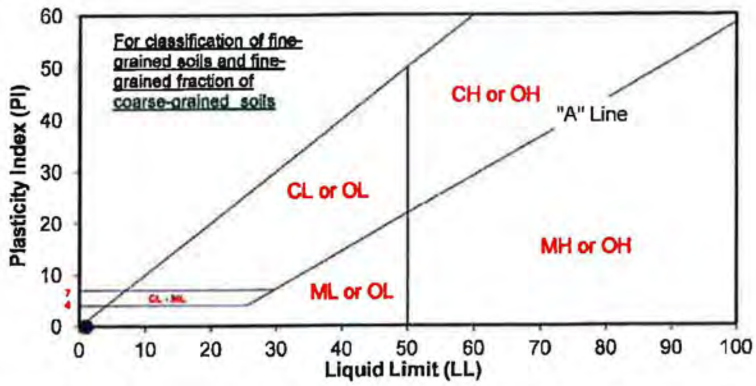
Project No.: 042343-002

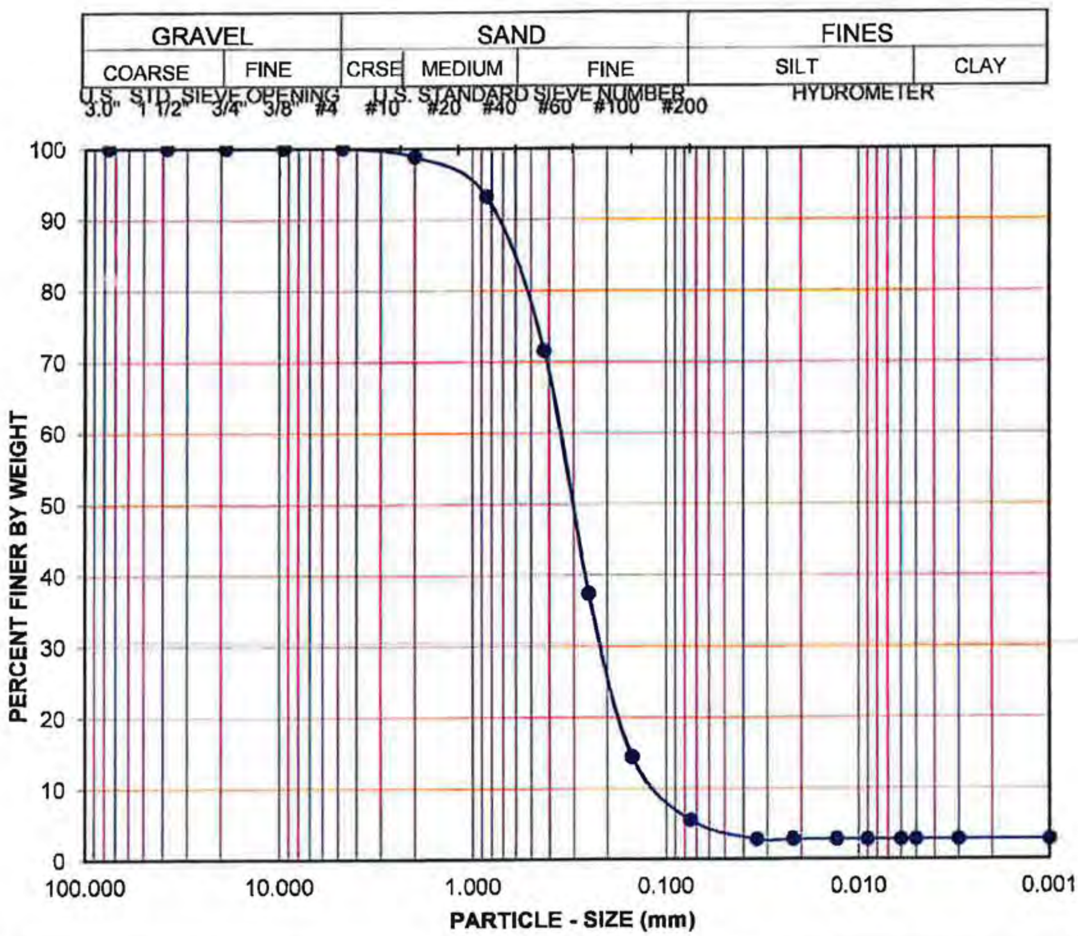
CAMPUS PARK SEWER LIFT

ATTERBERG LIMITS, PARTICLE - SIZE CURVE
ASTM D 4318, D 422

Leighton

Rev. 08-04





Boring No.	Sample No.	Depth (ft.)	Soil Type	GR:SA:FI (%)	LL,PL,PI
B-2	**S-8**	40.0	(SP-SM)	0:94:6	**,**,**

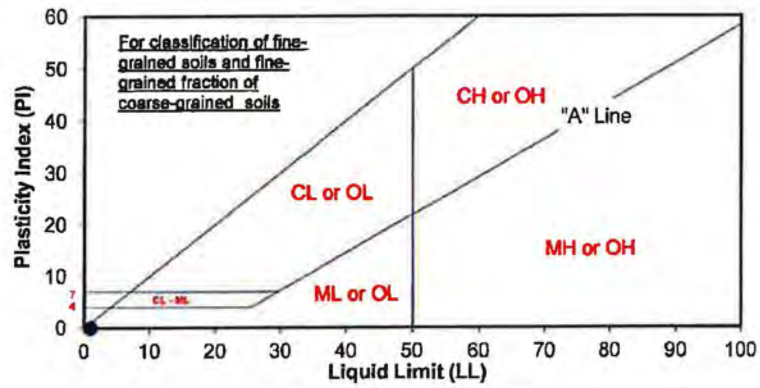
Sample Description:
POORLY GRADED SAND WITH SILT (SP-SM), black.

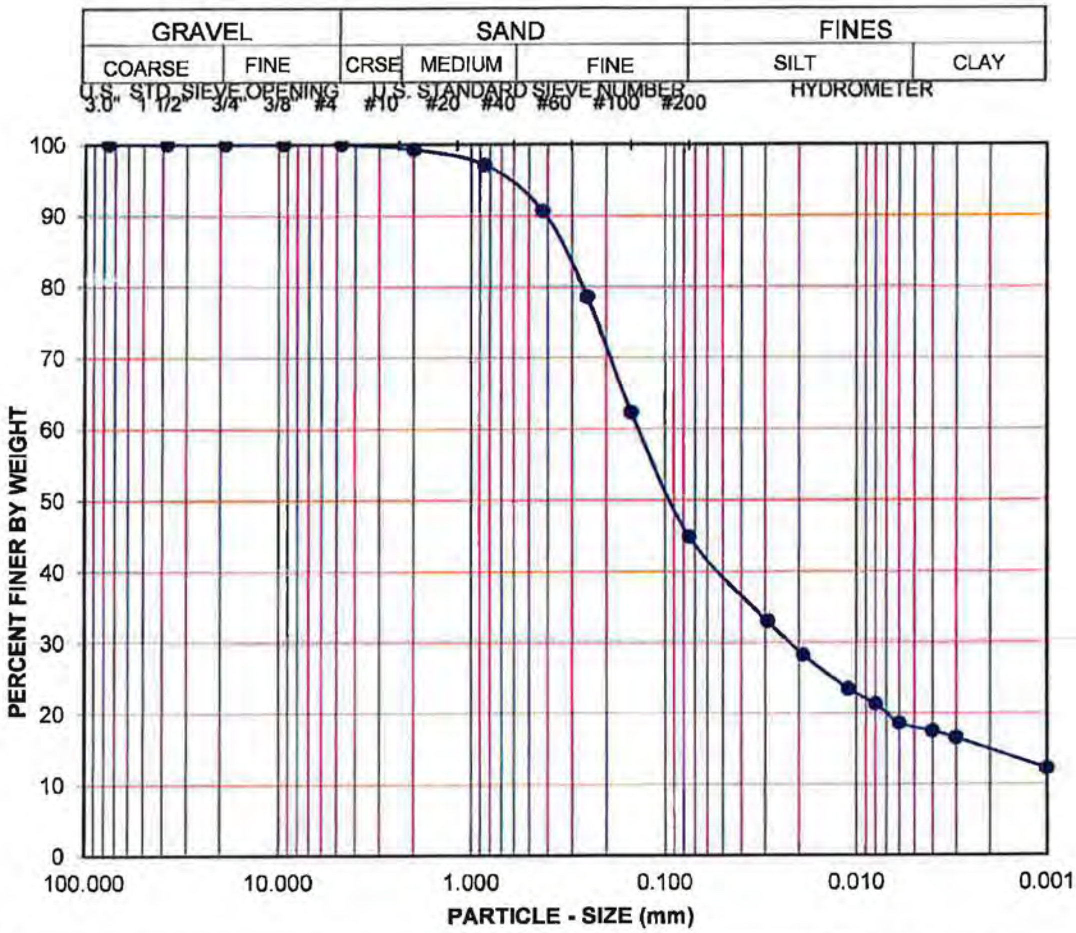
Leighton

Project No.: 042410-003
CAMPUS PARK WEST

ATTERBERG LIMITS, PARTICLE - SIZE CURVE
ASTM D 4318, D 422

Rev. 08-04





Boring No.	Sample No.	Depth (ft.)	Soil Type	GR:SA:FI (%)	LL,PL,PI
B-4	**S-6**	30.0	(SC-SM)	0:55:45	**, **, **

Sample Description:

SILTY, CLAYEY SAND (SC-SM), very dark grayish brown.

