



**UPDATE GEOTECHNICAL INVESTIGATION
PROPOSED SITE IMPROVEMENTS
9534 VIA ZAPADOR
SANTEE, CALIFORNIA**

Prepared For **CITY OF SANTEE**
10601 MAGNOLIA AVENUE
SANTEE, CA 92071

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Project Number 21762

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City of Santee
10601 Magnolia Avenue
Santee, California 92071

Attention: Mr. Antonio Sandoval

**Subject: Update Geotechnical Investigation
Proposed Site Improvements
9534 Via Zapador
Santee, California 92071**

In accordance with your request and authorization, Leighton Consulting, Inc. (Leighton) has conducted an update geotechnical investigation for the proposed site improvements located at the existing City Maintenance Yard and Facility at 9534 Via Zapador in Santee, California. Based on the results of our study, it is our professional opinion that the proposed site improvements are feasible provided the geotechnical recommendations contained in this report are implemented during design and construction. This report provides a summary of the current investigation and general geotechnical conclusions and recommendations for the project.

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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Distribution: Client via email

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Important Information about This Geotechnical-Engineering Report

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

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

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

Understand the Geotechnical-Engineering Services Provided for this Report

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

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

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

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

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

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

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

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

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

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer About Change

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

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

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

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

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

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

This Report’s Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

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

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

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



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1.0 INTRODUCTION

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

1.1 Purpose and Scope

This report presents the results of our update geotechnical investigation for the proposed site improvements located at the existing City Maintenance Yard and Facility at 9534 Via Zapador in Santee, California (Figure 1). We have performed an update geotechnical investigation to identify and evaluate the geologic hazards and significant geotechnical conditions present at the site in order to provide geotechnical recommendations for the proposed site improvements. Our scope of services included:

- Review of available pertinent, published and unpublished geotechnical literature and maps. References cited are listed in Appendix A;
- Field reconnaissance of the existing on-site geotechnical conditions;
- Subsurface exploration consisting of the excavation, logging, and sampling of four (4) hand auger borings and four (4) dynamic cone penetrometer (DCP) soundings. The exploration logs are presented in Appendix B and their approximate locations are shown on Figure 2;
- Laboratory testing of representative soil samples obtained from the subsurface exploration program. Results of these tests are presented in Appendix C;
- Development of a Geotechnical Map (Figure 2) using collected subsurface data;
- Assessment of geologic hazards;
- Development of seismic design parameters based on the 2022 California Building Code (CBC);
- Compilation and analysis of the geotechnical data obtained from the field investigation and laboratory testing; and
- Preparation of this report presenting our findings, conclusions, and geotechnical recommendations with respect to the proposed geotechnical design, site grading, and general construction considerations.

1.2 Site Location and Description

The site is irregular in shape and encompasses approximately 4.4 acres. Based on our review of historical aerial photographs, the site was graded followed by the construction of the existing site improvements sometime between 2006 and 2008. It should be noted that Leighton performed the preliminary geotechnical investigation for the subject site (Leighton, 2004). However, Leighton was not involved with the previous site grading operations. In general, the site is bounded by residential developments to the north and east, a dirt access road followed by an ascending 2:1 or flatter (horizontal:vertical) gradient slope that supports SR 52 to the south, Via Zapador to the southeast, and an irregular parcel of land with drainage improvements the west. Currently, the project site consists of a City Maintenance Yard and Facility comprised of an administration building, a shop/storage building, material storage bins, and a vehicle wash pad/structure. In addition, associated concrete and asphalt concrete paving, gravel paving and graded earth areas, retaining walls, landscaping, and hardscaping are located throughout the site. Site topography generally slopes from the northeast to the southwest with surface elevations ranging from approximately 338 to 335 feet above mean sea level (msl). An aerial image that shows the approximate site limits is presented on Figure 2.

Generalized Site Latitude and Longitude

32.8366 ° N

116.9900 ° W

1.3 Proposed Development

Based on our discussions with you, it is our understanding that the proposed site improvements will consist of construction of a permanent fire station, a decanting station for vector trucks, a vehicle wash station, a steel shade structure, a temporary apparatus bay, and living quarters on the property as presented on Figure 2. Additional improvements at the site may consist of utilities, landscape, and hardscape.

2.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

2.1 Site Investigation

Our subsurface exploration, performed on March 15, 2024, consisted of excavating four hand auger borings (HA-1 through HA-4) and advancing four DCP soundings (DCP-1 through DCP-4) to depths between approximately 5 to 10 feet bgs. The hand auger borings were excavated using conventional hand tools and the DCP soundings were advanced with a Triggs Technologies Wildcat Dynamic Cone Penetrometer using a 35-lb hammer. The purpose of our subsurface exploration was to evaluate the underlying stratigraphy, physical characteristics, and specific engineering properties of the soils located within the area of the proposed improvements.

During the exploration operations, a geologist from our firm prepared geologic logs and collected bulk samples for laboratory testing and evaluation. After logging and field testing, the exploration excavations were backfilled with soil cuttings and the surface was capped to match the existing ground surface. The exploration logs are presented in Appendix B and their approximate location are depicted on the Geotechnical Map (Figure 2).

As part of our study, we reviewed previous geotechnical data, obtained from our preliminary geotechnical investigation, for the subject site (Leighton, 2004). The previous investigation included ten (10) small diameter hollow-stem auger borings, B-1 through B-10. The field exploration logs are provided in Appendix B the locations of the current and previous explorations are also shown on Figure 2.

2.2 Laboratory Testing

Laboratory testing performed on representative soil samples obtained during the subsurface explorations included expansion index and geochemical analysis for corrosion. A discussion of the laboratory tests performed and a summary of the laboratory test results are presented in Appendix C.

Leighton also reviewed and utilized previous laboratory data which included consolidation and direct shear strength parameters, swell/collapse potential, expansion potential, corrosion suite, maximum density, sieve analysis, and R-value testing (Leighton, 2004). These test results are included in Appendix C.

3.0 SUMMARY OF GEOTECHNICAL CONDITIONS

3.1 Geologic Setting

The project area is situated in the Peninsular Ranges Geomorphic Province. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California and varies in width from approximately 30 to 100 miles (Norris and Webb, 1990). The province is characterized by mountainous terrain on the east composed mostly of Mesozoic igneous and metamorphic rocks, and relatively low-lying coastal terraces to the west underlain by late Cretaceous-age, Tertiary-age, and Quaternary-age sedimentary units. Most of the coastal region of the County of San Diego, including the site, occur within this coastal region and are underlain by sedimentary units. Specifically, the site is located within the coastal plain section of the Peninsular Range Geomorphic Province of California, which generally consists of subdued landforms underlain by sedimentary bedrock. The geologic setting is depicted on Figure 3.

3.2 Local Geologic Setting

Based on our subsurface exploration, and review of pertinent geologic literature and maps (Appendix A), the geologic units underlying the site consist of undocumented artificial fill overlying Quaternary-aged Alluvium, which in turn overlies Cretaceous-aged Granitic Rock. The approximate aerial distribution of these units is shown on the Geotechnical Map (Figure 2). A brief description of the geologic units encountered at the site is presented below.

3.2.1 Undocumented Artificial Fill (Afu)

Undocumented fill soils were encountered at the site to a depth of 1.5 to 2.5 feet bgs. Localized deeper unknown fills associated with past development may exist across the site. An as-graded report was not available for our review, and it is assumed that no engineering observations of these fill soils were provided at the time of grading. As encountered in the excavations, the fill soils generally consisted of loose to dense, grayish brown to very dark grayish brown, mottled dark reddish brown and very dark gray, light olive brown, yellowish brown, moist to very moist, clayey and silty sands, and medium stiff to stiff, mottled dark reddish brown and very dark gray, moist, sandy lean clay.

3.2.2 Quaternary-aged Alluvium (Qya)

Alluvial soils were visually observed during our updated geotechnical investigation at depths of 2.5 to 5 feet bgs. Based on our preliminary geotechnical investigation (Leighton, 2004), the alluvium extends to depths ranging between 20 to 28 feet bgs. The alluvial materials consist of medium dense to very dense, damp to wet, gravelly sands and silty sands.

3.2.3 Cretaceous-aged Granitic Rock (Kgr)

Cretaceous-aged Granitic Rock was not encountered during our update geotechnical investigation. However, based on our preliminary geotechnical investigation (Leighton, 2004), this material consists of damp to wet, moderately hard, highly weathered Granitic Rock.

3.3 Surface and Groundwater

No indication of surface water or evidence of surface water ponding was encountered during our update geotechnical investigation at the site. However, surface water may drain as sheet flow across the site during rainy periods.

Groundwater was not encountered during our recent subsurface exploration at the site; however, based on exploration data from our preliminary geotechnical investigation (Leighton, 2004), groundwater was encountered at depths ranging from 13 to 17 feet bgs. It should be noted that groundwater levels may fluctuate with seasonal variations and irrigation and local perched groundwater conditions may exist within sandy layers of the alluvium, and along the geologic contacts between the undocumented artificial fill, alluvium, and Granitic Rock. Nevertheless, based on the above information, we do not anticipate groundwater will be a constraint to the construction of the proposed improvements.

3.4 Engineering Characteristics of On-Site Soils

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

3.4.1 Compressible Soils

Based on the results of our subsurface exploration, the existing undocumented artificial fill materials and underlying alluvial deposits located at the project site are considered compressible. Recommendations for

treatment of these materials during construction are provided later in this report.

3.4.2 Expansion Potential

Expansion Index testing on representative soil samples taken from hand auger boring HA-2 and borings B-6 indicates that the onsite soil materials have a low to moderate potential for expansion. However, higher expansive soils may be encountered during the grading of the proposed site improvements. Geotechnical observation and/or laboratory testing upon completion of the anticipated fine grading operations is recommended to determine the actual expansion potential of finish grade soils on the site.

3.4.3 Soil Corrosivity

A screening of the onsite materials for corrosivity was performed to evaluate their potential effect on concrete and ferrous metals. The corrosion potential was evaluated using the results of laboratory testing on one representative soil sample obtained during our recent subsurface evaluation.

Laboratory testing was performed to evaluate pH, minimum electrical resistivity, and chloride and soluble sulfate content. The sample tested had a measured pH of 7.96 and a measured minimum electrical resistivity of 1,200 ohm-cm. The test results also indicated that the sample had a chloride content of 100 parts per million (ppm) and a soluble sulfate content of less than 150 ppm.

Corrosion testing was also performed during our previous site investigation. The samples tested had measured pH levels of 7.82 to 7.99 and measured minimum electrical resistivities of 1,099 to 1,305 ohm-cm. The test results also indicated that the samples had chloride contents of 360 and 480 parts per million (ppm) and soluble sulfate content 270 and 375 ppm.

3.4.4 Excavation Characteristics

The site is underlain by undocumented fill and alluvial soils which consists of silty to clayey sands with localized zones of gravel and cobble present within the alluvium. With regards to the proposed project, it is anticipated these on-site soils can be excavated with conventional heavy-duty construction equipment. Note that zones of poorly graded and friable sand and gravel/cobble will likely cave or slough during unsupported excavations. Oversize cobble material (typically over 6 inches in maximum dimension) could be present locally. Oversize cobble material should be placed in non-structural areas or hauled off-site.

4.0 SEISMICITY

4.1 Regional Tectonic Setting

The site is located within the Peninsular Ranges Geomorphic Province, which is traversed by several major active faults. The Whittier-Elsinore, San Jacinto, and the San Andreas faults are major active fault systems located east of the site, and the Rose Canyon, Newport-Inglewood (offshore), and Coronado Bank are active faults located west to southwest of the site (Jennings, 2010), see Figure 4. The primary seismic risk to the site area is the Rose Canyon fault zone located approximately 13 miles west of the site (USGS, 2008).

The Rose Canyon fault zone consists predominantly of right-lateral strike-slip faults that extend south-southeast bisecting the San Diego metropolitan area. Various fault strands display strike-slip, normal, oblique, or reverse components of displacement. The Rose Canyon fault zone extends offshore at La Jolla and continues north-northwest subparallel to the coastline. The offshore segments are poorly constrained regarding location and character. South of downtown, the fault zone splits into several splays that underlie San Diego Bay, Coronado, and the ocean floor south of Coronado (Treiman, 1993 and 2000; Kennedy and Clarke, 1999). Portions of the fault zone in the Mount Soledad, Rose Canyon, and downtown San Diego areas have been designated by the State of California (CGS, 2007) as being Earthquake Fault Zones.

4.2 Local Faulting

The California Geologic Survey (CGS, 2007) define a Holocene-active fault as a fault which has “had surface displacement within Holocene time (about the last 11,700 years).” Our review of available geologic literature (Appendix A) indicates that there are no known pre-Holocene or Holocene-active faults transecting the site. The site is also not located within any State mapped Earthquake Fault Zones or County of San Diego mapped fault zones. The nearest active fault is the Rose Canyon fault zone located approximately 13 miles west of the site (USGS, 2008).

4.3 Seismicity

The site is considered to lie within a seismically active region, as is all of Southern California. As previously mentioned above, the Rose Canyon fault zone located approximately 13 miles west of the site is considered the ‘active’ fault having the most significant effect at the site from a design standpoint.

4.4 Seismic Hazards

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

4.4.1 Shallow Ground Rupture

As mentioned above, no pre-Holocene or Holocene-active faults are mapped crossing or projecting toward the site. Due to the absence of faults at the site, surface rupture from faulting is considered low.

4.4.2 Mapped Fault Zones

The site is not located within a State mapped Earthquake Fault Zone (EFZ), nor is it located within a County of San Diego fault zone. As previously discussed, the site is not underlain by known faults.

4.4.3 Site Class

Utilizing 2022 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. It should be noted, per Section 11.4.8 of ASCE 7-16, a ground motion hazard analysis shall be performed in accordance with Section 21.2 for structures having a fundamental period of vibration greater than 0.5s on Site Class D sites where S_1 is greater than or equal to 0.2g. However, although S_1 is greater than 0.2g at the site, it is anticipated that the proposed improvements will have a fundamental period of vibration of less than 0.5s based on our current understanding. Therefore, a site-specific ground motion analysis is assumed to be not required according to ASCE 7-16 Section 11.4.8; however, the project structural engineer needs to confirm this assumption.

4.4.4 Building Code Mapped Spectral Acceleration Parameters

The effect of seismic shaking may be mitigated by adhering to the California Building Code and state-of-the-art seismic design practices of the Structural Engineers Association of California. Provided in Table 1 are the spectral acceleration parameters for the project determined in accordance with the

2022 CBC (CBSC, 2022) and SEAOC/OSHPD Seismic Design Maps Web Application (2022).

Table 1. CBC Mapped Spectral Acceleration Parameters

Site Class	D		
Site Coefficients	F_a	=	1.191
	F_v	=	Null
Mapped MCE Spectral Accelerations	S_s	=	0.773g
	S_1	=	0.284g
Site Modified MCE Spectral Accelerations	S_{MS}	=	0.920g
	S_{M1}	=	Null
Design Spectral Accelerations	S_{DS}	=	0.614g
	S_{D1}	=	Null
Transitional Period	F_v	=	2.032g
	S_{M1^*}	=	0.577g
	S_{D1^*}	=	0.385g
	$T_s = S_{D1}/S_{DS}$	=	0.627s

*Site-specific ground motion hazard analysis is required for determination of S_{M1} and S_{D1} for use in seismic design. The S_{D1} value is only presented for the purposes of calculating T_s as per Supplement 1 to ASCE 7-16 (ASCE, 2018).

Utilizing ASCE Standard 7-16, in accordance with Section 11.8, the following additional parameters for the peak horizontal ground acceleration are associated with the Geometric Mean Maximum Considered Earthquake (MCE_G). The mapped MCE_G peak ground acceleration (PGA) is 0.331g for the site. For a Site Class D, the F_{PGA} is 1.269 and the mapped peak ground acceleration adjusted for Site Class effects (PGA_M) is 0.42g for the site.

Since the mapped spectral response at 1-second period is less than 0.75g, then all structures subject to the criteria in Section 1613.2.5 of the 2022 CBC are assigned Seismic Design Category D.

4.4.5 Liquefaction Potential and Dynamic Settlement

Liquefaction and dynamic settlement of soils can be caused by strong vibratory motion due to earthquakes. Granular soils tend to densify when subjected to shear strains induced by ground shaking during earthquakes. Research and historical data indicate that loose granular soils underlain by

a near surface groundwater table are most susceptible to liquefaction, while the most clayey materials are not susceptible to liquefaction. Liquefaction is characterized 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 at the ground surface by settlement and, possibly, sand boils where insufficient confining overburden is present over liquefied layers. Where sloping ground conditions are present, liquefaction-induced instability can result.

Based on the relatively dense character of the underlying silty fine to coarse sands and gravelly sands comprising the alluvial soils beneath the groundwater elevation, it is our opinion that the potential for liquefaction and seismic related settlement across the site is low.

4.4.6 Lateral Spread

Empirical relationships have been derived (Youd et al., 1999) to estimate the magnitude of lateral spread due to liquefaction. These relationships include parameters such as earthquake magnitude, distance of the earthquake from the site, slope height and angle, the thickness of liquefiable soil, and gradation characteristics of the soil.

The susceptibility to earthquake-induced lateral spread is considered to be low for the site because of the low susceptibility to liquefaction and relatively level ground surface in the site vicinity.

4.4.7 Tsunamis and Seiches

Based upon the Tsunami Hazard Area Map of the County of San Diego (CGS, 2022), the site is not located within a tsunami inundation area. In addition, based on the generally strike-slip character of off-shore faulting and proposed elevation of the site with respect to sea level, the possibility of seiches and/or tsunamis is considered to be nil.

4.5 **Landslides**

Several formations within the San Diego region are particularly prone to landsliding. These formations generally have high clay content and mobilize when they become saturated with water. Other factors, such as steeply dipping bedding that project out of the face of the slope and/or the presence of fracture planes, will also increase the potential for landsliding.

No landslides or indications of deep-seated landsliding were indicated at the site during our field exploration or our review of available geologic literature, topographic maps, and stereoscopic aerial photographs. Furthermore, our field reconnaissance and the local geologic maps indicate the site is generally underlain by favorable oriented geologic structure consisting of Granitic Rock. Therefore, the potential for significant landslides or large-scale slope instability at the site is considered low.

