Construction Testing & Engineering, Inc.
Inspection | Testing | Geotechnical | Environmental & Construction Engineering | Civil Engineering | Surveying

GEOTECHNICAL INVESTIGATION
PROPOSED PSYCHIATRIC HEALTH FACILITY
TRI-CITY MEDICAL CENTER
4002 VISTA WAY
OCEANSIDE, CALIFORNIA

Prepared for:

COUNTY OF SAN DIEGO
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1.0 INTRODUCTION AND SCOPE OF SERVICES

1.1 Introduction

Construction Testing and Engineering, Inc. (CTE) has completed a geotechnical investigation and report providing conclusions and recommendations for the proposed psychiatric health facility with drive areas, flatwork, stormwater BMP's, utilities, and other associated improvements. CTE has performed this work in general accordance with the terms of proposal G-4860A dated December 6, 2019. Preliminary geotechnical recommendations for excavations, fill placement, and foundation design for the proposed improvements are presented herein.

1.2 Scope of Services

The scope of services provided included:

- Review of readily available geologic and soils reports.
- Coordination of USA utility mark-out and location.
- Excavation of exploratory borings and soil sampling utilizing a truck-mounted drill rig.
- Percolation testing in accordance with County of San Diego Department of Environmental Health (DEH) procedures.
- Establishing infiltration rates in general accordance with County of San Diego Storm Water Standards (2016).
- Laboratory testing of selected soil samples.
- Description of the site geology and evaluation of potential geologic hazards.
- Engineering and geologic analysis.
- Preparation of this preliminary geotechnical report.

2.0 SITE DESCRIPTION

The subject site is located in the northwestern portion of the Tri-City Medical Center campus at 4002 Vista Way in Oceanside, California (Figure 1). The proposed improvement area of the site is bounded by Waring Road to the west, a 36 foot high descending slope to the south, parking lot to the east, and residences to the north. The current site area is illustrated on Figure 1. The proposed improvement area is currently undeveloped. However previous grading has been performed at the site creating a stockpile that covers a large portion of the proposed structural footprint area. Other minor improvements that exist throughout the site area consist of utilities, light poles and landscaping. Based on reconnaissance and review of general site topography, the improvement area consists of a topographic high created by the stockpile and a 2:1 (horizontal: vertical) slope, approximately 36 feet high, that descends to the south. The proposed site improvements are depicted on Figure 2.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

3.1 Field Investigation

CTE performed the subsurface investigation on December 12, 2019 to evaluate underlying soil conditions. This fieldwork consisted of site reconnaissance, surface mapping of exposed geologic units throughout the site area, and the excavation of six exploratory soil borings and two percolation test holes. The borings were advanced to a maximum explored depth of approximately 20 feet below ground surface (bgs). Borings B-3 and B-4 were excavated by a CME-75 truck-mounted drill rig equipped with eight-inch-diameter, hollow-stem augers. Borings B-1, B-2, B-5, and B-6 as well as the percolation test holes were excavated with a manually advanced auger due to limited access. Bulk samples were collected from the cuttings, and relatively undisturbed samples were collected in borings B-3 and B-4 by driving Standard Penetration Test (SPT) and Modified California (CAL) samplers. Approximate locations of the exploratory soil borings and test holes are shown on the attached Figure 2.

Soils were logged in the field by a CTE Engineering Geologist, and were visually classified in general accordance with the Unified Soil Classification System (USCS). The field descriptions have been modified, where appropriate, to reflect laboratory test results. Boring logs, including descriptions of the soils encountered, are included in Appendix B.

3.2 Laboratory Testing

Laboratory tests were conducted on selected soil samples for classification purposes, and to evaluate physical properties and engineering characteristics. Laboratory tests included: In-place Moisture and Density, Modified Proctor, Expansion Index, Grain Size Analysis, Direct Shear, and Chemical Characteristics. Test descriptions and laboratory test results are included in Appendix C.

4.0 PERCOLATION TESTING

Two percolation tests were performed within the proposed bioretention area. The percolation test holes were excavated to depths of approximately 3.6 and 5.0 feet below the ground surface (bgs). The attached Figure 2 shows the approximate percolation test locations. The evaluation was performed in general accordance with Appendix C of the Model BMP Design Manual for the San Diego Region “Geotechnical and Groundwater Investigation Requirements”, dated January 2016.

4.1 Percolation Test Methods

The percolation tests were performed in general accordance with methods approved by the San Diego Region BMP Design Manual with a presoak period of approximately 20 to 21 hours. Percolation test results and calculated infiltration rates are presented below in Table 4.2. Field Data and percolation to infiltration calculations are included in Appendix E.

4.2 Calculated Infiltrated Rate

As per the San Diego Region BMP design documents (2016) infiltration rates are to be evaluated using the Porchet Method. San Diego BMP design documents utilized the Porchet Method through guidance of the County of Riverside (2011). The intent of calculating the infiltration rate is to take into account bias inherent in percolation test borehole sidewall infiltration that would not occur at a basin bottom where such sidewalls are not present.

The infiltration rate (I_t) is derived by the equation:

$$I_t = \frac{\Delta H \, \pi r2 \; 60}{\Delta t(\pi r2 + 2\pi r H_{avg})} = \frac{\Delta H \; 60 \; r}{\Delta t(r + 2H_{avg})}$$

Where:

I_t = tested infiltration rate, inches/hour
ΔH = change in head over the time interval, inches
Δt = time interval, minutes
* r = effective radius of test hole
H_{avg} = average head over the time interval, inches

Given the measured percolation rates, the calculated infiltration rates are presented with and without a Factor of Safety applied in Table 4.2 below. The civil engineer of record should determine an appropriate factor of safety to be applied via completion of Worksheet I-8 of County of San Diego "Best Management Practice Design Manual", Appendix D or other approved methods. CTE does not recommend using a factor of safety of less than 2.0.

TABLE 4.2
SUMMARY OF PERCOLATION AND INFILTRATION TEST RESULTS

Test Location	Soil Type	San Diego County Percolation Procedure	Depth (inches)	Percolation Rate (inches/hour)	Infiltration Rate (inches/hour)	Recommended Rate for Design* (inches/hour)
P-1	Stockpile	Case III	43	0.125	0.018	0.009
P-2	Qppf	Case III	60	0.875	0.143	0.072

NOTES
Water level was measured from a fixed point at the top of the hole.
Weather was sunny and warm during percolation testing.
Qppf = Quaternary Previously Placed Fill
The test holes were six inches in diameter.
* Factor of safety of 2 applied

5.0 GEOLOGY

5.1 General Setting

Oceanside is located with the Peninsular Ranges physiographic province that is characterized by northwest-trending mountain ranges, intervening valleys, and predominantly northwest trending active regional faults. The San Diego Region can be further subdivided into the coastal plain area, a central mountain–valley area, and the eastern mountain valley area. The project site is located within the coastal plain area. The coastal plain sub-province ranges in elevation from approximately sea level to 1200 feet above mean sea level (msl) and is characterized by Cretaceous and Tertiary sedimentary deposits that onlap an eroded basement surface consisting of Jurassic and Cretaceous crystalline rocks that have been repeatedly eroded and infilled and by alluvial processes throughout the Quaternary Period in response to regional uplift. This has resulted in a geomorphic landscape of uplifted alluvial and marine terraces that are dissected by current active alluvial drainages.

5.2 Geologic Conditions

Based on the regional geologic map prepared by Kennedy and Tan (2007), the Tertiary Santiago Formation is the near surface geologic unit that underlies the site (Figure 3). Based on recent explorations, stockpile material, Quaternary Previously Placed Fill and residual soil were observed overlying the Tertiary Santiago Formation. Descriptions of the geologic and soil units encountered during the investigation are presented below. Surficial geologic materials are depicted on Figure 2 and generalized geologic cross-sections are presented on Figure 2A.

5.2.1 Stockpile

Where observed, the stockpile material generally consists of loose to medium dense or stiff, olive brown, clayey fine to medium grained sand and sandy clay with gravel. This material was placed during previous site grading. The maximum thickness of the stockpile was observed to be at least 7.8 feet at the time of the subsurface investigation.

5.2.2 Quaternary Previously Placed Fill

Where observed, the Previously Placed Fill generally consists of loose to medium dense or stiff, brown, clayey fine to medium grained sand and sandy clay with gravel. Exploratory excavations encountered Previously Placed Fill to a maximum depth of approximately 5.0 feet (bgs). Localized areas with deeper fill may be encountered during site grading.

5.2.3 Residual Soil

Residual soil was observed in borings B-1, B-3, B-4, and B-5. Where observed, these materials generally consist of loose or very stiff, olive brown, silty fine grained sand and fine grained sandy clay. This unit is relatively thin and blankets the underlying Santiago Formation.

5.2.4 Tertiary Santiago Formation

The Tertiary Santiago Formation was observed in all the borings with the exception of Boring B-2. Where observed, these materials generally consist of dense to very dense, light gray, silty to clayey fine grained sandstone. This geologic unit is anticipated at depth throughout the site.

5.3 Groundwater Conditions

Groundwater was not encountered in the recent exploratory borings that extended to a maximum explored depth of approximately 20 feet bgs. While groundwater conditions may vary, especially following periods of sustained precipitation or irrigation, it is not generally anticipated to adversely affect shallow construction activities or the completed improvements, if proper site drainage is designed, installed, and maintained as per the recommendations of the project civil engineer of record.

5.4 Geologic Hazards

Geologic hazards considered to have potential impacts to site development were evaluated based on field observations, literature review, and laboratory test results. The following paragraphs discuss geologic hazards considered and associated potential risk to the site.

5.4.1 Surface Fault Rupture

In accordance with the Alquist-Priolo Earthquake Fault Zoning Act, (ACT), the State of California established Earthquake Fault Zones around known active faults. The purpose of the ACT is to regulate the development of structures intended for human occupancy near active fault traces in order to mitigate hazards associated with surface fault rupture. According to the California Geological Survey (Special Publication 42, Revised 2018), a fault that has had surface displacement within the last 11,700 years is defined as a Holocene-active fault and is either already zoned or is pending zonation in accordance with the ACT. There are several other definitions of fault activity that are used to regulate dams, power plants, and other critical facilities, and some agencies designate faults that are documented as older than Holocene (last 11,700 years) and younger than late Quaternary (1.6 million years) as potentially active faults that are subject to local jurisdictional regulations.

Based on the site reconnaissance and review of referenced literature, the site is not located within a State-designated Earthquake Fault Zone, no known active fault traces underlie or project toward the site, and no known potentially active fault traces project toward the site.