Leighton

Project No.: 042410-003
CAMPUS PARK WEST

ATTERBERG LIMITS, PARTICLE - SIZE CURVE
ASTM D 4318, D 422

Rev. 08-04

APPENDIX D

SEISMIC AND LIQUEFACTION ANALYSES

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*                     *
*   E Q F A U L T     *
*                     *
*    Version 3.00     *
*                     *
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DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 042410-003

DATE: 08-27-2012

JOB NAME: Campus

CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: C:\Program Files\EQFAULT1\CGSFLTE.DAT

SITE COORDINATES:
   SITE LATITUDE: 33.3363
   SITE LONGITUDE: 117.1513

SEARCH RADIUS: 100 mi

ATTENUATION RELATION: 17) Campbell & Bozorgnia (1994/1997) - Alluvium
   UNCERTAINTY (M=Median, S=Sigma): M   Number of Sigmas: 0.0
   DISTANCE MEASURE: cdist
   SCOND: 0
   Basement Depth: 5.00 km   Campbell SSR: 0   Campbell SHR: 0
   COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: C:\Program Files\EQFAULT1\CGSFLTE.DAT

MINIMUM DEPTH VALUE (km): 3.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
ELSINORE (TEMECULA)	7.8(12.5)	6.8	0.314	IX
ELSINORE (JULIAN)	8.6(13.9)	7.1	0.337	IX
NEWPORT-INGLEWOOD (Offshore)	20.4(32.8)	7.1	0.157	VIII
ROSE CANYON	21.6(34.8)	7.2	0.159	VIII
ELSINORE (GLEN IVY)	24.1(38.8)	6.8	0.102	VII
SAN JACINTO-ANZA	30.2(48.6)	7.2	0.108	VII
SAN JACINTO-SAN JACINTO VALLEY	31.1(50.0)	6.9	0.081	VII
EARTHQUAKE VALLEY	34.6(55.7)	6.5	0.050	VI
SAN JOAQUIN HILLS	35.5(57.2)	6.6	0.055	VI
CORONADO BANK	37.4(60.2)	7.6	0.116	VII
SAN JACINTO-COYOTE CREEK	38.1(61.3)	6.6	0.048	VI
CHINO-CENTRAL AVE. (Elsinore)	41.6(67.0)	6.7	0.048	VI
PALOS VERDES	45.7(73.5)	7.3	0.070	VI
WHITTIER	45.7(73.6)	6.8	0.045	VI
SAN JACINTO-SAN BERNARDINO	47.3(76.2)	6.7	0.040	V
NEWPORT-INGLEWOOD (L.A.Basin)	48.4(77.9)	7.1	0.055	VI
SAN ANDREAS - Whole M-1a	50.0(80.4)	8.0	0.114	VII
SAN ANDREAS - San Bernardino M-1	50.0(80.4)	7.5	0.075	VII
SAN ANDREAS - SB-Coach. M-1b-2	50.0(80.4)	7.7	0.089	VII
SAN ANDREAS - SB-Coach. M-2b	50.0(80.4)	7.7	0.089	VII
ELSINORE (COYOTE MOUNTAIN)	52.3(84.2)	6.8	0.038	V
PINTO MOUNTAIN	55.6(89.4)	7.2	0.050	VI
SAN JACINTO - BORREGO	56.1(90.3)	6.6	0.029	V
SAN ANDREAS - Coachella M-1c-5	56.5(90.9)	7.2	0.049	VI
PUENTE HILLS BLIND THRUST	58.2(93.6)	7.1	0.041	V
BURNT MTN.	60.3(97.1)	6.5	0.024	V
NORTH FRONTAL FAULT ZONE (West)	62.0(99.8)	7.2	0.040	V
SAN JOSE	62.1(100.0)	6.4	0.021	IV
CUCAMONGA	62.3(100.3)	6.9	0.032	V
EUREKA PEAK	63.6(102.4)	6.4	0.021	IV
NORTH FRONTAL FAULT ZONE (East)	64.7(104.1)	6.7	0.026	V
CLEGHORN	65.1(104.8)	6.5	0.022	IV
SIERRA MADRE	65.3(105.1)	7.2	0.037	V
SAN ANDREAS - Cho-Moj M-1b-1	68.8(110.8)	7.8	0.064	VI
SAN ANDREAS - Mojave M-1c-3	68.8(110.8)	7.4	0.045	VI
SAN ANDREAS - 1857 Rupture M-2a	68.8(110.8)	7.8	0.064	VI
LANDERS	70.7(113.8)	7.3	0.040	V
HELENDALE - S. LOCKHARDT	73.3(118.0)	7.3	0.038	V
UPPER ELYSIAN PARK BLIND THRUST	74.6(120.0)	6.4	0.016	IV
SUPERSTITION MTN. (San Jacinto)	74.8(120.4)	6.6	0.020	IV

DETERMINISTIC SITE PARAMETERS

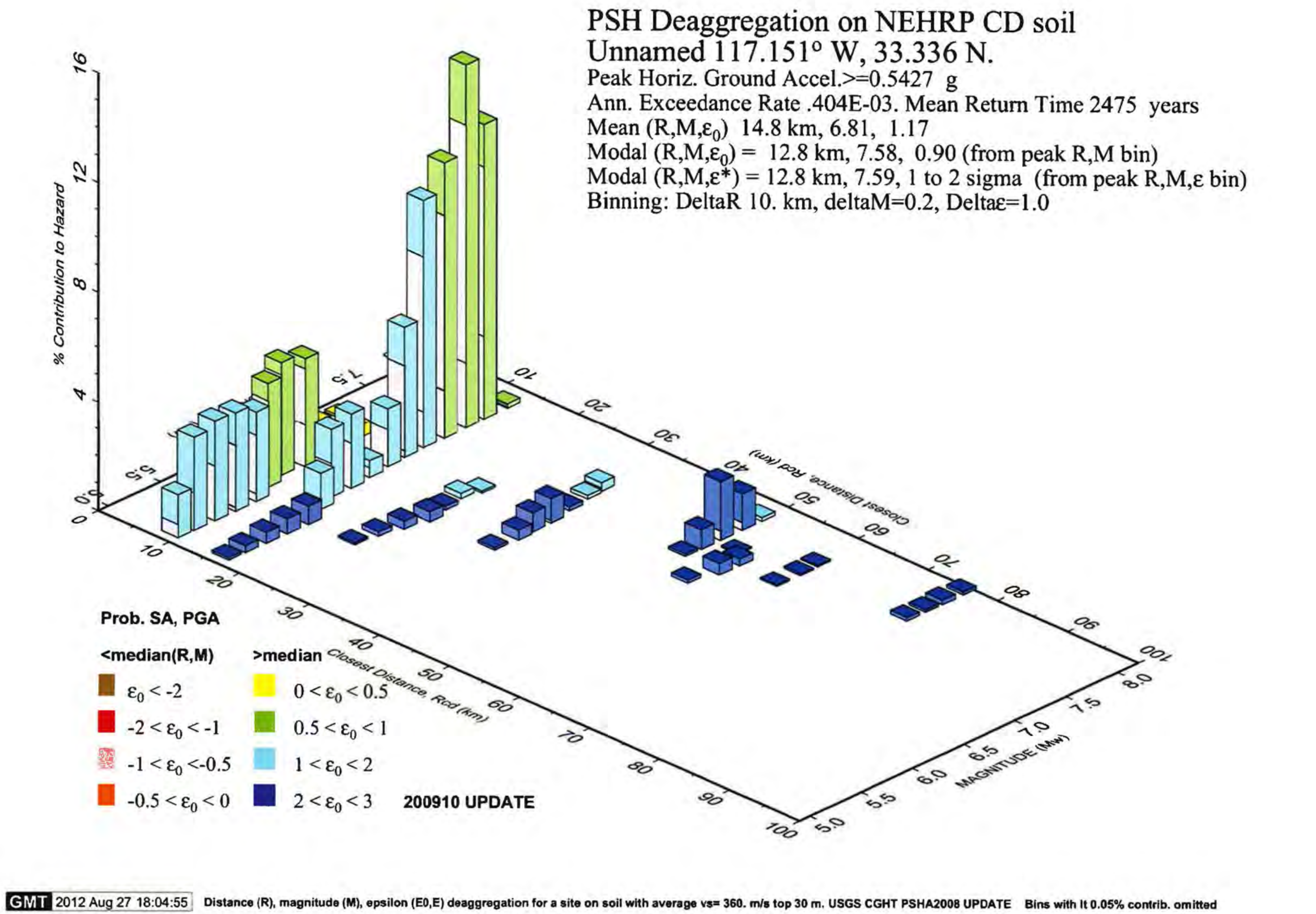
Page 2

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
RAYMOND	75.3(121.2)	6.5	0.017	IV
CLAMSHELL-SAWPIT	76.1(122.5)	6.5	0.017	IV
LENWOOD-LOCKHART-OLD WOMAN SPRGS	76.4(122.9)	7.5	0.043	VI
ELMORE RANCH	77.9(125.3)	6.6	0.019	IV
JOHNSON VALLEY (Northern)	78.5(126.3)	6.7	0.021	IV
SUPERSTITION HILLS (San Jacinto)	79.0(127.2)	6.6	0.019	IV
EMERSON So. - COPPER MTN.	79.5(127.9)	7.0	0.026	V
VERDUGO	80.2(129.0)	6.9	0.022	IV
HOLLYWOOD	83.0(133.6)	6.4	0.014	IV
BRAWLEY SEISMIC ZONE	83.4(134.2)	6.4	0.014	IV
LAGUNA SALADA	84.6(136.1)	7.0	0.024	V
PISGAH-BULLION MTN.-MESQUITE LK	86.0(138.4)	7.3	0.031	V
CALICO - HIDALGO	87.3(140.5)	7.3	0.031	V
SANTA MONICA	89.2(143.6)	6.6	0.015	IV
MALIBU COAST	93.3(150.1)	6.7	0.015	IV
SIERRA MADRE (San Fernando)	94.0(151.3)	6.7	0.015	IV
SAN GABRIEL	94.1(151.4)	7.2	0.025	V
IMPERIAL	96.0(154.5)	7.0	0.021	IV
NORTHRIDGE (E. Oak Ridge)	96.2(154.8)	7.0	0.018	IV