4.6 Flood Hazard

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2012); the site is located within Zone X, areas determined to be outside the 0.2% annual chance floodplain. However, it should be noted that the the site is located approximately 350 feet east of Zone A, special flood hazard areas subject to inundation by the 1% annual chance flood (100-year flood). In addition, based on our review of dam inundation and topographic maps, the site is not located within a dam inundation area.

5.0 CONCLUSIONS

Based on the results of our investigation of the site, it is our opinion that the proposed site improvements are considered feasible from a geotechnical standpoint, provided the following conclusions and recommendations are incorporated into the project plans and specifications. The following is a summary of the significant geotechnical factors that we expect may affect development of the site.

- As the site is located in the seismically active southern California area, all structures should be designed to tolerate the dynamic loading resulting from seismic ground motions.
- The site is not transected by either Holocene-active or pre-Holocene faults.
- The peak horizontal ground acceleration associated with the Maximum Considered Earthquake Ground Motion is 0.331g. The peak horizontal ground acceleration associated with the PGA_M is 0.42g.
- Exceptional geologic hazards are not anticipated to impact the site or the proposed site development.
- Due to the relatively dense characteristics of the onsite alluvial materials beneath the groundwater table, the potential for liquefaction is considered low.
- Based on our field observations and laboratory testing, the onsite soil materials possess a low potential for expansion and appear to be suitable for reuse as compacted fill provided they are moisture conditioned, relatively free of organic material, debris, and rock fragments larger than 6-inches in maximum dimension.
- Based on laboratory testing and site mapping, the site materials possess a low to moderate expansion potential. It is possible that higher expansion materials may be encountered in locations not explored.
- Based on our experience with similar sites in the area and the results of our subsurface investigation of the site, excavations within the underlying soil materials may encounter zones of poorly graded cohesionless and friable sands that will likely cave or slough during site excavation. Care in these cases should be exercised which may include the excavation of shorter open-face segments.

-
- Groundwater was encountered during our previous site investigation in 2003 in borings B-1 through B-10 and ranged in depth from 10 to 17 feet bgs. However, groundwater is not anticipated to be encountered during site excavation and construction except as possible seepage during/after episodes of precipitation or in areas of irrigation.
 - The undocumented fill and alluvial soils onsite are potentially compressible. These soils are not considered suitable for support of engineered fill soils or site improvements in their present condition. Recommendations for remedial grading are provided in Section 6.1.3 of this report.
 - If import soils are planned, the soils should be granular in nature, and have an expansion index less than 50 (per ASTM Test Method D 4829) and have a low corrosion impact to the proposed improvements.
 - Although Leighton does not practice corrosion engineering, laboratory test results indicate the soils present on the site have a moderate potential for sulfate attack on normal concrete. In addition, the onsite soils are considered to have a high corrosive potential to buried uncoated ferrous metal. A corrosion consultant may be consulted for additional recommendations.
 - Based on the results of our geotechnical study, we do not recommend the practice of surface water infiltration into near surface soils at the site due to existing settlement sensitive improvements and the dense nature of the underlying Tonalite.

6.0 PRELIMINARY RECOMMENDATIONS

6.1 Earthwork

We anticipate that earthwork at the site will consist of site preparation, removal of potentially compressible/collapsible soil, and placement of properly 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 D. In case of conflict, the following recommendations supersede those in Appendix D.

6.1.1 Site Preparation

Prior to grading, all areas to receive structural fill or engineered structures should be cleared of surface and subsurface obstructions, including any existing debris and undocumented or loose fill soils, and stripped of vegetation. Removed vegetation and debris should be properly disposed off-site. All areas to receive fill and/or other surface improvements should be scarified to a minimum depth of 6 inches, brought to above-optimum moisture conditions, and recompact to at least 90 percent relative compaction (based on ASTM Test Method D 1557).

6.1.2 Excavations and Oversized Material

Excavations of the onsite materials may generally be accomplished with conventional heavy-duty earthwork equipment. Surficial loose fill soils along with friable underlying sands present on site can be expected to cave during trenching and excavation operations. In accordance with OSHA requirements, excavations deeper than 5 feet should be shored or be laid back in accordance with Section 6.2 if workers are to enter such excavations.

6.1.3 Removal of Compressible Soils

Potentially compressible undocumented fill and alluvial soils at the site may settle as a result of wetting or settle under the surcharge of engineered fill and/or structural loads supported on shallow foundations.

Estimated removal depths of the surficial soils beneath the proposed improvements is approximately 6 feet below the existing ground surface and should extend 5 feet beyond the foundation footprint, if possible. The bottom of all removals should be evaluated by a Certified Engineering Geologist to confirm conditions are as anticipated.

In areas of proposed pavements, hardscape and landscaping features, removals should be performed to a depth of at least 3 feet below the proposed subgrade or existing site topography, whichever is deeper, and extend at least 3 feet beyond the limits of the proposed improvements, where possible. Isolated deeper removals may be necessary. The bottom of these removals should be scarified and moisture conditioned to a minimum depth of 6 inches and evaluated by a Certified Engineering Geologist to confirm conditions are as anticipated.

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

6.1.4 Engineered Fill

The onsite soils are generally suitable for use as compacted fill provided they are free of organic material, debris, and rock fragments larger than 6 inches in maximum dimension. All fill soils should be brought to at least 2 percent optimum moisture conditions (i.e., depending on the soil types) and compacted in uniform lifts to at least 90 percent relative compaction based on laboratory standard ASTM Test Method D 1557, 95 percent for wall backfill soils or if used for structural purposes (such as to support a footing, wall, etc.). The optimum lift thickness required to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in lifts not exceeding 8 inches in thickness.

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

6.1.5 Earthwork Shrinkage/Bulking

Based on the results of our investigation and our professional experience with similar projects in the general vicinity of the site, we have estimated bulking and shrinkage of the on-site soils. The volume change of excavated onsite materials upon recompaction as fill is expected to vary with materials and location. Typically, the surficial soils and bedrock materials vary significantly in natural and compacted density, and therefore, accurate earthwork shrinkage/bulking estimates cannot be determined. However, the following factors (based on the results of our investigation, geotechnical analysis and professional experience on nearby sites) are provided as guideline estimates. If possible, we suggest an area where site grades can be adjusted (during the later portion of the site grading operations) be provided as a balance area.

Artificial Fill	5 percent shrinkage
Alluvium	5 to 10 percent shrinkage

6.1.6 Import Soils

If import soils are used, the soil should be granular in nature, and have an expansion index less than 50 (per ASTM Test Method D 4829) and have a low corrosion impact to the proposed development. Import soils and/or the borrow site location should be evaluated by the geotechnical consultant prior to import.

6.1.7 Trench Backfill

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

6.1.8 Expansive Soils and Selective Grading

Based on our laboratory testing and observations, we anticipate the onsite soil materials possess a low expansion potential (Appendix C). Although not anticipated, should an abundance of highly expansive materials be

encountered, selective grading may need to be performed. In addition, to accommodate conventional foundation design, the upper 5 feet of materials within the building pad and 5 feet outside the limits of the building foundation should have a very low to low expansion potential ($EI < 50$).

6.2 Temporary Excavations

Sloping excavations may be utilized when adequate space allows. Based on the results of our investigation, we provide the following preliminary recommendations for sloped excavations in surficial soils or bedrock without seepage conditions.

Table 2. Temporary Slope Ratios

Excavation Depth (feet)	Maximum Slope Ratio In Fill Soils	Maximum Slope Ratio In Alluvial Soils
0 to 5	1:1 (Horizontal to Vertical)	1:1 (Horizontal to Vertical)
5 to 10	1.5:1 (Horizontal to Vertical)	1:1 (Horizontal to Vertical)

The above values are based on the assumption that no surcharge loading or equipment will be placed within 10 feet of the top of slope. Care should be taken during excavation adjacent to the existing structures so that undermining does not occur. An OSHA “competent person” should observe the slope on a daily basis for signs of instability. An engineered design will be needed where groundwater or surcharge conditions are present.

6.3 Shallow Spread Footing Considerations

The proposed buildings may be constructed with conventional foundations founded on properly compacted fill. Foundations and slabs should be designed in accordance with structural considerations and the following recommendations. These recommendations assume that the soils encountered within 5 feet of pad grade have a very low to low potential for expansion ($EI < 50$). If more expansive materials are encountered and selective grading cannot be accomplished, revised foundation recommendations may be necessary.

6.3.1 Shallow Conventional Foundation

The proposed buildings may be supported by conventional continuous or isolated spread footings, founded in compacted fill. 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). For footing founding in compacted fill soils, an allowable capacity increase of 500 psf for every 6 inches of additional embedment may be used, not exceeding 3,500 psf. 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 and have minimum reinforcement of four (4) No. 5 reinforcing bars (two top and two bottom).

6.3.2 Foundation Setback

We recommend a minimum horizontal setback distance from the face of slopes for all structural foundations, footings, and other settlement-sensitive structures as indicated on the Table 3 below. This distance is measured from the outside bottom edge of the footing, horizontally to the slope or retaining wall face, and is based on the overall slope or wall height. The foundation setback distance may be revised by the geotechnical consultant on a case-by-case basis if the geotechnical conditions are different than anticipated.

Table 3. Minimum Foundation Setback from Slope Faces

Slope Height	Minimum Recommended Foundation Setback
less than 5 feet	5 feet
5 to 15 feet	7 feet

Please note that the soils within the structural setback area possess poor lateral stability, and improvements (such as retaining walls, sidewalks, fences, pavements, etc.) constructed within this setback area may be subject to lateral movement and/or differential settlement. Potential distress to such improvements may be mitigated by providing a deepened footing or a pier and grade beam foundation system to support the improvement.

In addition, open or backfilled utility trenches that parallel or nearly parallel structure footings should not encroach within an imaginary 2:1 (horizontal to vertical) downward sloping line starting 9 inches above the bottom edge of the footing and should also not be located closer than 18 inches from the face of the footing.

Where pipes cross under footings, the footings should be specially designed. Pipe sleeves should be provided where pipes cross through footings or footing walls and sleeve clearances should provide for possible footing settlement.

6.3.3 Floor Slabs

Slab-on-grade floors associated with conventional foundations, 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 or No. 4 rebars at 24 inches on center (each way). The slab should be underlain by 2-inch layer of clean sand (S.E. greater than 30). A moisture barrier (10-mil non-recycled plastic sheeting) should be placed below the sand layer if reduction of moisture vapor up through the concrete slab is desired (such as below equipment, living/office areas, etc.), which is in turn underlain by an additional 2-inches of clean sand. If applicable, slabs should also be designed for the anticipated traffic loading using a modulus of subgrade reaction of 140 pounds per cubic inch. All waterproofing measures should be designed by the project architect.

6.3.4 Settlement

For conventional footings, the recommended allowable-bearing capacity is based on a maximum total and differential static settlement of 1 inch and $\frac{3}{4}$ -inch, respectively. Since settlements are a function of footing size and contact bearing pressures, some differential settlement can be expected where a large differential loading condition exists. However, for most cases, differential settlements are considered unlikely to exceed $\frac{3}{4}$ -inch.

6.4 **Lateral Earth Pressures and Retaining Wall Design**

Table 4 presents the lateral earth pressure values for level or sloping backfill for walls backfilled with free draining import soils of low expansion potential (less than

50 per ASTM D 4829) above the groundwater table. Soils used to backfill retaining walls should be classified as one of the following types according to ASTM D 2487: GW, GP, GM, GC, SW, SP, or SM. These backfill soils should be used within horizontal distance behind the wall equal to one-half the wall height.

Table 4. Static Equivalent Fluid Weight (pcf)

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

Walls up to 10 feet in height should be designed for the applicable pressure values provided above. If conditions other than those covered herein are anticipated, the equivalent fluid pressure values should be provided on an individual case-by-case basis by the geotechnical engineer. A surcharge load for a restrained or unrestrained wall resulting from automobile traffic may be assumed to be equivalent to a uniform lateral pressure of 75 psf which is in addition to the equivalent fluid pressure given above. For other uniform surcharge loads, a uniform pressure equal to $0.35q$ should be applied to the wall. The wall pressures assume walls are backfilled with free draining materials and water is not allowed to accumulate behind walls. A typical drainage design is contained in Appendix E. Wall backfill should be compacted by mechanical methods to at least 90 percent relative compaction (based on ASTM D 1557). If foundations are planned over the backfill, the backfill should be compacted to 95 percent. Wall footings should be designed in accordance with the foundation design recommendations and reinforced in accordance with structural considerations. For all retaining walls, we recommend a minimum horizontal distance from the outside base of the footing to daylight as outlined in Section 6.3.2.

Lateral soil resistance developed against lateral structural movement can be obtained from the passive pressure value provided above. Further, for sliding resistance, the friction coefficient of 0.35 may be used at the concrete and soil interface. This value may be increased by one-third when considering loads of short duration including wind or seismic loads. The total resistance may be taken

as the sum of the frictional and passive resistance provided that the passive portion does not exceed two-thirds of the total resistance.

To account for potential redistribution of forces during a seismic event, retaining walls providing lateral support where exterior grades on opposite sides differ by more than 6 feet fall under the requirements of 2022 CBC Section 1617A.1.11 and/or ASCE 7-16 Section 15.6.1 and should also be analyzed for seismic loading. For that analysis, an additional uniform lateral seismic pressure equal to 10H pounds per square foot should be considered for the design of restrained retaining walls with level backfill, where H is the height of the wall in feet. This increment should be added to the unfactored static active earth pressure to obtain the total seismic earth pressure on the wall. For restrained walls the value should be increased by 150%.

6.5 Geochemical Considerations

Concrete in direct contact with soil or water that contains a high concentration of soluble sulfates can be subject to chemical deterioration commonly known as “sulfate attack.” Soluble sulfate results (Appendix C) indicated a moderate exposure potential for sulfate attack on normal concrete. Test results by others indicate that sulfate contents of the subsurface soils are categorized as having a sulfate exposure Class “S1” utilizing ACI 318R-14 Table 19.3.1.1. Concrete in contact with earth materials should be designed in accordance with Section 4 of ACI 318-14 (ACI, 2014) Table 19.3.2.1.

Minimum resistivity and pH tests were performed on representative samples of subgrade soils (Appendix C). Based on laboratory test results, the site soils are considered highly corrosive to buried uncoated metal conduits (Caltrans, 2021). A corrosion consultant may be contacted for further recommendations.

6.6 Concrete Flatwork

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

Control joints should be provided at a distance equal to 24 times the slab thickness in inches, not to exceed 12 feet. Expansion joints should be incorporated where paving abuts a vertical surface, where paving changes direction and at 30 feet maximum spacing. Joints should be laid out so as to create square or nearly square areas.

6.7 Preliminary Pavement Design

Based on R-value data obtained from our preliminary geotechnical investigation (Leighton, 2004), we have utilized an R-value of 6 or less for preliminary pavement design. Actual subgrade R-value results should be verified during grading and an adjustment made to the base thicknesses as appropriate. If materials with lower R-value are placed as subgrade in proposed pavement areas, increased base thickness will be necessary. Flexible pavement sections have been evaluated in general accordance with the Caltrans method for flexible pavement design. The recommended flexible pavement section for this condition is given in Table 5 below:

Table 5. Preliminary Pavement Sections

Assumed Traffic Index (TI)	Asphalt Concrete (inches)	Aggregate Base (inches)
4.5	3.0	8.0
5.0	4.0	8.0
7.0	4.0	16.0

Prior to placement of the aggregate base, the upper 12 inches of subgrade soils should be scarified, moisture-conditioned to at least optimum moisture content and compacted to a minimum 95 percent relative compaction based on ASTM Test Method D 1557.

Class 2 Aggregate Base or Crushed Aggregate Base should then be placed and compacted at a minimum 95 percent relative compaction in accordance with ASTM Test Method D 1557. The aggregate base material (AB) should be a maximum of 6 inches thick below the curb and gutter and extend a minimum of 6 inches behind the back of the curb. The AB should conform to and be placed in accordance with the approved grading plans, and latest revision of the Standard Specifications Public Works Construction (Greenbook).

The Asphalt Concrete (AC) material should conform to Caltrans Standard Specifications, Sections 39 and 92, with a Performance Grade (PG) of 64-10, and City of Santee requirements. The placement of the AC should be in accordance with the approved grading plans, Section 203-6 of the “Greenbook” Standard Specifications for Public Works Construction, and City of Santee requirements.

For trash truck aprons, we recommend a full depth of Portland Cement Concrete section of 7 inches with No. 4 bars at 24 inches on center, each way steel and crack-control joints as designed by the project civil or structural engineer. We recommend that jointed sections be as nearly square as possible.

6.8 Surface Drainage and Erosion

Surface drainage should be controlled at all times and carefully taken into consideration during landscaping and construction of site improvements. The proposed development should have appropriate drainage systems to collect roof runoff. Positive surface drainage should be provided to direct surface water away from the structures toward the street or suitable drainage facilities. Planters should be designed with provisions for drainage to the storm drain. Ponding of water adjacent to structures or pavements should be avoided.

The impact of heavy irrigation or inadequate runoff gradient can create perched water conditions, resulting in seepage or shallow ground water conditions where previously none existed. Maintaining adequate surface drainage and controlled irrigation will significantly reduce the potential for nuisance-type moisture problems. To reduce differential earth movements such as heaving and shrinkage due to change in moisture content of foundation soils, which may cause distress to structures and improvements, moisture content of the soils surrounding the improvements should be kept as relatively constant as possible.

All area drain inlets should be maintained and kept clear of debris in order to function properly. In addition, landscaping should not cause any obstruction to site drainage.