5.4.2 Local and Regional Faulting

The United States Geological Survey (USGS), with support of State Geological Surveys, and reviewed published work by various researchers, have developed a Quaternary Fault and Fold Database of faults and associated folds that are believed to be sources of earthquakes with magnitudes greater than 6.0 that have occurred during the Quaternary (the past 1.6 million years). The faults and folds within the database have been categorized into four Classes (Class A-D) based on the level of evidence confirming that a Quaternary fault is of tectonic origin and whether the structure is exposed for mapping or inferred from fault related deformational features. Class A faults have been mapped and categorized based on age of documented activity ranging from Historical faults (activity within last 150 years), Latest Quaternary faults (activity within last 15,000 years), Late Quaternary (activity within last 130,000 years), to Middle to late Quaternary (activity within last 1.6 million years). The Class A faults are considered to have the highest potential to generate earthquakes and/or surface rupture, and the earthquakes and surface rupture potential generally increases from oldest to youngest. The evidence for Quaternary deformation and/or tectonic activity progressively decreases for Class B and Class C faults. When geologic evidence indicates that a fault is not of tectonic origin it is considered to be a Class D structure. Such evidence includes joints, fractures, landslides, or erosional and fluvial scarps that resemble fault features, but demonstrate a non-tectonic origin.

The nearest known Class A fault is the Newport-Inglewood-Rose Canyon Fault Zone (<130,000 years), which is approximately 10.5 kilometers west of the site. The attached Figure 4 shows regional faults and seismicity with respect to the site.

5.4.3 Historic Seismicity

The level of seismicity within recent history (last 50 years) of the San Diego area is relatively low compared to other areas of southern California and northwestern Baja California. Only a few small to moderate earthquakes have been reported in the greater San Diego area during the period of instrumental recordings, which began in the early 1900s. Most of the high seismic activity in the region is associated with the Elsinore Fault Zone and the San Jacinto Fault Zone, located approximately 29 and 65 kilometers northeast of the site respectively. In the western portion of San Diego County a series of small-to-moderate earthquakes in July 1985 were reportedly associated with the Rose Canyon Fault Zone (Reichle, 1985). The largest event in that series was M4.7, which was centered within San Diego Bay. A similar series of earthquakes in coastal San Diego occurred in 1964 (Simons, 1979).

Based on review of the USGS Earthquake Archives (http://earthquake.usgs.gov/earthquakes/search/) significant earthquakes within 100 kilometers of the site with magnitudes greater than M5.5 are provided in Table 5.3.3.

TABLE 5.4.3
Regional Earthquake History

EARTHQUAKE DATE (yr-mo-day)	EARTHQUAKE TIME (UTC)	MAGNITUDE	ESTIMATED DEPTH (km)	GENERAL LOCATION
1918-04-21	22:32:29	6.7	10.0	Southern California
1933-03-11	01:54:09	6.4	6.0	WNW of Newport Beach
1937-03-25	16:49:02	6.0	6.0	WSW of Oasis
1951-12-26	00:46:54	5.8	6.0	NNE of San Clemente Island

5.4.4 Liquefaction and Seismic Settlement Evaluation

Liquefaction occurs when saturated fine-grained sands or silts lose their physical strengths during earthquake-induced shaking and behave like a liquid. This is due to loss of point-to-point grain contact and transfer of normal stress to the pore water. Liquefaction potential varies with water level, soil type, material gradation, relative density, and probable intensity and duration of ground shaking. Seismic settlement can occur with or without liquefaction; it results from densification of loose soils.

The site is underlain at shallow depths by dense to very dense Tertiary formational materials. Based on the noted subsurface conditions, the potential for liquefaction or significant seismic settlement at the site is considered to be low.

5.4.5 Tsunamis and Seiche Evaluation

According to McCulloch (1985), the potential in the San Diego County coastal area for “100-year” and “500-year” tsunami waves is approximately five and eight feet, or less. This suggests that there is a negligible probability of a tsunami reaching the site based on elevation of the area and distance from the Pacific Ocean. The site is not located in a zone of potential tsunami inundation based on emergency planning maps prepared by California Emergency Management Agency and CGS. In addition, oscillatory waves (seiches) are considered unlikely due to the absence of nearby confined bodies of water.

5.4.6 Flooding

Based on Federal Emergency Management Agency mapping (FEMA 2012), site improvement areas are located within Zone X, which is defined as: "Areas determined to be outside of the 0.2% annual chance floodplain".

5.4.7 Landsliding

The project site is located near the top of an approximately 36 foot high, 2:1 (horizontal: vertical) slope that descends to the south. According to mapping by Tan (1995), the site is located in area 3-1, which is described as "Generally Susceptible" to landsliding. However, Kennedy and Tan (2007) do not indicate the presence of mapped landslides at the subject site. In addition, on-site field observations did not indicate the presence of deep gross instabilities. Based on the investigation findings, the potential for deep seated landslides at the subject site is considered to be low.

An evaluation of slope stability was performed using GeoStudio SLOPE/W software, based on laboratory determined soil parameters and geologic cross sections depicting existing subsurface conditions. The final input and output data from the limited evaluation of slope stability are presented in Appendix G. For the analysis, the existing slope was modeled based on topographic and geologic conditions. Based on laboratory direct shear testing, the Tertiary Santiago Formation yielded soil strength values of phi = 34.0° to 39.1° and cohesion values = 120 psf to 800 psf. Based on laboratory direct shear testing, the residual soil yielded soil strength values of phi = 42.3° and cohesion = 560 psf. Direct shear testing of the Previously Placed Fill was not performed due to its limited depth and the necessity for hand excavation within this unit for utility clearance. Therefore, estimated values for this soil were utilized for the analysis. To be conservative, Previously Placed Fill values of phi = 25.0° and cohesion = 200 psf, residual soil values of phi = 40.0° and cohesion = 500 psf, and Santiago Formation values of phi = 34.0° and cohesion = 200 psf were utilized for the analysis. Based on the findings, the existing slope condition is anticipated to exhibit a global static factor of safety well in excess of 1.5 and a pseudo-static factor of safety well in excess of 1.1. As such, the proposed slope conditions at the site are anticipated to adequate as planned. However, it is anticipated that surficial soils will continue to erode and may develop shallow slumps and failures on the slope face. Therefore, it would likely be prudent to properly plant and landscape the slopes at the site to minimize erosion and surface degradation.

5.4.8 Compressible and Expansive Soils

The stockpile, Previously Placed Fill and near surface soils are considered to be potentially compressible in their current condition. Therefore, it is recommended that these soils be overexcavated, where necessary, and properly compacted beneath proposed improvement areas as recommended herein and as determined to be necessary during construction. Based on the field data, site observations, and CTE's experience with similar soils in the vicinity of the site, dense native underlying soils are not considered to be subject to significant compressibility under the anticipated loads.

Based on laboratory testing, soils at the site may exhibit Low expansion potential (Expansion Index of 50 or less). However, medium to highly expansive clays are known to exist in the area and may be encountered during site excavations and grading. Additional evaluation of near-surface soils should be performed based on field observations performed during grading and excavation activities.

5.4.9 Corrosive Soils

Testing of representative site soils was performed to evaluate the potential corrosive effects on concrete foundations and buried metallic utilities. Soil environments detrimental to concrete generally have elevated levels of soluble sulfates and/or pH levels less than 5.5. According to the American Concrete Institute (ACI) Table 318 4.3.1, specific guidelines have been provided for concrete where concentrations of soluble sulfate (SO_4) in soil exceed 0.10 percent by weight. These guidelines include low water:cement ratios, increased compressive strength, and specific cement type requirements. A minimum resistivity value less than approximately 5,000 ohm-cm and/or soluble chloride levels in excess of 200 ppm generally indicate a corrosive environment for buried metallic utilities and untreated conduits.

Chemical test results indicate that near-surface soils at the site generally present a negligible corrosion potential for Portland cement concrete. Based on resistivity and chloride testing, the site soils have been interpreted to have a moderate corrosivity potential to buried metallic improvements.

Based on the results of the limited testing performed, it may be prudent to utilize plastic piping and conduits where buried and feasible. However, CTE does not practice corrosion engineering. Therefore, if corrosion of metallic or other improvements is of more significant concern, a qualified corrosion engineer could be consulted.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 General

CTE concludes that the proposed improvements on the site are feasible from a geotechnical standpoint, provided the preliminary recommendations in this report are incorporated into the design and construction of the project. Recommendations for the proposed earthwork and improvements are included in the following sections and Appendix D. However, recommendations in the text of this report supersede those presented in Appendix D should conflicts exist. These preliminary recommendations should either be confirmed as appropriate or updated following required excavations, demolition of existing improvements, and observations during site preparation.

6.2 Site Preparation

Prior to grading, areas to receive distress sensitive improvements should be cleared of existing debris and deleterious materials. Objectionable materials, such as construction or demolition debris and vegetation not suitable for structural backfill should be properly disposed of off-site.

In the area of the proposed structure, existing stockpile and fill soils should be removed in their entirety. The remedial excavations should be conducted to a minimum depth of two feet below the bottom of proposed foundations or to the depth of competent native materials, whichever is greatest. In order to provide relatively uniform conditions under the proposed structure, the minimum depth of proposed fill should also be one half of the maximum depth of fill beneath the structure footprint. If loose or otherwise unsuitable materials are encountered at the base of overexcavations, additional excavation to the depth of suitable material may be necessary. Remedial excavations should extend laterally at least five feet beyond the limits of the proposed improvements or the distance resulting from a 1:1 (horizontal: vertical) extended down to suitable material, where feasible. If overexcavations encroach upon property lines or adjacent structures the temporary excavation should generally be sloped at a 1:1 (horizontal to vertical) or flatter, to the prescribed overexcavation depth. Depending upon proximity and condition of exposed soils, overexcavation in slot cuts may be recommended by the geotechnical engineer.

Overexcavations for proposed surface improvement areas, such as pavement or flatwork should be conducted to a minimum depth of two feet below existing or proposed subgrade or to the depth of suitable material, whichever is deeper.

If encountered, existing below-ground utilities should be redirected around proposed structures. Existing utilities at an elevation to extend through the proposed footings should generally be sleeved and caulked to minimize the potential for moisture migration below the building slabs. Abandoned pipes exposed by grading should be securely capped or filled with minimum two-sack cement/sand slurry to help prevent moisture from migrating beneath foundation and slab soils.

A geotechnical representative from CTE should observe the exposed ground surface prior to placement of compacted fill or improvements, to verify the competency of exposed subgrade materials. After approval by this office, the exposed subgrades to receive fill should be scarified a minimum of eight inches, moisture conditioned, and properly compacted prior to fill placement.

6.3 Site Excavation

Based on CTE’s observations, shallow excavations at the site should generally be feasible using well-maintained heavy-duty construction equipment run by experienced operators. However, excavations within the underlying Tertiary Santiago Formation could encounter cemented zones resulting in difficult excavation.

6.4 Fill Placement and Compaction

Following the recommended overexcavation and removal of loose or disturbed soils, areas to receive fills should be scarified approximately eight inches, moisture conditioned, and properly compacted. Fill and backfill should be compacted to a minimum relative compaction of 90 percent at a minimum two percent above optimum moisture content (three percent above optimum for clayey soils), as evaluated by ASTM D 1557. The optimum lift thickness for fill soil depends on the type of compaction equipment used. Generally, backfill should be placed in uniform, horizontal lifts not exceeding eight inches in loose thickness. Fill placement and compaction should be conducted in conformance with local ordinances, and should be observed and tested by a CTE geotechnical representative.