-END OF SEARCH- 59 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ELSINORE (TEMECULA) FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 7.8 MILES (12.5 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.3367 g



PSH Deaggregation on NEHRP CD soil
Unnamed 117.151° W, 33.336 N.
Peak Horiz. Ground Accel.>=0.5427 g
Ann. Exceedance Rate .404E-03. Mean Return Time 2475 years
Mean (R,M,ε_0) 14.8 km, 6.81, 1.17
Modal (R,M,ε_0) = 12.8 km, 7.58, 0.90 (from peak R,M bin)
Modal (R,M,ε*) = 12.8 km, 7.59, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR 10. km, deltaM=0.2, Deltaε=1.0
% Contribution to Hazard
Closest Distance, Rcd (km)
MAGNITUDE (Mw)
Prob. SA, PGA
<median(R,M)
>median
$\varepsilon_0 < -2$
$-2 < \varepsilon_0 < -1$
$-1 < \varepsilon_0 < -0.5$
$-0.5 < \varepsilon_0 < 0$
$0 < \varepsilon_0 < 0.5$
$0.5 < \varepsilon_0 < 1$
$1 < \varepsilon_0 < 2$
$2 < \varepsilon_0 < 3$
200910 UPDATE
GMT 2012 Aug 27 18:04:55 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 360. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

