GEOTECHNICAL INVESTIGATION
PROPOSED MULITI-FAMILY
RESIDENTIAL DEVELOPMENT
845 SANTA FE DRIVE
ENCINITAS, CALIFORNIA

Prepared for:

The Swell Fund
1144 North Coast Highway 101
Encinitas, California 92121

Project No. 12980.001

December 21, 2020
(Revised November 29, 2021)

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The Swell Fund
1144 North Coast Highway 101
Encinitas, California 92121

Attention: Mr. Scott Travasos

Subject: Geotechnical Investigation
Proposed Multi-Family Residential Development
845 Santa Fe Drive, Encinitas, California

In accordance with your request and authorization, we have conducted a geotechnical investigation at the subject site to support the design and construction of the proposed multi-family residential development. The accompanying report presents a summary of our investigation and provides geotechnical conclusions and recommendations relative to the proposed site development.

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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Important Information about This

Geotechnical-Engineering Report

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

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

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

Understand the Geotechnical-Engineering Services Provided for this Report

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

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

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

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

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

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

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

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:
- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

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

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

This Report's Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are <u>Not</u> Covered

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

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

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

GEOPROFESSIONAL BUSINESS ASSOCIATION

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1.0 INTRODUCTION

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

1.1 Purpose and Scope

This report presents the results of our geotechnical investigation for the design and construction of proposed residential development at 845 Santa Fe Drive in Encinitas, California (Figure 1). The intent of this report is to provide specific geotechnical conclusions and recommendations for the currently proposed project.

1.2 Site Location and Description

In general, the site is a rectangular-shaped 2-acre lot that is bounded by Santa Fe Drive to the north, a tennis club to the east, Munevar Road to the south, and residential properties to the west. Currently, Pacific View Baptist Church occupies the eastern portion of the site, while an open grassy field occupies the western portion of the site. The northwestern portion of the site contains an asphaltic concrete parking lot for the church.

Site topography slopes from the northeast to the southwest with surface elevations ranging between approximately 256 to 230 feet above mean sea level (msl). The eastern and western portions of the site are separated by a 10-foot high 2:1 (horizontal:vertical) slope that bisects the site.

Site Latitude and Longitude

33.0353º N

117.2745º W

1.3 Proposed Development

Based on review of a preliminary grading plan and site diagram, we understand that the project will consist of the design and construction of construction of 42 individual lots of which 33 will be single family residences and 8 lots will be multi-family townhomes. The development includes an entryway and driveway with access to all the residential units and parking areas, utilities, a bio-retention basin, landscape and hardscape.

2.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

2.1 Site Investigation

Our field exploration performed on December 12 and 14, 2020, consisted of advancement of 5 hollow-stem auger (HSA) borings to depths between 17 to 26 feet below the existing ground surface with a truck-mounted drill rig and one hand auger excavation to a maximum depth of 5.5 feet. The purpose of our subsurface exploration was to evaluate the underlying stratigraphy, physical characteristics, and specific engineering properties of the soils within the area of the proposed improvements. Bulk samples of the subgrade soils were collected for laboratory testing and evaluation. After logging, the boring locations were backfilled with soil cuttings to match the existing finished surface.

In addition, 4 percolation tests were excavated to a depth of approximately 3 to 4 feet below the existing ground surface. The percolation test well locations were presoaked overnight, and the testing was performed the following day by the falling head method.

The geotechnical boring logs, hand auger log, and percolation tests and are provided in Appendix B. In addition, the boring and percolation test locations are depicted on Figure 2.

2.2 Laboratory Testing

Laboratory testing performed on representative subgrade soils obtained during the recent subsurface exploration included direct shear, expansion index, corrosion, sulfate content, and in-place moisture and density. A summary of the laboratory test results by our office and others is presented in Appendix C.

3.0 SUMMARY OF GEOTECHNICAL CONDITIONS

3.1 Geologic Setting

The project area is situated in the Peninsular Ranges Geomorphic Province. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California, and varies in width from approximately 30 to 100 miles (Norris and Webb, 1990). The province is characterized by mountainous terrain on the east composed mostly of Mesozoic igneous and metamorphic rocks, and relatively low-lying coastal terraces to the west underlain by late Cretaceous-age, Tertiary-age, and Quaternary-age sedimentary units. Most of the coastal region of the County of San Diego occurs within this coastal region and is underlain by sedimentary units. The subject 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. Specifically, the site is located in an area underlain by undocumented artificial fill, which in turn is underlain by the Quaternary-aged Old Paralic Deposits.

3.2 Site-Specific Geology

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 Very Old Paralic Deposits. The approximate areal distribution of the geologic units is depicted on Figure 2. A brief description of the geologic units encountered at the site is presented below. The geotechnical logs with detailed soils descriptions are presented in Appendix B.

3.2.1 Undocumented Artificial Fill – Afu

Based on our subsurface exploration, fill soils were encountered within Borings B-1 to B-3 and HA-1 with a thickness ranging from 2 to 3.5 feet bgs. Where observed in our exploration, the fill materials consisted of loose to medium dense, light brown to grayish brown, moist, clayey sand and silty sand. 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. Therefore, these fills are considered undocumented and may settle under the placement of additional fill and improvement loads.

3.2.2 Quaternary Very Old Paralic Deposits – Qvop13

Underlying existing undocumented artificial fill soils and topsoil, the Quaternary-aged Paralic Deposits were observed within each boring. As encountered, the Paralic Deposits generally consisted of light to grayish-brown, dry, dense to very dense, silty sandstone, clayey sandstone and siltstone. The upper 2 feet of the exposed this formational soil is weathered and considered compressible.

3.3 Surface Water and Groundwater

Evidence of surface water was observed within the earthen drainage ditch along the west side of the property. During rainy periods, surface water may drain across the site and collect within the berm.

Groundwater was not encountered during our subsurface exploration at the site. It should be noted that groundwater levels may fluctuate with seasonal variations and irrigation and local perched groundwater conditions may exist at the contact between the undocumented artificial fill and the Very Old Paralic Deposits. Beyond nuisance seepage into open holes, we do not anticipate groundwater will be a constraint to the development of the site. Seepage may be present at geologic contacts of sandy material and fine-grained material (siltstone). If encountered in cut slopes a subdrain system may be required.

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 soil conditions, the engineering characteristics of the on-site soils are discussed below.

3.4.1 Compressible Soils

The site is underlain by undocumented topsoil, artificial fill, and weathered paralic deposits which are considered compressible. Recommendations for remedial grading and/or ground improvements of these soils are provided in the following sections of this report.

3.4.2 Expansion Potential

Expansion index testing on representative soil samples indicated that the onsite soils generally have a very low to low potential (EI < 50) for expansion (Appendix C). However, higher expansive soils may be encountered during the grading of the site and during foundation excavation. Expansive soils are not anticipated to significantly impact the proposed site improvements.

3.4.3 Soil Corrosivity

A preliminary screening of the on-site soils was performed to evaluate their potential corrosive effect on concrete and ferrous metals. In summary, laboratory testing on one representative soil sample obtained during our subsurface exploration evaluated pH, minimum electrical resistivity, and chloride and soluble sulfate content. The sample tested had measured pH value of 7.9, and a measured minimum electrical resistivity of 600 ohm-cm. Test results also indicated that the sample had a chloride content of 140 parts per million (ppm), and soluble a sulfate content of 270 ppm.

3.4.4 Excavation Characteristics

It is anticipated the onsite soils can be excavated with conventional heavy-duty construction equipment. Localized cemented zones located within the Paralic Deposits, if encountered, may require heavy ripping or breaking. If oversize material (larger than 8 inches in maximum dimensions) is generated, it should be placed in non-structural areas or hauled off site. Zones of friable sands may be encountered within the Paralic Deposits which may experience caving during unsupported excavation or drilling.

3.4.5 Infiltration

Field percolation tests were performed in general accordance with the City of Encinitas BMP Design Manual (2016). Based on our field percolation testing, the in-situ percolation rates and calculated infiltration rates at tested locations and depths are summarized in Table 1. We have used the following equation based upon the Porchet Method to convert measured percolation rates to infiltration rates in accordance with the County of San Diego BMP Design Manual (2020). In addition, we have included a factor of safety of 2 for the evaluation of existing site conditions. The storm water design factor of safety should be determined by the civil engineer and reviewed by the geotechnical consultant. Also, additional field percolation tests may be required within storm water retention areas once final locations are determined by the civil engineer.

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

Where:

I_t = calculated infiltration rate, inches/hour

ΔH = change in head over the time interval, inches

Δt = time interval, minutes

r = radius of test hole

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

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

Table 1
Percolation and Infiltration Rates

Test No.	Depth (ft)	Soil Type	Measured Percolation Rate (mins/in)	Calculated Infiltration Rate (inches/hr)	Recommended Infiltration Rate w/ FS of 2 (inches/hr)
P-1	3.5	Paralic Deposits ($Qvop_{13}$)	250.0	0.013	0.007
P-2	3.6	Paralic Deposits ($Qvop_{13}$)	41.7	0.098	0.049
P-3	3.4	Paralic Deposits ($Qvop_{13}$)	27.8	0.197	0.098
P-4	3.4	Paralic Deposits ($Qvop_{13}$)	50.0	0.083	0.042

Based on the field percolation testing and the recommended calculated infiltration rates, the tested locations are categorized as "Partial Infiltration" conditions, as determined by the City of Encinitas Infiltration Form I-8, Categorization of Infiltration Feasibility Condition, which has been completed and is presented in Appendix D.

It is important to note that percolation rates are not equal to infiltration rates. As a result, we have made a distinction between percolation rates where water movement is considered laterally and vertically versus infiltration rates where only the vertical direction is considered. It should also be noted that the above percolation test results are representative of the tested locations and depths where they were performed, and that percolation test field measurements are accurate to 0.01 feet. Varying subsurface conditions may exist outside of the test locations, which could alter the calculated percolation rate indicated below.

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

4.0 SEISMIC AND GEOLOGIC HAZARDS

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). The primary seismic risk to the site area is the Rose Canyon fault zone located approximately 4 miles west of the site.

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, 2003) as being Earthquake Fault Zones.

4.2 Local Faulting

Our review of available geologic literature (Appendix A) indicates that there are no known Holocene-active or pre-Holocene 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 Holocene-active fault is the Rose Canyon fault zone located approximately 4 miles west of the site (USGS, 2014).

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 4 miles west of the site is considered the 'Holocene-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 previously discussed, no active faults are mapped transecting or projecting toward the site. Therefore, surface rupture hazard due to faulting is considered very low. Ground cracking due to shaking from a seismic event is not considered a significant hazard either, since the site is not located near slopes.

4.4.2 Mapped Fault Zones

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

4.4.3 Site Class

The site is underlain at shallow depth by Quaternary-aged Very Old Paralic Deposits. Based on our experience with similar sites, regional shear wave velocity mapping, and the results of our subsurface evaluation, the site class is characterized by the Site C description of very dense soil and soft rock. However, it has also been our experience that shear wave velocity measurements within these materials can be found to be at the boundary between Site Class D and C materials. For that reason, we have elected to select Site Class D as the default site class, with the constraint that F_a not be less than 1.2 as specified in ASCE 7-16 Section 11.4.4.

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 2 are the spectral acceleration parameters for the project determined in accordance with the 2019 CBC (CBSC, 2019) and SEAOC/OSHPD Seismic Design Maps Web Application (2019). Since the site has an S_1 value greater than 0.2g and site specific ground motion hazard analysis has not been performed, increased values of C_s are required for analysis as summarized in EXCEPTION 2 of ASCE 7-16 Section 11.4.8.

Table 2
2019 CBC Mapped Spectral Acceleration Parameters

Site Class	D (default)		
Site Coefficients	F_a	=	1.200
	F_v	=	1.885
Mapped MCE Spectral Accelerations	S_S	=	1.164g
	S_1	=	0.415g
Site Modified MCE Spectral Accelerations	S_{MS}	=	1.396g
	S_{M1}	=	0.782g
Design Spectral Accelerations	S_{DS}	=	0.931g
	S_{D1}	=	0.522g

If the requirements of EXCEPTION 2 are found to be a significant design constraint, we recommend using the shear wave velocity measurements be at the site for use in performing site specific ground motion analysis.

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 0.525g for the site. For a Site Class D, the F_{PGA} is 1.2 and the mapped peak ground acceleration adjusted for Site Class effects (PGA_M) is 0.624g 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 1613A.2.5 of the 2019 CBC are assigned Seismic Design Category D.

4.5 Secondary Seismic Hazards

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

4.5.1 Liquefaction 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.

Most of the site is underlain at depth by Very Old Paralic Deposits with surficial potentially compressible undocumented artificial fill recommended for removal. Based on the underlying dense character of the Very Old Paralic Deposits and the lack of a shallow ground water table, it is our opinion that the potential for liquefaction and seismic related settlement across the site is low.

4.5.2 Lateral Spread

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

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.

4.5.3 Tsunamis and Seiches

Based upon the California Emergency Management Agency Tsunami Inundation Map (CalEMA, 2009), 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.

4.6 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 massively bedded sandstone. Therefore, the potential for significant landslides or large-scale slope instability at the site is considered low.