6.5 Fill Materials

Properly moisture conditioned, low expansion potential soils derived from the on-site materials are considered suitable for reuse on the site as compacted fill. If used, these materials should be screened of organics and materials generally greater than three inches in maximum dimension. Irreducible materials greater than three inches in maximum dimension should not be used in shallow fills (within three feet of proposed grades). In utility trenches, adequate bedding should surround pipes.

Imported fill beneath structures and flatwork should have an Expansion Index of 20 or less (ASTM D 4829). Imported fill soils for use in structural or slope areas should be evaluated by the soils engineer a minimum of two weeks before being imported to the site.

If retaining walls are proposed, backfill located within a 45-degree wedge extending up from the bottom of the heel foundation of the wall should consist of soil having an Expansion Index of 20 or less (ASTM D 4829) with less than 30 percent passing the No. 200 sieve. The upper 12 to 18 inches of wall backfill should consist of lower permeability soils, in order to reduce surface water infiltration behind walls. The project structural engineer and/or architect should detail proper wall backdrains, including gravel drain zones, fills, filter fabric and perforated drain pipes. A conceptual wall drainage detail is provided in Figure 5.

6.6 Temporary Construction Slopes

The following recommended slopes should be relatively stable against deep-seated failure, but may experience localized sloughing. On-site soils are considered Type B and Type C soils with recommended slope ratios as set forth in Table 6.6.

TABLE 6.6
RECOMMENDED TEMPORARY SLOPE RATIOS

SOIL TYPE	SLOPE RATIO (Horizontal: vertical)	MAXIMUM HEIGHT
B (Tertiary Santiago Formation)	1:1 (OR FLATTER)	10 Feet
C (Stockpile, Previously Placed Fill, and Residual Soil)	1.5:1 (OR FLATTER)	10 Feet

Actual field conditions and soil type designations must be verified by a "competent person" while excavations exist, according to Cal-OSHA regulations. In addition, the above sloping recommendations do not allow for surcharge loading at the top of slopes by vehicular traffic, equipment or materials. Appropriate surcharge setbacks must be maintained from the top of all unshored slopes.

6.7 Foundation and Slab Recommendations

The following recommendations are for preliminary design purposes only. These foundation recommendations should be re-evaluated after review of the project grading and foundation plans, and after completion of rough grading of the building pad areas. Upon completion of rough pad grading, Expansion Index of near surface soils should be verified, and these recommendations should be updated, if necessary.

6.7.1 Foundations

Foundation recommendations presented herein are based on the anticipated low expansion potential of near surface soils after remedial site grading is performed (Expansion Index of 50 or less).

Following the recommended preparatory grading, continuous and isolated spread footings are anticipated to be suitable for use at this site. Foundation dimensions and reinforcement should be based on allowable bearing values of 3,000 pounds per square foot (psf) for minimum 15-inch wide footings embedded a minimum of 30-inches below lowest adjacent subgrade elevation. Isolated footings should be at least 24 inches in minimum dimension. The provided bearing value may be increased by 250 psf for each additional six inches of embedment up to a maximum static value of 3,500 psf. The allowable bearing value may be increased by one-third for short-duration loading, which includes the effects of wind or seismic forces. Based on the recommended preparatory grading, it is anticipated that all footings will be founded entirely in properly recompacted fill materials. Footings should not span cut to fill interfaces.

Minimum reinforcement for continuous footings should consist of four No. 6 reinforcing bars; two placed near the top and two placed near the bottom, or as per the project structural engineer. The structural engineer should design isolated footing reinforcement. An uncorrected subgrade modulus of 130 pounds per cubic inch is considered suitable for elastic foundation design.

The structural engineer should provide recommendations for reinforcement of any spread footings and footings with pipe penetrations. Footing excavations should generally be maintained at above optimum moisture content until concrete placement.

6.7.2 Foundation Settlement

The maximum total static settlement is expected to be on the order of one inch and the maximum differential settlement is expected to be on the order of 0.5 inch. Due to the generally dense nature of underlying native materials, dynamic settlement is not expected to adversely affect the proposed buildings.

6.7.3 Foundation Setback

Footings for structures should be designed such that the horizontal distance from the face of adjacent slopes to the outer edge of the footing is at least 12 feet. In addition, footings should bear beneath a 1:1 plane extended up from the nearest bottom edge of adjacent trenches and/or excavations. Deepening of affected footings may be a suitable means of attaining the prescribed setbacks.

6.7.4 Interior Concrete Slabs

Lightly loaded concrete slabs for non-traffic areas should be a minimum of 5.0 inches thick. Minimum slab reinforcement should consist of #4 reinforcing bars placed on maximum 18-

inch centers, each way, at or above mid-slab height, but with proper cover. More stringent recommendations per the project structural engineer supersede these recommendations, as applicable.

In moisture-sensitive floor areas, a suitable vapor retarder of at least 15-mil thickness (with all laps or penetrations sealed or taped) overlying a four-inch layer of consolidated aggregate base or gravel (with SE of 30 or more) should be installed. An optional maximum two-inch layer of similar material may be placed above the vapor retarder to help protect the membrane during steel and concrete placement. This recommended protection is generally considered typical in the industry. If proposed floor areas or coverings are considered especially sensitive to moisture emissions, additional recommendations from a specialty consultant could be obtained. CTE is not an expert at preventing moisture penetration through slabs. A qualified architect or other experienced professional should be contacted if moisture penetration is a more significant concern.

Slabs subjected to heavier loads or traffic will require thicker slab sections and/or increased reinforcement. A 110-pci subgrade modulus is considered suitable for elastic design of minimally embedded improvements such as slabs-on-grade.

Subgrade materials should be maintained or brought to a minimum of two percent or greater above optimum moisture content until slab underlayment and concrete are placed.

6.8 Seismic Design Criteria

The seismic ground motion values listed in the table below were derived in accordance with the ASCE 7-16 Standard. This was accomplished by establishing the Site Class based on the soil properties at the site, and calculating the site coefficients and parameters using the United States Geological Survey Seismic Design Maps application. These values are intended for the design of structures to resist the effects of earthquake ground motions for the site coordinates 33.1864° latitude and –117.2940° longitude, as underlain by soils corresponding to site Class C.

TABLE 6.8
SEISMIC GROUND MOTION VALUES (CODE-BASED)
2019 CBC AND ASCE 7-16

PARAMETER	VALUE	2019 CBC/ASCE 7-16 REFERENCE
Site Class	C	ASCE 7, Chapter 20
Mapped Spectral Response Acceleration Parameter, S_S	0.94	Figure 1613.3.1 (1)
Mapped Spectral Response Acceleration Parameter, S_1	0.346	Figure 1613.3.1 (2)
Seismic Coefficient, F_a	1.200	Table 1613.3.3 (1)
Seismic Coefficient, F_v	1.500	Table 1613.3.3 (2)
MCE Spectral Response Acceleration Parameter, S_{MS}	1.128	Section 1613.3.3
MCE Spectral Response Acceleration Parameter, S_{M1}	0.520	Section 1613.3.3
Design Spectral Response Acceleration, Parameter S_{DS}	0.752	Section 1613.3.4
Design Spectral Response Acceleration, Parameter S_{D1}	0.346	Section 1613.3.4
Peak Ground Acceleration PGA_M	0.490	ASCE 7, Section 11.8.3

6.9 Lateral Resistance and Earth Pressures

Lateral loads acting against structures may be resisted by friction between the footings and the supporting soil or passive pressure acting against structures. If frictional resistance is used, allowable coefficients of friction of 0.30 (total frictional resistance equals the coefficient of friction multiplied by the dead load) for concrete cast directly against compacted fill or native material is recommended. A design passive resistance value of 250 pounds per square foot per foot of depth (with a maximum value of 2,000 pounds per square foot) may be used. The allowable lateral resistance can be taken as the sum of the frictional resistance and the passive resistance, provided the passive resistance does not exceed two-thirds of the total allowable resistance.

If proposed, retaining walls backfilled using granular soils may be designed using the equivalent fluid unit weights given in Table 6.9 below.

TABLE 6.9
EQUIVALENT FLUID UNIT WEIGHTS (G_h)
(pounds per cubic foot)

WALL TYPE	LEVEL BACKFILL	SLOPE BACKFILL 2:1 (HORIZONTAL: VERTICAL)
CANTILEVER WALL (YIELDING)	35	55
RESTRAINED WALL	55	65

Lateral pressures on cantilever retaining walls (yielding walls) over six feet high due to earthquake motions may be calculated based on work by Seed and Whitman (1970). The total lateral earth pressure against a properly drained and backfilled cantilever retaining wall above the groundwater level can be expressed as:

$P_{AE} = P_A + \Delta P_{AE}$

For non-yielding (or "restrained") walls, the total lateral earth pressure may be similarly calculated based on work by Wood (1973):

$P_{KE} = P_K + \Delta P_{KE}$

Where P_A/b = Static Active Earth Pressure = $G_h H^2/2$
P_K/b = Static Restrained Wall Earth Pressure = $G_h H^2/2$
$\Delta P_{AE}/b$ = Dynamic Active Earth Pressure Increment = $(3/8)\ k_h\ \gamma H^2$
$\Delta P_{KE}/b$ = Dynamic Restrained Earth Pressure Increment = $k_h\ \gamma H^2$
b = unit length of wall (usually 1 foot)
$k_h = 1/2^*\ PGA_m$ (PGA_m given previously Table 6.8)
G_h = Equivalent Fluid Unit Weight (given previously Table 6.9)
H = Total Height of the retained soil
γ = Total Unit Weight of Soil ≈ 135 pounds per cubic foot

*It is anticipated that the 1/2 reduction factor will be appropriate for proposed walls that are not substantially sensitive to movement during the design seismic event. Proposed walls that are more sensitive to such movement could utilize a 2/3 reduction factor. If any proposed walls require minimal to no movement during the design seismic event, no reduction factor to the peak ground acceleration should be used. The project structural engineer of record should determine the appropriate reduction factor to use (if any) based on the specific proposed wall characteristics.

The static and increment of dynamic earth pressure in both cases may be applied with a line of action located at H/3 above the bottom of the wall (SEAOC, 2013).

These values assume non-expansive backfill and free-draining conditions. Measures should be taken to prevent moisture buildup behind all retaining walls. Drainage measures should include free-draining backfill materials and sloped, perforated drains. These drains should discharge to an appropriate off-site location. Waterproofing should be as specified by the project architect.

6.10 Exterior Flatwork

Flatwork should be installed with crack-control joints at appropriate spacing as designed by the project architect to reduce the potential for cracking in exterior flatwork caused by minor movement of subgrade soils and concrete shrinkage. Additionally, it is recommended that flatwork be installed with at least number 4 reinforcing bars at 18-inch centers, each way, at or above mid-height of slab, but with proper concrete cover, or with other reinforcement per the applicable project designer. Flatwork that should be installed with crack control joints, includes driveways, sidewalks, and architectural features. All subgrades should be prepared according to the earthwork recommendations previously given before placing concrete. Positive drainage should be established and maintained next to all flatwork. Subgrade materials should be maintained at a minimum of two percent above optimum moisture content until the time of concrete placement.