6.9 Construction Observation

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

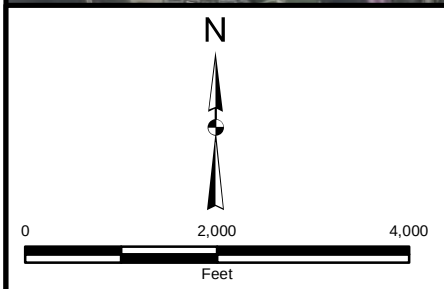
6.10 Plan Review

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

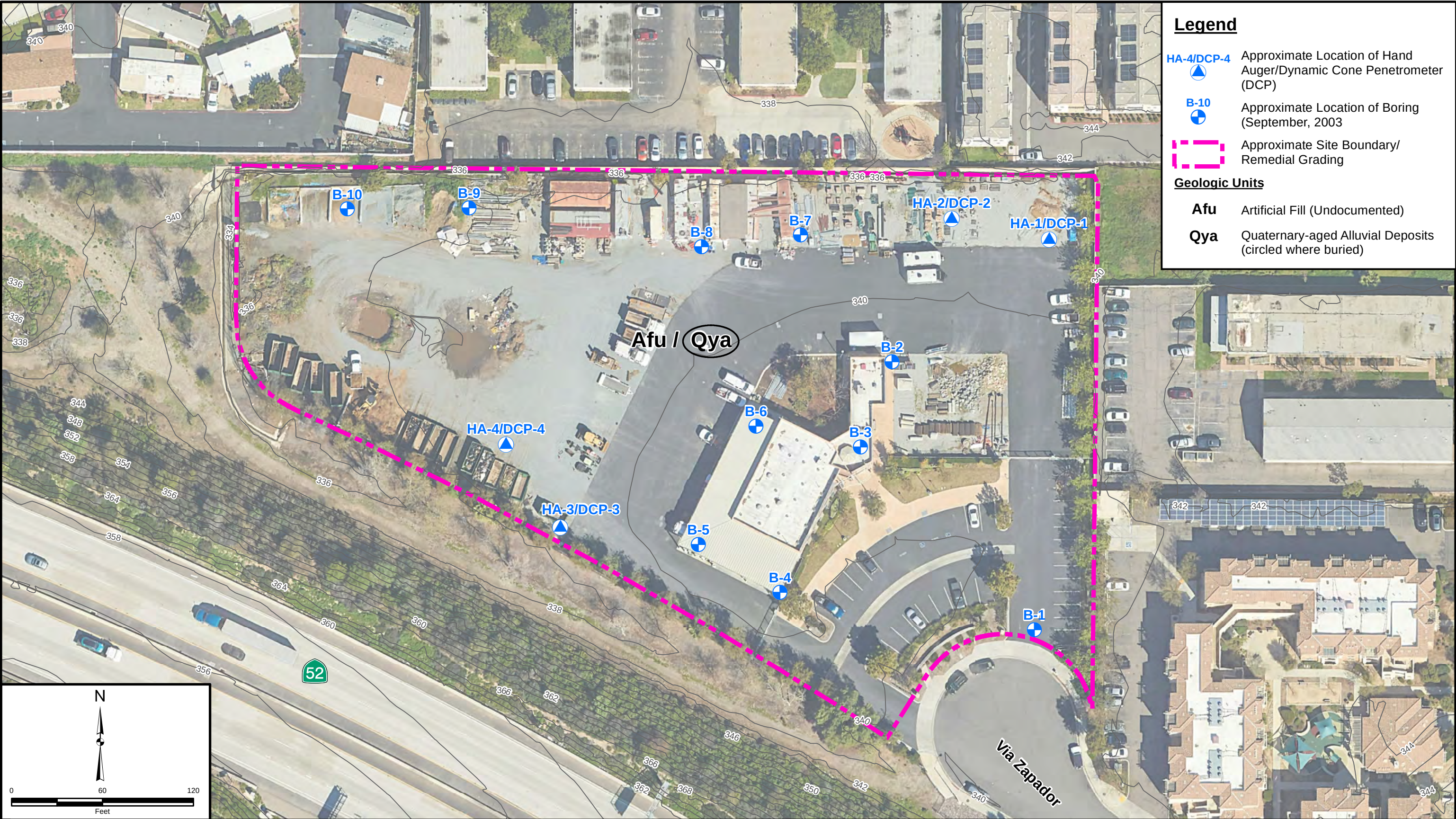
7.0 LIMITATIONS

The findings, conclusions and recommendations in this report are based in part upon data that were obtained from widely spaced subsurface investigations and limited geotechnical analysis. 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 review final grading plans and to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site.

FIGURES



Project: 21762	Eng/Geol: SAN/TCS	SITE LOCATION MAP City of Santee Maintenance Yard and Facility Improvements 9534 Via Zapador Santee, CA 92071	FIGURE 1
Scale: 1" = 2,000'	Date: March 2024		
Reference: © 2024 Microsoft Corporation © 2024 Maxar ©CNES (2024) Distribution Airbus Author: (mmurphy)			

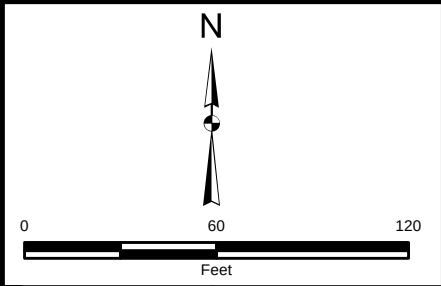


Legend

- HA-4/DCP-4** Approximate Location of Hand Auger/Dynamic Cone Penetrometer (DCP)
- B-10** Approximate Location of Boring (September, 2003)
- Approximate Site Boundary/ Remedial Grading

Geologic Units

- Afu** Artificial Fill (Undocumented)
- Qya** Quaternary-aged Alluvial Deposits (circled where buried)

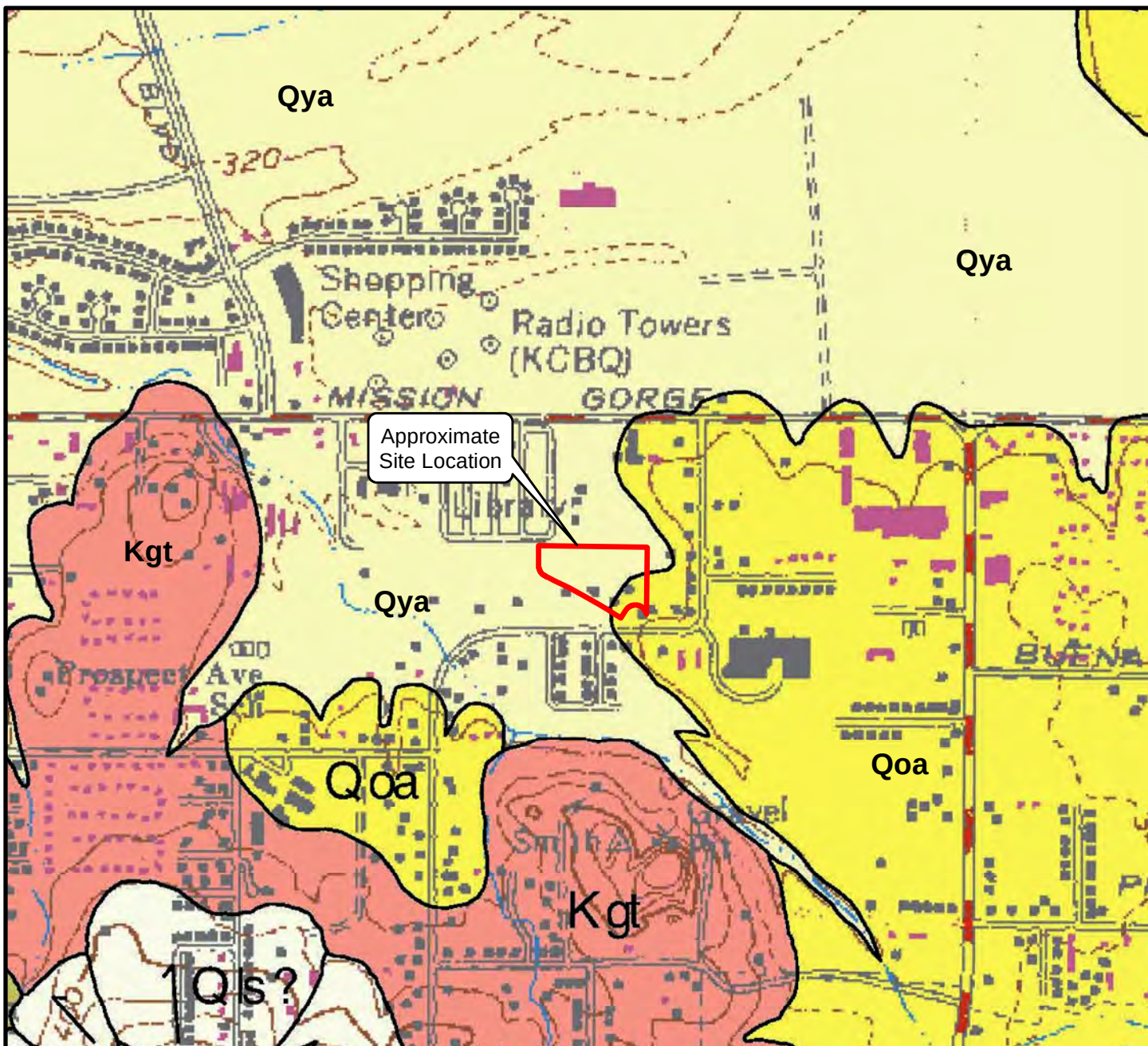


Project: 21762	Eng/Geol: SAN/TCS
Scale: 1" = 60'	Date: April 2024
Base Map: Google Earth, 2024.	
Author: (mmurphy)	

GEOTECHNICAL MAP
City of Santee
Maintenance Yard and Facility Improvements
9534 Via Zapador
Santee, CA 92071

FIGURE 2





Legend

- Qya** Holocene alluvial deposits unconsolidated to poorly consolidated silt, clay, sand and gravel. Includes sediments along small, active drainage channels.
- Qls** Landslide deposits (Holocene to Pleistocene). Landslide slump and rock fall deposits. On map, the deposit is depicted by landslide arrows
- Qoa** Late Pleistocene alluvial deposits; moderately consolidated, poorly-sorted flood plain deposits consisting of gravelly, sandy silt and clay.
- Kgt** Tonalite (Cretaceous); includes some granodiorite and quartz diorite; medium-grained; generally dark colored and severely weathered.

Project: 21762

Eng/Geol: SAN/TCS

Scale: 1" = 800'

Date: April 2024

Base Map: Geologic Map of the El Cajon 7.5' Quadrangle
San Diego County, California by Siang S. Tan, 2002.

Author: (mmurphy)

GEOLOGIC MAP

City of Santee

Maintenance Yard and Facility Improvements
9534 Via Zapador
Santee, CA 92071

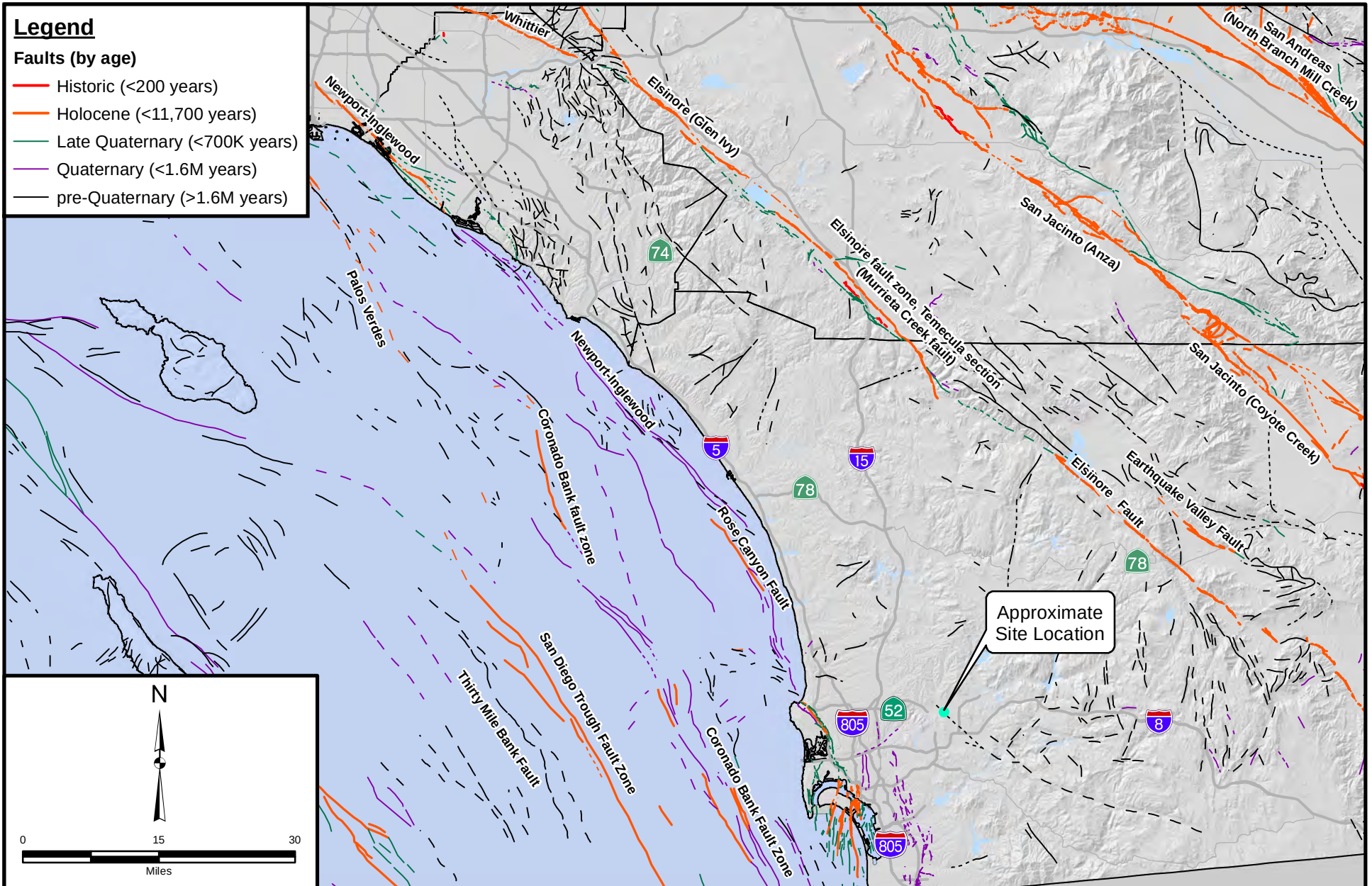
FIGURE 3



Legend

Faults (by age)

- Historic (<200 years)
- Holocene (<11,700 years)
- Late Quaternary (<700K years)
- Quaternary (<1.6M years)
- pre-Quaternary (>1.6M years)



Project: 21762

Eng/Geol: SAN/TCS

Scale: 1" = 15 miles

Date: March 2024

Faults: Bryant, Bryant CGS 2010

Author: Leighton Geomatics (mmurphy)

REGIONAL FAULT MAP

City of Santee
Maintenance Yard and Facility Improvements
9534 Via Zapador
Santee, CA 92071

FIGURE 4



APPENDIX A

References

APPENDIX A

References

- American Concrete Institute, 2014, Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary.
- Associated Society of Civil Engineers (ASCE), 2016, ASCE/SEI 7-16, Minimum Design Loads for Buildings and Other Structures.
- California Building Standards Commission (CBSC), 2022, California Building Code, Volumes 1 and 2.
- Caltrans, 2021, Corrosion Guidelines, Version 3.2, May 2021.
- California Geologic Survey (CGS), 2000, Digital Images of Official Maps of Alquist-Priolo Earthquake Fault Zones of California, Southern Region, DMG CD 2000-02.
- _____, 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California Special Public 117a.
- _____, 2013, Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Building – Note 48, dated October.
- _____, 2018, Special Publication 42, Earthquake Fault Zones, revised 2018.
- FEMA, 2012, Flood Insurance Map, Panel 1653 of 2375, dated May 16, 2012.
- Graehl, N.G., and Wilson, R.I., 2022. Tsunami Hazard Area Map, San Diego County; produced by the California Geological Survey and the California Governor's Office of Emergency Services; dated 2022, displayed at multiple scales.
- Jennings, 2010 Fault Activity Map of California, Geological Survey 150th Anniversary, 1:750,000 scale.
- Kennedy, M.P., 1975, Geology of the San Diego Metropolitan area, California, California, California Division of Mines and Geology, Bulletin 200.
- Kennedy, M.P., and Tan, S.S., 2008, Geologic Map of the San Diego Quadrangle, California, California Geologic Survey, 1:100,000 scale.

Leighton and Associates, Inc., 2004, Geotechnical Investigation, Proposed Santee City Maintenance Yard/Facility, Santee, California, Leighton Project No. 040986-001, dated January 28, 2004.

Norris, R.M., and Webb, R.W., 1990, Geology of California, Second Edition: John Wiley & Sons, Inc.

SEAOC, 2024, USGS Seismic Design Data via SEAOC/OSHPD Seismic Design Maps Web Application, accessed March 20, 2024, at <https://seismicmaps.org>

Tan, S.S., 2002, Geologic Map of the El Cajon 7.5' Quadrangle, San Diego County, California, 2002.

United States Geologic Survey, 2008, National Seismic Hazard Maps – Source Parameters, Interactive Fault Map.

Youd, T. L., Hanson C. M., and Bartlett, S. F., 1999, Revised MLR Equations for Predicting Lateral Spread Displacement, Proceedings of the 7th U.S.-Japan Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures Against Soil Liquefaction, November 19, 1999, pp. 99-114.