4.7 Flood Hazard

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2012); the site is not located within a floodplain. Based on our review of topographic maps, the site is not located downstream of a dam or within a dam inundation area. Based on this review and our site reconnaissance, the potential for flooding of the site is considered low.

5.0 CONCLUSIONS

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

- The undocumented fill and weathered formational materials are potentially compressible in their present state and will require removal and recompaction in areas of proposed improvement or future fill (i.e., remedial grading). Unknown objects such as buried concrete footings and debris left from previous site uses should be anticipated and are common on sites where previous structures existed;
- The site is not transected by either Holocene-active or pre-Holocene faults;
- Based on laboratory testing and site mapping, the site materials possess a very low to low expansion potential. It is possible that higher expansion materials may be encountered in locations not explored;
- The existing onsite granular soils are generally suitable for use as engineered fill, provided they are free of organic material, debris, and rock fragments larger than 6 inches in maximum dimension;
- Based on the results of our subsurface exploration, we anticipate that the onsite materials should be generally excavatable with conventional heavy-duty earthwork equipment. Localized cemented zones within the Paralic Deposits, if encountered, may be difficult to excavate and may require heavy ripping which can produce oversized rock fragments;
- Groundwater was not encountered during our investigation, nor is groundwater anticipated to be encountered during site excavation and construction except as possible seepage during/after episodes of precipitation or in areas of irrigation;
- Based on the results of our geotechnical evaluation, it is our opinion that the proposed improvements can be supported on conventional foundations founded on compacted fill or competent undisturbed Paralic Deposits.
- Although Leighton does not practice corrosion engineering, laboratory test results indicate the soils present on the site have a low potential for sulfate attack on normal concrete. However, the onsite soils are considered to have a corrosive potential for corrosion to buried uncoated ferrous metal. A corrosion consultant may be consulted to provide additional recommendations.

6.0 RECOMMENDATIONS

6.1 Earthwork

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

6.1.1 Site Preparation

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

6.1.2 Remedial Grading

Potentially compressible undocumented fill and weathered formational materials at the site may settle as a result of wetting or settle under the surcharge of engineered fill and/or structure loads supported on shallow foundations. Therefore, remedial grading or removals of the compressible materials is recommended beneath buildings and improvements that are not founded on the underlying competent formation.

Removals should extend to a depth at least 2 feet below the bottom of the footings and at least 5 feet beyond the limits of building footprints. In areas of proposed pavements, vehicular pavers, and hardscape, removals should be performed to a depth of at least 18 inches feet below proposed subgrade or existing site topography, whichever is deeper, and extend at least 2 feet beyond the limits of the proposed improvements. Isolated deeper removals may be necessary depending on the differential fill thickness, pad over excavations, and depth to competent formational material. The bottom of all removals should be evaluated by a Certified Engineering Geologist to confirm conditions are as anticipated. In addition, the actual depth and extent of the required removals should be confirmed during grading operations by the geotechnical consultant.

6.1.3 Fill Placement and Compaction

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 D1557; 95 percent for wall backfill soils or when wall backfill soils are 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.

In vehicle pavement areas, the upper 12 inches of subgrade soils should be scarified then moisture conditioned to a moisture content above optimum content and compacted to 95 percent or more relative to the maximum laboratory dry density, as evaluated by ASTM D 1557.

Placement and compaction of fill should be performed in general accordance with current City of Encinitas grading ordinances, California Building Code and sound construction practices, these recommendations, and the General Earthwork and Grading Specifications for Rough Grading presented in Appendix E.

6.1.4 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 3-inches. Compaction by jetting or flooding may not be performed.

6.1.5 Import Soils

If import soils are necessary to bring the site up to the proposed grades, these soils should be granular in nature, and have an expansion index less than 50 (per ASTM Test Method D4829) and have a low corrosion impact to the proposed improvements. Beneath pavements, subgrade materials should possess an R-value of 30, or greater. Import soils and/or the borrow site location should be evaluated by the geotechnical consultant prior to import.

6.1.6 Earthwork Shrinkage/Bulking

The volume change of excavated onsite materials upon recompaction as fill is expected to vary with material and location. The undocumented fill, alluvium should consider 5 to 10 percent of shrinkage. Typically, the Paralic Deposits vary significantly in natural and compacted density, and therefore, accurate earthwork shrinkage/bulking estimates cannot be determined. However, based on the results of our geotechnical analysis and our experience, a 3 to 5 percent bulking factor is considered appropriate for the Paralic Deposits.

6.2 Cut/Fill Transition Mitigation

Based on review of preliminary grading plans, it is our understanding that some of the proposed structures will be situated where a cut/fill transition occurs beneath the structure. To mitigate the impact of the underlying cut/fill transition condition beneath a structure, the shallow formational materials should be over-excavated to at least 1/3 of the removal depth below finish grade, or 2 feet below the bottoms of proposed foundations, whichever is deeper. Alternatively, all footings for the proposed structures can be extended through the engineered fill and a minimum of 6 inches into competent formational material. The additional depth can be filled with concrete or controlled low-strength material (CLSM) prior to placement of foundation reinforcing steel and concrete.

6.3 Temporary Excavations

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

Table 3
Maximum Slope Ratios

Excavation Depth (feet)	Maximum Slope Ratio In Fill Soils	Maximum Slope Ratio In Competent Formation
0 to 5	1:1 (Horizontal to Vertical)	Vertical
5 to 20	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 is present within 10 feet of the top of slope. Care should be taken during design of excavations adjacent to the existing structures so that foundation support is preserved. A "competent person" should observe the slope on a daily basis for signs of instability. All excavations should comply with current OSHA requirements.

6.4 Slope Stability

Based on our experience, permanent cut slopes within the paralic deposits with maximum heights of roughly 25 feet and gradients of 2:1 (horizontal to vertical) are generally considered stable provided they are free of adverse geologic conditions. Cut slopes should be geologically mapped during grading to evaluate the exposed conditions. Care should be taken not to over excavate proposed cut slopes. Care should be taken to not "paste" fill back onto these areas.

We anticipate the project development plans have fill slopes proposed at inclinations of 2:1 (horizontal to vertical), or flatter, with maximum heights on the order of 10 feet with proper surface drainage benches.

Slope stability analyses on all cut slopes, fill slopes and existing slopes to remain in place should be performed once final grading plans are complete.

Cut and fill slopes should be provided with appropriate surface drainage features and landscaped with drought-tolerant, slope-stabilizing vegetation as soon as possible after grading to reduce the potential for erosion. Berms should be provided at the top of fill slopes, and brow ditches should be constructed at the top of cut slopes. Inadvertent oversteepening of cut and fill slopes should be avoided during fine grading. If seepage is encountered in slopes, special drainage features may be recommended by the geotechnical consultant.

We recommend against exclusive use of generally cohesionless sand in the slope faces, as these materials are prone to erosive rilling. In addition, expansive clayey soils, if placed within 15 feet of the slope face, may be subject to surficial instability. We recommend that clayey soils be thoroughly mixed with poorly graded sands to produce better quality fill material which will be more effective in reducing erosion and increasing surficial stability.

6.4.1 Setback from Slopes

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 following table. This distance is measured from the outside bottom edge of the footing, horizontally to the slope or retaining wall face and is based on the slope or wall height. The foundation setback distance may be revised by the geotechnical consultant on a case-by-case basis if the geotechnical conditions are different than anticipated.

Table 4

Minimum Foundation Setback from Slope Faces

Slope Height	Minimum Recommended Foundation Setback
less than 5 feet	7 feet
5 to 20 feet	10 feet
greater than 20 feet	H/2, where H is slope height; not to exceed 15 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. The deepened footing should meet the setback as described above.

6.5 Foundation and Slab Considerations

Conventional Foundations

The proposed structures and buildings may be supported by conventional, continuous or isolated spread footings. Footings should extend a minimum of 24 inches beneath the lowest adjacent soil grade. At these depths, footings may be designed for a maximum allowable bearing pressure of 3,000 pounds per square foot (psf) if founded in properly compacted fill soils. For buildings founded entirely within undisturbed Paralic Deposits, footings may be designed for a maximum allowable bearing pressure of 5,000 psf. The bearing pressure for miscellaneous site retaining walls and other at-grade improvements should be limited to 2,000 psf. The allowable pressures may 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.

The recommended allowable-bearing capacity is based on maximum total and differential settlements of 1 inch, and ¾ of an inch, respectively. Since settlements are a function of footing size and contact bearing pressures, some differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists.

Floor Slabs

Slab-on-grade should be at least 5 inches thick and be reinforced with No. 4 rebars 18 inches on center each way (minimum) placed at mid-height in the slab. The slab should be underlain by a moisture barrier which consists of 2-inch layer of clean sand (S.E. greater than 30) over a 10-mil non-recycled plastic sheeting, which is in turn underlain by an additional 2-inches of clean sand. Note that moisture barriers can retard, but not eliminate moisture vapor movement from the underlying soils up through the slabs.

We also recommend that the floor covering installer test the moisture vapor flux rate prior to attempting applications of the flooring. "Breathable" floor coverings should be considered if the vapor flux rates are high. A slip-sheet or equivalent should be utilized above the concrete slab if crack-sensitive floor coverings (such as ceramic tiles, etc.) are to be placed directly on the concrete slab. Additional guidance is provided in ACI Publications 302.1R-04 Guide for Concrete Floor and Slab Construction and 302.2R-06 Guide for Concrete Slabs that Receive Moisture-Sensitive Floor Materials.

We also recommend that soil-moisture around the immediate perimeter of the slab be maintained at above optimum-moisture content during construction and up to occupancy of the homes. Future building owners should be informed and educated regarding the importance of maintaining a constant level of soil-moisture. The owners should be made aware of the potential negative consequences of both excessive watering, as well as allowing expansive soils to lose moisture (i.e., the soil will undergo shrinkage as it dries up, followed by swelling during the winter, rainy season or when irrigation is resumed, resulting in distress to improvements and structures).

The potential for slab cracking may be reduced by careful control of water/cement ratios. The contractor should take appropriate curing precautions during the pouring of concrete in hot weather to minimize cracking of the slabs. We recommend that a slipsheet (or equivalent) be utilized if grouted tile, marble tile, or other crack-sensitive floor covering is planned directly on concrete slabs. All slabs should be designed in accordance with structural considerations.

If heavy vehicle or equipment loading is proposed for the slabs, greater thickness and increased reinforcing may be required. The additional measures should be designed by the structural engineer using a modulus of subgrade reaction of 150 pounds per cubic inch. Additional moisture/waterproofing measures that may be needed to accomplish desired serviceability of the building finishes and should be designed by the project architect.

6.6 Lateral Resistance and Retaining Wall Design Parameters

Retaining walls should be designed for the lateral soil pressures exerted on them, the magnitude of which depends primarily on the type of soil used as backfill and the amount of deformation the wall can yield under the lateral load. If a retaining wall can yield enough to mobilize the full shear strength of the soil, it can be designed for the 'active' pressure condition. Walls that are under restrained conditions and cannot yield under the applied load (e.g., basement walls) should be designed for the 'at-rest' pressure condition. If a wall tends to move towards the soils, the resulting resistance developed by the soil is the 'passive' resistance.

For design purposes, the following lateral earth pressure values for level or sloping backfill are recommended for walls backfilled with onsite soils of very low to low (EI<50) expansion potential or undisturbed in-place materials.

Table 5
Static Equivalent Fluid Weight (pcf)

Conditions	Level	2:1 Slope
Active	35	65
At-Rest	55	80
Passive	350 (Maximum of 3 ksf)	200 (sloping down)

If conditions other than those covered herein are anticipated, the equivalent fluid pressure values should be provided on an individual case basis by the geotechnical engineer. 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. Surcharge loading from adjacent structures to the east should also be taken into account during wall design. For other uniform surcharge loads, a uniform pressure equal to 0.35q should be applied to the wall (where q is the surcharge pressure in psf).

The provided wall pressures assume walls are backfilled with free draining materials and water is not allowed to accumulate behind walls. Specifically, where walls are not designed to consider hydrostatic conditions, in order to mitigate the potential for hydrostatic build-up behind the basement walls, drainage board should be extended from 2 feet below the ground surface to outlet drain or by piping to a sump at the lowest wall elevations. Waterproofing should be designed by the structural engineer and/or architect.

Where wall backfill is utilized, it should be compacted by mechanical methods to at least 90 percent relative compaction (based on ASTM D1557). We recommend compaction effort be increased to 95 percent where backfill will support structural foundations. Wall footings should be designed in accordance with the foundation design recommendations and reinforced in accordance with structural considerations.

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

The account for potential redistribution of forces during a seismic event, walls should also be checked considering an additional seismic pressure distribution equal to 9H psf applied as a uniform pressure, where H equals the overall retained height in feet. If conditions other than those covered herein are anticipated, the equivalent fluid pressure values should be provided on an individual case basis by the geotechnical engineer.