LIQUEFACTION ANALYSIS

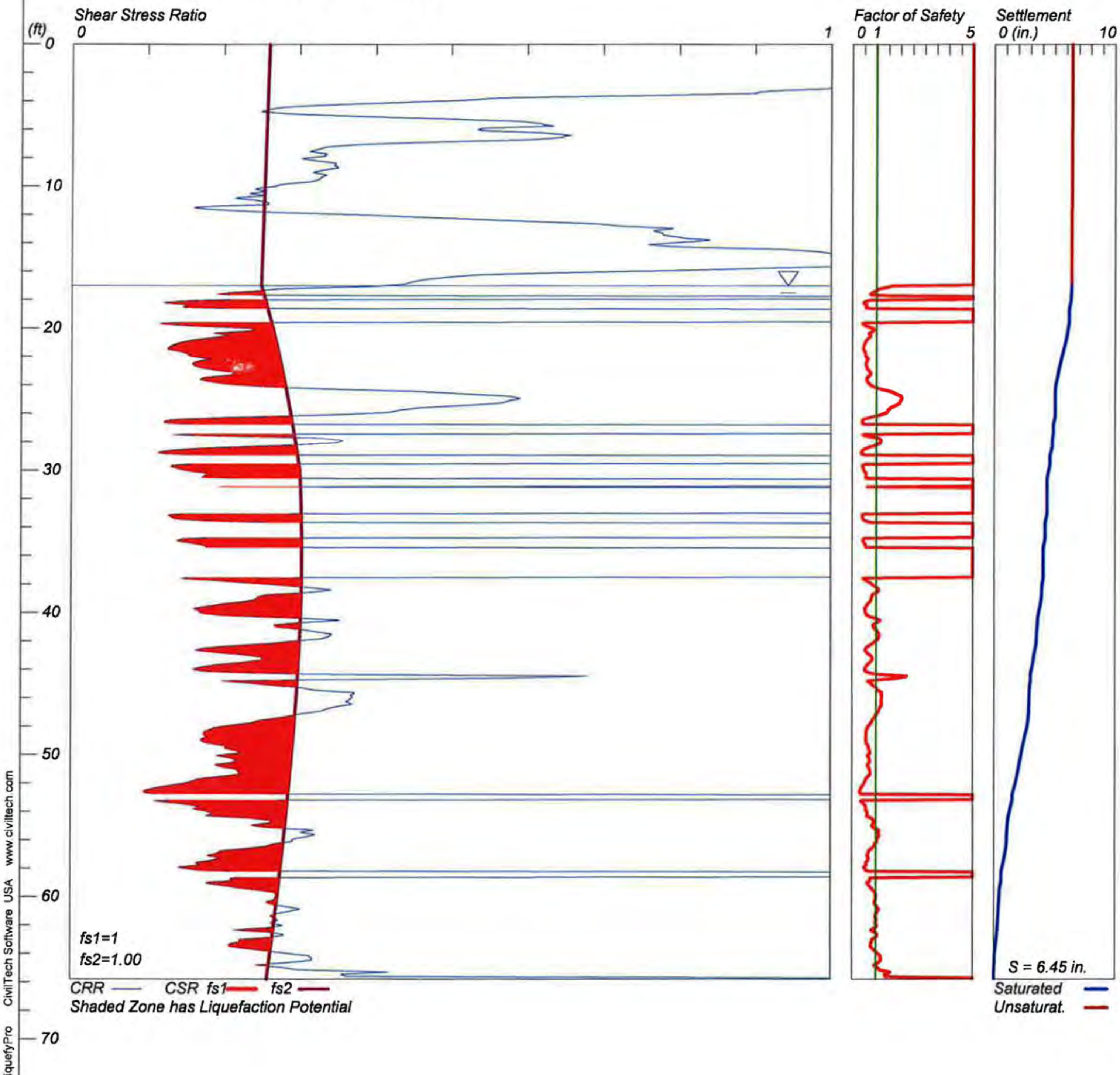
Campus Park West

Hole No.=CPT-1 Water Depth=17 ft Surface Elev.=260

Ground Improvement of Fill=4 ft

Magnitude=7.6

Acceleration=0.4g



LIQUEFACTION ANALYSIS

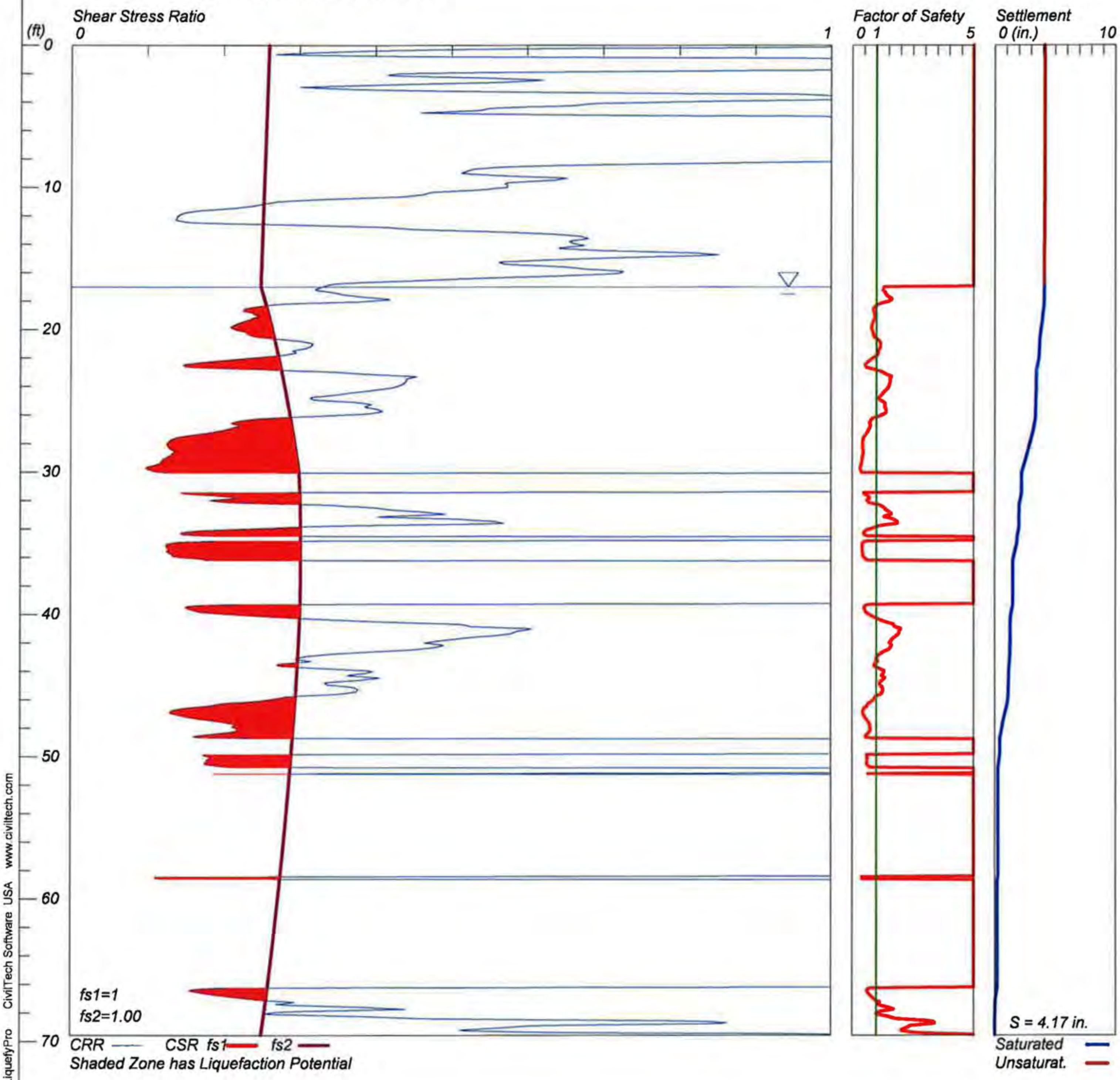
Campus Park West

Hole No.=CPT-2 Water Depth=17 ft Surface Elev.=260

Ground Improvement of Fill=4 ft

Magnitude=7.6

Acceleration=0.4g



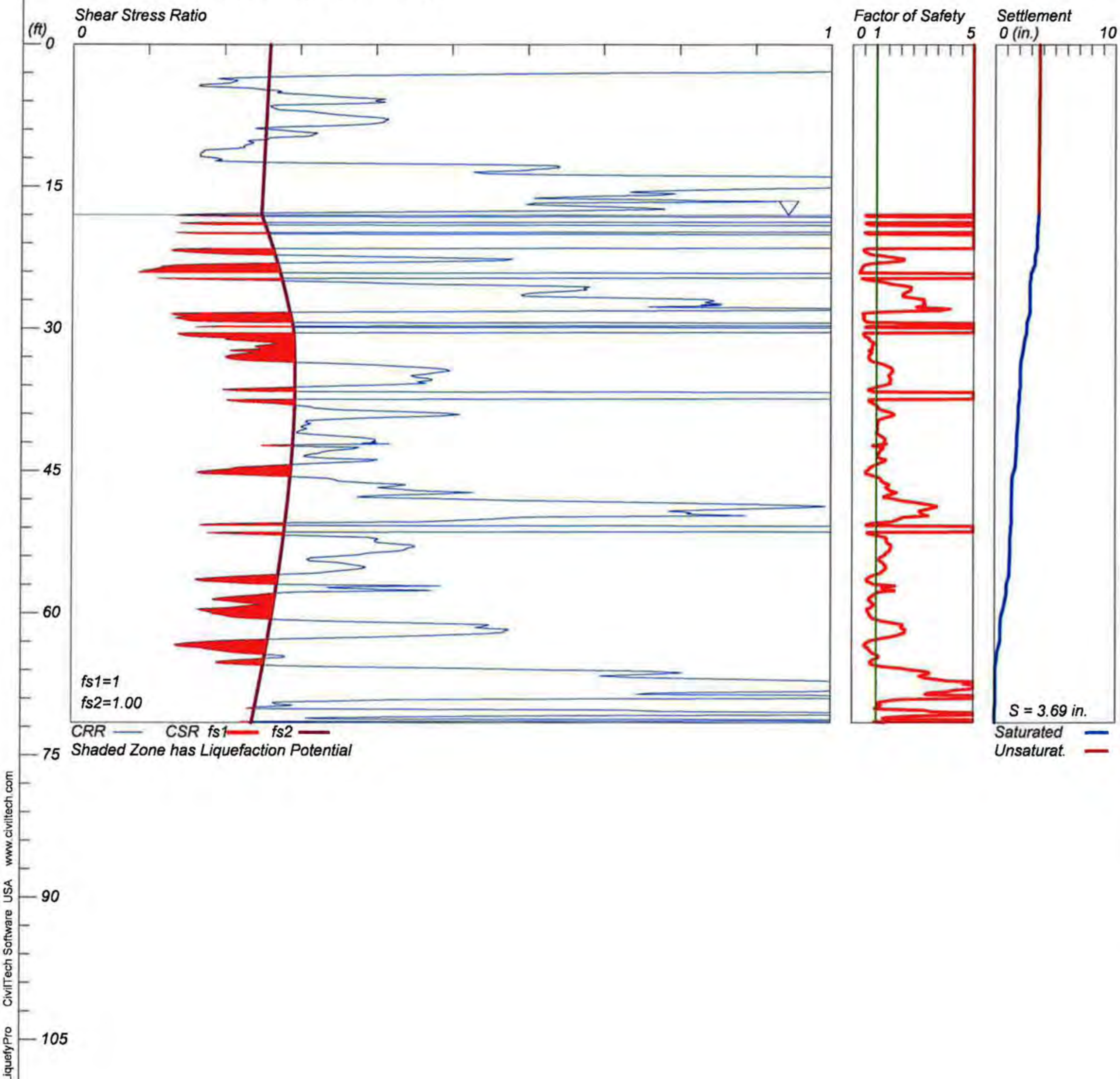
CivilTech Corporation **042410-003** **Plate A-1**

LIQUEFACTION ANALYSIS

Campus Park West

Hole No.=CPT-3   Water Depth=18 ft   Surface Elev.=260
Ground Improvement of Fill=6 ft