APPENDIX B
Exploration Logs

GEOTECHNICAL BORING LOG HA-1

Project No.	21762
Project	Proposed Santee City Yard/Facility Improvements
Drilling Co.	N/A
Drilling Method	Hand Auger
Location	See Figure 2

Date Drilled	3-15-24
Logged By	CA
Hole Diameter	3.25"
Ground Elevation	337' msl
Sampled By	CA

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0	N S							<i>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.</i>	
				B-1 (0.5'-5')				GM	UNDOCUMENTED ARTIFICIAL FILL (Afu) @ 0-0.5': Silty GRAVEL with SAND, loose, very dark grayish brown (10YR, 3/2), very moist, fine to medium SAND, fine gravel.	
								CL	@ 0.5': Sandy Lean CLAY, medium stiff to stiff, mottled dark reddish brown (2.5YR, 3/3), very dark gray (5YR, 3/1), moist, fine to medium SAND, low to medium plasticity, trace organics.	
335								SC	@ 2': Clayey SAND, medium dense, mottled dark reddish brown (5YR, 2.5/2), very dark gray (5YR, 3/1), moist, fine to medium SAND, trace organics.	
								SM	QUATERNARY ALLUVIUM (Qya) @ 3.5': Silty SAND, dense, light olive brown (2.5Y, 5/3), moist, fine to medium SAND.	
	5								Total Depth = 5 Feet (bgs) No Groundwater or Seepage Encountered Backfilled on 3/15/24	
330										

WILDCAT DYNAMIC CONE LOG

Page 1 of 1

Leighton

PROJECT NUMBER: 21762
DATE STARTED: 03-15-2024
DATE COMPLETED: 03-15-2024

HOLE #: DCP-1
CREW: CA
PROJECT: Proposed Santee City Yard/Facility Improvements
ADDRESS: 9534 Via Zapador
LOCATION: Santee, CA 92071

SURFACE ELEVATION: 337' (msl)
WATER ON COMPLETION: N/A
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm ²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY	
					NON-COHESIVE	COHESIVE
-	9	40.0		11	MEDIUM DENSE	STIFF
-	9	40.0		11	MEDIUM DENSE	STIFF
- 1 ft	5	22.2		6	LOOSE	MEDIUM STIFF
-	5	22.2		6	LOOSE	MEDIUM STIFF
-	6	26.6		7	LOOSE	MEDIUM STIFF
- 2 ft	13	57.7		16	MEDIUM DENSE	VERY STIFF
-	14	62.2		17	MEDIUM DENSE	VERY STIFF
-	14	62.2		17	MEDIUM DENSE	VERY STIFF
- 3 ft	19	84.4		24	MEDIUM DENSE	VERY STIFF
- 1 m	20	88.8		25	MEDIUM DENSE	VERY STIFF
-	33	127.4		25+	DENSE	HARD
- 4 ft	40	154.4		25+	DENSE	HARD
-	45	173.7		25+	DENSE	HARD
-	43	166.0		25+	DENSE	HARD
- 5 ft	38	146.7		25+	DENSE	HARD
-	40	154.4		25+	DENSE	HARD
-	43	166.0		25+	DENSE	HARD
- 6 ft	42	162.1		25+	DENSE	HARD
-	39	150.5		25+	DENSE	HARD
- 2 m	42	162.1		25+	DENSE	HARD
- 7 ft	41	140.2		25+	DENSE	HARD
-	38	130.0		25+	DENSE	HARD
-	38	130.0		25+	DENSE	HARD
- 8 ft	43	147.1		25+	DENSE	HARD
-	40	136.8		25+	DENSE	HARD
-	40	136.8		25+	DENSE	HARD
- 9 ft	41	140.2		25+	DENSE	HARD
-						
- 3 m 10 ft						
-						
-						
- 11 ft						
-						
-						
- 12 ft						
-						
- 4 m 13 ft						

GEOTECHNICAL BORING LOG HA-2

Project No.	21762	Date Drilled	3-15-24
Project	Proposed Santee City Yard/Facility Improvements	Logged By	CA
Drilling Co.	N/A	Hole Diameter	3.25"
Drilling Method	Hand Auger	Ground Elevation	337' msl
Location	See Figure 2	Sampled By	CA

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION <i>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.</i>	Type of Tests
	0	N S						GM	UNDOCUMENTED ARTIFICIAL FILL (Afu) @ 0-0.5': Silty GRAVEL with SAND, loose, very dark grayish brown (10YR, 3/2), very moist, fine to medium SAND, fine gravel.	EI, CR
				B-1 (0.5'-5')				CL	@ 0.5': Sandy Lean CLAY, medium stiff to stiff, mottled dark reddish brown (2.5YR, 3/3), very dark gray (5YR, 3/1), moist, fine to medium SAND, low to medium plasticity, trace organics.	
335								SC	@ 2': Clayey SAND, medium dense, mottled dark reddish brown (5YR, 2.5/2), very dark gray (5YR, 3/1), moist, fine to medium SAND, trace organics.	
								SM	QUATERNARY ALLUVIUM (Qya) @ 3.5': Silty SAND, dense, dark grayish brown (10YRY, 4/2), moist, fine to medium SAND.	
5									Total Depth = 5 Feet (bgs) No Groundwater or Seepage Encountered Backfilled on 3/15/24	
330										
10										

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



WILDCAT DYNAMIC CONE LOG

Page 1 of 1

Leighton

PROJECT NUMBER: 21762

DATE STARTED: 03-15-2024

DATE COMPLETED: 03-15-2024

HOLE #: DCP-2

CREW: CA

PROJECT: Proposed Santee City Yard/Facility Improvements

ADDRESS: 9534 Via Zapador

LOCATION: Santee, CA 92071

SURFACE ELEVATION: 337' (msl)

WATER ON COMPLETION: N/A

HAMMER WEIGHT: 35 lbs.

CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm ²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY	
					NON-COHESIVE	COHESIVE
-	11	48.8		13	MEDIUM DENSE	STIFF
-	8	35.5		10	LOOSE	STIFF
- 1 ft	9	40.0		11	MEDIUM DENSE	STIFF
-	7	31.1		8	LOOSE	MEDIUM STIFF
-	7	31.1		8	LOOSE	MEDIUM STIFF
- 2 ft	5	22.2		6	LOOSE	MEDIUM STIFF
-	12	53.3		15	MEDIUM DENSE	STIFF
-	15	66.6		19	MEDIUM DENSE	VERY STIFF
- 3 ft	14	62.2		17	MEDIUM DENSE	VERY STIFF
- 1 m	35	155.4		25+	DENSE	HARD
-	40	154.4		25+	DENSE	HARD
- 4 ft	38	146.7		25+	DENSE	HARD
-	43	166.0		25+	DENSE	HARD
-	45	173.7		25+	DENSE	HARD
- 5 ft						
-						
- 6 ft						
-						
- 2 m						
- 7 ft						
-						
- 8 ft						
-						
- 9 ft						
-						
- 3 m 10 ft						
-						
-						
- 11 ft						
-						
- 12 ft						
-						
- 4 m 13 ft						

GEOTECHNICAL BORING LOG HA-3

Project No.	21762
Project	Proposed Santee City Yard/Facility Improvements
Drilling Co.	N/A
Drilling Method	Hand Auger
Location	See Figure 2

Date Drilled	3-15-24
Logged By	CA
Hole Diameter	3.25"
Ground Elevation	337' msl
Sampled By	CA

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0	N S						GM	<i>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.</i> UNDOCUMENTED ARTIFICIAL FILL (Afu) @ 0-0.5': Silty GRAVEL with SAND, dense, grayish brown (10YR, 5/2), moist, fine to medium SAND, fine gravel. @ 0.5'-1.5': Poorly-graded SAND with SILT, loose, yellowish brown (10YR, 5/6), moist, fine to medium SAND, friable, trace fine to coarse gravel. @ 1.5': Sandy Lean CLAY, medium stiff to stiff, mottled dark reddish brown (2.5YR, 3/3), very dark gray (5YR, 3/1), moist, fine to medium SAND, low to medium plasticity, trace organics.	
				B-1 (0.5'-5')				SP-SM		
335								CL		
	5								Total Depth = 5 Feet (bgs) No Groundwater or Seepage Encountered Backfilled on 3/15/24	
	330									
	10									

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



WILDCAT DYNAMIC CONE LOG

Leighton

PROJECT NUMBER: 21762

DATE STARTED: 03-15-2024

DATE COMPLETED: 03-15-2024

HOLE #: DCP-3

CREW: CA

PROJECT: Proposed Santee City Yard/Facility Improvements

ADDRESS: 9534 Via Zapador

LOCATION: Santee, CA 92071

SURFACE ELEVATION: 337' (msl)

WATER ON COMPLETION:	N/A
----------------------	-----

HAMMER WEIGHT: 35 lbs.

CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm ²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY	
					NON-COHESIVE	COHESIVE
1 ft	30	133.2		25+	DENSE	HARD
	38	168.7		25+	DENSE	HARD
	41	182.0		25+	VERY DENSE	HARD
	23	102.1		25+	MEDIUM DENSE	VERY STIFF
2 ft	7	31.1		8	LOOSE	MEDIUM STIFF
	8	35.5		10	LOOSE	STIFF
	8	35.5		10	LOOSE	STIFF
3 ft	7	31.1		8	LOOSE	MEDIUM STIFF
	8	35.5		10	LOOSE	STIFF
	7	31.1		8	LOOSE	MEDIUM STIFF
4 ft	7	27.0		7	LOOSE	MEDIUM STIFF
	7	27.0		7	LOOSE	MEDIUM STIFF
	7	27.0		7	LOOSE	MEDIUM STIFF
5 ft	6	23.2		6	LOOSE	MEDIUM STIFF
	10	38.6		11	MEDIUM DENSE	STIFF
	8	30.9		8	LOOSE	MEDIUM STIFF
6 ft	6	23.2		6	LOOSE	MEDIUM STIFF
	5	19.3		5	LOOSE	MEDIUM STIFF
	7	27.0		7	LOOSE	MEDIUM STIFF
7 ft	13	50.2		14	MEDIUM DENSE	STIFF
	17	58.1		16	MEDIUM DENSE	VERY STIFF
	8	27.4		7	LOOSE	MEDIUM STIFF
8 ft	12	41.0		11	MEDIUM DENSE	STIFF
	11	37.6		10	LOOSE	STIFF
	11	37.6		10	LOOSE	STIFF
9 ft	10	34.2		9	LOOSE	STIFF
	11	37.6		10	LOOSE	STIFF
	12	41.0		11	MEDIUM DENSE	STIFF
10 ft	24	82.1		23	MEDIUM DENSE	VERY STIFF
	21	71.8		20	MEDIUM DENSE	VERY STIFF
11 ft						
12 ft						
13 ft						

GEOTECHNICAL BORING LOG HA-4

Project No.	21762	Date Drilled	3-15-24
Project	Proposed Santee City Yard/Facility Improvements	Logged By	CA
Drilling Co.	N/A	Hole Diameter	3.25"
Drilling Method	Hand Auger	Ground Elevation	336' msl
Location	See Figure 2	Sampled By	CA

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0	N S							<i>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.</i>	
335				B-1 (0.5'-5')				GM	UNDOCUMENTED ARTIFICIAL FILL (Afu) @ 0-0.5': Silty GRAVEL with SAND, dense, grayish brown (10YR, 5/2), moist, fine to medium SAND, fine gravel. @ 0.5'-2': Poorly-graded SAND with SILT, loose, yellowish brown (10YR, 5/6), moist, fine to medium SAND, friable, trace fine to coarse gravel.	
								CL	@ 2': Sandy Lean CLAY, medium stiff to stiff, dark reddish brown (2.5YR, 3/3) to reddish brown (2.5YR, 4/4), moist, fine to medium SAND, low to medium plasticity, trace organics, trace debris.	
330	5								Total Depth = 5 Feet (bgs) No Groundwater or Seepage Encountered Backfilled on 3/15/24	
10										

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



Page 1 of 1

PROJECT NUMBER:	21762
DATE STARTED:	03-15-2024
DATE COMPLETED:	03-15-2024

SURFACE ELEVATION:	336' (msl)
WATER ON COMPLETION:	N/A
HAMMER WEIGHT:	35 lbs.
CONE AREA:	10 sq. cm

HOLE #: DCP-4
 CREW: CA
 PROJECT: Proposed Santee City Yard/Facility Improvements
 ADDRESS: 9534 Via Zapador
 LOCATION: Santee, CA 92071

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm ²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY	
					NON-COHESIVE	COHESIVE
1 ft	25	111.0		25+	DENSE	HARD
	27	119.9		25+	DENSE	HARD
	29	128.8		25+	DENSE	HARD
	30	133.2		25+	DENSE	HARD
2 ft	5	22.2		6	LOOSE	MEDIUM STIFF
	5	22.2		6	LOOSE	MEDIUM STIFF
	7	31.1		8	LOOSE	MEDIUM STIFF
3 ft	7	31.1		8	LOOSE	MEDIUM STIFF
	6	26.6		7	LOOSE	MEDIUM STIFF
	6	26.6		7	LOOSE	MEDIUM STIFF
	8	30.9		8	LOOSE	MEDIUM STIFF
4 ft	7	27.0		7	LOOSE	MEDIUM STIFF
	10	38.6		11	MEDIUM DENSE	STIFF
5 ft	11	42.5		12	MEDIUM DENSE	STIFF
	7	27.0		7	LOOSE	MEDIUM STIFF
	8	30.9		8	LOOSE	MEDIUM STIFF
	8	30.9		8	LOOSE	MEDIUM STIFF
6 ft	9	34.7		9	LOOSE	STIFF
	8	30.9		8	LOOSE	MEDIUM STIFF
7 ft	6	23.2		6	LOOSE	MEDIUM STIFF
	6	20.5		5	LOOSE	MEDIUM STIFF
8 ft	5	17.1		4	VERY LOOSE	SOFT
	7	23.9		6	LOOSE	MEDIUM STIFF
	8	27.4		7	LOOSE	MEDIUM STIFF
	8	27.4		7	LOOSE	MEDIUM STIFF
9 ft	8	27.4		7	LOOSE	MEDIUM STIFF
	10	34.2		9	LOOSE	STIFF
	10	34.2		9	LOOSE	STIFF
10 ft	22	75.2		21	MEDIUM DENSE	VERY STIFF
	25	85.5		24	MEDIUM DENSE	VERY STIFF
11 ft						
12 ft						
13 ft						

APPENDIX B

Previous Exploration Logs (Leighton, 2004)

GEOTECHNICAL BORING LOG B-1

Date 8-28-03 Sheet 1 of 2
 Project Santee Maintenance Facility Project No. 040986-001
 Drilling Co. Tri-County Type of Rig Hollow-Stem Auger
 Hole Diameter 8 Drive Weight 140 pound hammer Drop 30"
 Elevation Top of Hole _____ Location See Map

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	0	N S						SM/ML	Logged By <u>GJM</u> Sampled By <u>GJM</u> QUATERNARY ALLUVIUM (Qal) @ 0': Fine silty SAND to sandy SILT: Dark brown to red-brown, dry to damp, loose to medium dense	
	5			R-1	29	118.6	12.1	SM	@ 5': Fine silty SAND: Red-brown to brown, damp to moist, medium dense	
				Bag-2 @ 5'-10'						
	10			R-3	62	113.3	17.2	SM	@ 10': Fine to medium silty SAND: Light gray to gray, damp to moist, dense; some minor pinhole pores in upper rings	
	15			R-4	35	125.7	9.8	SW	@ 15': Fine to coarse SAND to gravelly SAND: Gray to dark gray, wet to saturated, medium dense; gravels up to 3/4" to 1" subangular	
	20			R-5	52	115.5	18.7	SW	@ 20': Fine to coarse SAND to gravelly SAND: Light gray to gray-brown, wet, dense; minor micas	
	25			R-6	43	103.0	22.3		@ 25': Fine to coarse SAND to gravelly SAND: Light gray to gray-brown, wet, dense; minor micas	
	30								WEATHERED GRANITIC ROCK @ 28': Gray to gray-brown, moist, very dense; weathered	

SAMPLE TYPES:

S SPLIT SPOON
 R RING SAMPLE
 B BULK SAMPLE
 T TUBE SAMPLE

G GRAB SAMPLE
 SH SHELBY TUBE

TYPE OF TESTS:

DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 CN CONSOLIDATION
 CR CORROSION

SA SIEVE ANALYSIS
 CU TRIAXIAL SHEAR
 EI EXPANSION INDEX
 RV R-VALUE



LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-1

Date 8-28-03

Sheet 2 of 2

Project Santee Maintenance Facility

Project No. 040986-001

Drilling Co. Tri-County

Type of Rig Hollow-Stem Auger

Hole Diameter 8

Drive Weight 140 pound hammer

Drop 30"

Elevation Top of Hole

Location

See Map

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	30	N S			50/1"				Logged By <u>GJM</u> Sampled By <u>GJM</u> WEATHERED GRANITIC ROCK @ 30': Dark gray to gray-brown, moist, very dense; no sample recovery, weathered Total Depth = 30 Feet Ground water encountered at 13 feet at time of drilling Backfilled with bentonite on 8/28/03	
	35									
	40									
	45									
	50									
	55									
	60									

SAMPLE TYPES:

S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE

G GRAB SAMPLE
SH SHELBY TUBE

TYPE OF TESTS:

DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION

SA SIEVE ANALYSIS
CU TRIAXIAL SHEAR
EI EXPANSION INDEX
RV R-VALUE



LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-2

Date 8-28-03 Sheet 1 of 2
 Project Santee Maintenance Facility Project No. 040986-001
 Drilling Co. Tri-County Type of Rig Hollow-Stem Auger
 Hole Diameter 8 Drive Weight 140 pound hammer Drop 30"
 Elevation Top of Hole _____ Location See Map

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
		N S							Logged By <u>GJM</u> Sampled By <u>GJM</u>	
	0							ML	<u>QUATERNARY ALLUVIUM (Qal)</u> @ 0': SILT: Dark brown to red-brown, dry to damp, loose to medium dense	
	5			R-1	52	109.3	18.3	ML	@ 5': SILT: Brown to red-brown, damp to moist, hard	
	10			R-2	34	112.4	14.8	SM	@ 10': Fine to medium silty SAND: Light gray to light gray-brown, moist to wet, medium dense	
	15			R-3	28			SM	@ 15': Disturbed sample	
	20			R-4	90/11"	135.3	9.1	SP	@ 20': Fine to coarse SAND to gravelly SAND: Light brown to brown, moist to wet, very dense; poorly graded, angular gravels up to 1", micas present	
	25			R-5	50/3"				@ 25': Fine to coarse SAND to gravelly SAND: Dark gray to gray-brown, saturated, very dense; abundant micas; no sample recovery	
	30								<u>WEATHERED GRANITIC ROCK</u> @ 26': Light brown to gray-brown, moist to wet, very dense, weathered micas	

SAMPLE TYPES:

S SPLIT SPOON
 R RING SAMPLE
 B BULK SAMPLE
 T TUBE SAMPLE

G GRAB SAMPLE
 SH SHELBY TUBE

TYPE OF TESTS:

DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 CN CONSOLIDATION
 CR CORROSION

SA SIEVE ANALYSIS
 CU TRIAXIAL SHEAR
 EI EXPANSION INDEX
 RV R-VALUE



LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-2

Date 8-28-03 Sheet 2 of 2
 Project Santee Maintenance Facility Project No. 040986-001
 Drilling Co. Tri-County Type of Rig Hollow-Stem Auger
 Hole Diameter 8 Drive Weight 140 pound hammer Drop 30"
 Elevation Top of Hole _____ Location See Map

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	30	N S			50/2"				Logged By <u>GJM</u> Sampled By <u>GJM</u> <u>@ 30': No recovery</u> Total Depth = 30.5 Feet Ground water encountered at 10 feet at time of drilling Backfilled with bentonite on 8/28/03	
	35									
	40									
	45									
	50									
	55									
	60									

SAMPLE TYPES:

S SPLIT SPOON
 R RING SAMPLE
 B BULK SAMPLE
 T TUBE SAMPLE

G GRAB SAMPLE
 SH SHELBY TUBE

TYPE OF TESTS:

DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 CN CONSOLIDATION
 CR CORROSION

SA SIEVE ANALYSIS
 CU TRIAXIAL SHEAR
 EI EXPANSION INDEX
 RV R-VALUE



LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-3

Date 8-28-03 Sheet 1 of 1
 Project Santee Maintenance Facility Project No. 040986-001
 Drilling Co. Tri-County Type of Rig Hollow-Stem Auger
 Hole Diameter 8 Drive Weight 140 pound hammer Drop 30"
 Elevation Top of Hole _____ Location See Map

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	0	N S						ML	Logged By <u>GJM</u> Sampled By <u>GJM</u>	
	5			R-1	53	113.5	15.0	ML	QUATERNARY ALLUVIUM (Qal) @ 0': SILT: Brown to dark brown, dry to damp, medium dense @ 5': SILT: Red-brown to brown, damp to moist, hard	
	10			R-2	24			SP	@ 10': Fine to coarse SAND to gravelly SAND: Brown, wet, medium dense; poorly graded, angular gravels up to 1", disturbed sample	
	15			R-3	50/5"	134.2	7.9	SP	@ 15': Fine to coarse gravelly SAND: Brown to gray-brown, wet, very dense; gravels up to 1" angular	
	20			R-4	41	114.2	17.1	SM	@ 20': Fine to coarse silty SAND: Light gray to light gray-brown, wet, medium dense; no gravels	
	25			R-5	50/1"				WEATHERED GRANITIC ROCK @ 24': No sample recovery, granitics black and white	
	30								Total Depth = 25.5 Feet Ground water encountered at 12 feet at time of drilling Backfilled with bentonite on 8/28/03	

SAMPLE TYPES:
 S SPLIT SPOON
 R RING SAMPLE
 B BULK SAMPLE
 T TUBE SAMPLE

G GRAB SAMPLE
 SH SHELBY TUBE

TYPE OF TESTS:

DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 CN CONSOLIDATION
 CR CORROSION

SA SIEVE ANALYSIS
 CU TRIAXIAL SHEAR
 EI EXPANSION INDEX
 RV R-VALUE



LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-4

Date 8-28-03 Sheet 1 of 1
 Project Santee Maintenance Facility Project No. 040986-001
 Drilling Co. Tri-County Type of Rig Hollow-Stem Auger
 Hole Diameter 8 Drive Weight 140 pound hammer Drop 30"
 Elevation Top of Hole _____ Location See Map

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
		N S							Logged By <u>GJM</u> Sampled By <u>GJM</u>	
0								ML	<u>QUATERNARY ALLUVIUM (Qal)</u> @ 0': SILT: Red-brown to brown, dry to damp, medium dense	
5				R-1	19	107.3	6.2	SM	@ 5': Fine to medium silty SAND: Light brown to orange-brown, damp to moist, medium dense	
10				R-2	26	106.5	17.9	SW	@ 10': Fine to coarse gravelly SAND to silty SAND: Gray to medium gray, wet, medium dense	
15				R-3	33	109.3	13.3	SW	@ 15': Fine to coarse gravelly SAND to clayey silty SAND: Gray to medium gray, wet to saturated, medium dense; disturbed sample; gravelly up to 3/4"-subangular	
20				R-4	52			SW	@ 20': Fine to coarse gravelly SAND to silty SAND: Gray to gray-brown, wet to saturated, dense; gravelly up to 3/4" subangular to subrounded; friable, minor micas present	
25				R-5	50/4"				<u>WEATHERED GRANITIC ROCK</u> @ 22': Brown to gray-brown, moist to wet, very dense; weathered @ 25': No recovery Total Depth = 25 Feet Ground water encountered at 13 feet at time of drilling Backfilled with bentonite on 8/28/03	
30										

SAMPLE TYPES:
 S SPLIT SPOON
 R RING SAMPLE
 B BULK SAMPLE
 T TUBE SAMPLE

G GRAB SAMPLE
 SH SHELBY TUBE

TYPE OF TESTS:
 DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 CN CONSOLIDATION
 CR CORROSION

SA SIEVE ANALYSIS
 CU TRIAXIAL SHEAR
 EI EXPANSION INDEX
 RV R-VALUE



LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-5

Date 8-28-03 Sheet 1 of 1
 Project Santee Maintenance Facility Project No. 040986-001
 Drilling Co. Tri-County Type of Rig Hollow-Stem Auger
 Hole Diameter 8 Drive Weight 140 pound hammer Drop 30"
 Elevation Top of Hole _____ Location See Map

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	0	N S						ML	Logged By <u>GJM</u> Sampled By <u>GJM</u>	
	5			R-1	25	113.2	5.2	SM	QUATERNARY ALLUVIUM (Qal) @ 0': SILT: Dark brown to dark red-brown, dry to damp, loose to medium dense	
	10			R-2	39	108.5	19.9	SM	@ 5': Fine to medium silty SAND: Red-brown to brown, dry to damp, medium dense	
	15			R-3	56	110.1	20.4	SM	@ 10': Fine silty SAND, gray to light gray, damp to moist, medium dense	
	20			R-4	85	118.1	15.6		@ 15': Fine to medium silty SAND: Moist to wet, light gray to dark gray, dense	
	25				50/1"				WEATHERED GRANITIC ROCK @ 20': Gray brown to brown: Moist, very dense; very weathered, micas	
									@ 25': No sample recovery	
	30								Total Depth = 25 Feet Ground water encountered at 15 feet at time of drilling Backfilled with bentonite on 8/28/03	

SAMPLE TYPES:

S SPLIT SPOON
 R RING SAMPLE
 B BULK SAMPLE
 T TUBE SAMPLE

G GRAB SAMPLE
 SH SHELBY TUBE

TYPE OF TESTS:

DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 CN CONSOLIDATION
 CR CORROSION

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LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-6

Date 8-28-03 Sheet 1 of 2
 Project Santee Maintenance Facility Project No. 040986-001
 Drilling Co. Tri-County Type of Rig Hollow-Stem Auger
 Hole Diameter 8 Drive Weight 140 pound hammer Drop 30"
 Elevation Top of Hole _____ Location See Map

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
		N S							Logged By <u>GJM</u> Sampled By <u>GJM</u>	
0				Bag-1 @ 0-5'				ML	<u>QUATERNARY ALLUVIUM (Qal)</u> @ 0': SILT: Brown to red-brown, dry to damp, medium dense	
5				R-2	61	115.8	15.2		@ 5': SILT to silty clay: Light brown to red-brown, damp to moist, hard; minor pinhole pores	
10				R-3	32	121.0	12.6	SM	@ 10': Fine to coarse silty SAND to gravelly SAND: Red-brown to orange-brown, moist, to wet, medium dense; minor 3/4" gravels subangular	
15				R-4	57	125.4	13.1	SW	@ 15': Fine to coarse gravelly SAND to silty SAND: Light gray to medium gray, wet to saturated, dense; some gravels up to 3/4" subangular	
20				R-5	88/11"	122.2	11.1	SW	@ 20': Fine to coarse gravelly SAND to silty SAND: Light gray to medium gray, wet to saturated, very dense; minor gravels to 3/4" subrounded	
25				R-6	50/4"					
30									<u>WEATHERED GRANITIC ROCK</u> @ 25.5': Dark gray to light gray, damp to moist, medium dense	

SAMPLE TYPES:

S SPLIT SPOON
 R RING SAMPLE
 B BULK SAMPLE
 T TUBE SAMPLE

G GRAB SAMPLE
 SH SHELBY TUBE

TYPE OF TESTS:

DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 CN CONSOLIDATION
 CR CORROSION

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LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-6

Date 8-28-03

Sheet 2 of 2

Project Santee Maintenance Facility

Project No. 040986-001

Drilling Co. Tri-County

Type of Rig Hollow-Stem Auger

Hole Diameter 8 Drive Weight 140 pound hammer

Drop 30"

Elevation Top of Hole Location See Map

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	30	N S		R-7	50/1"				Logged By GJM Sampled By GJM <u>WEATHERED GRANITIC ROCK</u> @ 30': No sample recovery Total Depth = 30 Feet Ground water encountered at 15 feet at time of drilling Backfilled with bentonite on 8/28/03	
	35									
	40									
	45									
	50									
	55									
	60									

SAMPLE TYPES:

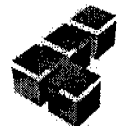
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE

G GRAB SAMPLE
SH SHELBY TUBE

TYPE OF TESTS:

DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION

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LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-7

Date 8-28-03 Sheet 1 of 2
 Project Santee Maintenance Facility Project No. 040986-001
 Drilling Co. Tri-County Type of Rig Hollow-Stem Auger
 Hole Diameter 8 Drive Weight 140 pound hammer Drop 30"
 Elevation Top of Hole Location See Map

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
		N S							Logged By <u>GJM</u> Sampled By <u>GJM</u>	
	0							ML	QUATERNARY ALLUVIUM (Qal) @ 0': SILT: Dark brown, dry to damp, loose to medium dense	
	5			R-1	51	120.3	12.9	ML	@ 5': Clayey SILT: Red-brown to brown, damp to moist, hard	
	10			R-2	29	106.9	20.0	ML	@ 10': SILT: Gray brown to red-brown, damp to moist, very stiff; abundant pinhole sized micas	
	15			R-3	35	99.5	24.5	SM	@ 15': Fine to medium silty SAND: Gray to gray-brown, moist to wet, medium dense; micas present; friable	
	20			R-4	89/11"	130.9	10.4	SW	@ 20': Fine to coarse SAND to gravelly SAND: Medium gray to gray-brown, wet to saturated, very dense; gravels up to 3/4" cobble up to 2", both subangular to subrounded, well graded	
	25			R-5	50/5"				WEATHERED GRANITIC ROCK @ 25': Yellow-gray to gray, damp, very dense; micas present; no sample recovery	
	30									

SAMPLE TYPES:
 S SPLIT SPOON
 R RING SAMPLE
 B BULK SAMPLE
 T TUBE SAMPLE

G GRAB SAMPLE
 SH SHELBY TUBE

TYPE OF TESTS:
 DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 CN CONSOLIDATION
 CR CORROSION

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LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-7

Date 8-28-03 Sheet 2 of 2
 Project Santee Maintenance Facility Project No. 040986-001
 Drilling Co. Tri-County Type of Rig Hollow-Stem Auger
 Hole Diameter 8 Drive Weight 140 pound hammer Drop 30"
 Elevation Top of Hole _____ Location See Map

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	30	N S			50/2"				Logged By <u>GJM</u> Sampled By <u>GJM</u> <u>WEATHERED GRANITIC ROCK</u> @ 30': Yellow gray to gray, damp, very dense; micas present; no sample Total Depth = 30.5 Feet Ground water encountered at 17 feet at time of drilling Backfilled with bentonite on 8/27/03	
	35									
	40									
	45									
	50									
	55									
	60									

SAMPLE TYPES:

S SPLIT SPOON
 R RING SAMPLE
 B BULK SAMPLE
 T TUBE SAMPLE

G GRAB SAMPLE
 SH SHELBY TUBE

TYPE OF TESTS:

DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 CN CONSOLIDATION
 CR CORROSION

SA SIEVE ANALYSIS
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LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-8

Date 8-28-03 Sheet 1 of 1
 Project Santee Maintenance Facility Project No. 040986-001
 Drilling Co. Tri-County Type of Rig Hollow-Stem Auger
 Hole Diameter 8 Drive Weight 140 pound hammer Drop 30"
 Elevation Top of Hole _____ Location See Map

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	0	N S		Bag-1 @ 0'-5'				ML	QUATERNARY ALLUVIUM (Qal) @ 0': SILT: Dark red-brown to dark brown, dry, damp, medium dense	
	5			R-2	52	122.0	13.5	SM/SC	@ 5': Fine to medium slightly clayey silty SAND: Red-brown to dark brown, damp to moist, dense; minor gravel up to 3/4"; subangular	
	10			R-3	50	116.1	16.2	SM	@ 10': Fine to medium silty SAND: Red-brown to gray-brown, damp to moist, dense; micas	
	15			R-4	43	110.6	19.1	SW	@ 15': Fine to coarse SAND to gravelly SAND: Gray-brown to reddish gray-brown, wet, medium dense; minor micas, minor gravel up to 1/2" subangular; friable, well graded	
	20			R-5	78/9"	131.5	9.7		WEATHERED GRANITIC ROCK @ 20.5': Gray to dark gray-brown, damp to moist; very dense; very weathered; micas	
	25			R-6	50/4"				@ 25': No sample recovery, dark gray, moist, very dense, micas	
	30								Total Depth = 25.5 Feet Ground water encountered at 17 feet at time of drilling Backfilled with bentonite on 8/28/03	

SAMPLE TYPES:

S SPLIT SPOON
 R RING SAMPLE
 B BULK SAMPLE
 T TUBE SAMPLE

G GRAB SAMPLE
 SH SHELBY TUBE

TYPE OF TESTS:

DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 CN CONSOLIDATION
 CR CORROSION

SA SIEVE ANALYSIS
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LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-9

Date 8-28-03 Sheet 1 of 2
 Project Santee Maintenance Facility Project No. 040986-001
 Drilling Co. Tri-County Type of Rig Hollow-Stem Auger
 Hole Diameter 8 Drive Weight 140 pound hammer Drop 30"
 Elevation Top of Hole _____ Location See Map

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	0	N S						ML	QUATERNARY ALLUVIUM (Qal) @ 0': SILT: Dark red-brown to dark brown, dry to damp, medium dense	
	5			R-1	72/10"	103.0	12.4	ML	@ 5': SILT: Dark red-brown to dark brown, dry to damp, hard	
	10			R-2	47	121.0	12.9	SW	@ 10': Fine to coarse SAND: Red-brown to brown, damp to moist, medium dense; friable, minor micas present	
	15			R-3	50/5"	114.7	20.1	SW	@ 15': Fine to coarse SAND: Red-brown to brown, damp to moist, medium dense; friable, minor micas present	
	20			R-4	57	130.2	12.9	SP	@ 20': Medium to coarse SAND to gravelly SAND: Dark gray, wet to saturated, medium dense; friable, some fines, minor micas; some gravels up to 1/2" subangular-subrounded	
	25			R-5	50/5"	122.5	16.4		WEATHERED GRANITIC ROCK @ 25.5': Yellow-brown, light gray, damp to moist, very dense; micas	
	30									

SAMPLE TYPES:

S SPLIT SPOON
 R RING SAMPLE
 B BULK SAMPLE
 T TUBE SAMPLE

G GRAB SAMPLE
 SH SHELBY TUBE

TYPE OF TESTS:

DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 CN CONSOLIDATION
 CR CORROSION

SA SIEVE ANALYSIS
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LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-9

Date 8-28-03 Sheet 2 of 2
 Project Santee Maintenance Facility Project No. 040986-001
 Drilling Co. Tri-County Type of Rig Hollow-Stem Auger
 Hole Diameter 8 Drive Weight 140 pound hammer Drop 30"
 Elevation Top of Hole _____ Location See Map

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	30	N S		R-6	50/1"				Logged By <u>GJM</u> Sampled By <u>GJM</u> <u>WEATHERED GRANITIC ROCK</u> <u>@ 30'. No sample recovery</u> Total Depth = 30 Feet Ground water encountered at 15 feet Backfilled with bentonite on 8/28/03	
	35									
	40									
	45									
	50									
	55									
	60									

SAMPLE TYPES:

S SPLIT SPOON
 R RING SAMPLE
 B BULK SAMPLE
 T TUBE SAMPLE

G GRAB SAMPLE
 SH SHELBY TUBE

TYPE OF TESTS:

DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 CN CONSOLIDATION
 CR CORROSION

SA SIEVE ANALYSIS
 CU TRIAXIAL SHEAR
 EI EXPANSION INDEX
 RV R-VALUE



LEIGHTON AND ASSOCIATES, INC.

GEOTECHNICAL BORING LOG B-10

Date 8-27-03 Sheet 1 of 1
 Project Santee Maintenance Facility Project No. 040986-001
 Drilling Co. Tri-County Type of Rig Hollow-Stem Auger
 Hole Diameter 8 Drive Weight 140 pound hammer Drop 30"
 Elevation Top of Hole _____ Location See Map

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
									Logged By <u>GJM</u> Sampled By <u>GJM</u>	
0								SP-SC	<u>QUATERNARY ALLUVIUM (Qal)</u> @ 0': SILT: Dark red-brown to dark brown, dry to damp, loose to medium dense	
5				R-1	70/11"	121.1	10.5	ML	@ 5': Clayey SILT: Red-brown to brown, dry to damp, hard; minor pinhole pores	
10				R-2	38	110.8	20.0	SM	@ 9': Fine to medium silty SAND: Red-brown to brown, moist, medium dense, micas	
15				R-3	53	119.7	14.1	SW	@ 10': Fine to coarse SAND: Orange-brown to light brown, moist to wet, medium dense; micas	
20				R-4	74/11"	113.3	15.7	SW	@ 15': Fine to coarse SAND to gravelly SAND: Gray-brown to orange-brown, wet, dense; minor micas, minor gravel up to 1/2"	
25				R-5	50/5"	124.0	16.6		<u>WEATHERED GRANITIC ROCK</u> @ 20': Gray to dark gray, damp to moist, dense, micas @ 25': Gray to dark gray, damp to moist, dense, not as weathered	
30									Total Depth = 25.5 Feet Ground water encountered at 17 feet at time of drilling Backfilled with bentonite on 8/27/03	

SAMPLE TYPES:
 S SPLIT SPOON
 R RING SAMPLE
 B BULK SAMPLE
 T TUBE SAMPLE

G GRAB SAMPLE
 SH SHELBY TUBE

TYPE OF TESTS:

DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 CN CONSOLIDATION
 CR CORROSION

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LEIGHTON AND ASSOCIATES, INC.