6.7 Control of Ground Water and Surface Water

Our experience indicates that surface or near-surface ground water conditions can develop in areas where ground water conditions did not exist prior to site development, especially in areas where a substantial increase in surface water infiltration results from landscape irrigation. This sometimes occurs where relatively impermeable bedrock materials are overlain by granular fill soils. In addition, during slope excavations, seepage in cut slopes may be encountered. We recommend than an engineering geologist be present during grading operations to evaluate seepage areas. Drainage devices for reduction of water accumulation can be recommended when these conditions are observed.

We recommend that measures be taken to properly finish grade each building area, such that drainage water from the building area is directed away from building foundations (2 percent minimum grade for a distance of 5 feet), floor slabs, and tops of slopes. Ponding of water should not be permitted, and installation of roof gutters which outlet into a drainage system is considered prudent. Planting areas at grade should be provided with positive drainage directed away from the building. Drainage and subdrain design for these facilities should be provided by the design civil engineer.

Where desilting basins are proposed, seepage may occur along the outflow structure. To minimize the potential for seepage in this area, we recommend that cut-off walls be incorporated into the design. Cut-off walls should have a minimum width and extend for at least 12 inches beyond the sides of the trench excavated for the outlet pipe. Cut off walls should extend at least 24 inches above the top of pipe, at least 12 inches below the trench bottom, and consist of poured-in-place concrete.

Regarding Best Management Practices (BMP) and Low Impact Development (LID) measures, we are of the opinion that infiltration basins, and other on-site storm water retention and infiltration systems can potentially create adverse perched groundwater conditions, both on-site and off-site, when not installed using proper design recommendations (such as the use of liners) and infiltration design parameters. Due to the dense nature of the Paralic Deposits and existing site constraints and conditions, we do not recommend infiltration of surface storm water into the existing site soils without mitigation measures. Low Impact Development (LID) BMPs that contain, and filter surface waters (flow-through planters and bioretention areas) are acceptable provided that the side walls are lined with an impermeable liner and have subdrain systems that tie into an approved existing or proposed storm drain system.

6.8 Preliminary Pavement Design

The preliminary pavement section design below is based on an assumed Traffic Index (TI), our visual classification of the subject site soils, experience with other projects in the area, and our limited laboratory testing. Actual pavement recommendations should be based on R-value tests performed on bulk samples of the soils that are exposed at the finished subgrade elevations across the site at the completion of the mass grading operations. Preliminary flexible pavement sections have been evaluated in general accordance with the Caltrans method for flexible pavement design. Based on an assumed R-value of 10, preliminary pavement sections for planning purposes is given in table below:

Table 6 Preliminary Pavement Sections		
Assumed Traffic Index (TI)	Asphalt Concrete (inches)	Aggregate Base (inches)
4.5	4.0	5.0
5.0	4.0	7.0
6.0	4.0	11.0

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 American Standard of Testing and Materials (ASTM) Test Method D1557.

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

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

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

For areas subject to regular truck loading (i.e., trash truck apron), we recommend a full depth of Portland Cement Concrete (PCC) section of 7 inches with appropriate steel reinforcement and crack-control joints as designed by the project structural engineer. We recommend that sections be as nearly square as possible. A 3,500-psi mix that produces a 550-psi modulus of rupture should be utilized.

All pavement section materials should conform to and be placed in accordance with the latest revision of the California Department of Transportation Standard Specifications (Caltrans) and American Concrete Institute (ACI) codes. The upper 12 inches of subgrade soil and all aggregate base should be compacted to a relative compaction of at least 95 percent (based on ASTM Test Method D1557) and to a moisture content above optimum content.

6.9 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. If expansive soil (EI greater than 20) is encountered, flatwork should be reinforced with No. 4 bars at 24 inches on center. In addition, flatwork near curbs, Americans with Disabilities Act (ADA) ramps, and entry ways, should be dowelled into curbs and flatwork.

6.10 Construction Observation

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

6.11 Plan Review

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

7.0 LIMITATIONS

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

Figures

Project: 12980.001	Eng/Geol: NT/MDJ
Scale: 1 " = 2,000 '	Date: December 2020

Base Map: Bing Maps 2020
Thematic Information: Leighton
Author: (mmurphy)

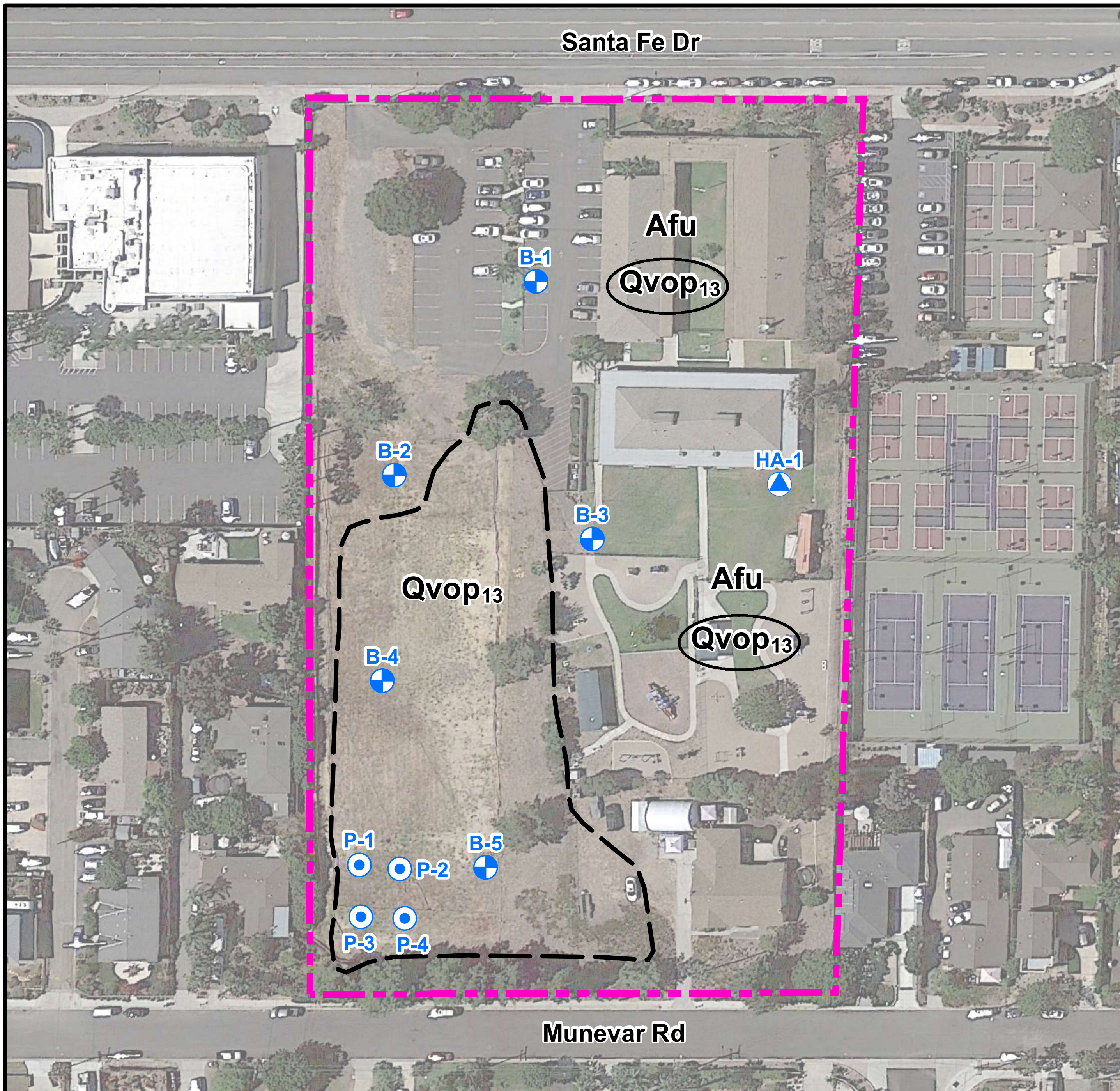
SITE LOCATION MAP

845 Santa Fe Drive Multi-Family
Residential Development
Encinitas, California

Figure 1

Leighton

Map Saved as V:\Drafting\12980\001\Maps\12980-001_F01_SLM_2020-12-16.mxd on 12/16/2020 11:47:56 AM



Legend

B-5 Approximate Location of Boring

HA-1 Approximate Location of Hand Auger

P-4 Approximate Location of Percolation Test

Approximate Geologic Contact

Approximate Site Boundary

Geologic Units

Afu Artificial Fill (Undocumented)

Qvop$_{13}$ Quaternary Very Old Paralic Deposits (circled where buried)

N

0 100 200

Feet

Project: 12980.001	Eng/Geol: NT/MDJ
Scale: 1 " = 100 '	Date: December 2020

Base Map: Googlel Earth, 2020.
Thematic Information: Leighton
Author: (mmurphy)

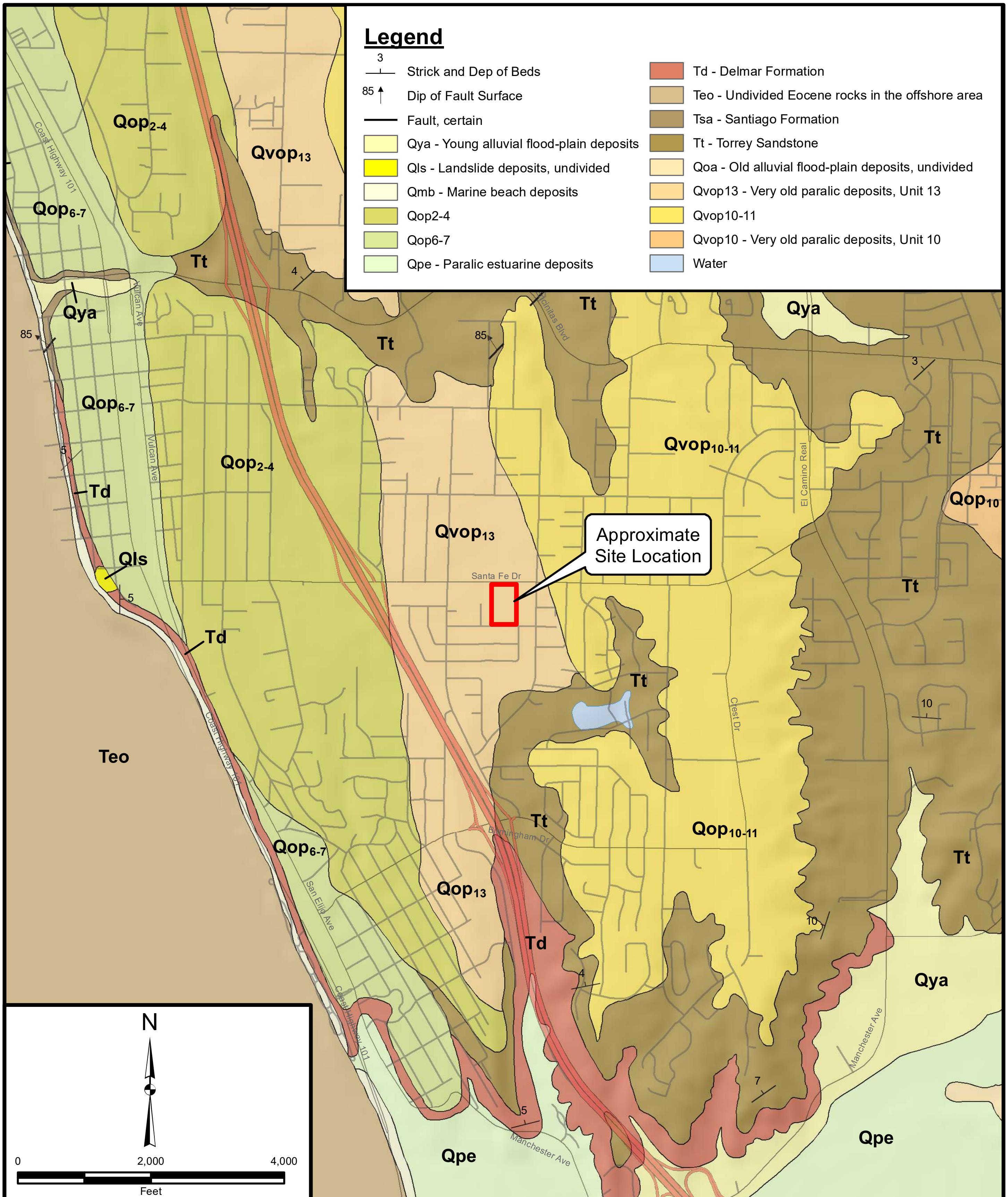
GEOTECHNICAL MAP

845 Santa Fe Drive Multi-Family
Residential Development
Encinitas, California

Figure 2

Leighton

Map Saved as V:\Drafting\12980\001\Maps\12980-001_F02_GM_2020-12-16.mxd on 12/18/2020 10:19:24 AM



Project: 12980.001	Eng/Geol: NT/MDJ
Scale: 1 " = 2,000 '	Date: December 2020

Base map: Kennedy, M.P. and Tan, S.S, 2008, Geologic Map of the Oceanside 30' X 60' Quadrangle, California Compiled by Michael P. Digital Preparation by Kelly R. Bovard, Anne G. Garcia and Diane Burns, California Geological Survey.
Author: (mmurphy)