Magnitude=7.6
Acceleration=0.4g

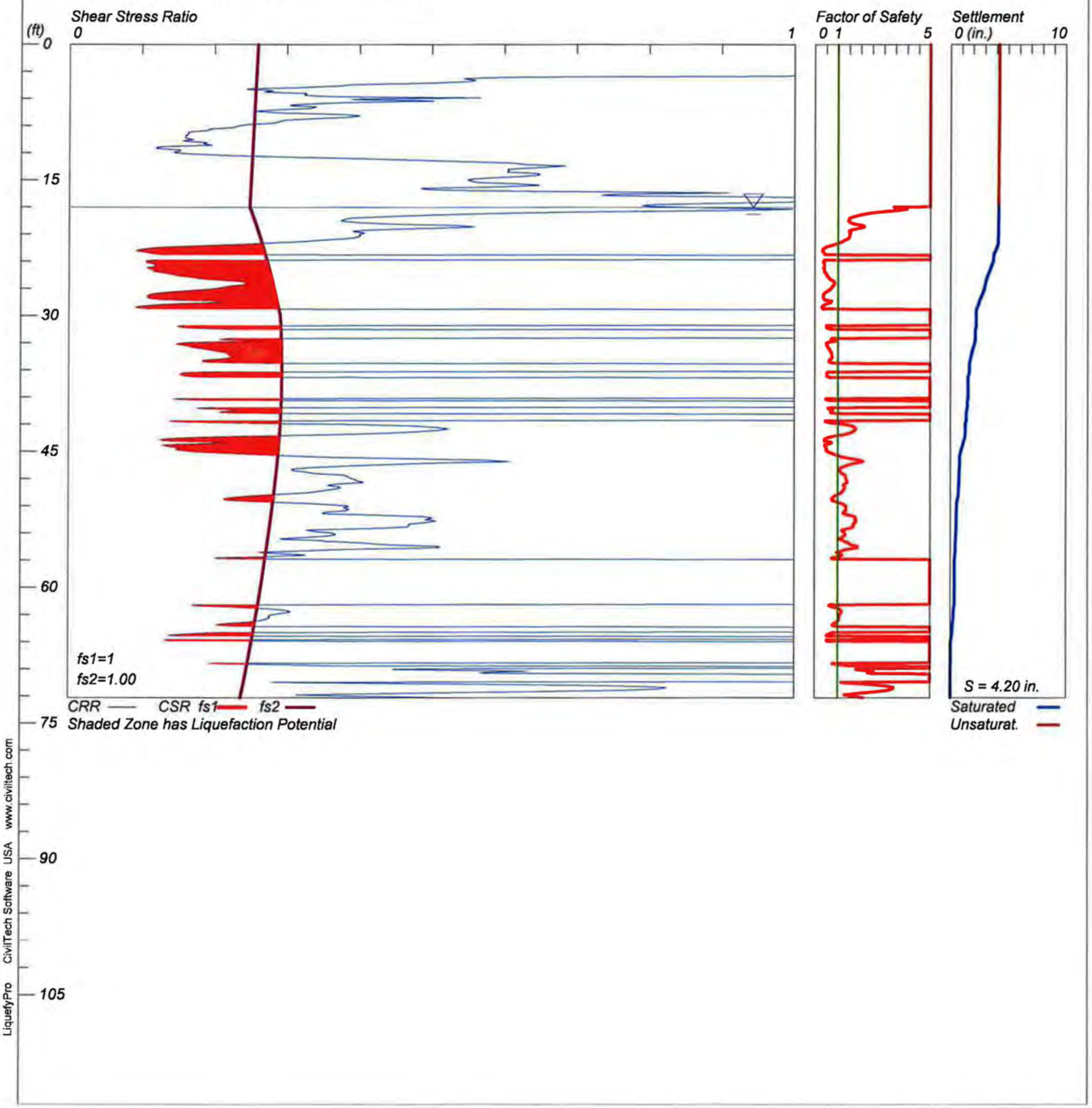


LIQUEFACTION ANALYSIS

Campus Park West

Hole No.=CPT-4   Water Depth=18 ft   Surface Elev.=260   ***Magnitude=7.6***

Ground Improvement of Fill=6 ft   ***Acceleration=0.4g***

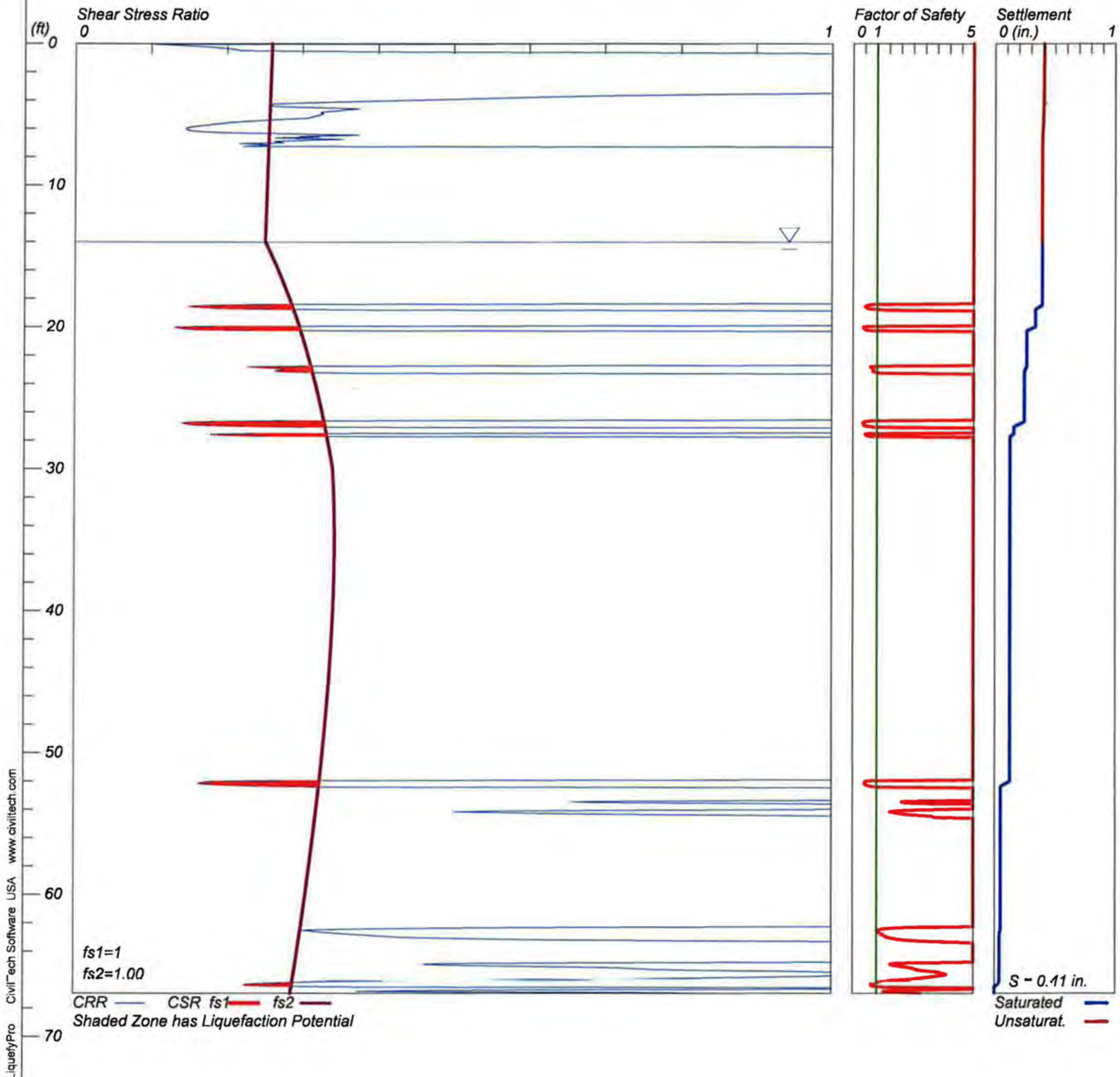


LIQUEFACTION ANALYSIS

Campus Park West

Hole No.=CPT-5 Water Depth=14 ft Surface Elev.=260

Magnitude=7.6
Acceleration=0.4g



LIQUEFACTION ANALYSIS

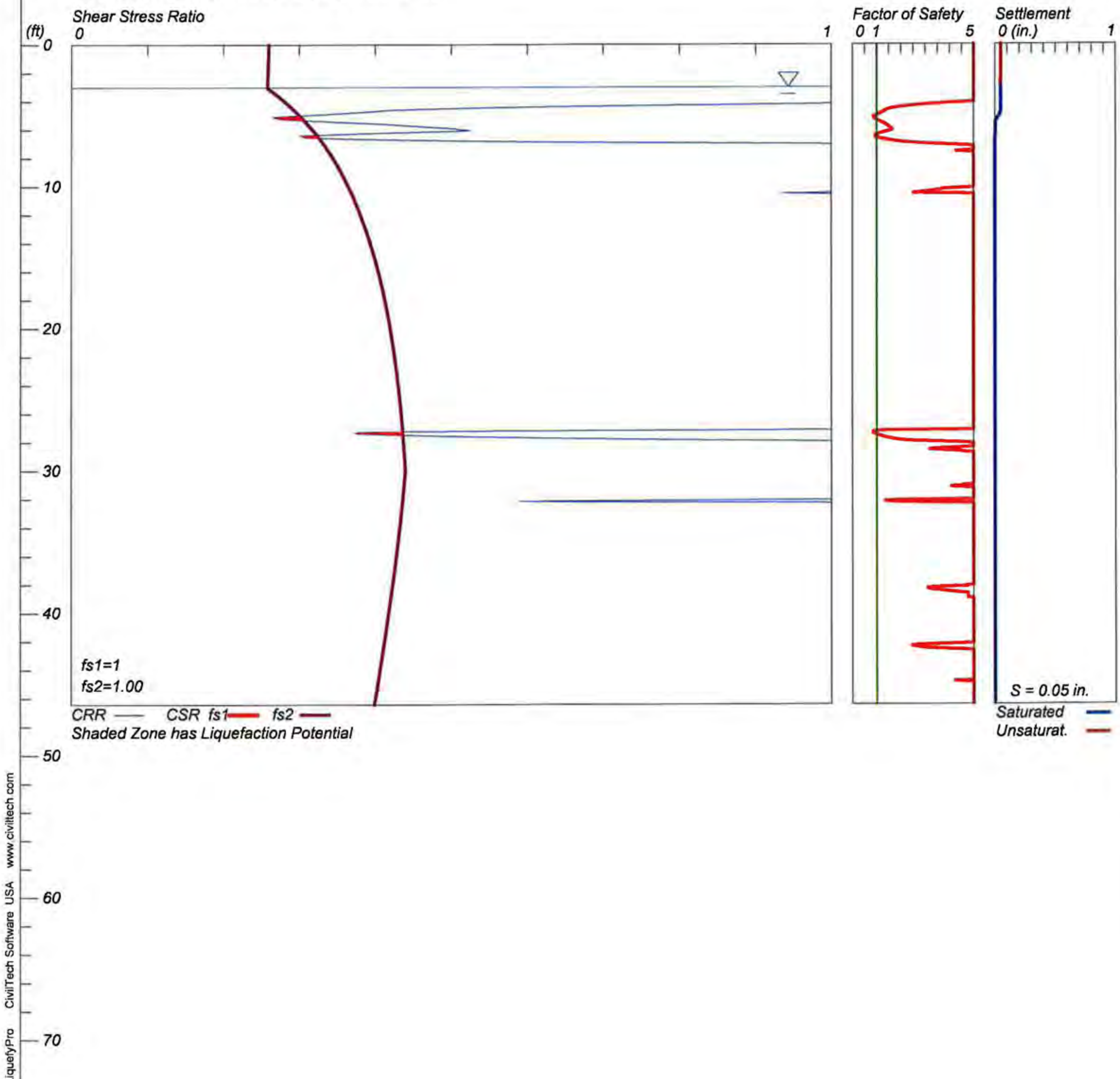
Campus Park West

Hole No.=CPT-6 Water Depth=3 ft Surface Elev.=260

Ground Improvement of Fill=2 ft

Magnitude=7.6

Acceleration=0.4g



APPENDIX E

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

1.0 General

1.1 Intent

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

1.2 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.