APPENDIX C

Laboratory Testing Procedures and Test Results

APPENDIX C

Laboratory Testing Procedures and Test Results

Expansion Index Test: The expansion potential of selected sample was evaluated by the Expansion Index Text, ASTM Test Method D 4829. The specimen was molded under a given compactive energy to approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimen was then loaded to an equivalent 144 psf surcharge and was inundated with water until volumetric equilibrium was reached. The result of this test is presented in the table below:

Sample Location	Description	Expansion Index	Expansion Potential
HA-1 @ 0-2.5 Feet	Clayey SAND	21	Low

Soluble Sulfates: The soluble content of a selected sample was determined by standard geochemical methods. The test result is presented in the table below:

Sample Location	Sulfate Content, ppm
HA-2 @ 0-3 Feet	150

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

Sample Location	Chloride Content, ppm
HA-2 @ 0-3 Feet	100

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)
HA-2 @ 0-3 Feet	7.96	1,200

APPENDIX C (continued)

Previous Laboratory Testing Procedures and Test Results

Expansion Index Test: The expansion potential of selected sample was evaluated by the Expansion Index Text, ASTM Test Method D 4829. The specimen was molded under a given compactive energy to approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimen was then loaded to an equivalent 144 psf surcharge and was inundated with water until volumetric equilibrium was reached. The result of this test is presented in the table below:

Sample Location	Description	Expansion Index	Expansion Potential
B-6 @ 0-5 Feet	Silty CLAY with SAND	76	Medium

Soluble Sulfates: The soluble content of a selected sample was determined by standard geochemical methods. The test result is presented in the table below:

Sample Location	Sulfate Content, ppm
B-1 @ 5-10 Feet	270
B-6 @ 0-5 Feet	375

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

Sample Location	Chloride Content, ppm
B-1 @ 5-10 Feet	360
B-6 @ 0-5 Feet	480

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

Sample Location	pH	Minimum Resistivity (ohms-cm)
B-1 @ 5-10 Feet	7.99	1305
B-6 @ 0-5 Feet	7.82	1099

Maximum Dry Density and Optimum Moisture Content Tests: The maximum dry density and optimum moisture content of a selected representative soil sample was evaluated in general accordance with ASTM D 1557. The test result is presented in the table below.

Sample Location	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-1 @ 5-10 Feet	Silty SAND	131.5	9.5
B-6 @ 0-5 Feet	Silty CLAY with SAND	124.5	11.0

"R"-Value: The resistance "R"-value was determined by the California Materials Method CT301 for base, subbase, and basement soils. The samples were prepared and exudation pressure and "R"-value determined. The graphically determined "R"-value at exudation pressure of 300 psi is reported.

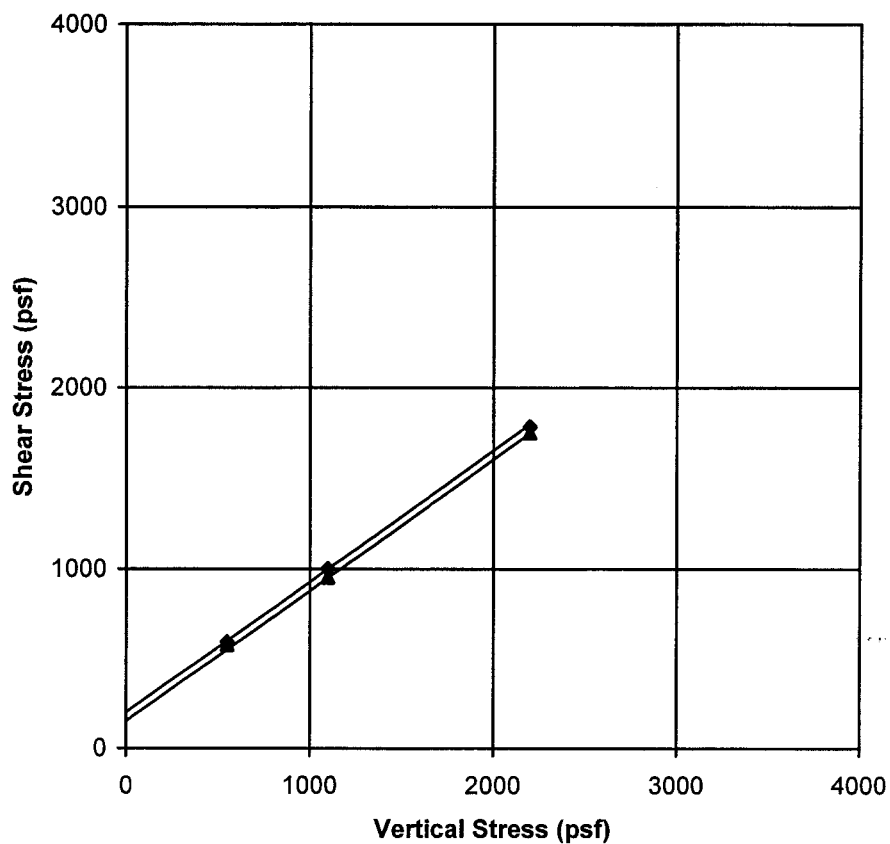
Sample Location	Sample Description	R-Value
B-6 @ 0-5 Feet	Silty CLAY with SAND	<5
B-8 @ 0-5 Feet	Silty CLAY with SAND	6

Particle/Grain Size Analysis (ASTM D 422): Particle size analysis was performed by mechanical sieving methods according to ASTM D 422, D 1440, and C 136. This test was performed to assist in the classification of the soil and to determine grain size distributions of the tested soil.

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

Swell/Collapse Potential: The one-dimensional swell or collapse potential of cohesive soil for selected materials was evaluated in accordance with ASTM Test Method D 4546. The samples were placed in a consolidometer and a load approximately equal to the insitu overburden pressure was applied. The percent swell or collapse for the load cycles were recorded as the ratio of the amount of vertical compression to the original 1-inch height. The one-dimensional swell or collapse curves are presented on the attached figures.

Consolidation Test: Consolidation testing was performed as per ASTM D2435 on a selected relatively undisturbed ring sample. The sample was placed in a consolidometer and loads were applied in geometric progression. The percent consolidation for each load cycle was recorded as the ratio of the amount of vertical compression to the original 1-inch height. The consolidation curve is presented on the attached figure.



Boring Location B-4

Sample Depth (feet) 5

Sample Description Brown silty SAND

Average Strength Parameters

Friction Angle, ϕ'_{peak} (deg) 36

Cohesion, c'_{peak} (psf) 200

Friction Angle, ϕ'_{ult} (deg) 36

Cohesion, c'_{ult} (psf) 150

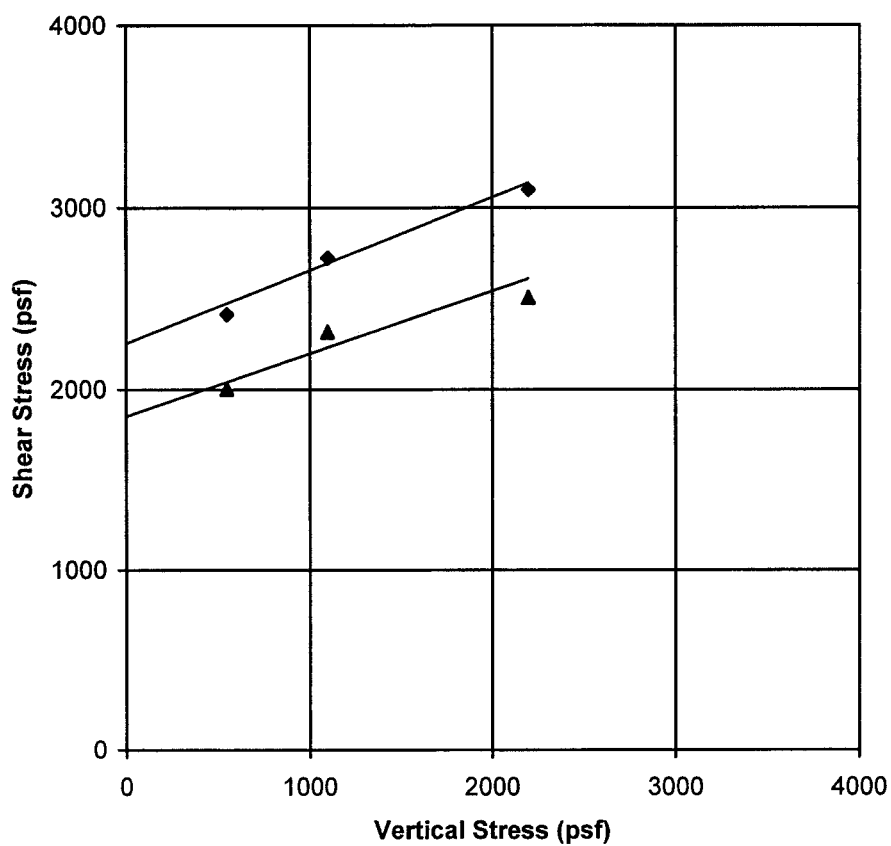
DIRECT SHEAR SUMMARY

Project No. 040986-001

Project Name Gillis/Santee Maint. Fac.

Figure No. C-1





Boring Location	<u>B-6</u>
Sample Depth (feet)	<u>5</u>
Sample Description	<u>Red-Brown Silty Clay with Sand</u>

Average Strength Parameters

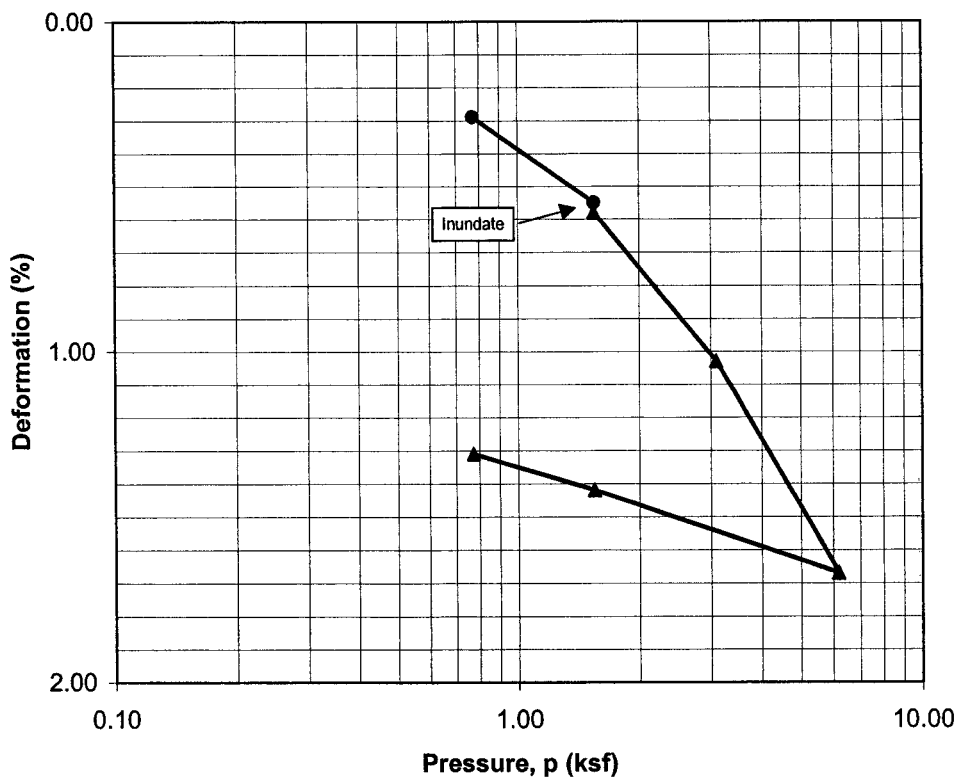
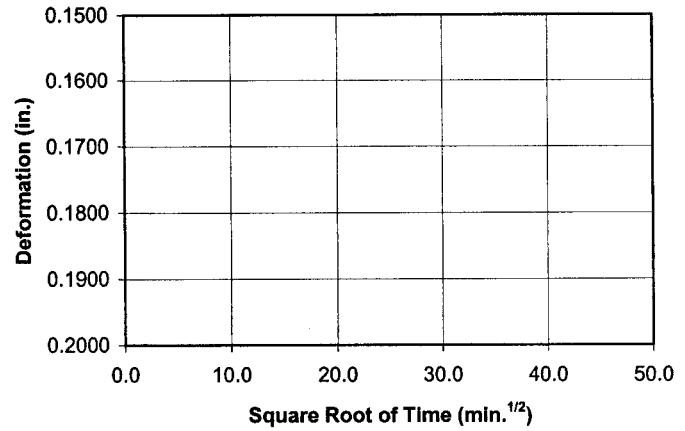
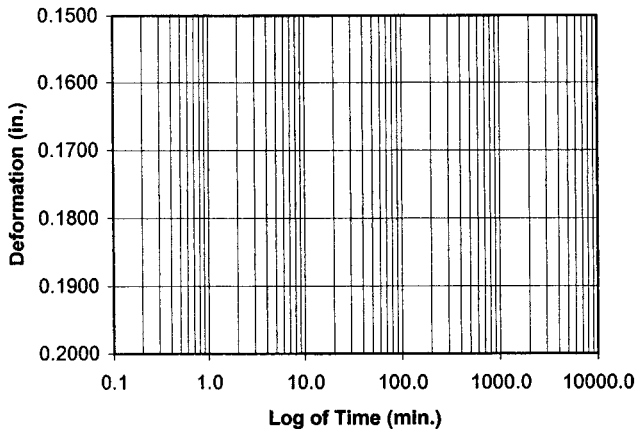
Friction Angle, ϕ'_{peak} (deg)	<u>22</u>
Cohesion, c'_{peak} (psf)	<u>2250</u>
Friction Angle, ϕ'_{ult} (deg)	<u>19</u>
Cohesion, c'_{ult} (psf)	<u>1850</u>

DIRECT SHEAR SUMMARY

Project No.	<u>040986-001</u>
Project Name	<u>Gillis/Santee Maint. Fac.</u>
Figure No.	<u>C-2</u>



Time Readings @ 0 ksf



Boring Number	Sample Number:	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
B-4	2	10.0	21.9	20.6	103.4	103.4	0.626	0.609	94	88

Sample Description:
SP-SM, BROWN POORLY-GRADED SAND
WITH SILT



Teratest Labs, Inc.

Project Name: SANTEE MAINTENANCE FACILITY

Project Number: 040986-001

**ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOIL
ASTM D 2435**



Teratest Labs, Inc.

**ONE-DIMENSIONAL SWELL OR SETTLEMENT
POTENTIAL OF COHESIVE SOILS
ASTM D 4546**

Project Name: SANTEE MAINTENANCE FACILITY
Project Number: 040986-001
Boring Number: B-5
Sample Number: 1
Sample Description: SM, RED-BROWN SILTY SAND

Tested By: BCC Date: 9/17/2003
Checked By: Date:
Sample Type: RING
Depth (ft.): 5.0

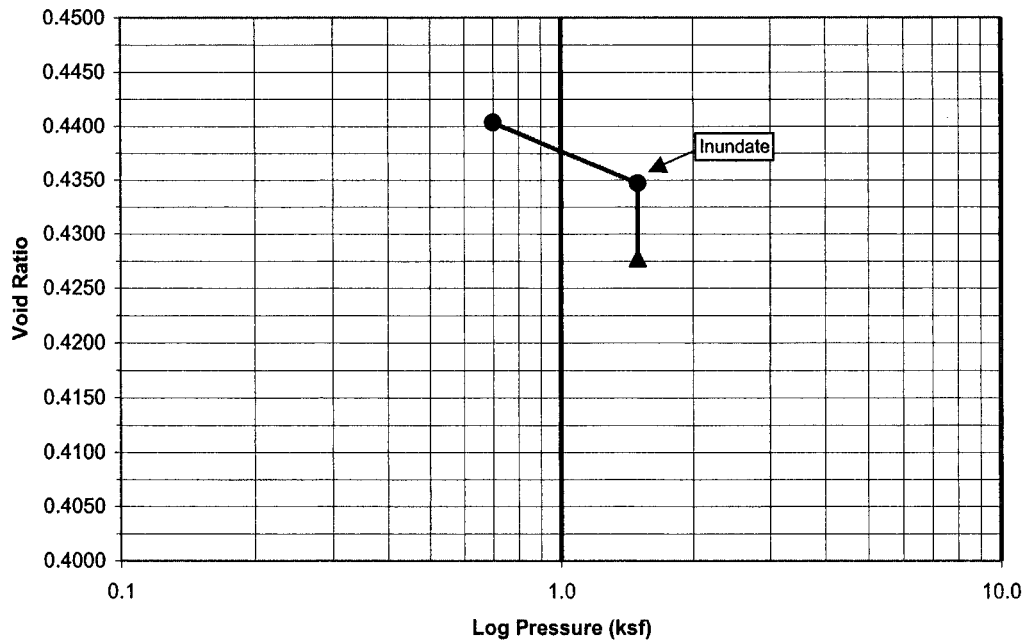
Initial Dry Density (pcf)	116.4
Initial Moisture (%)	5.7
Initial Height (in.)	1.0000
Initial Dial Reading	0.0500
Diameter (in.)	2.416

Final Dry Density (pcf)	78.0
Final Moisture (%)	25.3
Initial Void Ratio	0.4478
Specific Gravity (assumed)	2.70
Initial Saturation (%)	34

Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Swell (+) Settlement (-) Sample Thickness (%)	Void Ratio	Corrected Deformation (%)
0.70	0.0552	0.9948	0.00	-0.52	0.4403	-0.52
1.50	0.0591	0.9909	0.00	-0.91	0.4347	-0.91
H ₂ O	0.0639	0.9861	0.00	-1.39	0.4277	-1.39

Percent Swell (+) / Settlement (-) After Inundation = **-0.48**

Void Ratio - Log Pressure Curve





ONE-DIMENSIONAL SWELL OR SETTLEMENT
POTENTIAL OF COHESIVE SOILS
ASTM D 4546

Project Name: SANTEE MAINTENANCE FACILITY
Project Number: 040986-001
Boring Number: B-7
Sample Number: 1
Sample Description: s(ML), BROWN SANDY LEAN SILT

Tested By: BCC Date: 9/17/2003
Checked By: _____ Date: _____
Sample Type: RING
Depth (ft.): 5.0

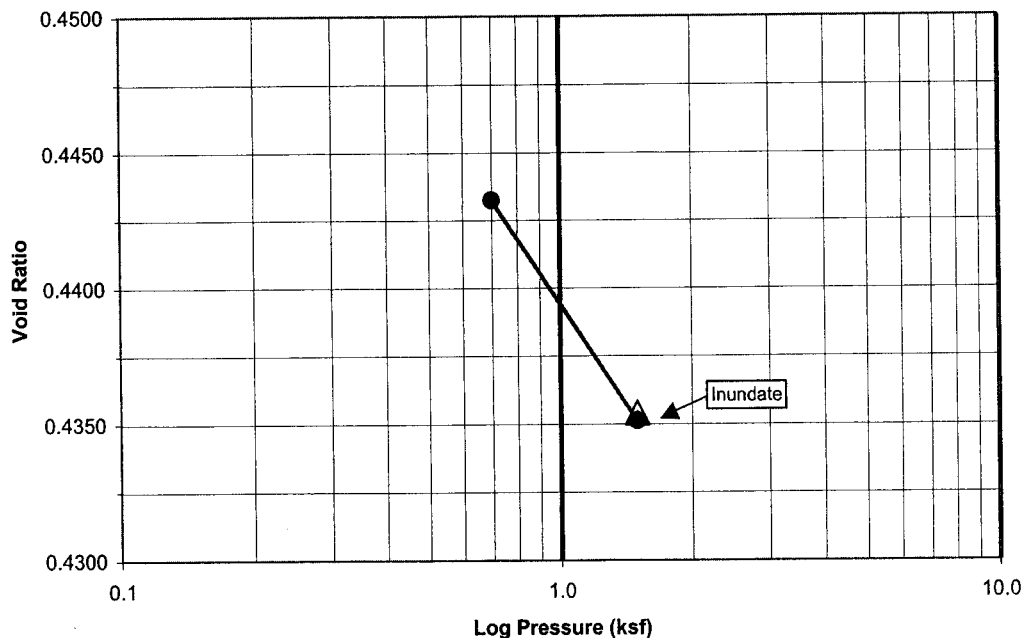
Initial Dry Density (pcf)	116.3
Initial Moisture (%)	15.5
Initial Height (in.)	1.0000
Initial Dial Reading	0.0500
Diameter (in.)	2.416

Final Dry Density (pcf)	79.5
Final Moisture (%)	24.7
Initial Void Ratio	0.4489
Specific Gravity (assumed)	2.70
Initial Saturation (%)	93

Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Swell (+) Settlement (-) Sample Thickness (%)	Void Ratio	Corrected Deformation (%)
0.70	0.0539	0.9961	0.00	-0.39	0.4432	-0.39
1.50	0.0595	0.9905	0.00	-0.95	0.4351	-0.95
H ₂ O	0.0593	0.9907	0.00	-0.93	0.4354	-0.93

Percent Swell (+) / Settlement (-) After Inundation = **0.02**

Void Ratio - Log Pressure Curve





Teratest Labs, Inc.

**ONE-DIMENSIONAL SWELL OR SETTLEMENT
POTENTIAL OF COHESIVE SOILS
ASTM D 4546**

Project Name: SANTEE MAINTENANCE FACILITY
Project Number: 040986-001
Boring Number: B-7
Sample Number: 2
Sample Description: s(ML), OLIVE-BROWN SANDY LEAN SILT

Tested By: BCC Date: 9/17/2003
Checked By: Date:
Sample Type: RING
Depth (ft.): 10.0

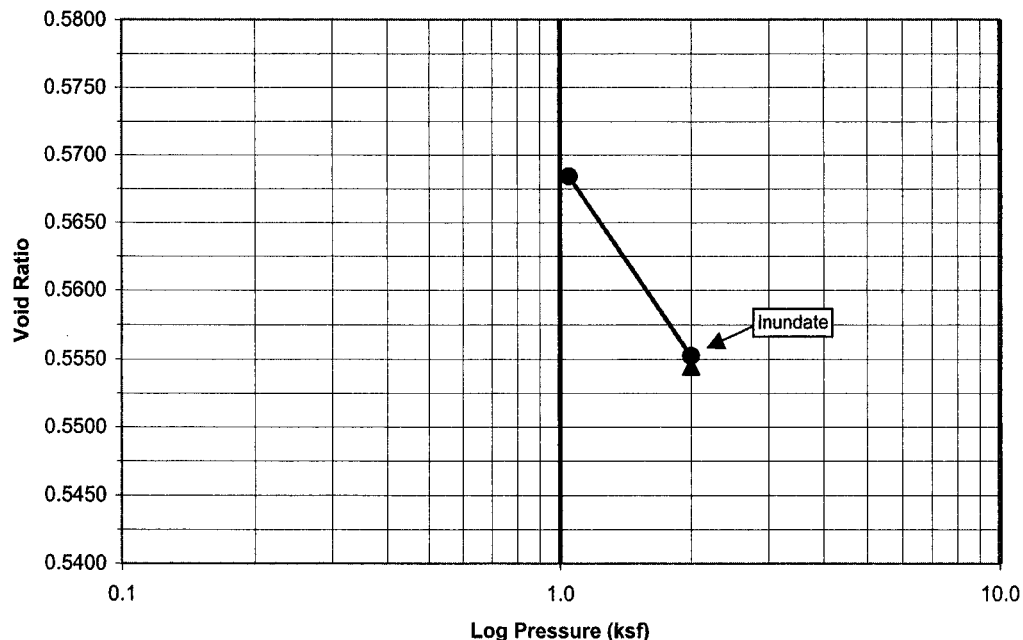
Initial Dry Density (pcf)	106.4
Initial Moisture (%)	22.9
Initial Height (in.)	1.0000
Initial Dial Reading	0.0500
Diameter (in.)	2.416

Final Dry Density (pcf)	70.4
Final Moisture (%)	34.5
Initial Void Ratio	0.5847
Specific Gravity (assumed)	2.70
Initial Saturation (%)	100

Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Swell (+) Settlement (-) Sample Thickness (%)	Void Ratio	Corrected Deformation (%)
1.05	0.0603	0.9897	0.00	-1.03	0.5684	-1.03
2.00	0.0686	0.9814	0.00	-1.86	0.5552	-1.86
H ₂ O	0.0691	0.9809	0.00	-1.91	0.5544	-1.91

Percent Swell (+) / Settlement (-) After Inundation = **-0.05**

Void Ratio - Log Pressure Curve



APPENDIX D

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.

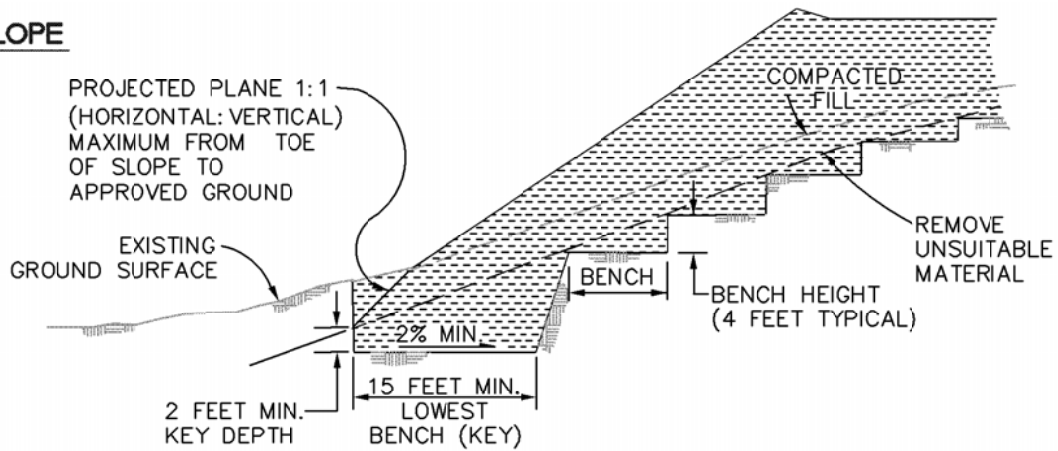
7.3 Lift Thickness

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.

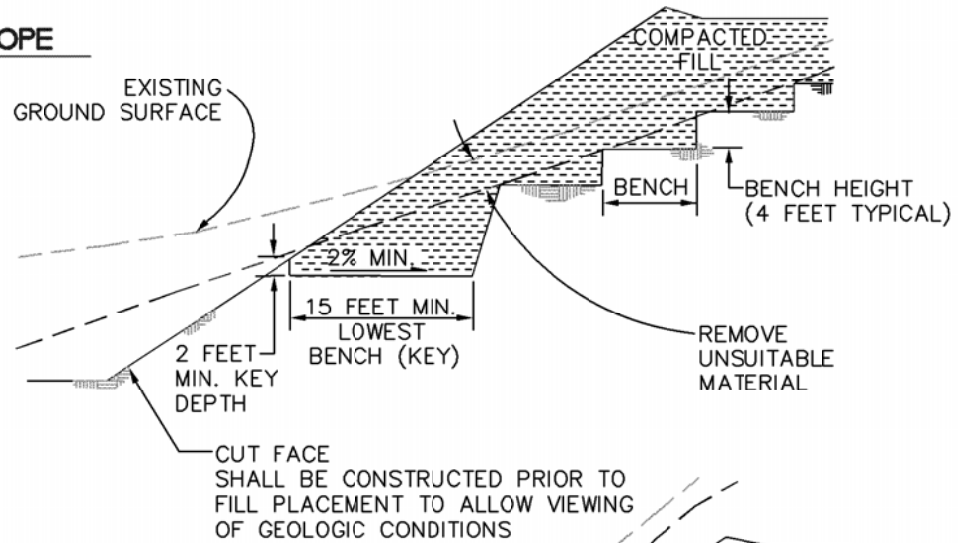
7.4 Observation and Testing

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

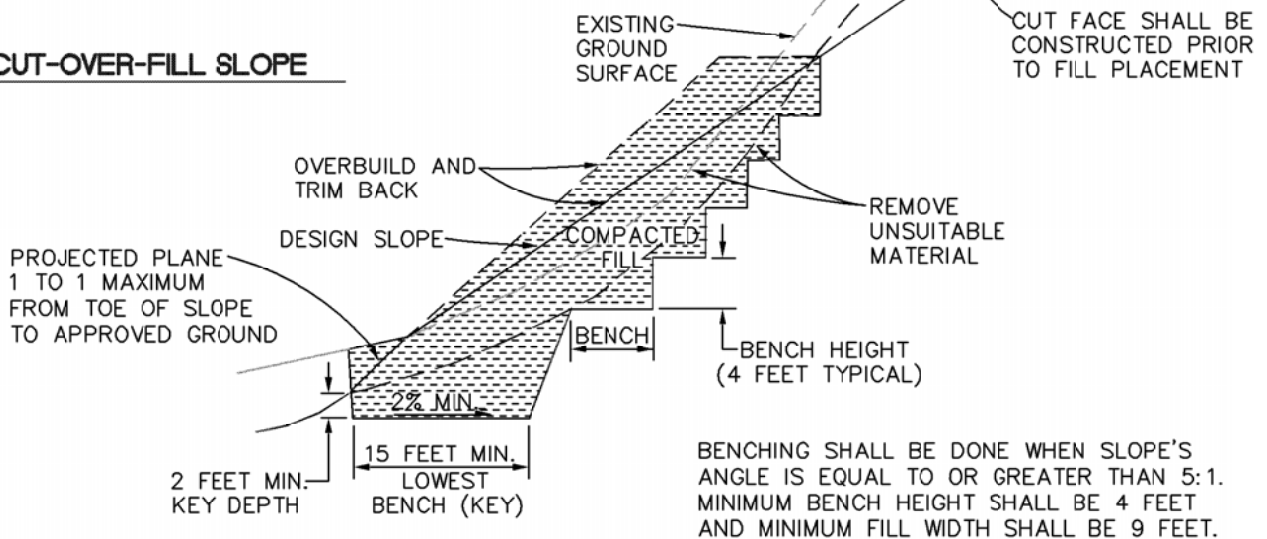
FILL SLOPE



FILL-OVER-CUT SLOPE



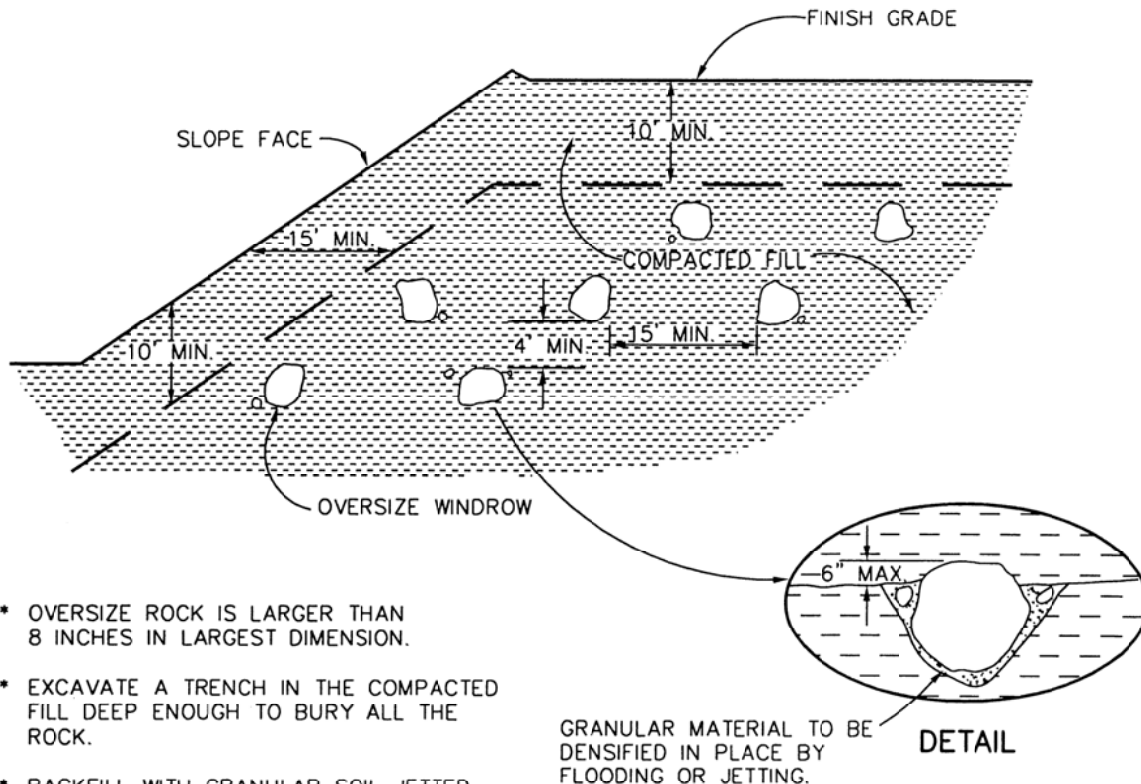
CUT-OVER-FILL SLOPE



KEYING AND BENCHING

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL A

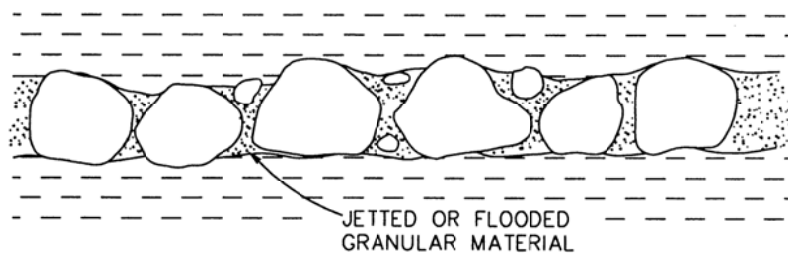




- * OVERSIZE ROCK IS LARGER THAN 8 INCHES IN LARGEST DIMENSION.
- * EXCAVATE A TRENCH IN THE COMPACTED FILL DEEP ENOUGH TO BURY ALL THE ROCK.
- * BACKFILL WITH GRANULAR SOIL JETTED OR FLOODED IN PLACE TO FILL ALL THE VOIDS.
- * DO NOT BURY ROCK WITHIN 10 FEET OF FINISH GRADE.
- * WINDROW OF BURIED ROCK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