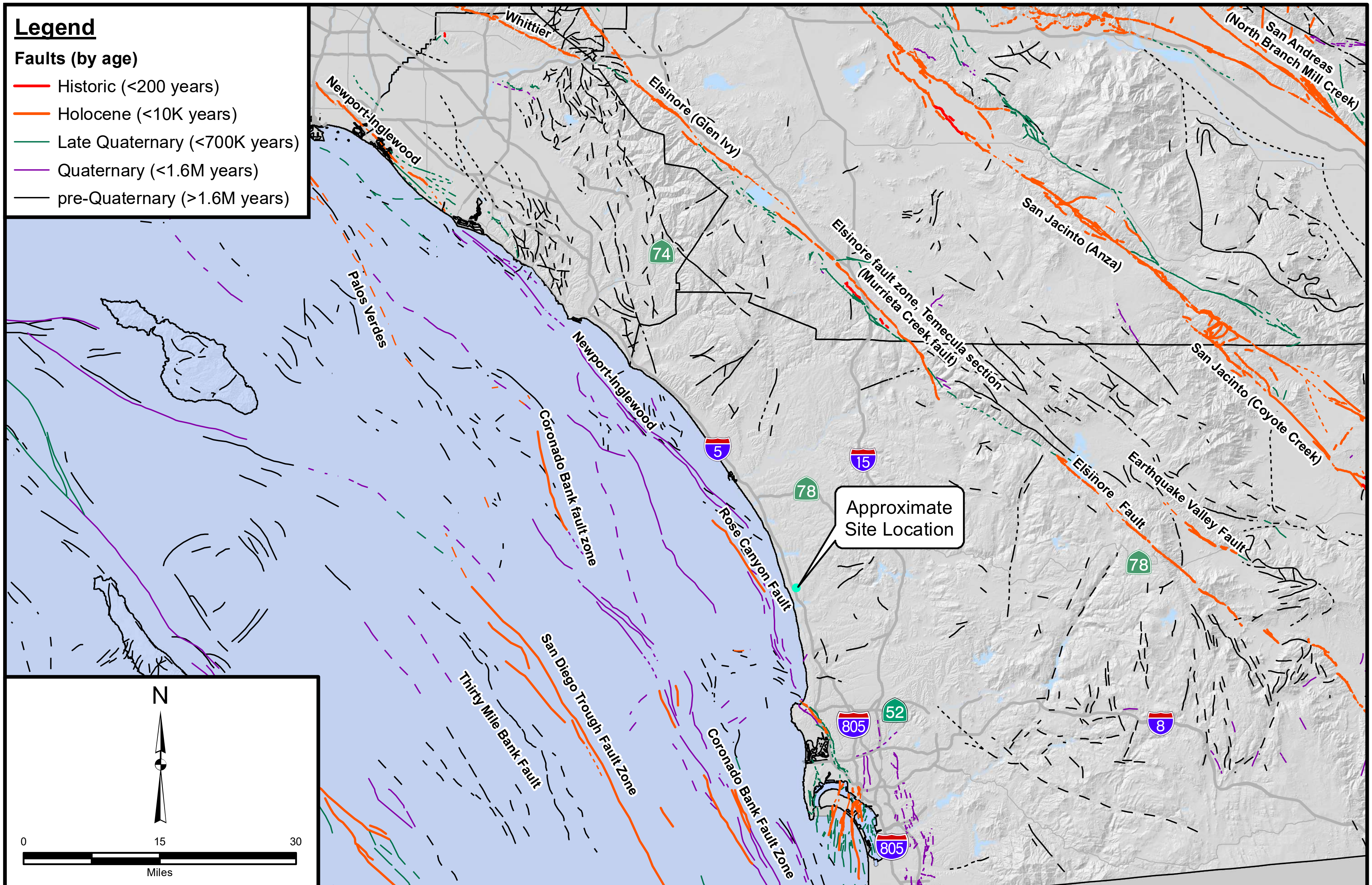
GEOLOGIC MAP

845 Santa Fe Drive Multi-Family Residential Development
Encinitas, California

Figure 3

Leighton

Map Saved as V:\Drafting\12980\001\Maps\12980-001_F03_GM_2020-12-16.mxd on 12/16/2020 1:25:32 PM



Project: 12980.001	Eng/Geol: NT/MDJ
Scale: 1 " = 15 miles	Date: December 2020

Faults: Bryant, Bryant CGS 2010
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

REGIONAL FAULT MAP

845 Santa Fe Drive Multi-Family
Residential Development
Encinitas, California

Figure 4

Leighton

Map Saved as V:\Drafting\12980\001\Maps\12980-001_F04_RFM_2020-12-16.mxd on 12/16/2020 1:28:58 PM

Appendix A
References

APPENDIX A

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Appendix B
Geotechnical Boring and Percolation Logs

GEOTECHNICAL BORING LOG KEY

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

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

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

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

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GEOTECHNICAL BORING LOG B-1

Project No.	12980.001	**Date Drilled**	12-12-20
Project	The Swell Fund/ 845 Santa Fe Drive	**Logged By**	NT
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	245' msl
Location	See Figure 2	**Sampled By**	NT

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
245	0								5" of ASPHALT CONCRETE	
				B-1 1-5'				SC	ARTIFICIAL FILL (Afu) @ 5": Clayey SAND, medium dense, reddish brown, moist, fine-grained	
								SC	VERY OLD PARALIC DEPOSITS (Qvop13) @ 2': Clayey SANDSTONE, grayish brown, medium dense, some minor orange colored iron oxide staining, highly weathered	
240	5			R-1	2 5 13					
235	10			R-2	50/6"			SM	@ 10': Silty SANDSTONE, very dense, gray with abundance of orange colored iron oxide @ 12': hard drilling	
230	15			R-3	22 50/3"			SC	@ 15': Clayey SANDSTONE, very dense, gray in color	
225	20			R-4	50/6"			SM	@ 20': Silty SANDSTONE, becomes very dense	
									Total Depth = 20.5 Feet (bgs) **No Groundwater Encountered at Time of Drilling** **Backfilled with Bentonite on 12/12/2020**	
220	25									
215	30									

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

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

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GEOTECHNICAL BORING LOG B-2

Project No.	12980.001	**Date Drilled**	12-12-20
Project	The Swell Fund/ 845 Santa Fe Drive	**Logged By**	NT
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	237' msl
Location	See Figure 2	**Sampled By**	NT

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			B-1 2"-5'				SM	2" Topsoil ARTIFICIAL FILL (Afu) @ 2": Silty SAND, medium dense, light brown, damp	
235								SM	VERY OLD PARALIC DEPOSITS (Qvop13) @ 1.5': Silty SANDSTONE, light brown, dense, medium-grained	
	5			R-1	18 25 27					
230									@ 8': hard drilling	
	10			S-1	13 19 40				@ 10': Becomes very dense, dark gray	
225									@ 13': slow drilling	
	15			S-2	32 40 50/3"				@ 15': Becomes brown in color, fine-grained	
220									**Total Depth = 17 Feet (bgs)** **No Groundwater Encountered at Time of Drilling** **Backfilled with Bentonite on 12/12/2020**	
	20									
215										
	25									
210										
	30									

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

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

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

GEOTECHNICAL BORING LOG B-3

Project No.	12980.001	**Date Drilled**	12-12-20
Project	The Swell Fund/ 845 Santa Fe Drive	**Logged By**	NT
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	245' msl
Location	See Figure 2	**Sampled By**	NT

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
245	0								2" Topsoil	
				B-1 1-5'				SC	ARTIFICIAL FILL (Afu) @ 2": Clayey SAND, loose to medium dense, brown, moist to very moist	
								SC	VERY OLD PARALIC DEPOSITS (Qvop13) @ 4': Clayey SANDSTONE, gray with orange colored iron oxide, very dense	
240	5			R-1	50/5"					
235	10			R-2	50/6"					
230	15			S-1	31 50/4"					
									@ 16': slow drilling	
				R-3	50/5"					
									Total Depth = 18.5 Feet (bgs) **No Groundwater Encountered at Time of Drilling** **Backfilled with Bentonite on 12/12/2020**	
225	20									
220	25									
215	30									

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

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

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

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GEOTECHNICAL BORING LOG B-4

Project No.	12980.001	**Date Drilled**	12-12-20
Project	The Swell Fund/ 845 Santa Fe Drive	**Logged By**	NT
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	234' msl
Location	See Figure 2	**Sampled By**	NT

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0							SM	2" Topsoil	
				B-1 1-5'					VERY OLD PARALIC DEPOSITS (Qvop13) @ 2": Silty SANDSTONE, light brown, damp, dense, fine-grained	
230	5			R-1	10 34 34				@5': Becomes medium-grained @ 6': hard drilling	
225	10			S-1	50/5"				@ 10': Becomes very dense, light gray with orange colored iron oxide	
220	15			R-2	12 22 50			ML	@ 15': Sandy SILTSTONE, Hard, light gray with orange colored iron oxide	
215	20			S-2	50/4"				@ 20': No recovery	
210	25			S-3	34 50/4"				@ 25': SILTSTONE, gray, some iron oxide steaks	
									Total Depth = 26 Feet (bgs) **No Groundwater Encountered at Time of Drilling** **Backfilled with Bentonite on 12/12/2020**	
205	30									

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

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

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GEOTECHNICAL BORING LOG B-5

Project No.	12980.001	**Date Drilled**	12-12-20
Project	The Swell Fund/ 845 Santa Fe Drive	**Logged By**	NT
Drilling Co.	Baja Exploration	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	234' msl
Location	See Figure 2	**Sampled By**	NT

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0								2" Topsoil	
				B-1 1-5'				SM	VERY OLD PARALIC DEPOSITS (Qvop13) @ 2": Silty SANDSTONE, light brown, damp, medium dense to dense, fine-grained	
230	5			R-1	20 18 29			SC	@ 5': Clayey SANDSTONE, dense, light brown	
225	10			S-1	15 27 20			SM	@ 10': Becomes Silty SANDSTONE, light brown	
									@ 12': hard drilling	
220	15			R-2	33 50/4"				@ 15': Becomes gray in color with some iron oxide steaks	
215				S-2	10 27 50				@ 18.5': Becomes fine-grained	
	20								**Total Depth = 20 Feet (bgs)** **No Groundwater Encountered at Time of Drilling** **Backfilled with Bentonite on 12/12/2020**	
210	25									
205	30									

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

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

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GEOTECHNICAL BORING LOG HA-1

Project No.	12980.001	**Date Drilled**	12-12-20
Project	The Swell Fund/ 845 Santa Fe Drive	**Logged By**	NT
Drilling Co.	Baja Exploration	**Hole Diameter**	3"
Drilling Method	Hand Auger	**Ground Elevation**	245' msl
Location	See Figure 2	**Sampled By**	NT

SOIL DESCRIPTION

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

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
245	0								6" Topsoil	
								SC	ARTIFICIAL FILL (Afu) @ 6": Clayey SAND, loose, brown, very moist	
								SC	VERY OLD PARALIC DEPOSITS (Qvop13) @ 3.5': Clayey SANDSTONE, gray brown, medium dense, moist	
240	5									
									Total Depth = 5.5 Feet (bgs) **No Groundwater Encountered at Time of Drilling** **Backfilled with Bentonite on 12/12/2020**	
235	10									
230	15									
225	20									
220	25									
215	30									

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

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

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FIELD PERCOLATION TEST DATA SHEET

Project Name:	The Swell Fund - 845 Santa Fe Drive Multi-Family	Project No.:	12980.001
Proj. Address:	845 Santa Fe Drive, Encinitas		

SOIL TYPE / TEST LOCATION / BOREHOLE

Soil Type: SM — Hole #: P-1

Location: See Map

Hole Dia: 8"

Depth 3.46'

Tested by: Reese Davis — Test Date: 12.14.2020

Notes: Measurements in 1/100ths of feet (ft)

Time of Day	Interval / Notes	Initial Depth to Water (ft)	Final Depth of Water (ft)	Δ in Water Level (ft)	Percolation Rate (min/inch)
11:08 AM	Start	0.55			-
11:38 AM	30	0.55	0.56	0.01	250.00
12:08 PM	30	0.56	0.58	0.02	125.00
12:38 PM	30	0.58	0.59	0.01	250.00
1:08 PM	30	0.59	0.60	0.01	250.00
1:38 PM	30	0.60	0.61	0.01	250.00
2:08 PM	30	0.61	0.63	0.02	125.00
2:38 PM	30	0.63	0.65	0.02	125.00
3:08 PM	30	0.65	0.66	0.01	250.00

Leighton

FIELD PERCOLATION TEST DATA SHEET

Project Name:	The Swell Fund - 845 Santa Fe Drive Multi-Family	Project No.:	12980.001
Proj. Address:	845 Santa Fe Drive, Encinitas		

SOIL TYPE / TEST LOCATION / BOREHOLE

Soil Type: SM   Hole #: P-2

Location: See Map

Hole Dia: 8"

Depth 3.55'

Tested by: Reese Davis   Test Date: 12.14.2020

Notes: Measurements in 1/100ths of feet (ft)