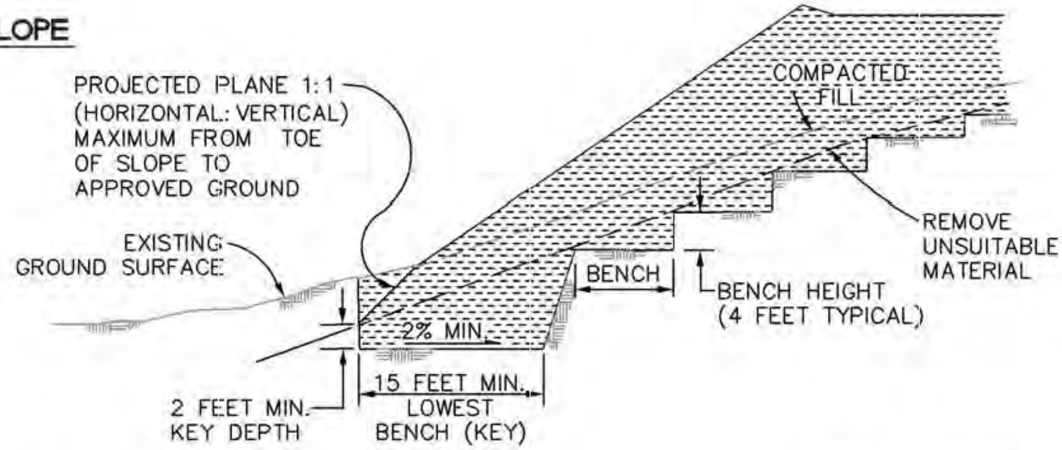
7.3 Lift Thickness

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.

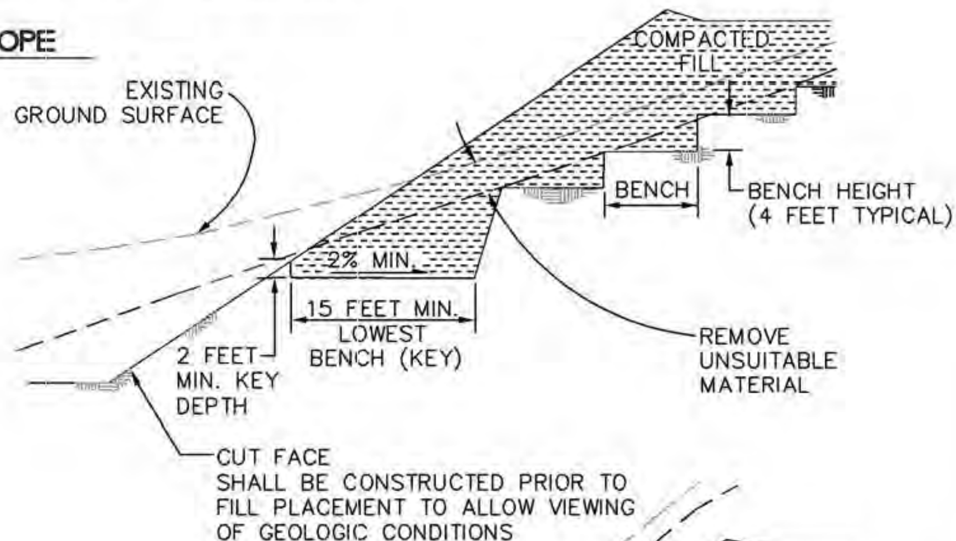
7.4 Observation and Testing

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

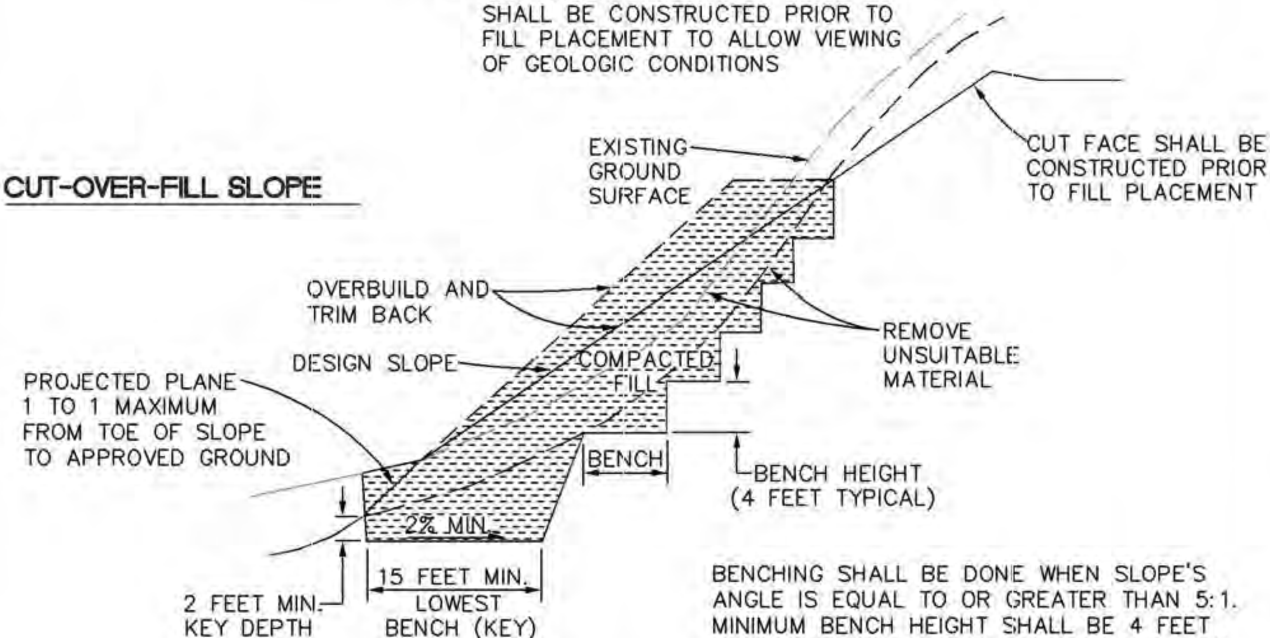
FILL SLOPE



FILL-OVER-CUT SLOPE



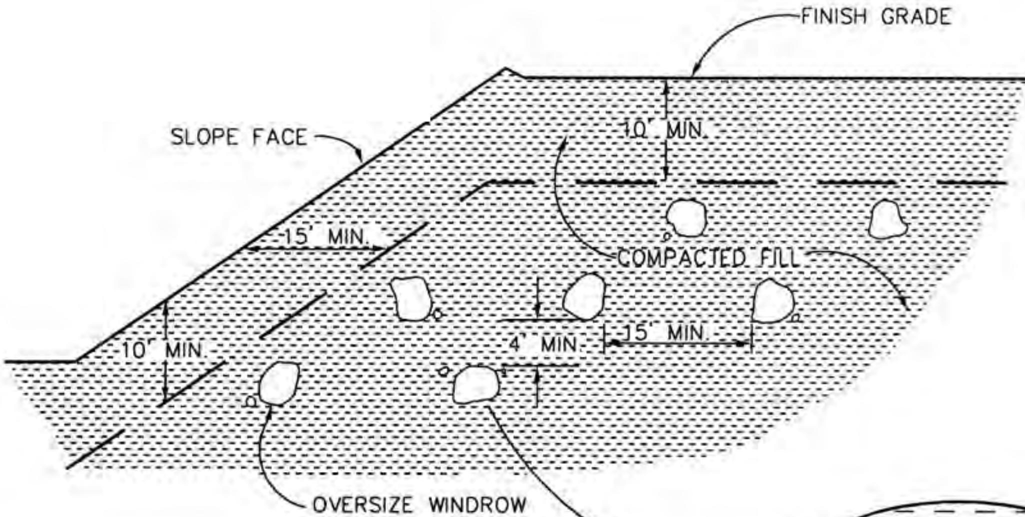
CUT-OVER-FILL SLOPE

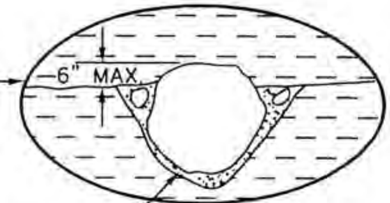


BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

KEYING AND BENCHING

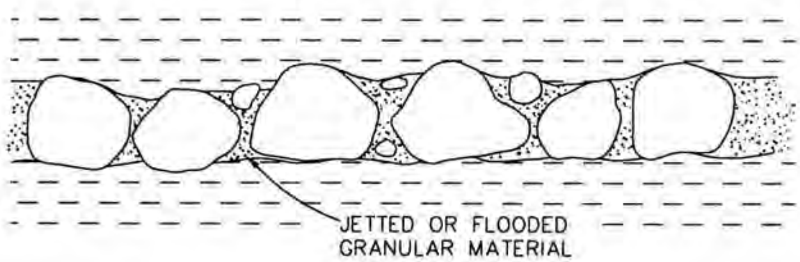
GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL A