6.11 Vehicular Pavement

The proposed improvements include paved vehicle drive and parking areas. Presented in Table 6.11 are preliminary pavement sections utilizing laboratory determined Resistance "R" Value. Actual traffic area slab sections to be provided by the structural designer. Beneath proposed pavement areas, the upper 12 inches of subgrade and all base materials should be compacted to 95% relative compaction in accordance with ASTM D1557, and at a minimum of two percent above optimum moisture content.

TABLE 6.11
RECOMMENDED PAVEMENT THICKNESS

Traffic Area	Assumed Traffic Index	Preliminary Subgrade "R"-Value	Asphalt Pavements		Portland Cement Concrete Pavements, on Subgrade Soils (inches)
			AC Thickness (inches)	Class II Aggregate Base Thickness (inches)	
Drive Areas & Infrequent Emergency Vehicle Access	6.0	5+	4.0	12.0	7.5
Parking Areas	5.0	5+	3.0	10.0	7.0

* Caltrans Class 2 aggregate base
** Concrete should have a modulus of rupture of at least 600 psi

Following rough site grading, CTE laboratory testing of representative subgrade soils for as-graded "R"-Value should be performed to verify adequacy of pavement sections.

Asphalt paved areas should be designed, constructed, and maintained in accordance with the recommendations of the Asphalt Institute, or other widely recognized authority. Concrete paved areas should be designed and constructed in accordance with the recommendations of the American Concrete Institute or other widely recognized authority, particularly with regard to thickened edges, joints, and drainage. The Standard Specifications for Public Works construction (“Greenbook”) or Caltrans Standard Specifications may be referenced for pavement materials specifications.

6.12 Drainage

Surface runoff should be collected and directed away from improvements by means of appropriate erosion-reducing devices and positive drainage should be established around the proposed improvements. Positive drainage should be directed away from improvements at a gradient of at least two percent for a distance of at least five feet. However, the project civil engineers should evaluate the on-site drainage and make necessary provisions to keep surface water from affecting the site.

Generally, CTE recommends against allowing water to infiltrate building pads or adjacent to slopes. CTE understands that some agencies are encouraging the use of storm-water cleansing devices. Use of such devices tends to increase the possibility of adverse effects associated with high groundwater including slope instability and liquefaction. See Appendix E for further discussion of site infiltration.

6.13 Slopes

Based on anticipated soil strength characteristics slopes, if proposed, should be constructed at ratios of 2:1 (horizontal: vertical) or flatter. These slope inclinations should exhibit factors of safety greater than 1.5.

Although properly constructed slopes on this site should be grossly stable, the soils will be somewhat erodible. Therefore, runoff water should not be permitted to drain over the edges of slopes unless that water is confined to properly designed and constructed drainage facilities. Erosion-resistant vegetation should be maintained on the face of all slopes.

Typically, soils along the top portion of a fill slope face will creep laterally. CTE recommends against building distress-sensitive hardscape improvements within five feet of slope crests, and against using thickened edges in this area.

6.14 Controlled Low Strength Materials (CLSM)

Controlled Low Strength Materials (CLSM) may be used in deepened footing excavation areas, building pads, and/or adjacent to retaining walls or other structures, provided the appropriate following recommendations are also incorporated. Minimum overexcavation depths recommended herein beneath slabs, flatwork, and other areas may be applicable beneath CLSM if/where CLSM is to be used, and excavation bottoms should be observed by CTE prior to placement of CLSM. Prior to CLSM placement, the excavation should be free of debris, loose soil materials, and water. Once specific areas to utilize CLSM have been determined, CTE should review the locations to determine if additional recommendations are appropriate.

CLSM should consist of a minimum three-sack cement/sand slurry with a minimum 28-day compressive strength of 100 psi (or equal to or greater than the maximum allowable short term soil bearing pressure provided herein, whichever is higher) as determined by ASTM D4832. If re-excavation is anticipated, the compressive strength of CLSM should generally be limited to a maximum of 150 psi per ACI 229R-99. Where re-excavation is required, two-sack cement/sand slurry may be used to help limit the compressive strength. The allowable soils bearing pressure and coefficient of friction provided herein should still govern foundation design. CLSM may not be used in lieu of structural concrete where required by the structural engineer.

6.15 Plan Review

CTE should be authorized to review the project grading and foundation plans prior to commencement of earthwork in order to provide additional recommendations, if necessary.

6.16 Construction Observation

The recommendations provided in this report are based on preliminary design information for the proposed construction and the subsurface conditions observed in the soil borings. The interpolated subsurface conditions should be confirmed by CTE during construction with respect to anticipated conditions. Upon completion of precise grading, if necessary, soil samples will be collected to evaluate as-built Expansion Index. Foundation recommendations may be revised upon completion of grading, and as-built laboratory tests results. Additionally, soil samples should be taken in pavement subgrade areas upon rough grading to refine pavement recommendations as necessary.

Recommendations provided in this report are based on the understanding and assumption that CTE will provide the observation and testing services for the project. All earthwork should be observed and tested in accordance with recommendations contained within this report. CTE should evaluate footing excavations before reinforcing steel placement.

7.0 LIMITATIONS OF INVESTIGATION

The field evaluation, laboratory testing and geotechnical analysis presented in this report have been conducted according to current engineering practice and the standard of care exercised by reputable geotechnical consultants performing similar tasks in this area. No other warranty, expressed or implied, is made regarding the conclusions, recommendations and opinions expressed in this report. Variations may exist and conditions not observed or described in this report may be encountered during construction. This report is prepared for the project as described. It is not prepared for any other property or party.

The recommendations provided herein have been developed in order to reduce the post-construction movement of site improvements related to soil settlement and expansion. However, even with the design and construction recommendations presented herein, some post-construction movement and associated distress may occur.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside CTE's involvement. Therefore, this report is subject to review and should not be relied upon after a period of three years.

CTE's conclusions and recommendations are based on an analysis of the observed conditions. If conditions different from those described in this report are encountered, CTE should be notified and additional recommendations, if required, will be provided subject to CTE remaining as authorized geotechnical consultant of record. This report is for use of the project as described. It should not be utilized for any other project.

The percolation test results were obtained in accordance with regional standards and were performed with the standard of care practiced by other professionals practicing in the area. However, percolation test results can significantly vary laterally and vertically due to slight changes in soil type, degree of weathering, secondary mineralization, and other physical and chemical variabilities. As such, the test results are only considered as an estimate of percolation and converted infiltration rates for design purposes. No guarantee is made based on the percolation testing to the actual functionality or longevity of associated infiltration basins or other BMP devices designed from the presented infiltration rates.

CTE appreciates this opportunity to be of service on this project. If you have any questions regarding this report, please do not hesitate to contact the undersigned.

Respectfully submitted,

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No.1890
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DANIEL T. MATH
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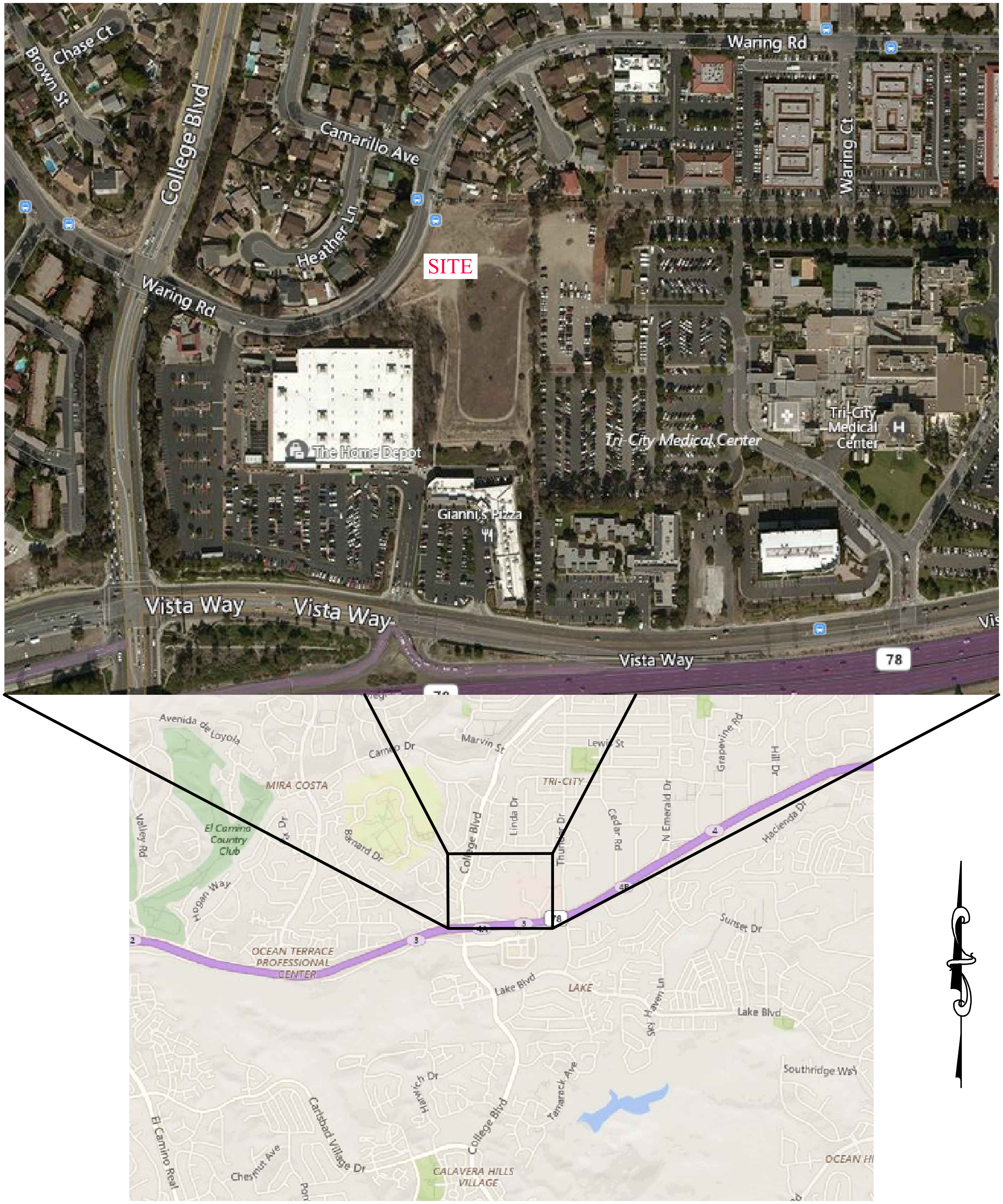
AARON J. BEEBY
No.2603
Exp. 3/31/20