Time of Day	Interval / Notes	Initial Depth to Water (ft)	Final Depth of Water (ft)	Δ in Water Level (ft)	Percolation Rate (min/inch)
11:11 AM	Start	0.85			-
11:41 AM	30	0.85	0.87	0.02	125.00
12:11 PM	30	0.87	0.96	0.09	27.78
12:41 PM	30	0.96	1.03	0.07	35.71
1:11 PM	30	1.03	1.09	0.06	41.67
1:41 PM	30	1.09	1.14	0.05	50.00
2:11 PM	30	1.14	1.20	0.06	41.67
2:41 PM	30	1.20	1.25	0.05	50.00
3:11 PM	30	1.25	1.31	0.06	41.67

Leighton

FIELD PERCOLATION TEST DATA SHEET

Project Name:	The Swell Fund - 845 Santa Fe Drive Multi-Family	Project No.:	12980.001
Proj. Address:	845 Santa Fe Drive, Encinitas		

SOIL TYPE / TEST LOCATION / BOREHOLE

Soil Type: SM   Hole #: P-3

Location: See Map

Hole Dia: 8"

Depth 3.43'

Tested by: Reese Davis   Test Date: 12.14.2020

Notes: Measurements in 1/100ths of feet (ft)

Time of Day	Interval / Notes	Initial Depth to Water (ft)	Final Depth of Water (ft)	Δ in Water Level (ft)	Percolation Rate (min/inch)
11:14 AM	Start	0.80			-
11:44 AM	30	0.80	1.06	0.26	9.62
12:14 PM	30	1.06	1.22	0.16	15.63
12:44 PM	30	1.20	1.33	0.13	19.23
1:14 PM	30	1.33	1.46	0.13	19.23
1:44 PM	30	1.46	1.55	0.09	27.78
2:14 PM	30	1.55	1.63	0.08	31.25
2:44 PM	30	1.63	1.72	0.09	27.78
3:14 PM	30	1.72	1.81	0.09	27.78

Leighton

FIELD PERCOLATION TEST DATA SHEET

Project Name:	The Swell Fund - 845 Santa Fe Drive	Project No.:	12980.001
Proj. Address:	845 Santa Fe Drive, Encinitas		

SOIL TYPE / TEST LOCATION / BOREHOLE

Soil Type: SM Hole #: P-4

Location: See Map

Hole Dia: 8"

Depth 3.36'

Tested by: Reese Davis Test Date: 12.14.2020

Notes: Measurements in 1/100ths of feet (ft)

Time of Day	Interval / Notes	Initial Depth to Water (ft)	Final Depth of Water (ft)	Δ in Water Level (ft)	Percolation Rate (min/inch)
11:16 AM	Start	0.57			-
11:46 AM	30	0.57	0.68	0.11	22.73
12:16 PM	30	0.68	0.78	0.10	25.00
12:46 PM	30	0.78	0.87	0.09	27.78
1:16 PM	30	0.87	0.93	0.06	41.67
1:46 PM	30	0.93	0.98	0.05	50.00
2:16 PM	30	0.98	1.04	0.06	41.67
2:46 PM	30	1.04	1.10	0.06	41.67
3:16 PM	30	1.10	1.15	0.05	50.00

Appendix C
Laboratory Testing Procedures & Results

APPENDIX C

Laboratory Testing Procedures and Test Results

Expansion Index Test: The expansion potential of selected material was evaluated by the Expansion Index Text, ASTM Test Method 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 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	Sample Description	Expansion Index	Expansion Potential
B-1 @ 1 to 5 feet	Clayey SAND	3	Very Low
B-3 @ 1 to 5 feet	Clayey SAND	17	Very Low

Particle Size Analysis (ASTM D1140): Particle size analyses were performed by mechanical sieving methods according to ASTM D1140. These tests were performed to assist in the classification of the soil and to determine grain size distributions of the tested soil. The percent fine particles from the analyses are summarized below:

Sample Location	Percent Passing No. 200 Sieve
B-2 at 1 to 5 Feet	46.0
B-4 at 1 to 5 Feet	33.5

APPENDIX C (continued)

Minimum Resistivity and pH Tests: Minimum resistivity and pH tests were performed in general accordance with Caltrans Test Method CT643 and standard geochemical methods. The results are presented in the table below:

Sample Location	Sample Description	pH	Minimum Resistivity (ohms-cm)
B-2 @ 1 to 5 feet	Silty SAND	7.9	600

Chloride Content: Chloride content was tested in accordance with Caltrans Test Method CT422. The results are presented below:

Sample Location	Sample Description	Chloride Content, ppm
B-2 @ 1 to 5 feet	Silty SAND	140

Soluble Sulfates: The soluble sulfate contents of selected samples were determined by standard geochemical methods (Caltrans Test Method CT417). The test results are presented in the table below:

Sample Location	Sample Description	Sulfate Content, ppm	Exposure Class*
B-2 @ 1 to 5 feet	Silty SAND	270	Not Applicable

*Based on the 2014 edition of American Concrete Institute (ACI) Committee 318R, Table No. 19.3.1.1

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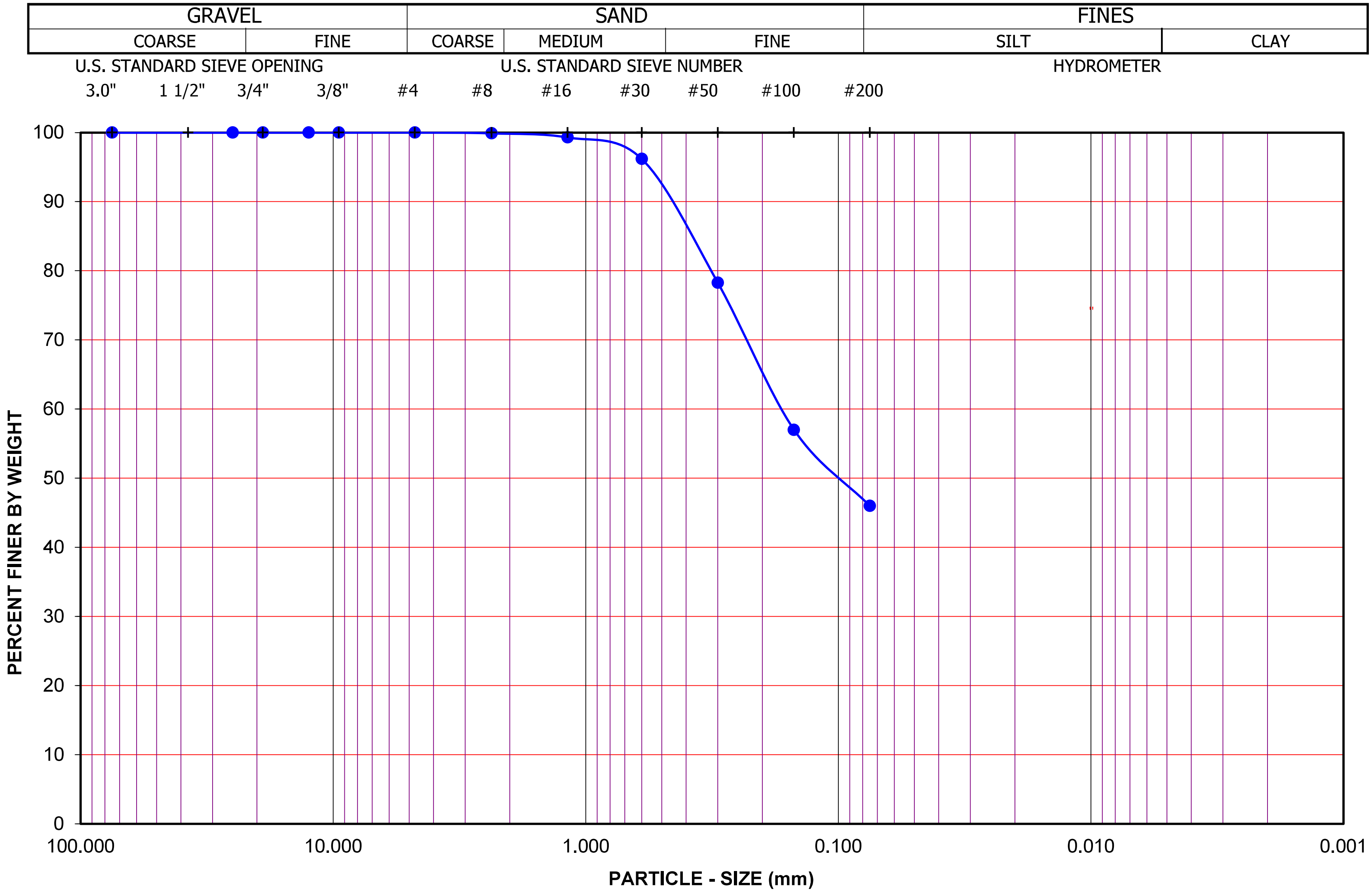
APPENDIX C (continued)

Direct Shear Strength Test: Direct shear testing, in accordance with ASTM D3080, was performed on two 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 the samples, 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, using a motor-driven, strain-controlled, direct-shear testing apparatus. The test results are presented in the accompanying plots.



Project Name: 845 Santa Fe

Project No.: 12980.001

Boring No.: B2 Sample No.: B1

Depth (feet): 0.0 Soil Type : 0

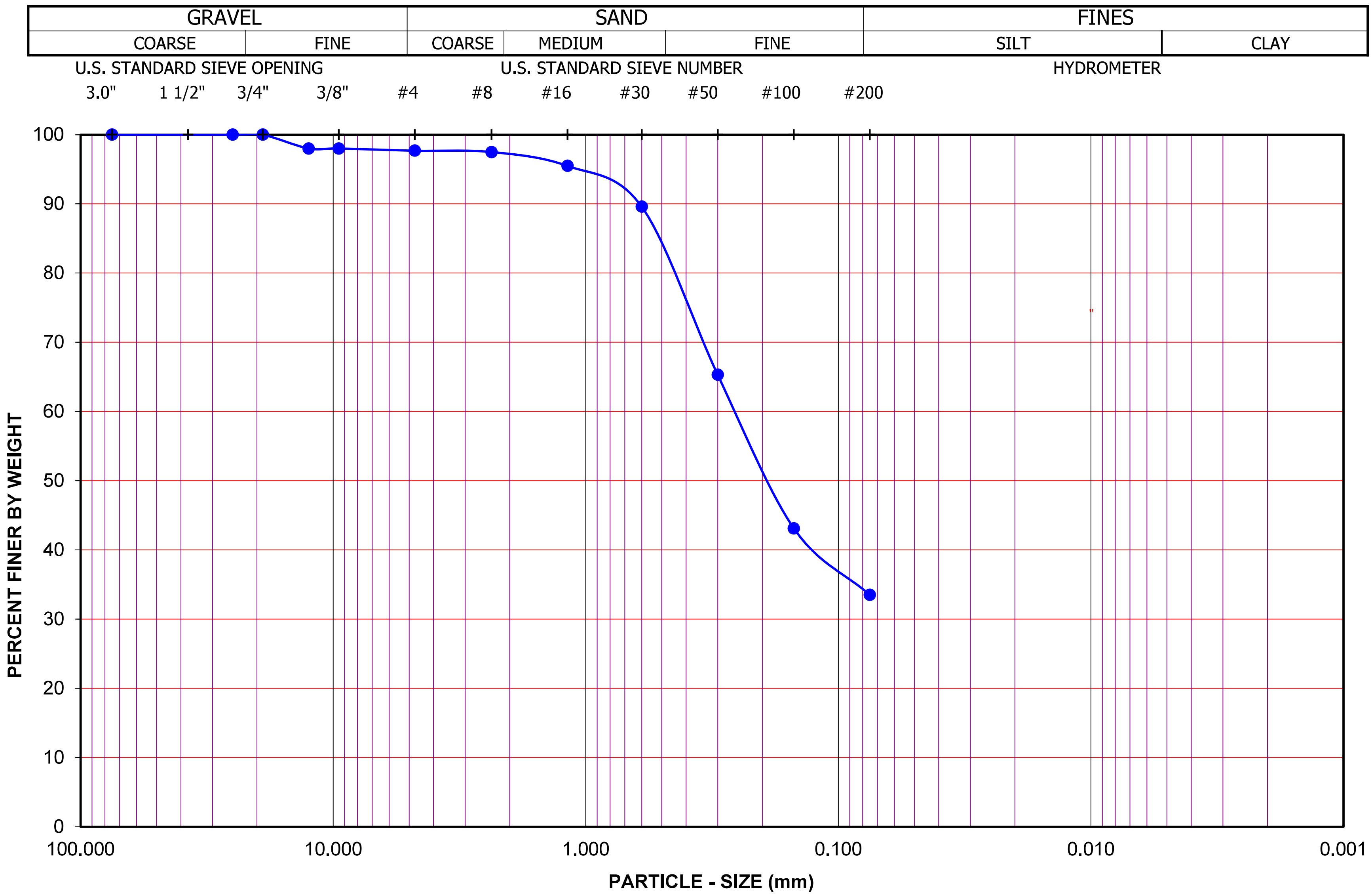
Soil Identification: Brown Silty Sand (SM)