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

DETAIL

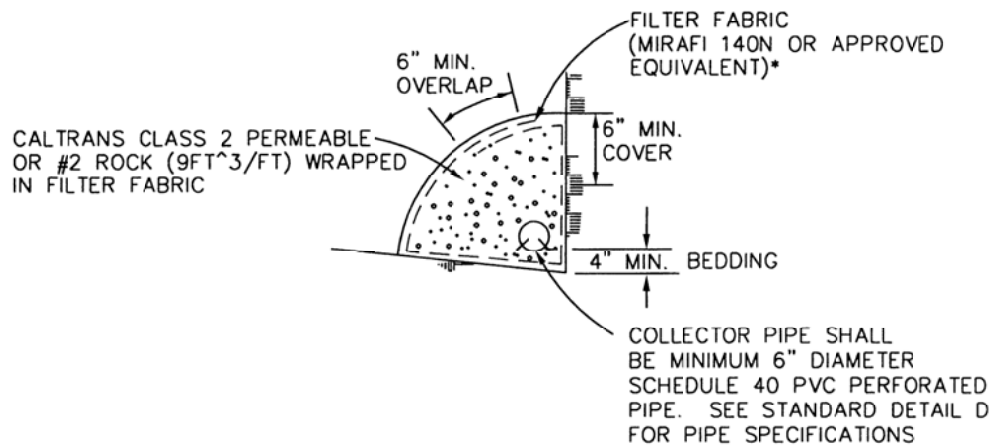
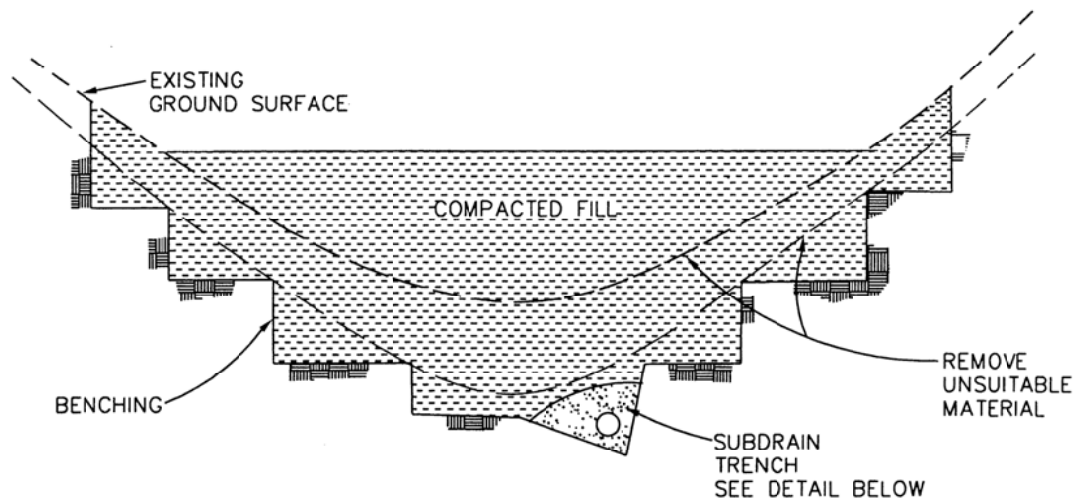


TYPICAL PROFILE ALONG WINDROW

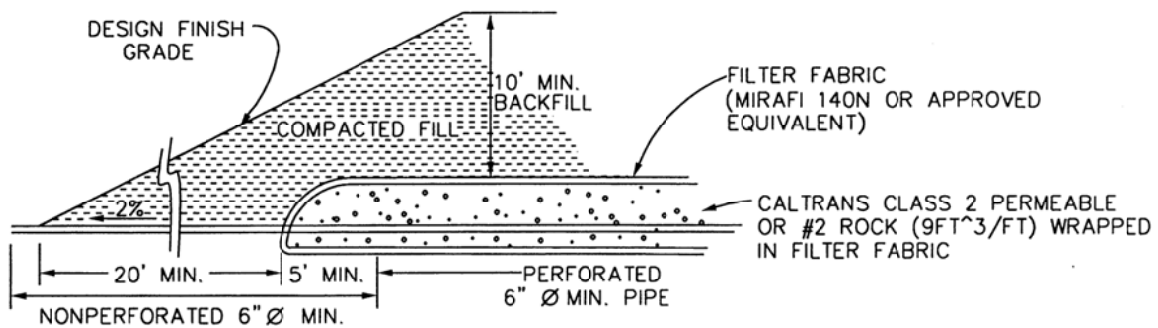
OVERSIZE ROCK DISPOSAL

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL B





SUBDRAIN DETAIL

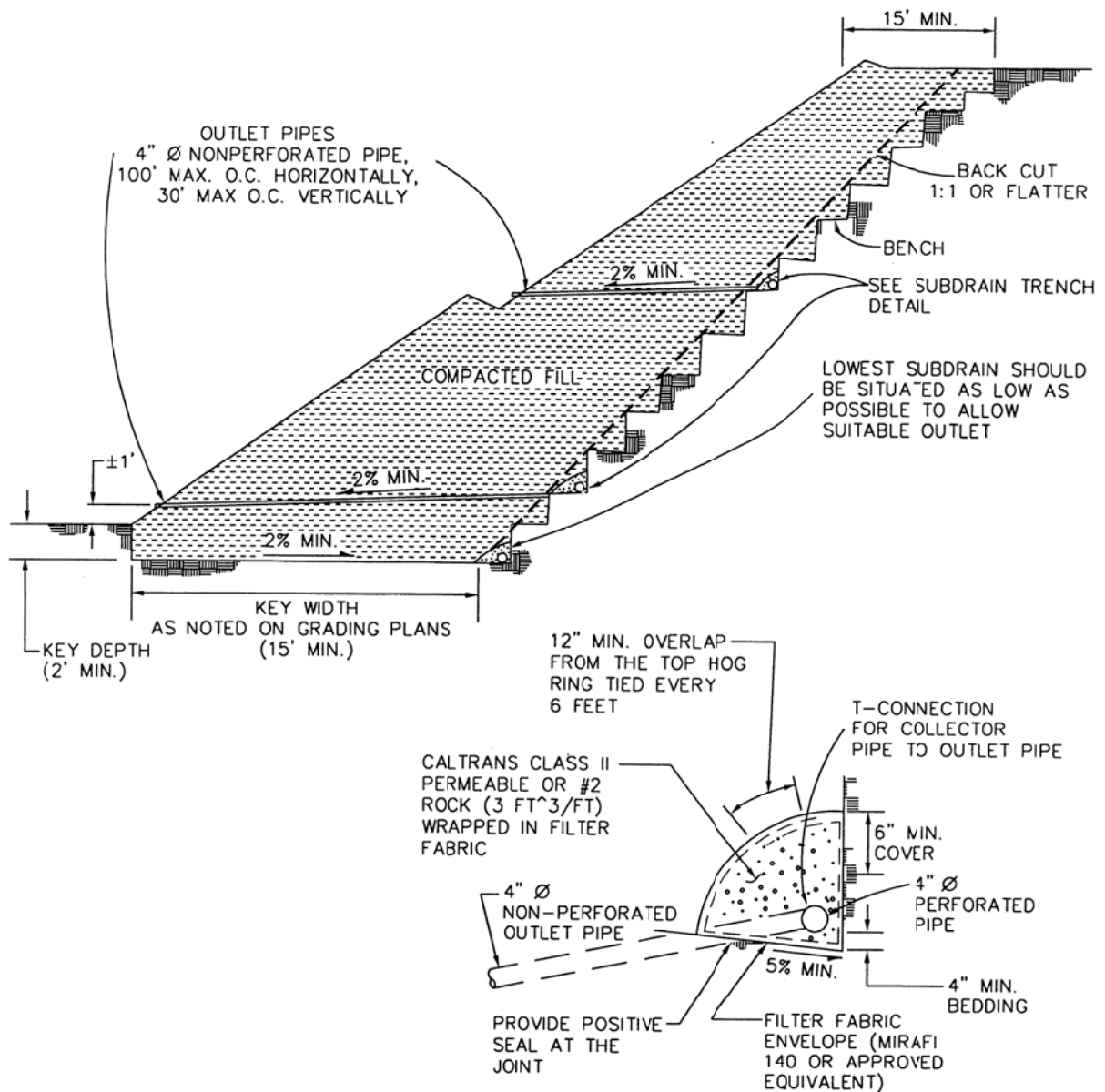


DETAIL OF CANYON SUBDRAIN OUTLET

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.

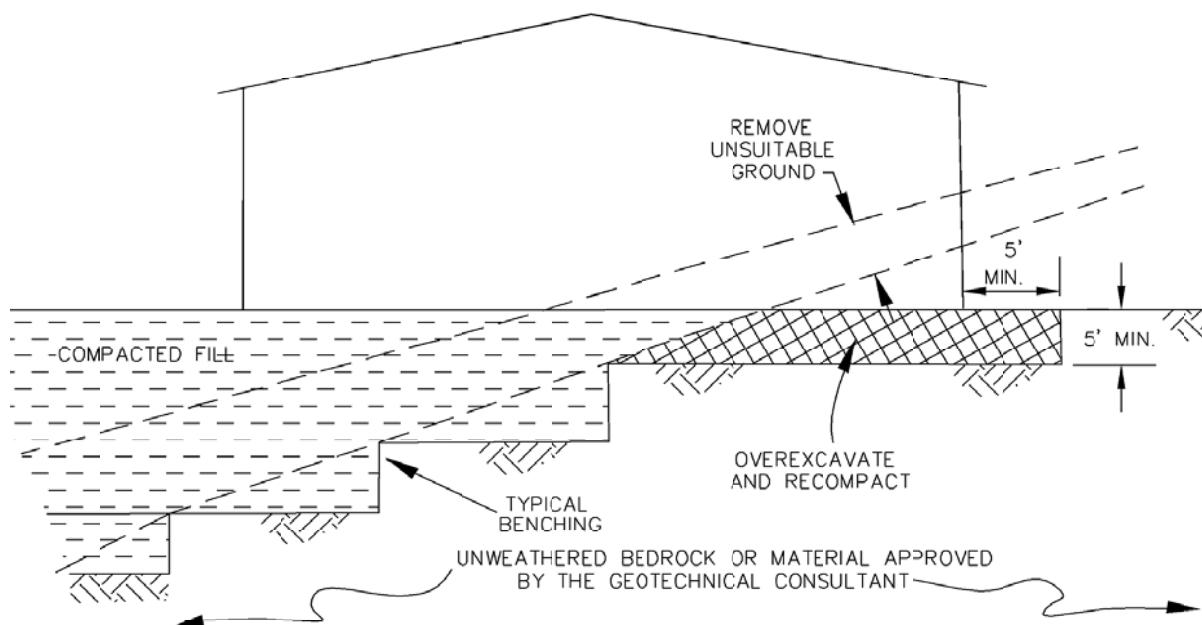
All outlet pipe shall be placed in a trench no wider than twice the subdrain pipe.

**BUTTRESS OR
REPLACEMENT
FILL SUBDRAINS**

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



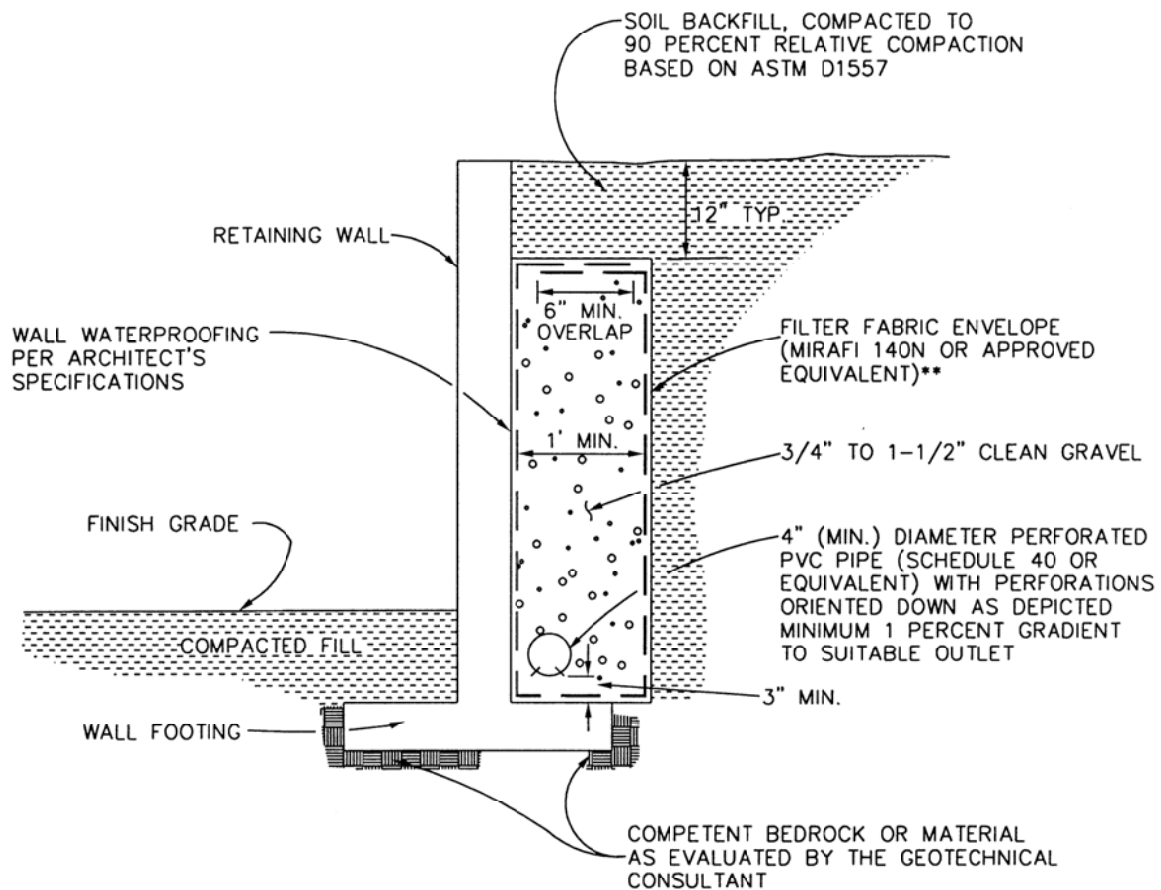
CUT-FILL TRANSITION LOT OVEREXCAVATION



TRANSITION LOT FILLS

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL E



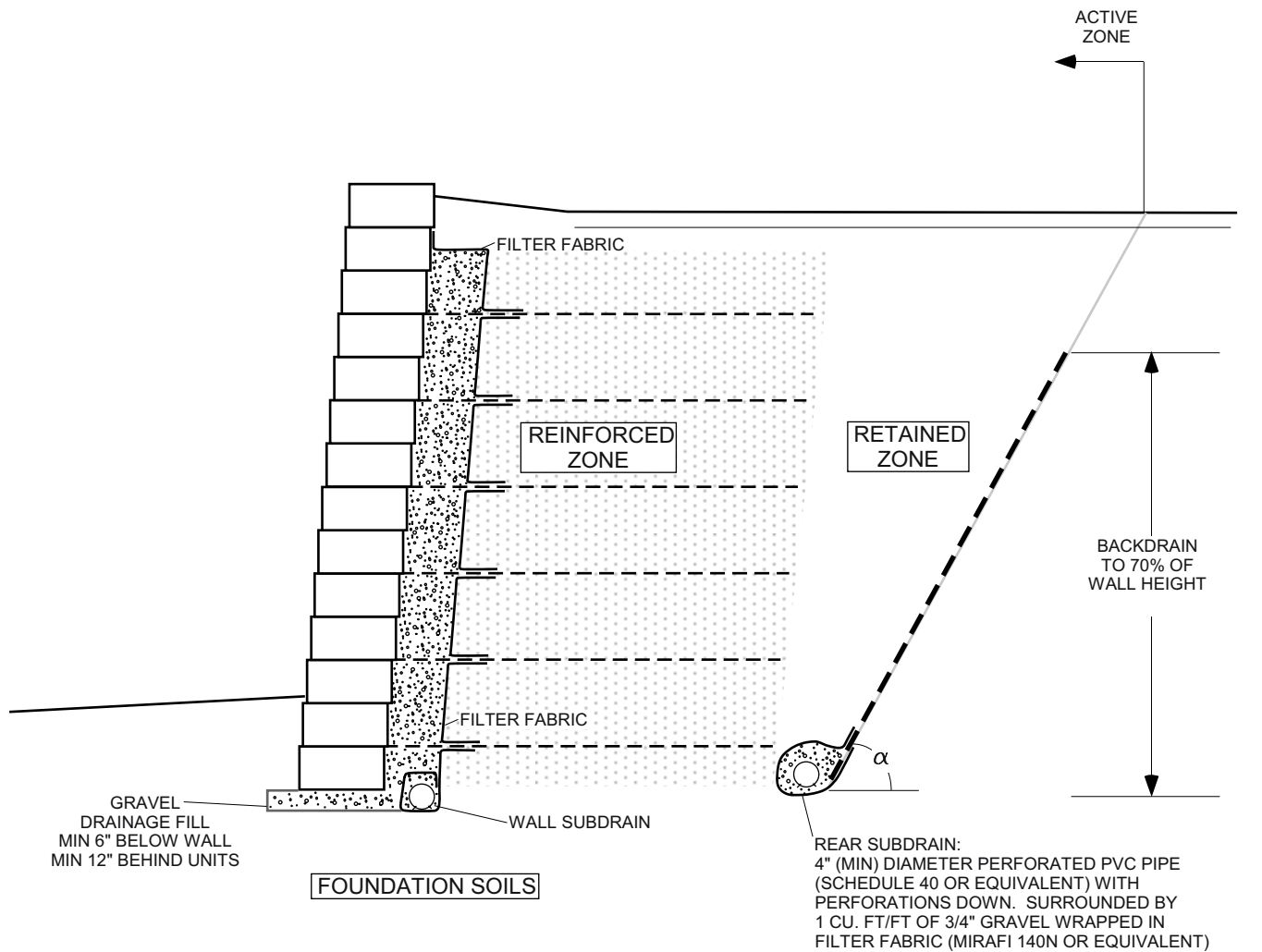


NOTE: UPON REVIEW BY THE GEOTECHNICAL CONSULTANT, COMPOSITE DRAINAGE PRODUCTS SUCH AS MIRADRAIN OR J-DRAIN MAY BE USED AS AN ALTERNATIVE TO GRAVEL OR CLASS 2 PERMEABLE MATERIAL. INSTALLATION SHOULD BE PERFORMED IN ACCORDANCE WITH MANUFACTURER'S SPECIFICATIONS.

RETAINING WALL DRAINAGE

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL F





NOTES:

1) MATERIAL GRADATION AND PLASTICITY
REINFORCED ZONE:

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

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

GRAVEL DRAINAGE FILL:

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

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

- CONTRACTOR TO USE SOILS WITHIN THE RETAINED AND REINFORCED ZONES THAT MEET THE STRENGTH REQUIREMENTS OF WALL DESIGN.
- GEOGRID REINFORCEMENT TO BE DESIGNED BY WALL DESIGNER CONSIDERING INTERNAL, EXTERNAL, AND COMPOUND STABILITY.
- GEOGRID TO BE PRETENSIONED DURING INSTALLATION.
- 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.
- 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