DTM/JFL/AJB:ack



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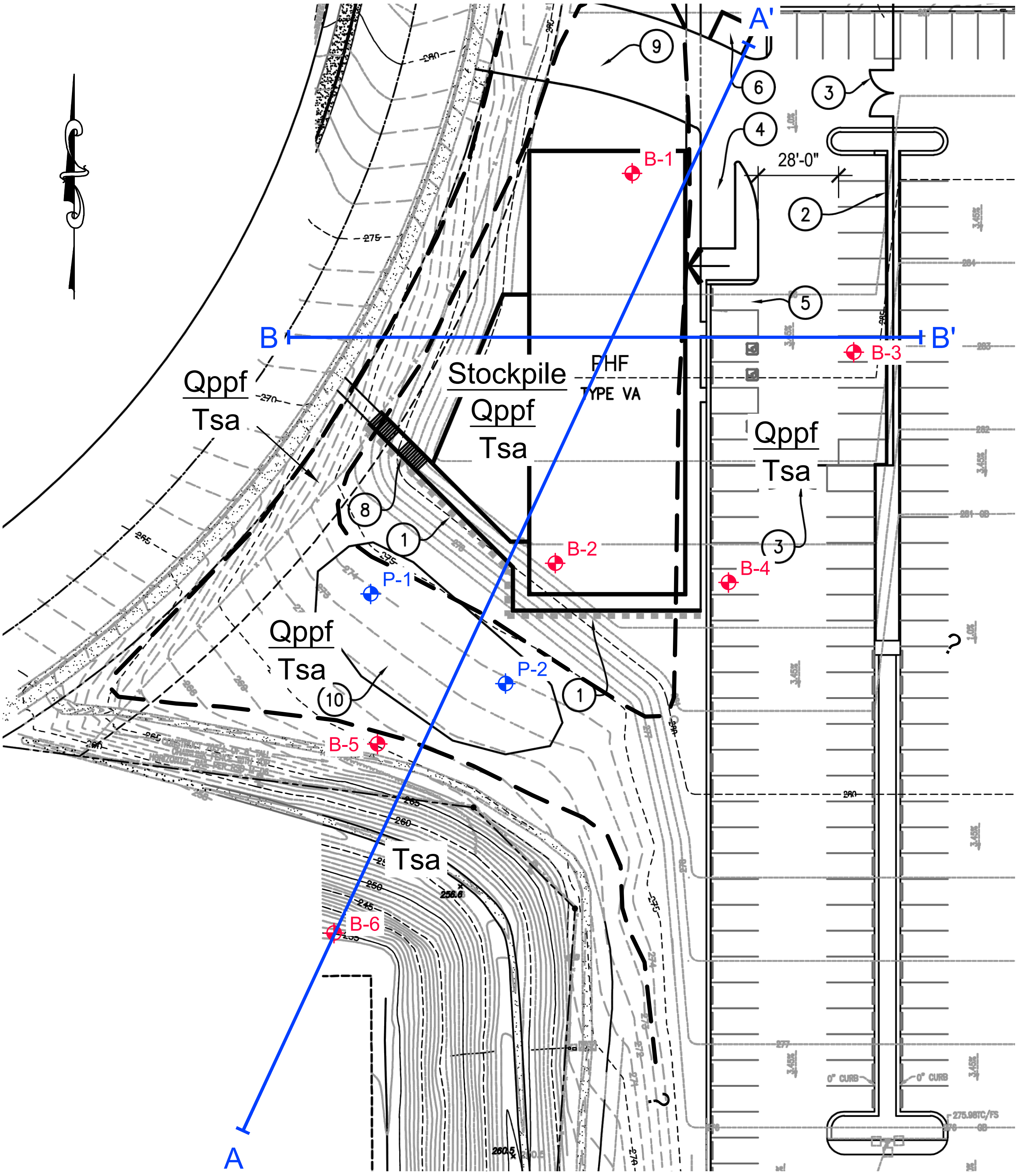
SITE INDEX MAP

PROPOSED TCMC PSYCHIATRIC HEALTH FACILITY
4002 VISTA WAY
OCEANSIDE, CALIFORNIA

SCALE: AS SHOWN	DATE: 1/20
CTE JOB NO.: 10-15341G	FIGURE: 1



50' 0 25' 50'

LEGEND

B-6 ⊕	Approximate Boring Location
P-2 ⊕	Approximte Perc Test Location
Qppf / Tsa	Quaternary Previously Placed Fill over Tertiary Santiago Formation
— — —	Approximate Geologic Contact
● ● ●	Approximate Buried Geologic Contact
B⊢⊣B'	Cross Section

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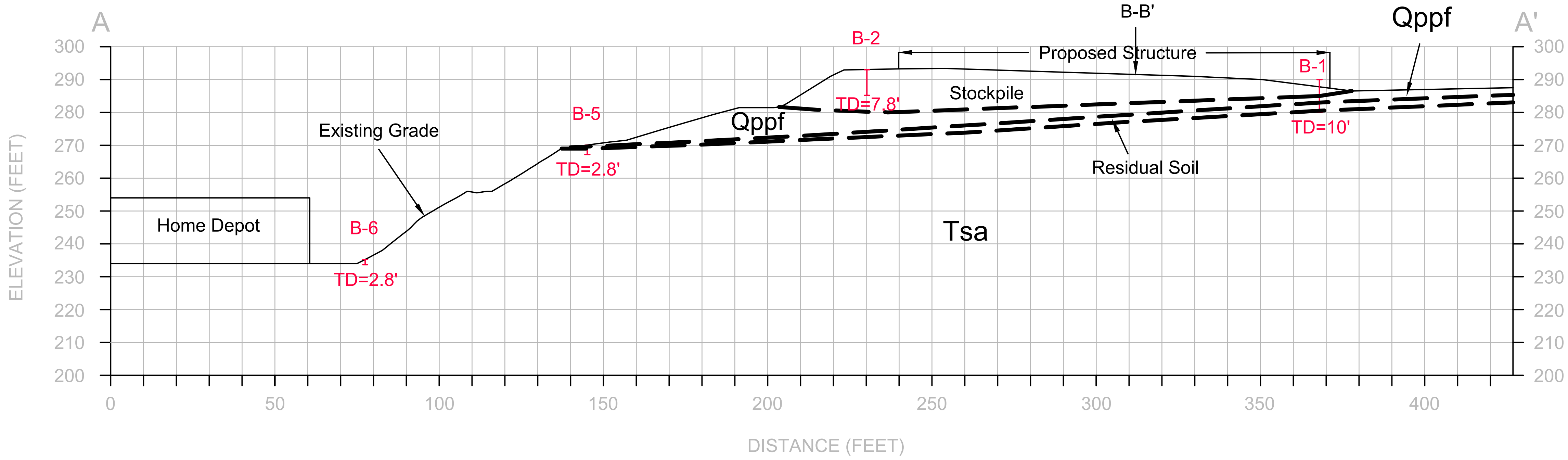
GEOLOGIC/EXPLORATION LOCATION MAP

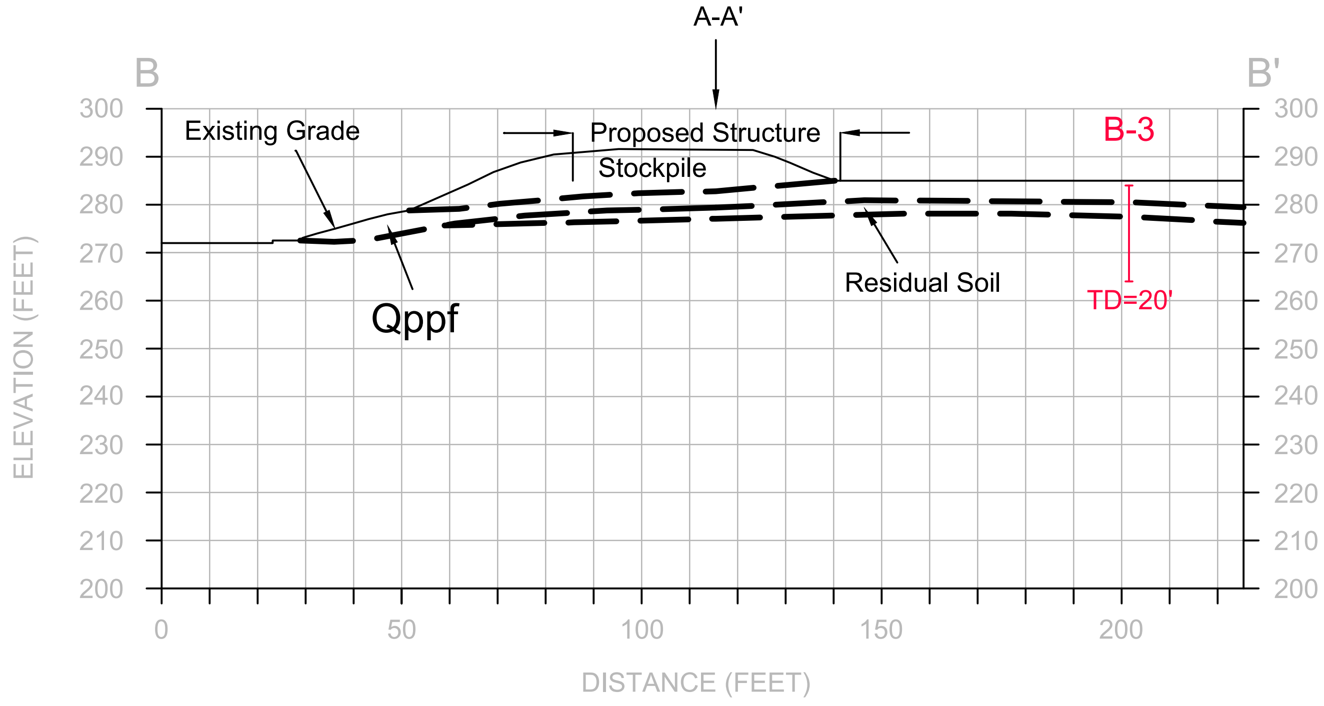
PROPOSED TCMC PSYCHIATRIC HEALTH FACILITY
4002 VISTA WAY
OCEANSIDE, CALIFORNIA

SCALE:	1"=50'
DATE:	1/20
CTE JOB NO.:	10-15341G
FIGURE:	2

\\Esc_server\projects\10-15341G\Figure 2.dwg



CROSS SECTION A-A'



CROSS SECTION B-B'