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 0 : 54 : 46

Jan-00



Project Name: 845 Santa Fe

Project No.: 12980.001

Boring No.: B4

Sample No.: B1

Depth (feet): 0.0

Soil Type : 0

Soil Identification: Brown Silty Sand (SM)

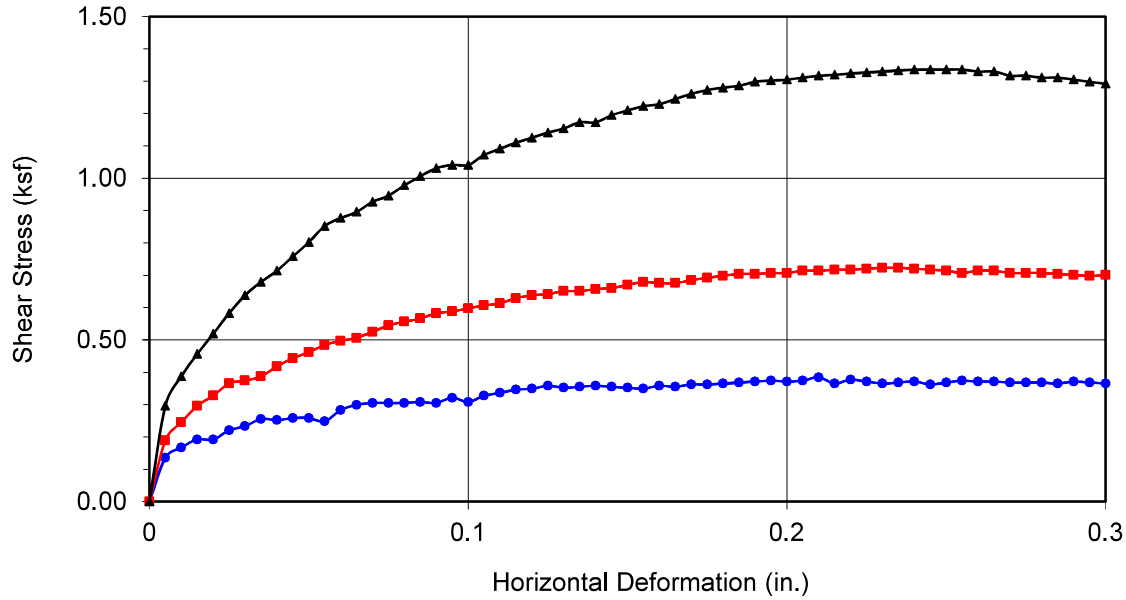
Leighton

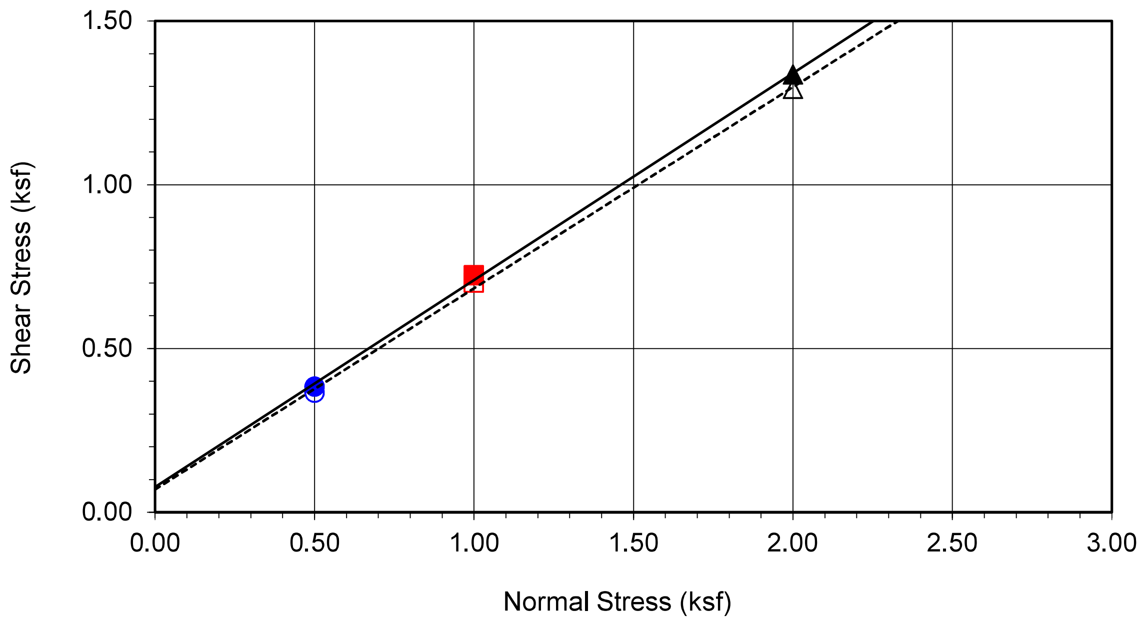
PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 2 : 64 : 34

Jan-00

12980.001 Santa Fe B4-B1@1-5' SA





Boring No.	**B-2**
Sample No.	**R-1**
Depth (ft)	**5**
Sample Type:	Ring
Soil Identification: Light olive brown clayey sand (SC)	

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	78	32
Ultimate	69	32

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.384	■ 0.723	▲ 1.336
Shear Stress @ End of Test (ksf)	○ 0.365	□ 0.701	△ 1.292
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	8.13	8.13	8.13
Dry Density (pcf)	100.8	102.9	103.8
Saturation (%)	32.6	34.4	35.2
Soil Height Before Shearing (in.)	0.9853	0.9669	0.9745
Final Moisture Content (%)	21.5	19.3	17.9

Leighton

DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 12980.001

845 Santa Fe

12-20

Appendix D
City of Encinitas Infiltration Form I-8

Categorization of Infiltration Feasibility Condition			FORM I-8
Part 1 - Full Infiltration Feasibility Screening Criteria **Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?**			
Criteria	Screening Question	**Yes**	**No**
1	**Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X
Provide basis: Based on our field percolation testing, the in-situ infiltration rates of the soils within the limits of proposed residential development are generally less than 0.5 inches per hour (Leighton, 2020). The calculated infiltration rates via the Porchet Method and applied safety factor of 2 ranges from 0.007 to 0.098 inches per hour. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
2	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.	X	
Provide basis: The geotechnical hazards would not be increased provided mitigation is performed for any underground utilities/structures, slopes (i.e., setbacks) and undocumented fill depths greater than 5 feet within the proposed limits of Hydromodification Basins at the subject site. The calculated infiltration rates via the Porchet Method and applied safety factor of 2 ranges from 0.007 to 0.098 inches per hour. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			

<table>
<tr><th colspan="4">FORM I-8 Page 2 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>3</td><td>Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:

If the infiltration rates were greater than 0.5 inches per hour, it may be possible that the risk of groundwater contamination would not be increased provided there are no known contaminated soil or groundwater sites within 250 feet of the proposed Hydromodification Basins at the subject site. The calculated infiltration rates via the Porchet Method and applied safety factor of 2 ranges from 0.007 to 0.098 inches per hour.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>4</td><td>Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:

If the infiltration rates were greater than 0.5 inches per hour, it may be possible that potential water balance issues would not be affected provided there are no unlined site drainages/creeks/streams within 250 feet of the proposed Hydromodification Basins at the subject site. The calculated infiltration rates via the Porchet Method and applied safety factor of 2 ranges from 0.007 to 0.098 inches per hour.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>Part 1 Result*</td><td colspan="2">If all answers to rows 1 - 4 are "Yes" a full infiltration design is potentially feasible. The feasibility screening category is Full Infiltration

If any answer from row 1-4 is "No", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design. Proceed to Part 2</td><td>Go to Part 2</td></tr>
</table>

FORM I-8 Page 3 of 4			
Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria **Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?**			
Criteria	Screening Question	**Yes**	**No**
5	**Do soil and geologic conditions allow for infiltration in any appreciable rate or volume?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	X	
Provide basis: Based on our field percolation testing, the in-situ infiltration rates of the soils within the limits of proposed the site are less than 0.5 inches per hour (Leighton, 2020), but greater than 0.01 inches per hour. The calculated infiltration rates via the Porchet Method and applied safety factor of 2 ranges from 0.007 to 0.098 inches per hour. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.			
6	**Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level**? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.	X	
Provide basis: For a partial infiltration condition (greater than 0.01 inches per hour), the risk of geotechnical hazards will not be increased by partial infiltration provided mitigation is performed for any underground utilities/structures, slopes (i.e., setbacks) and undocumented fill depths greater than 5 feet within the vicinity of proposed Hydromodification Basins at the subject site. Mitigation includes subsurface vertical barriers and subdrains to limit perched ground water mounding conditions. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.			

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

For a partial infiltration condition (greater than 0.01 inches per hour), the risk of groundwater contamination will not be increased by partial infiltration provided there are no known contaminated soil or groundwater sites within 250 feet of the proposed Hydromodification Basins at the subject site.

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

For a partial infiltration condition (greater than 0.01 inches per hour), violation of downstream water rights is not anticipated based on the site location and that there are no unlined site drainages/creeks/streams within 250 feet of the proposed Hydromodification Basins at the subject site.