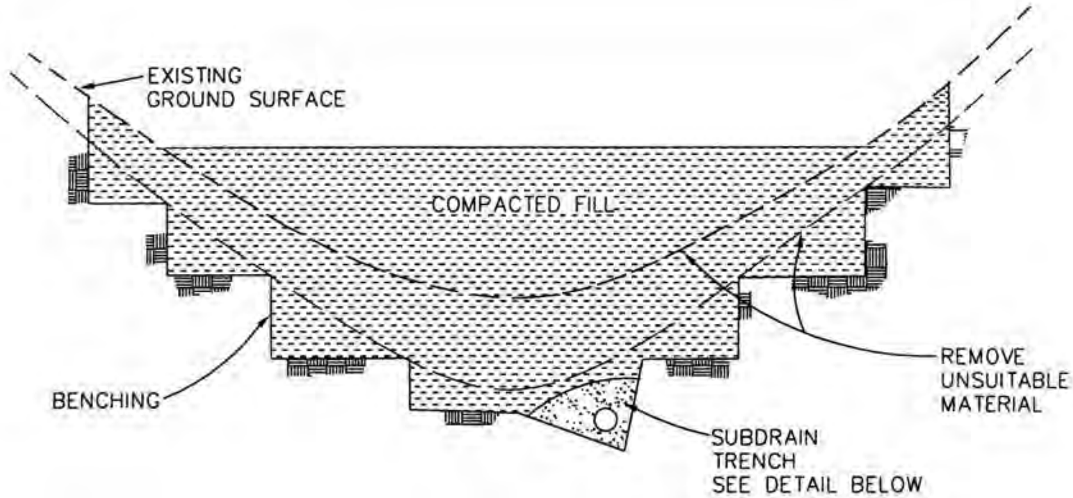
DETAIL

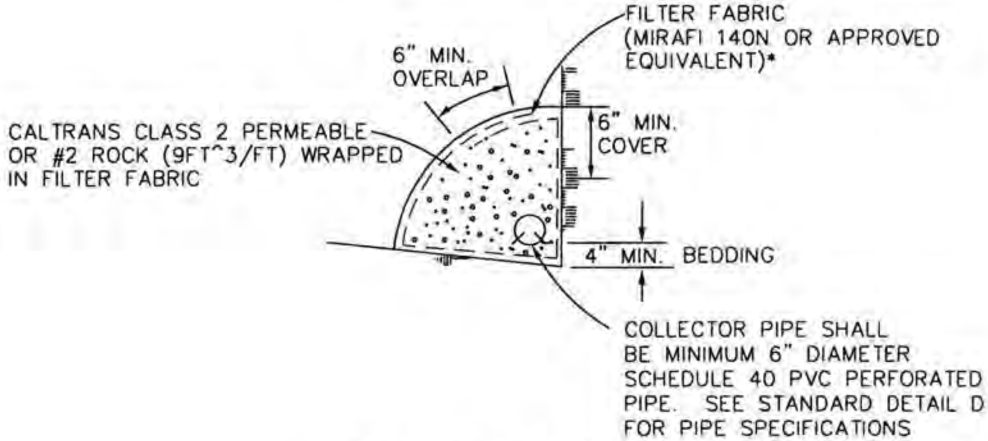
- OVERSIZE ROCK IS LARGER THAN 8 INCHES IN LARGEST DIMENSION.
- EXCAVATE A TRENCH IN THE COMPACTED FILL DEEP ENOUGH TO BURY ALL THE ROCK.
- BACKFILL WITH GRANULAR SOIL JETTED OR FLOODED IN PLACE TO FILL ALL THE VOIDS.
- DO NOT BURY ROCK WITHIN 10 FEET OF FINISH GRADE.
- WINDROW OF BURIED ROCK SHALL BE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