LEGEND

Qppf	QUATERNARY PREVIOUSLY PLACED FILL
Tsa	TERTIARY SANTIAGO FORMATION
— — —	APPROXIMATE GEOLOGIC CONTACT

\\Esc_server\projects\10-15341G\Figure 2B.dwg

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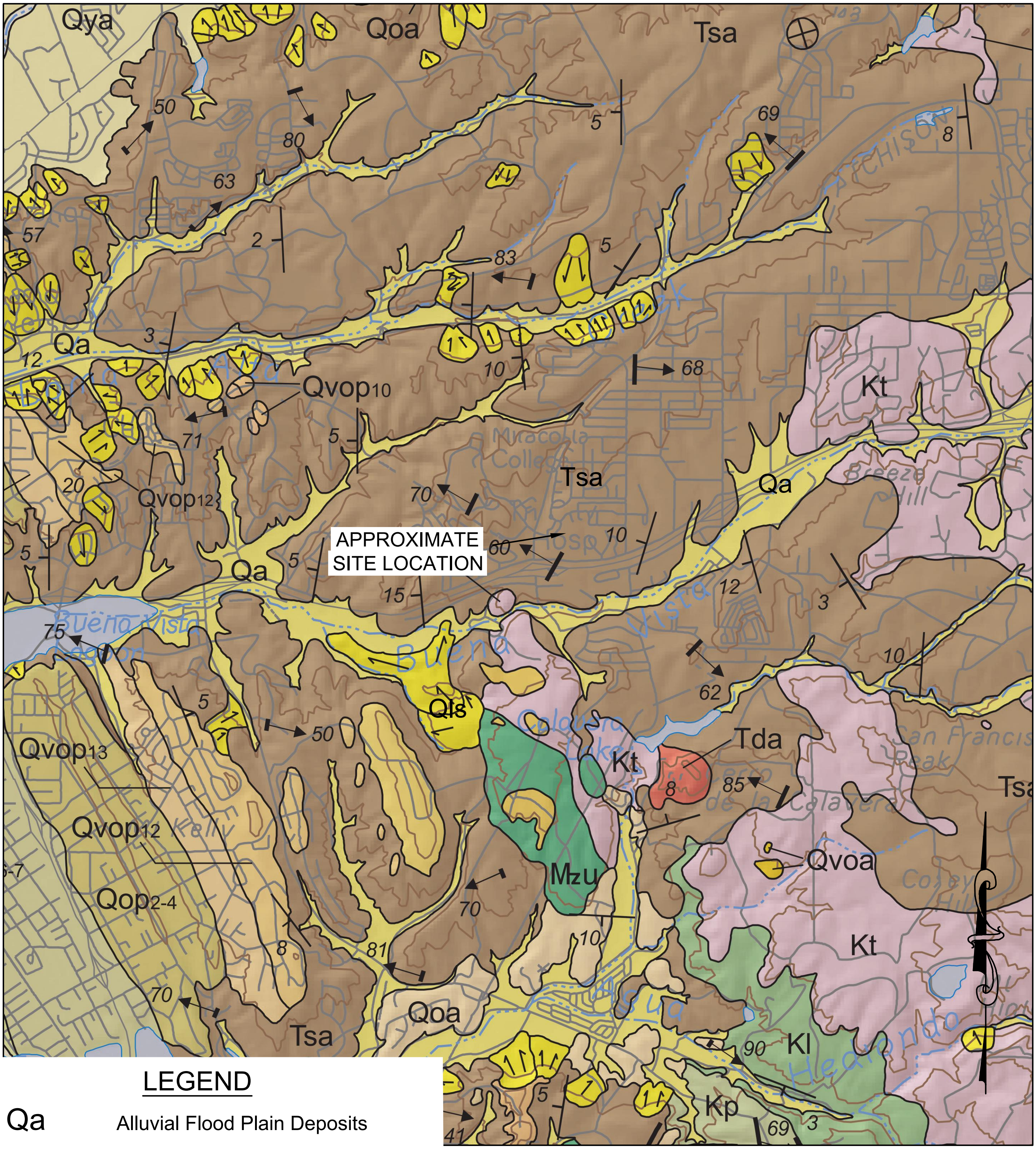
CROSS SECTIONS A-A' and B-B'
PROPOSED TCMC PSYCHIATRIC HEALTH CENTER
4002 VISTA WAY
OCEANSIDE, CALIFORNIA

CTE JOB NO: 10-15341G
SCALE: 1" = 40'
DATE: 1/20
FIGURE: 2A



LEGEND

Qa	Alluvial Flood Plain Deposits
Qls	Landslide Deposits
Qop	Old Paralic Deposits
Qvop	Very Old Paralic Deposits
Tsa	Santiago Formation
Kp	Point Ioma Formation
Kt	Tonalite
Mzu	Metasedimentary and Metavolcanic Rock

NOTE: Base Map by Kennedy and Tan, 2007, Geologic Map of the Oceanside 30' x 60' Quadrangle, California.

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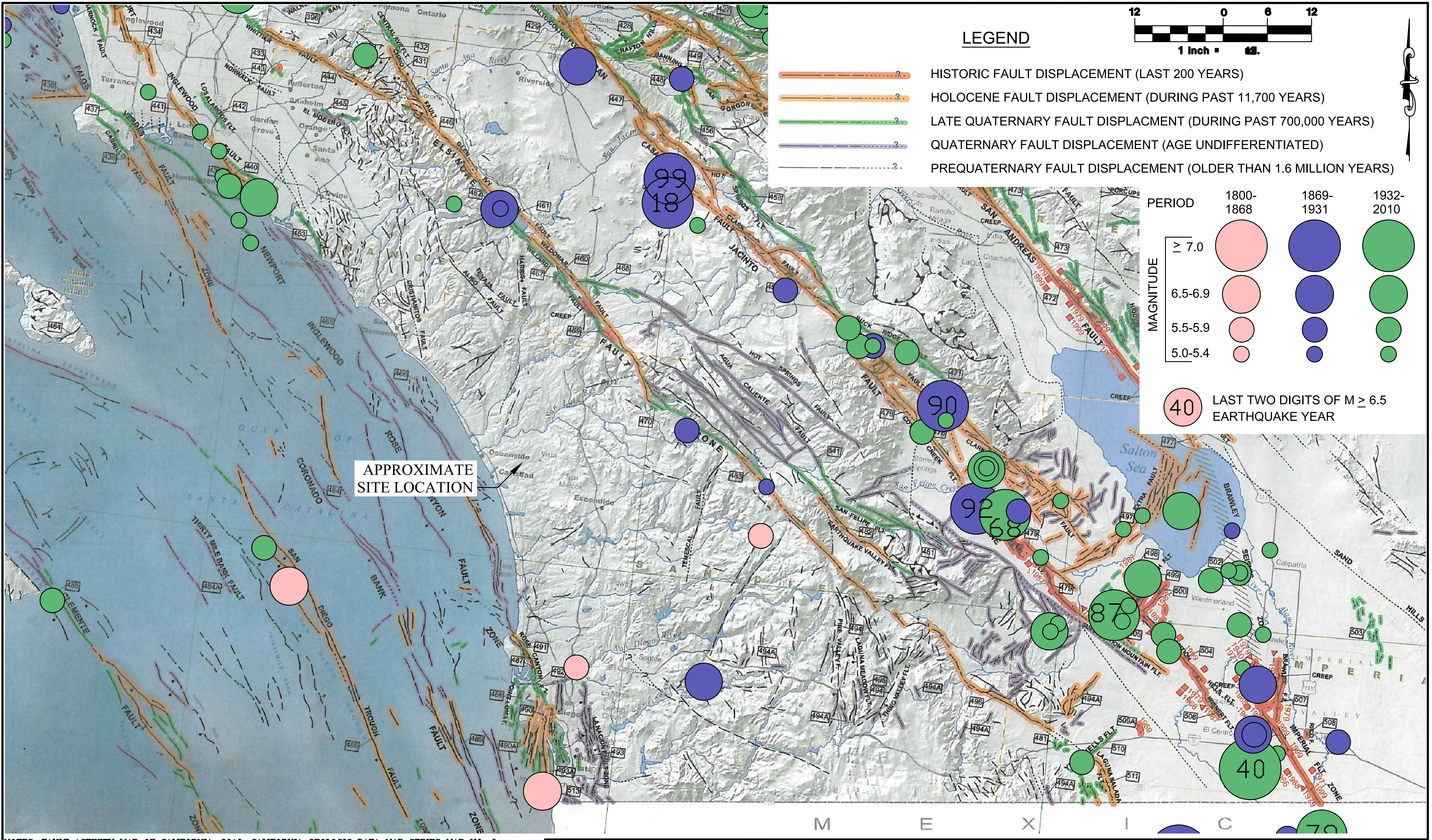
REGIONAL GEOLOGIC MAP

PROPOSED TCMC PSYCHIATRIC HEALTH FACILITY
4002 VISTA WAY
OCEANSIDE, CALIFORNIA

SCALE: 1"~4,000'	DATE: 1/20
CTE JOB NO.: 10-15341G	FIGURE: 3

\\Esc_server\projects\10-15341G\Figure 3 (Geo Map).dwg



NOTES: FAULT ACTIVITY MAP OF CALIFORNIA, 2010, CALIFORNIA GEOLOGIC DATA MAP SERIES MAP NO. 6;
EPICENTERS OF AND AREAS DAMAGED BY M≥5 CALIFORNIA EARTHQUAKES, 1800-1999 ADAPTED AFTER TOPPOZADA, BRANUM, PETERSEN, HALLSTORM, CRAMER, AND REICHLE, 2000, CDMG MAP SHEET 49
REFERENCE FOR ADDITIONAL EXPLANATION; MODIFIED WITH CISN AND USGS SEISMIC MAPS