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

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

Appendix E
General Earthwork and Grading Specifications for Rough Grading

1.0 General

1.1 Intent

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

1.2 The Geotechnical Consultant of Record

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

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

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

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

1.3 The Earthwork Contractor

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

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

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

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing

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

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

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

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

2.2 <u>Processing</u>

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 <u>Overexcavation</u>

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 <u>Benching</u>

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical

Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas

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

3.0 Fill Material

3.1 General

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

3.2 Oversize

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

3.3 Import

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

4.0 Fill Placement and Compaction

4.1 Fill Layers

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

4.2 Fill Moisture Conditioning

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

4.3 Compaction of Fill

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

4.4 Compaction of Fill Slopes

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

4.5 Compaction Testing

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

4.6 Frequency of Compaction Testing

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

4.7 Compaction Test Locations

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

5.0 Subdrain Installation

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

6.0 Excavation

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

7.0 Trench Backfills

7.1 Safety

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

7.2 Bedding and Backfill

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

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

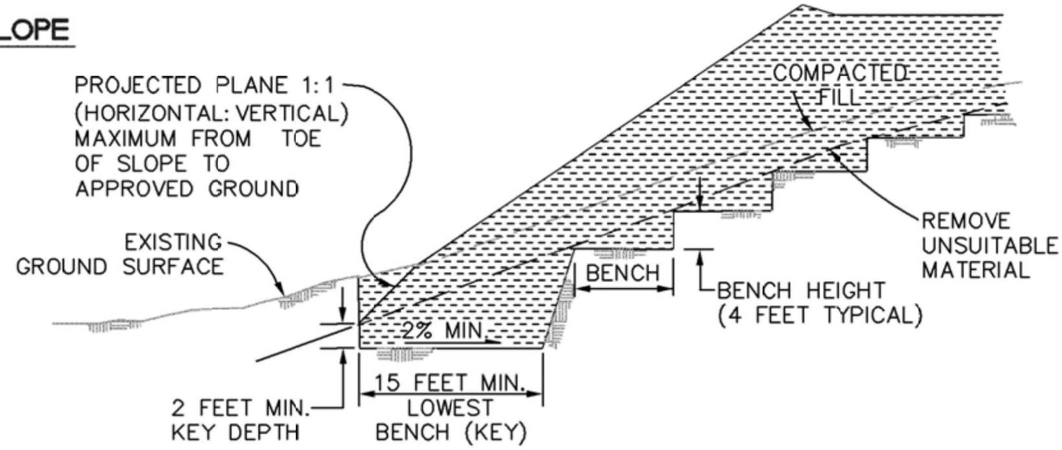
7.3 Lift Thickness

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

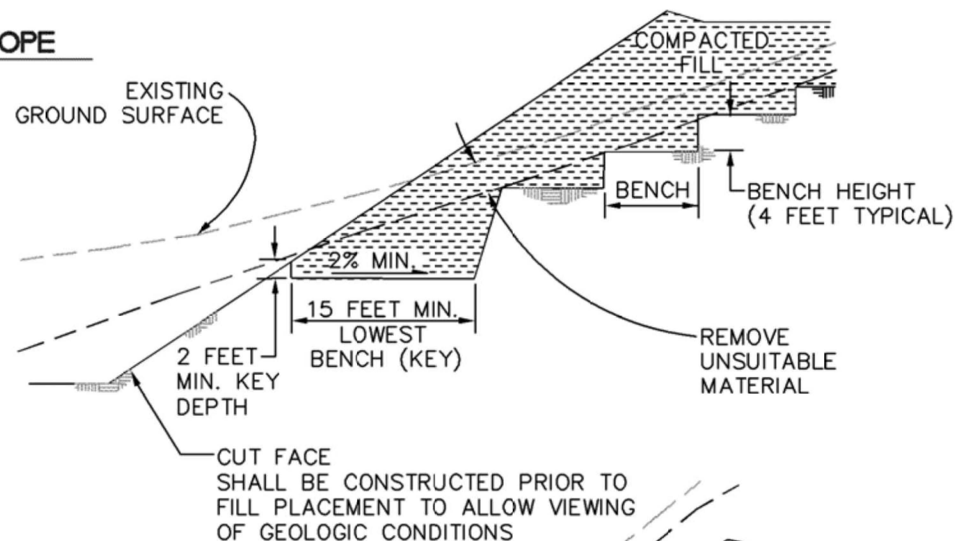
7.4 Observation and Testing

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

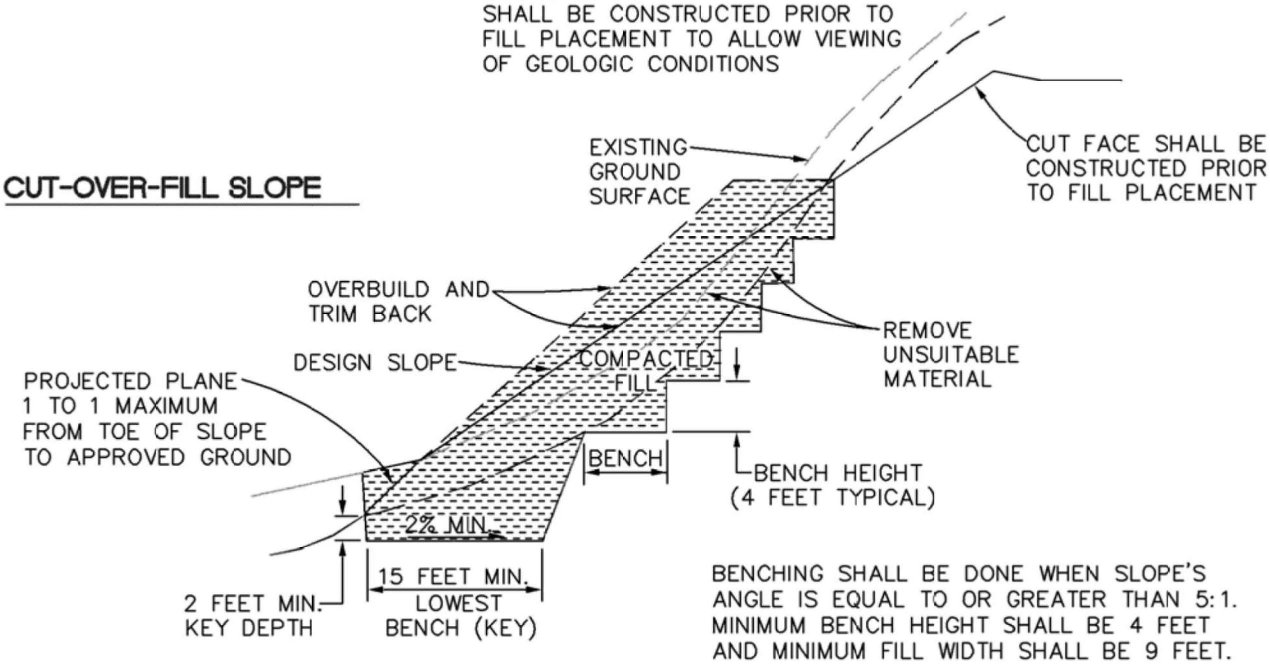
FILL SLOPE



FILL-OVER-CUT SLOPE

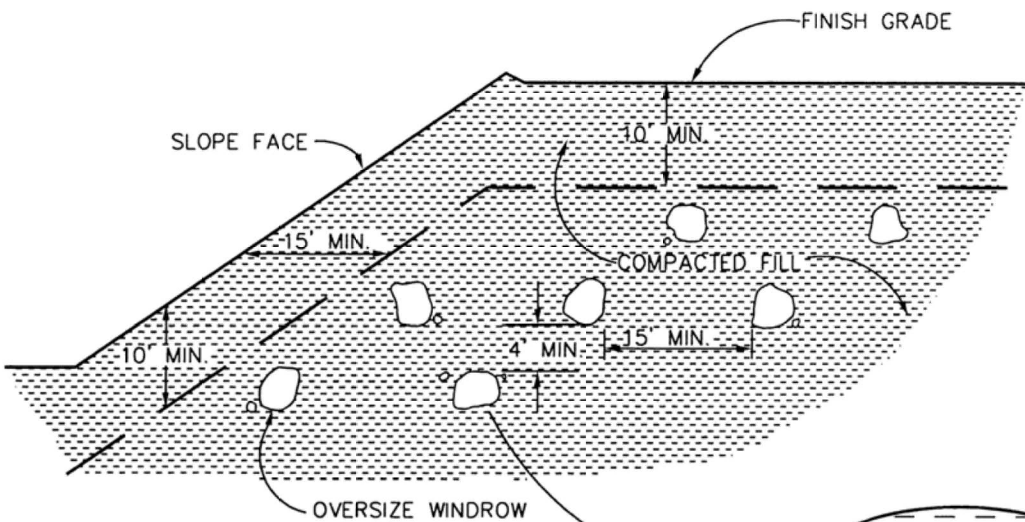


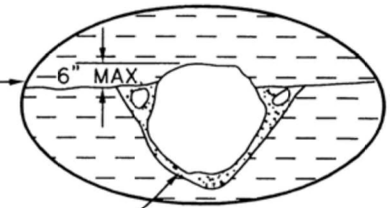
CUT-OVER-FILL SLOPE



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

KEYING AND BENCHING	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL A

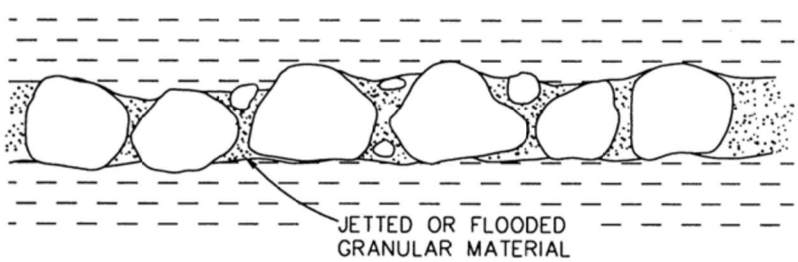




GRANULAR MATERIAL TO BE DENSIFIED IN PLACE BY FLOODING OR JETTING.

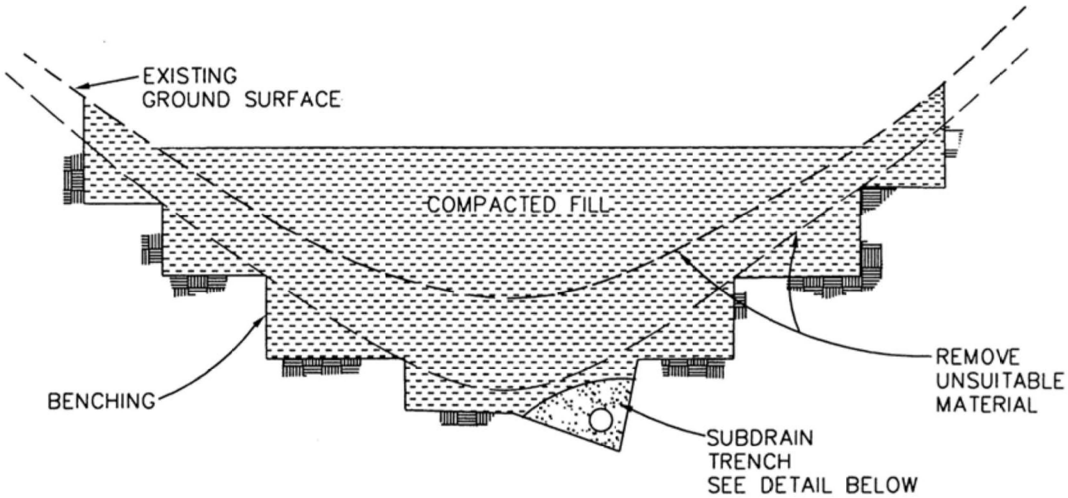
DETAIL

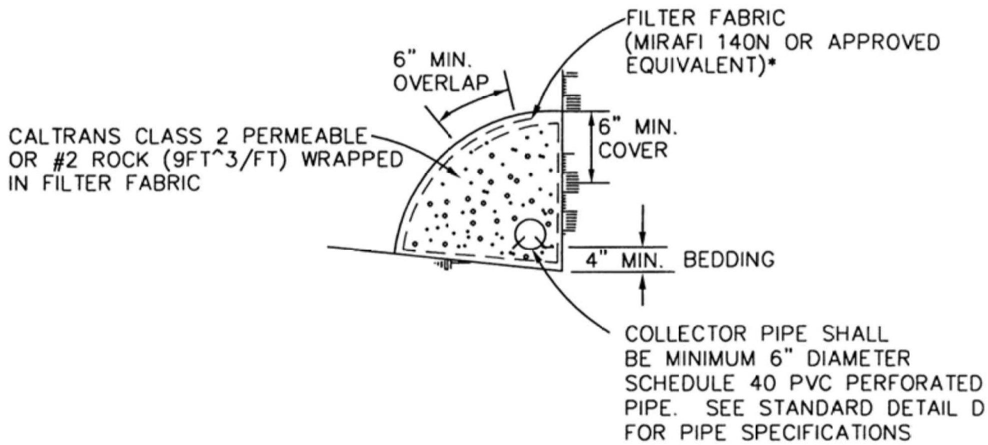
- OVERSIZE ROCK IS LARGER THAN 8 INCHES IN LARGEST DIMENSION.
- EXCAVATE A TRENCH IN THE COMPACTED FILL DEEP ENOUGH TO BURY ALL THE ROCK.
- BACKFILL WITH GRANULAR SOIL JETTED OR FLOODED IN PLACE TO FILL ALL THE VOIDS.
- DO NOT BURY ROCK WITHIN 10 FEET OF FINISH GRADE.
- WINDROW OF BURIED ROCK SHALL BE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