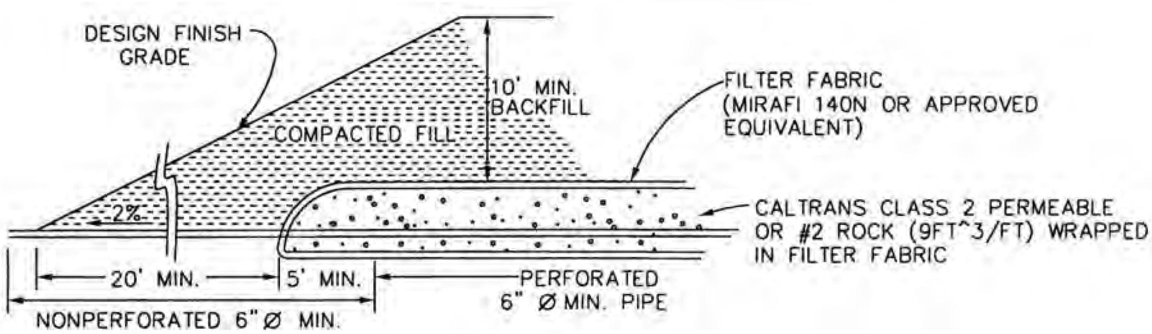
TYPICAL PROFILE ALONG WINDROW

OVERSIZE ROCK DISPOSAL	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL B



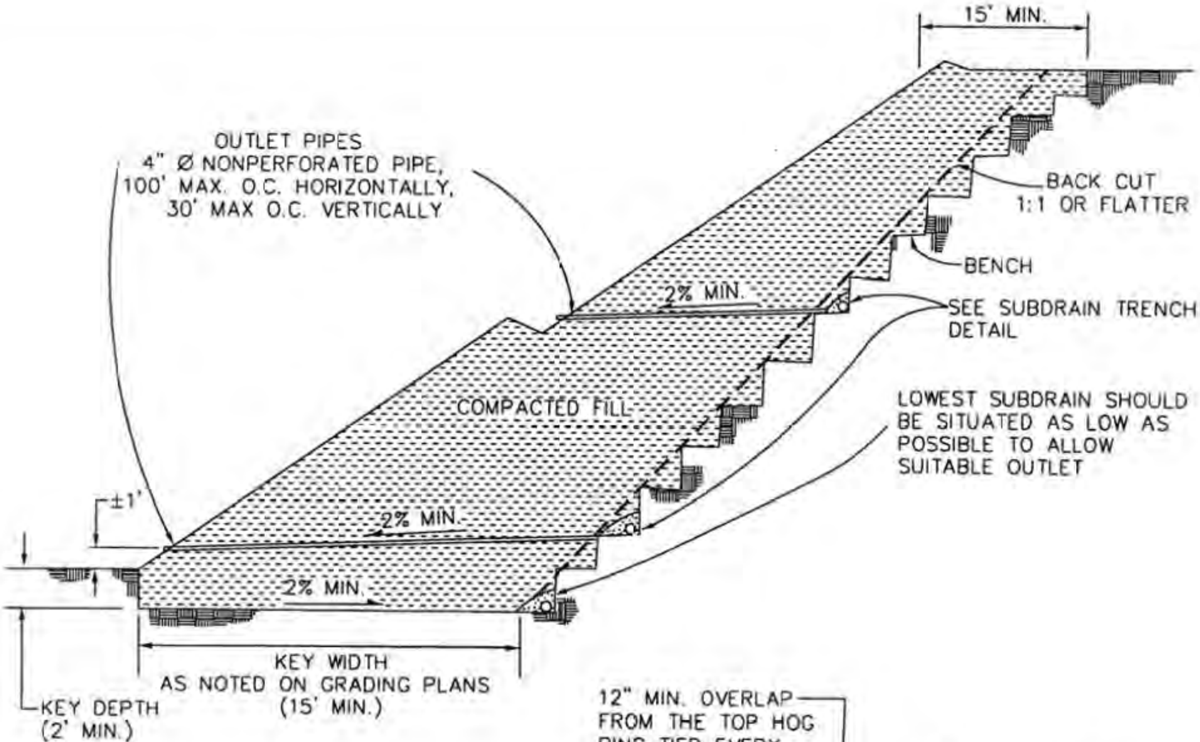


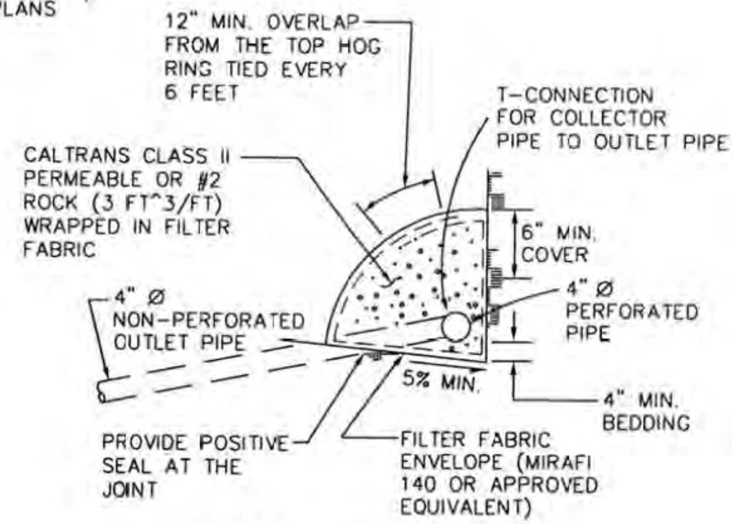
SUBDRAIN DETAIL



DETAIL OF CANYON SUBDRAIN OUTLET

CANYON SUBDRAINS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL C





SUBDRAIN TRENCH DETAIL

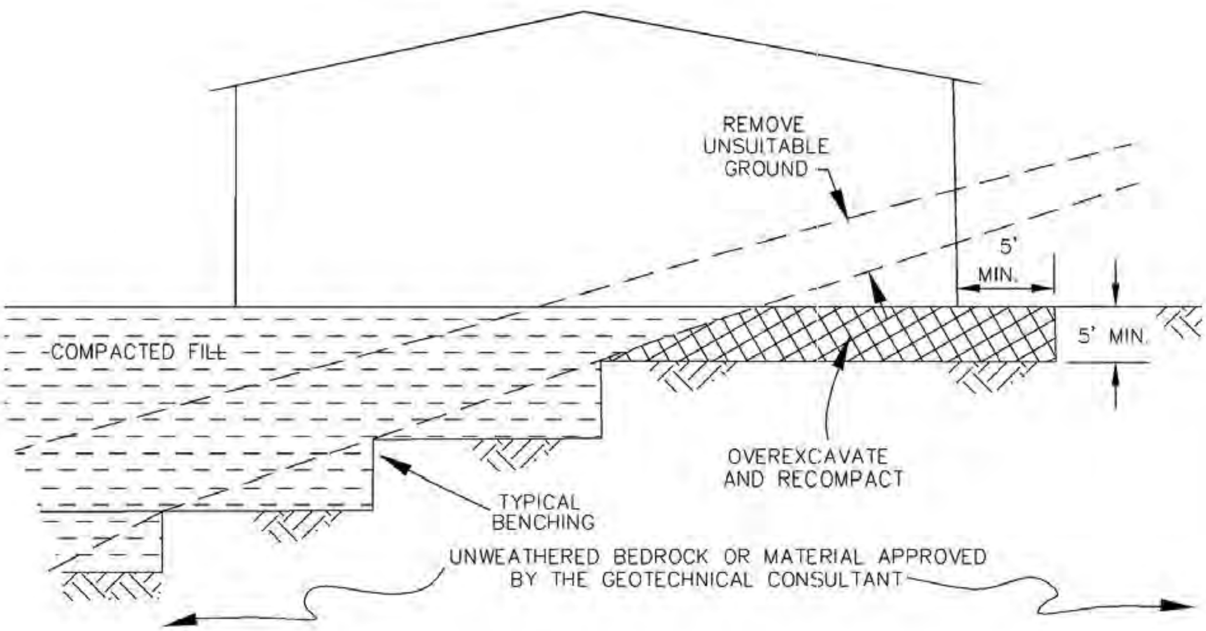
SUBDRAIN INSTALLATION – subdrain collector pipe shall be installed with perforation down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" 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.

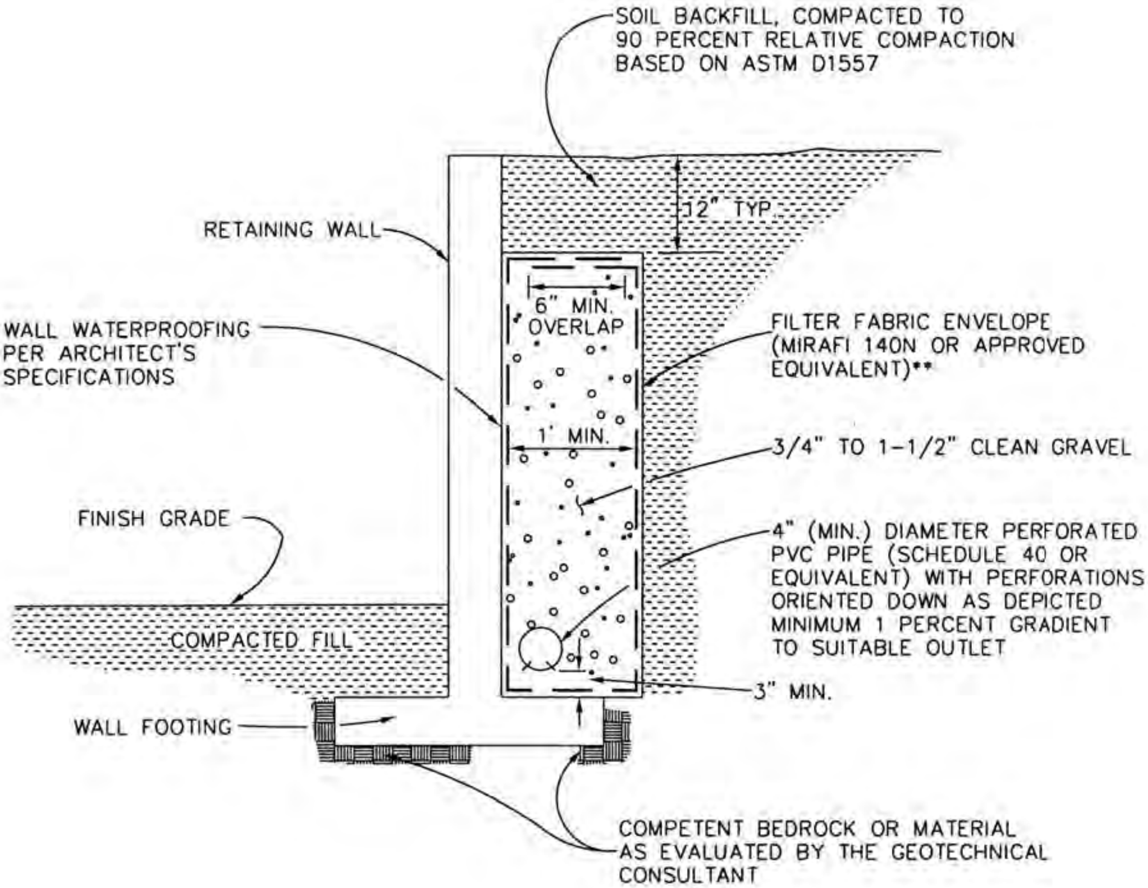
All outlet pipe shall be placed in a trench no wider than twice the subdrain pipe.

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

CUT-FILL TRANSITION LOT OVEREXCAVATION



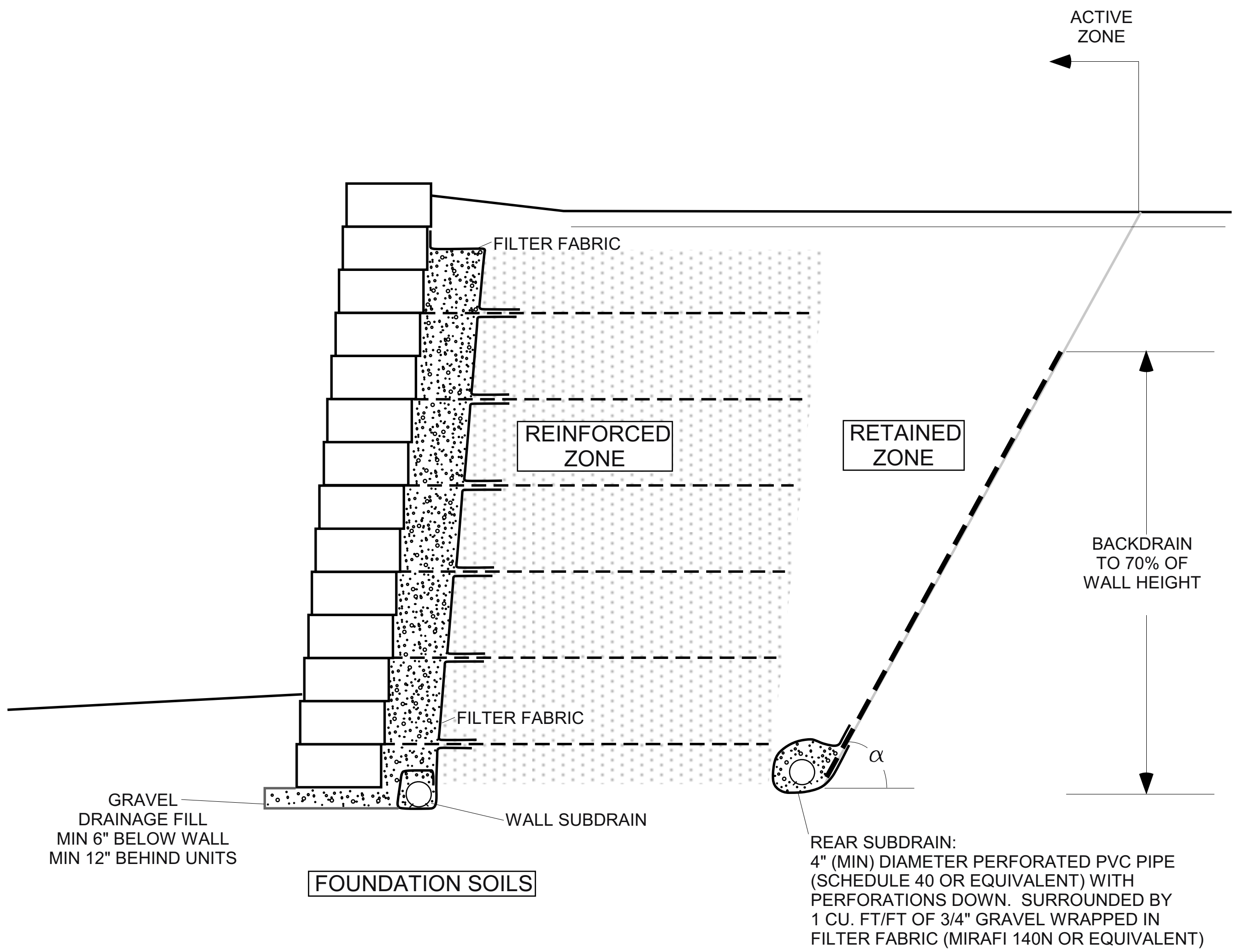
TRANSITION LOT FILLS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL E



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



NOTES:

1) MATERIAL GRADATION AND PLASTICITY

<u>REINFORCED ZONE:</u>

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

<u>GRAVEL DRAINAGE FILL:</u>

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 F

ASFE

Important Information about Your

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.

Geotechnical 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 civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- 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,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- 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. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of 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 equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE THE GEOPROFESSIONAL BUSINESS ASSOCIATION

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