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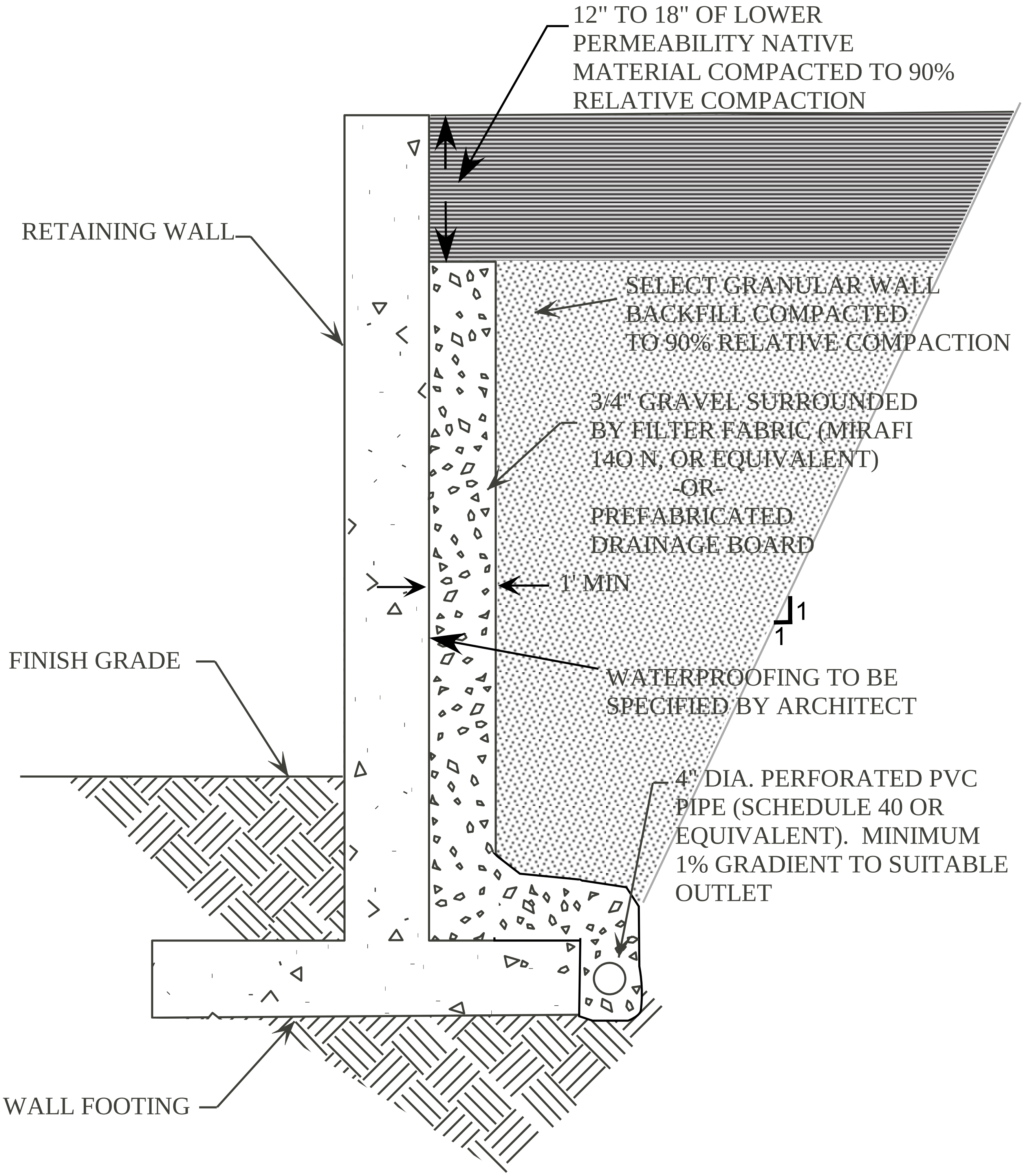
REGIONAL FAULT AND SEISMICITY MAP
PROPOSED TCMC PSYCHIATRIC HEALTH FACILITY
4002 VISTA WAY
OCEANSIDE, CALIFORNIA

CTE JOB NO: 10-15341G
SCALE: 1 inch = 12 miles
DATE: 1/20
FIGURE: 4

\\Esc_server\projects\10-15341G\Figure 4 (Regional Fault Map).dwg



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RETAINING WALL DRAINAGE DETAIL

CTE JOB NO: 10-15341G
SCALE: NO SCALE
DATE: 1/20
FIGURE: 5

APPENDIX A

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CITED REFERENCES

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APPENDIX B

EXPLORATION LOGS

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DEFINITION OF TERMS

PRIMARY DIVISIONS			SYMBOLS	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	**GRAVELS** MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS < 5% FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS OR GRAVEL SAND MIXTURES, LITTLE OF NO FINES
		GRAVELS WITH FINES	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES, NON-PLASTIC FINES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES, PLASTIC FINES
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS < 5% FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES	SM	SILTY SANDS, SAND-SILT MIXTURES, NON-PLASTIC FINES
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES, PLASTIC FINES
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	**SILTS AND CLAYS** LIQUID LIMIT IS LESS THAN 50		ML	INORGANIC SILTS, VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, SLIGHTLY PLASTIC CLAYEY SILTS
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY, SANDY, SILTS OR LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTY CLAYS
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

GRAIN SIZES

BOULDERS	COBBLES	GRAVEL		SAND			SILTS AND CLAYS
		COARSE	FINE	COARSE	MEDIUM	FINE	

12" 3" 3/4" 4 10 40 200

CLEAR SQUARE SIEVE OPENING U.S. STANDARD SIEVE SIZE

ADDITIONAL TESTS

(OTHER THAN TEST PIT AND BORING LOG COLUMN HEADINGS)

MAX- Maximum Dry Density
GS- Grain Size Distribution
SE- Sand Equivalent
EI- Expansion Index
CHM- Sulfate and Chloride Content , pH, Resistivity
COR - Corrosivity
SD- Sample Disturbed

PM- Permeability
SG- Specific Gravity
HA- Hydrometer Analysis
AL- Atterberg Limits
RV- R-Value
CN- Consolidation
CP- Collapse Potential
HC- Hydrocollapse
REM- Remolded

PP- Pocket Penetrometer
WA- Wash Analysis
DS- Direct Shear
UC- Unconfined Compression
MD- Moisture/Density
M- Moisture
SC- Swell Compression
OI- Organic Impurities

FIGURE: BL1

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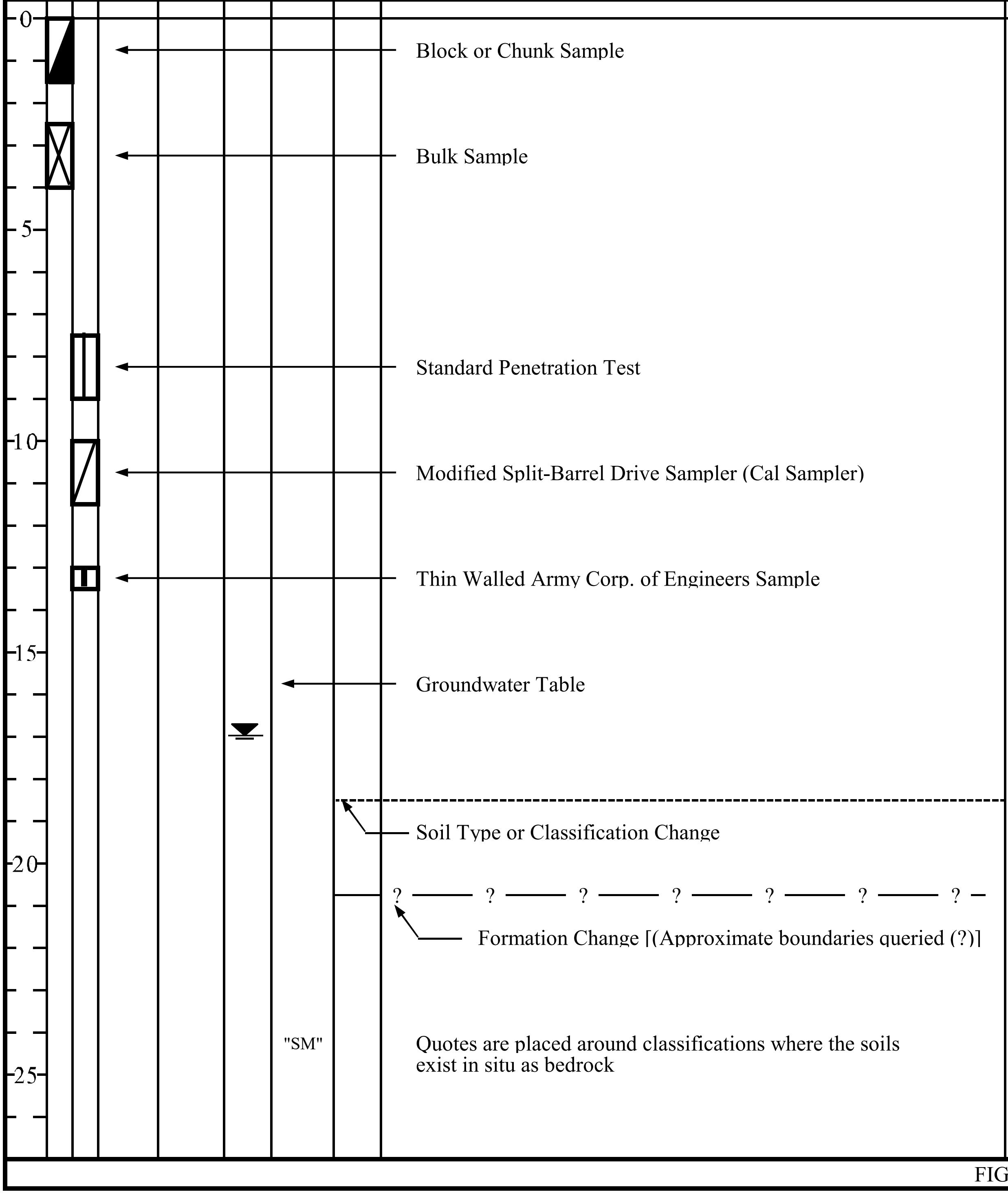
PROJECT: DRILLER: SHEET: of

CTE JOB NO: DRILL METHOD: DRILLING DATE:

LOGGED BY: SAMPLE METHOD: ELEVATION:

Depth (Feet)	Sample	Type	Blows/Foot	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	BORING LEGEND	Laboratory Tests
	Bulk	Driven						DESCRIPTION	



Block or Chunk Sample

Bulk Sample

Standard Penetration Test

Modified Split-Barrel Drive Sampler (Cal Sampler)

Thin Walled Army Corp. of Engineers Sample

Groundwater Table

Soil Type or Classification Change

? ——— ? ——— ? ——— ? ——— ? ——— ? ——— ? –

Formation Change [(Approximate boundaries queried (?)]

Quotes are placed around classifications where the soils exist in situ as bedrock

FIGURE: BL2

PROJECT:	TCMC PSYCHIATRIC HEALTH FACILITY	DRILLER:	AJB	SHEET:	1 of 1
CTE JOB NO:	10-15341G	DRILL METHOD:	HAND AUGER	DRILLING DATE:	12/12/2019
LOGGED BY:	AJB	SAMPLE METHOD:	BULK	ELEVATION:	~290 FEET

BORING: B-1

Depth (Feet)	Bulk Sample	Driven Type	Blows/6"	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0	☒					CL		**STOCKPILE:** Stiff, moist, olive brown, fine to medium grained sandy CLAY with gravel.	
5						SC		**QUATERNARY PREVIOUSLY PLACED FILL:** Medium dense, moist, olive gray, clayey fine to medium grained SAND with abundant fine gravel.	CHM
						SM		**RESIDUAL SOIL:** Loose to medium dense, slightly moist, light olive, silty fine grained SAND.	
						CL		Very stiff, moist, olive, fine grained sandy CLAY.	
10						"SM"		**TERTIARY SANTIAGO FORMATION:** Dense to very dense, slightly moist, light gray, silty fine grained SANDSTONE, oxidized mottling.	
								Total Depth: 10' No Groundwater Encountered	
15									
20									
25									

PROJECT:	TCMC PSYCHIATRIC HEALTH FACILITY	DRILLER:	AJB	SHEET:	1 of 1
CTE JOB NO:	10-15341G	DRILL METHOD:	HAND AUGER	DRILLING DATE:	12/12/2019
LOGGED BY:	AJB	SAMPLE METHOD:	BULK	ELEVATION:	~293 FEET