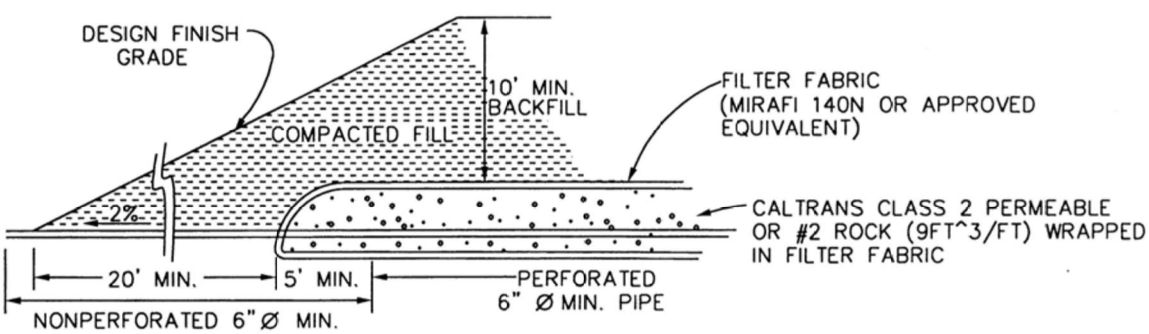
TYPICAL PROFILE ALONG WINDROW

OVERSIZE ROCK DISPOSAL	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL B



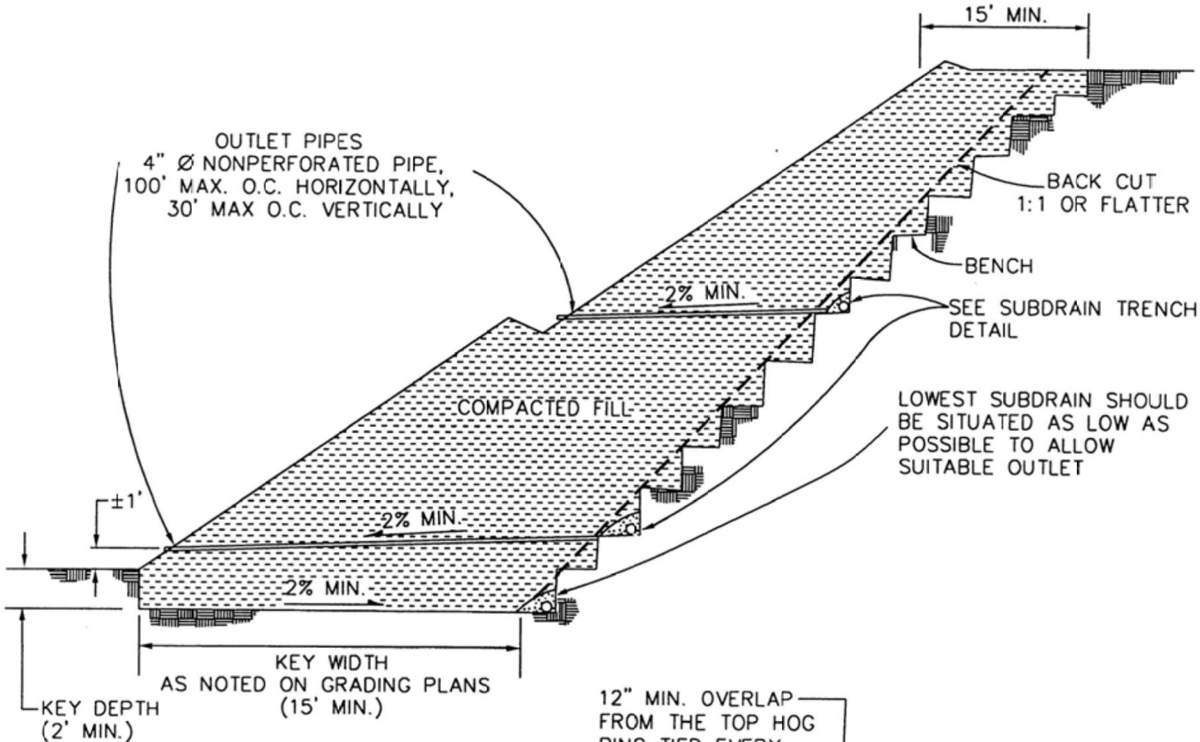


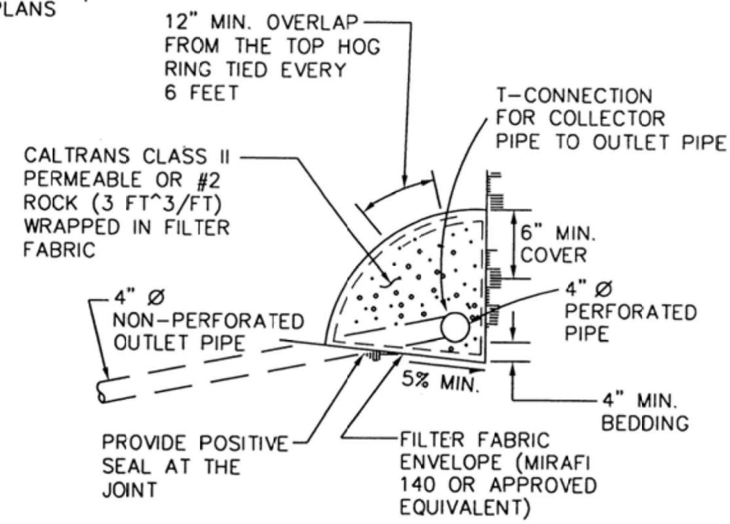
SUBDRAIN DETAIL



DETAIL OF CANYON SUBDRAIN OUTLET

CANYON SUBDRAINS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL C





SUBDRAIN TRENCH DETAIL

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

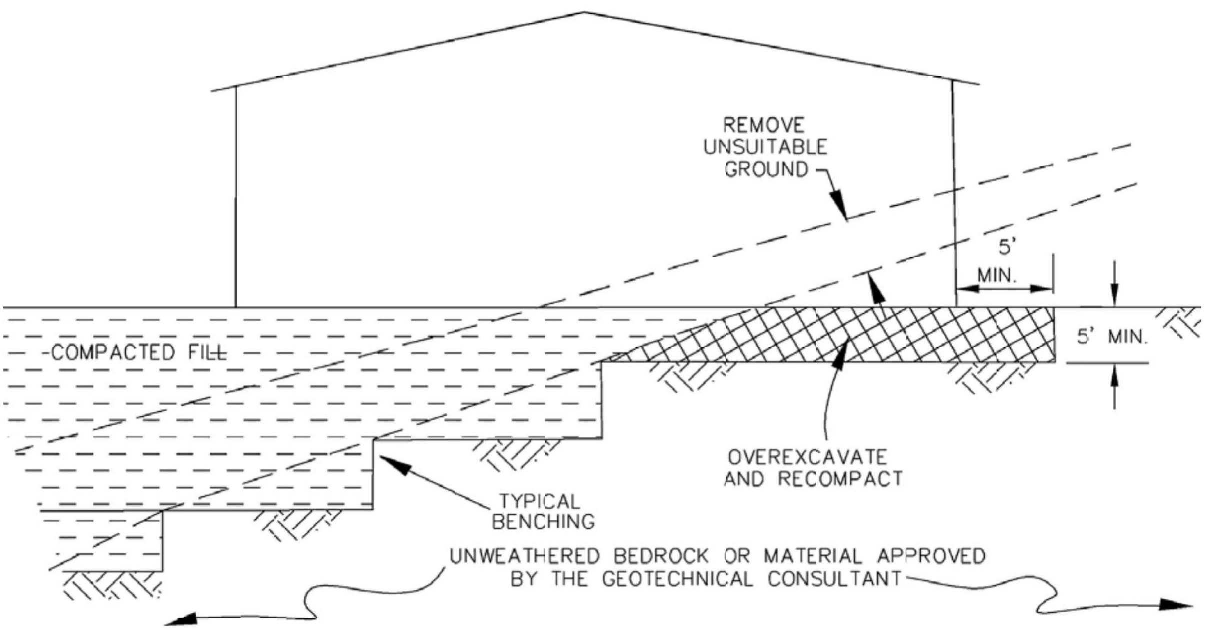
SUBDRAIN PIPE – Subdrain pipe shall be ASTM D2751, SDR 23.5 or ASTM D1527, Schedule 40, or ASTM D3034, SDR 23.5, Schedule 40 Polyvinyl Chloride Plastic (PVC) pipe.

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

BUTTRESS OR REPLACEMENT FILL SUBDRAINS

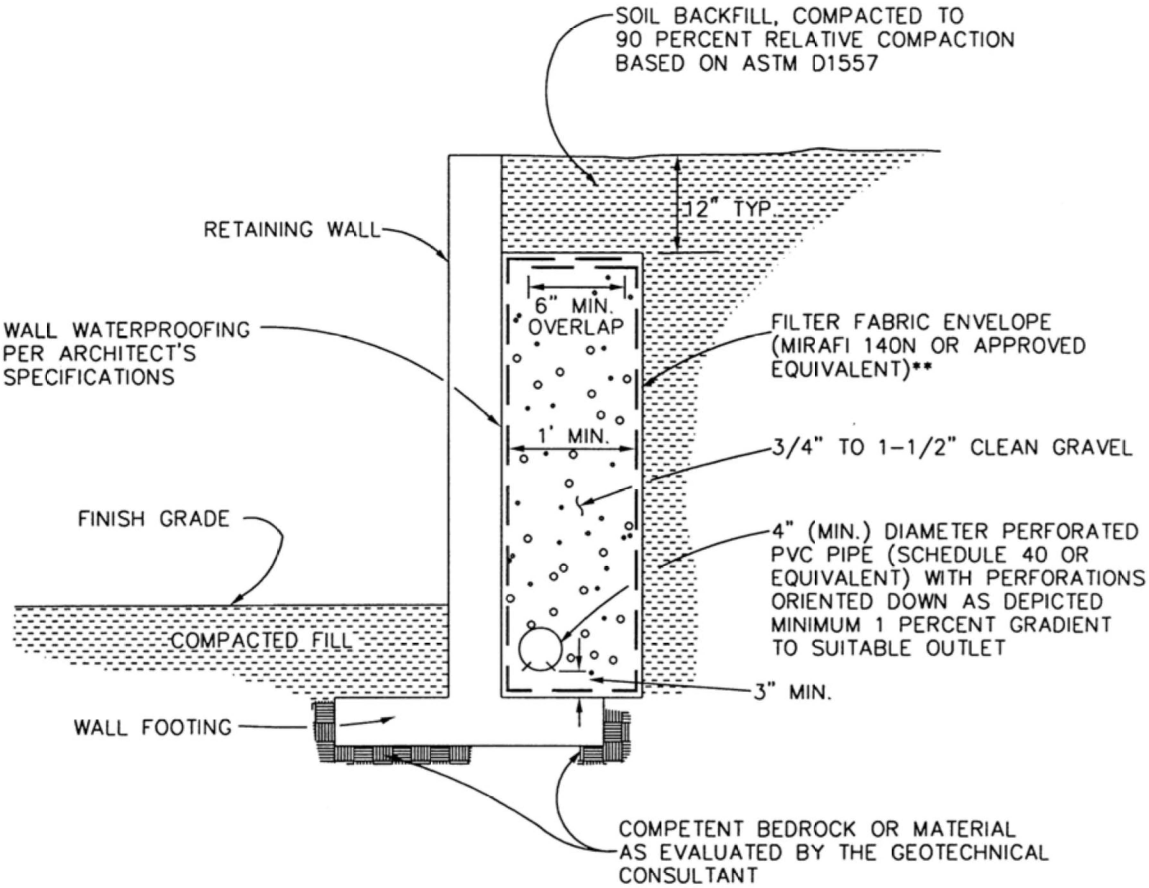
GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL D

CUT-FILL TRANSITION LOT OVEREXCAVATION



TRANSITION LOT FILLS

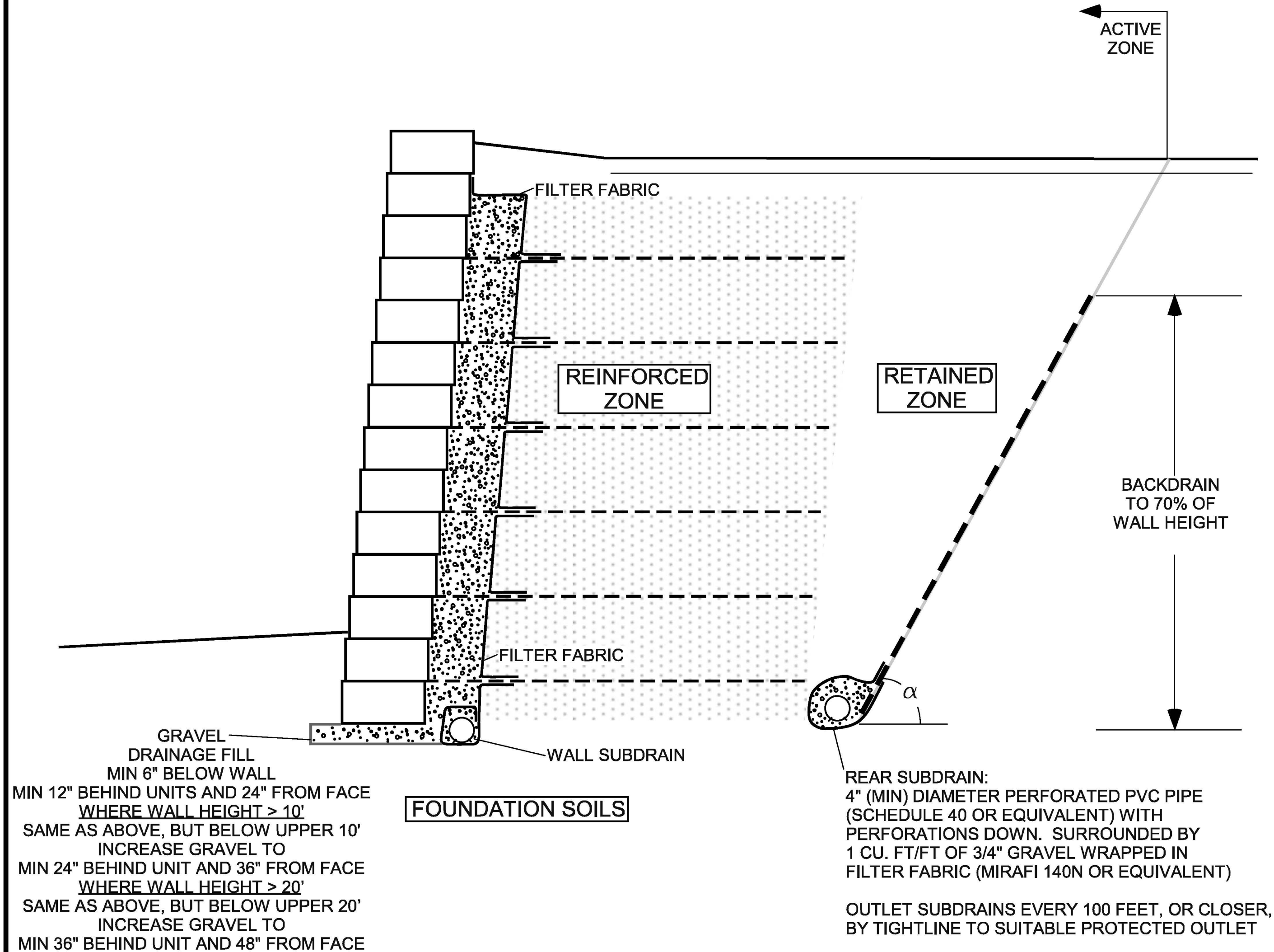
GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL E



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

RETAINING WALL DRAINAGE

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL F



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 AND LIQUID LIMIT < 40
FOR WALL HEIGHT 10 FEET OR TALLER, PLASTICITY INDEX < 6
FOR TIERED WALLS, USE COMBINED WALL HEIGHTS
FOR WALL HEIGHT > 20 FEET, REDUCE ALLOWABLE RANGE % PASSING NO. 200 SIEVE TO 0-15

GRAVEL DRAINAGE FILL:

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

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

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

3) GEOGRID TO BE PRETENSIONED DURING INSTALLATION.

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

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

SEGMENTAL RETAINING WALLS

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAIL G

REVISED 11/16