BORING: B-2

Depth (Feet)	Bulk Sample	Driven Type	Blows/6"	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0	☒					CL/SC		**STOCKPILE:** Stiff or loose, moist, olive brown, fine to medium grained sandy CLAY/ clayey SAND with gravel.	
5									EI
								Total Depth: 7.8' (Refusal on gravel) No Groundwater Encountered	
10									
15									
20									
25									

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PROJECT:	TCMC PSYCHIATRIC HEALTH FACILITY	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-15341G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	12/12/2019
LOGGED BY:	AJB	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~284 FEET

BORING: B-3

Depth (Feet)	Bulk Sample	Driven Type	Blows/6"	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0	☒					SC/CL		Asphalt: 0-4" Base Material: 4-9" **QUATERNARY PREVIOUSLY PLACED FILL:** Loose to medium dense, slightly moist, brown, clayey fine grained SAND/ sandy CLAY with fine gravel. Organic material	MAX
						SM		**RESIDUAL SOIL:** Loose, slightly moist, gray, silty fine grained SAND.	
						CL		Very stiff, slightly moist, gray, silty fine grained SAND.	
5		◩	9 24 50/4"						
						"SM"		**TERTIARY SANTIAGO FORMATION:** Dense, slightly moist, light yellowish gray, fine grained SANDSTONE oxidized mottling. Becomes less oxidized	
10		▥	15 45 42						GS
						"SC"		Very dense, slightly moist, light olive gray, clayey fine grained SANDSTONE, oxidized.	
15		▥	18 35 50/5"						
		▥	23 39 50/4"						GS
20								Total Depth: 19.9' No Groundwater Encountered	
25									

PROJECT:	TCMC PSYCHIATRIC HEALTH FACILITY	DRILLER:	BAJA EXPLORATION	SHEET:	1 of 1
CTE JOB NO:	10-15341G	DRILL METHOD:	HOLLOW-STEM AUGER	DRILLING DATE:	12/12/2019
LOGGED BY:	AJB	SAMPLE METHOD:	RING, SPT and BULK	ELEVATION:	~283 FEET

BORING: B-4

Depth (Feet)	Bulk Sample	Driven Type	Blows/6"	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0						SC		Asphalt: 0-4" Base Material: 4-8" **QUATERNARY PREVIOUSLY PLACED FILL:** Loose, slightly moist, brown, fine grained sandy CLAY with gravel.	
						CL		Stiff, moist, brown, fine grained sandy CLAY with gravel.	
5		⧄	14 22 20			CL		**RESIDUAL SOIL:** Very stiff, moist, olive gray, fine grained sandy CLAY.	MD, DS
						"SC"		**TERTIARY SANTIAGO FORMATION:** Very dense, slightly moist, light yellowish gray, clayey fine to medium grained SANDSTONE.	
10		⧄	11 20 33						MD, DS
						"SM"		Very dense, slightly moist, light gray, silty fine grained SANDSTONE, oxidized mottling, micaceous.	
15		⧄	23 50/5"						MD, DS
		◫	18 34 50/6"						
20								Total Depth: 20' No Groundwater Encountered	
25									

PROJECT:	TCMC PSYCHIATRIC HEALTH FACILITY	DRILLER:	AJB	SHEET:	1 of 1
CTE JOB NO:	10-15341G	DRILL METHOD:	HAND AUGER	DRILLING DATE:	12/12/2019
LOGGED BY:	AJB	SAMPLE METHOD:	BULK	ELEVATION:	~270 FEET

BORING: B-5

Depth (Feet)	Bulk Sample	Driven Type	Blows/6"	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0						SC SM		**QUATERNARY PREVIOUSLY PLACED FILL:** Loose to medium dense, moist, olive brown, clayey fine to medium grained SAND.	
						"SM"		**RESIDUAL SOIL:** Loose, moist, olive gray, silty fine grained SAND.	
								TERTIARY SANTIAGO FORMATION: Dense to very dense, slightly moist, light gray, silty fine grained SANDSTONE.	
5								Total Depth: 2.8' No Groundwater Encountered	
10									
15									
20									
25									

PROJECT:	TCMC PSYCHIATRIC HEALTH FACILITY	DRILLER:	AJB	SHEET:	1 of 1
CTE JOB NO:	10-15341G	DRILL METHOD:	HAND AUGER	DRILLING DATE:	12/12/2019
LOGGED BY:	AJB	SAMPLE METHOD:	BULK	ELEVATION:	~235 FEET

BORING: B-6

Depth (Feet)	Bulk Sample	Driven Type	Blows/6"	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests
0						SC		**QUATERNARY PREVIOUSLY PLACED FILL:** Loose, moist, brown, silty fine grained SAND, plastic debris.	
						"SM"		**TERTIARY SANTIAGO FORMATION:** Dense to very dense, slightly moist, light gray, silty fine grained SANDSTONE.	
								Total Depth: 1.6' No Groundwater Encountered	
5									
10									
15									
20									
25									

APPENDIX C

LABORATORY METHODS AND RESULTS

LABORATORY METHODS AND RESULTS

Laboratory Testing Program

Laboratory tests were performed on representative soil samples to detect their relative engineering properties. Tests were performed following test methods of the American Society for Testing Materials or other accepted standards. The following presents a brief description of the various test methods used.

Classification

Soils were classified visually according to the Unified Soil Classification System. Visual classifications were supplemented by laboratory testing of selected samples according to ASTM D2487. The soil classifications are shown on the Exploration Logs in Appendix B.

In-Place Moisture and Density

To determine the moisture and density of in-place site soils, a representative sample was tested for the moisture and density at time of sampling.

Modified Proctor

Laboratory maximum dry density and optimum moisture content were evaluated according to ASTM D 1557, Method A. A mechanically operated rammer was used during the compaction process.

Expansion Index

Expansion testing was performed on selected samples of the matrix of the on-site soils according to ASTM D 4829.

Particle-Size Analysis

Particle-size analyses were performed on selected representative samples according to ASTM D 422.

Direct Shear

Direct shear tests were performed on either samples direct from the field or on samples recompacted to a specific density. Direct shear testing was performed in accordance with ASTM D 3080. The samples were inundated during shearing to represent adverse field conditions.

Chemical Analysis

Soil materials were collected with sterile sampling equipment and tested for Sulfate and Chloride content, pH, Corrosivity, and Resistivity.

EXPANSION INDEX TEST

ASTM D 4829

LOCATION	DEPTH (feet)	EXPANSION INDEX	EXPANSION POTENTIAL
B-2	0-7.8	19	VERY LOW

IN-PLACE MOISTURE AND DENSITY

LOCATION	DEPTH (feet)	% MOISTURE	DRY DENSITY
B-4	5	6.4	117.9
B-4	10	9.3	117.0
B-4	15	9.5	99.3

SULFATE

LOCATION	DEPTH (feet)	RESULTS ppm
B-1	0-10	70.9

CHLORIDE

LOCATION	DEPTH (feet)	RESULTS ppm
B-1	0-10	34.9

p.H.

LOCATION	DEPTH (feet)	RESULTS
B-1	0-10	8.35

RESISTIVITY

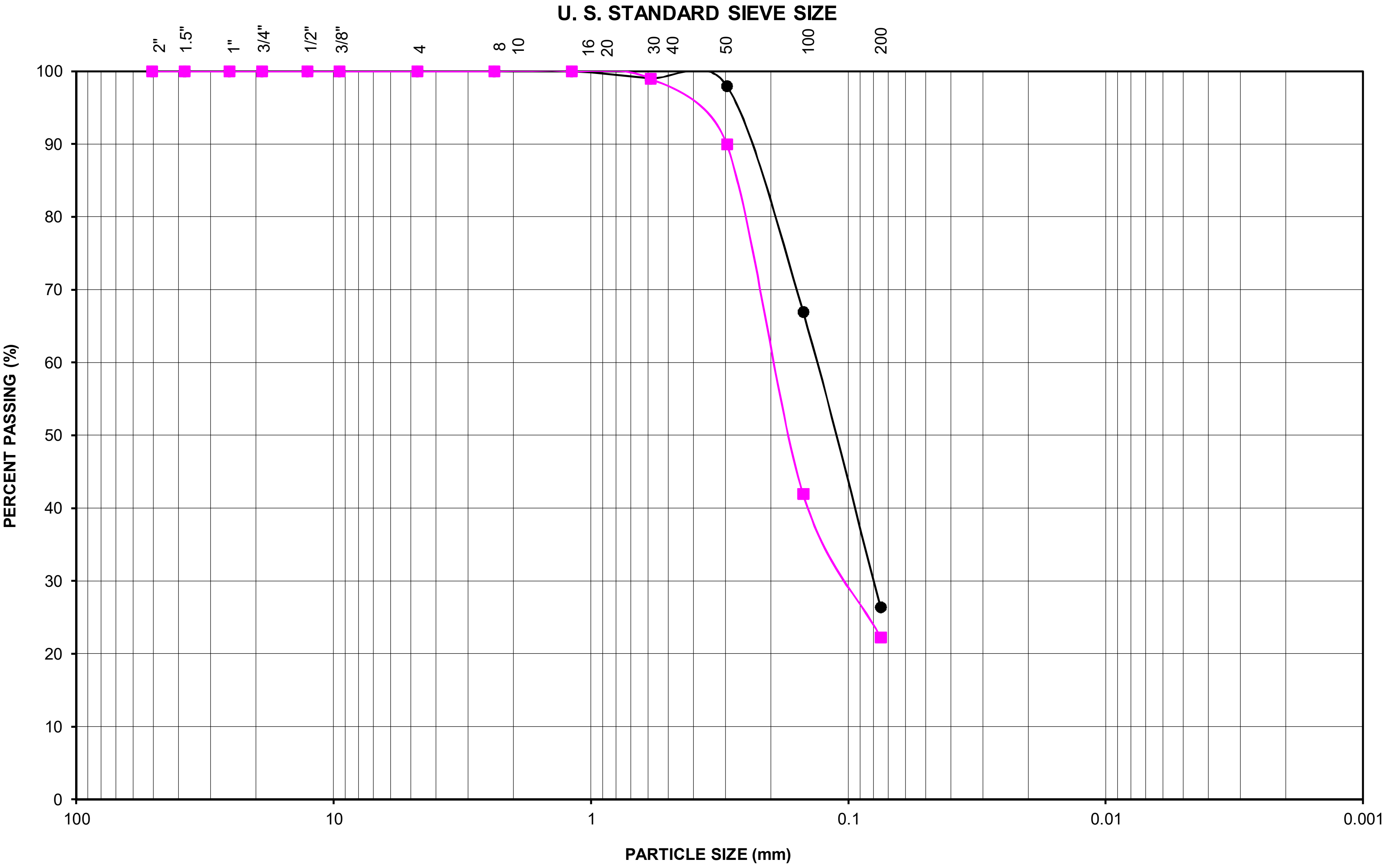
CALIFORNIA TEST 424

LOCATION	DEPTH (feet)	RESULTS ohms-cm
B-1	0-10	5220

MODIFIED PROCTOR

ASTM D 1557

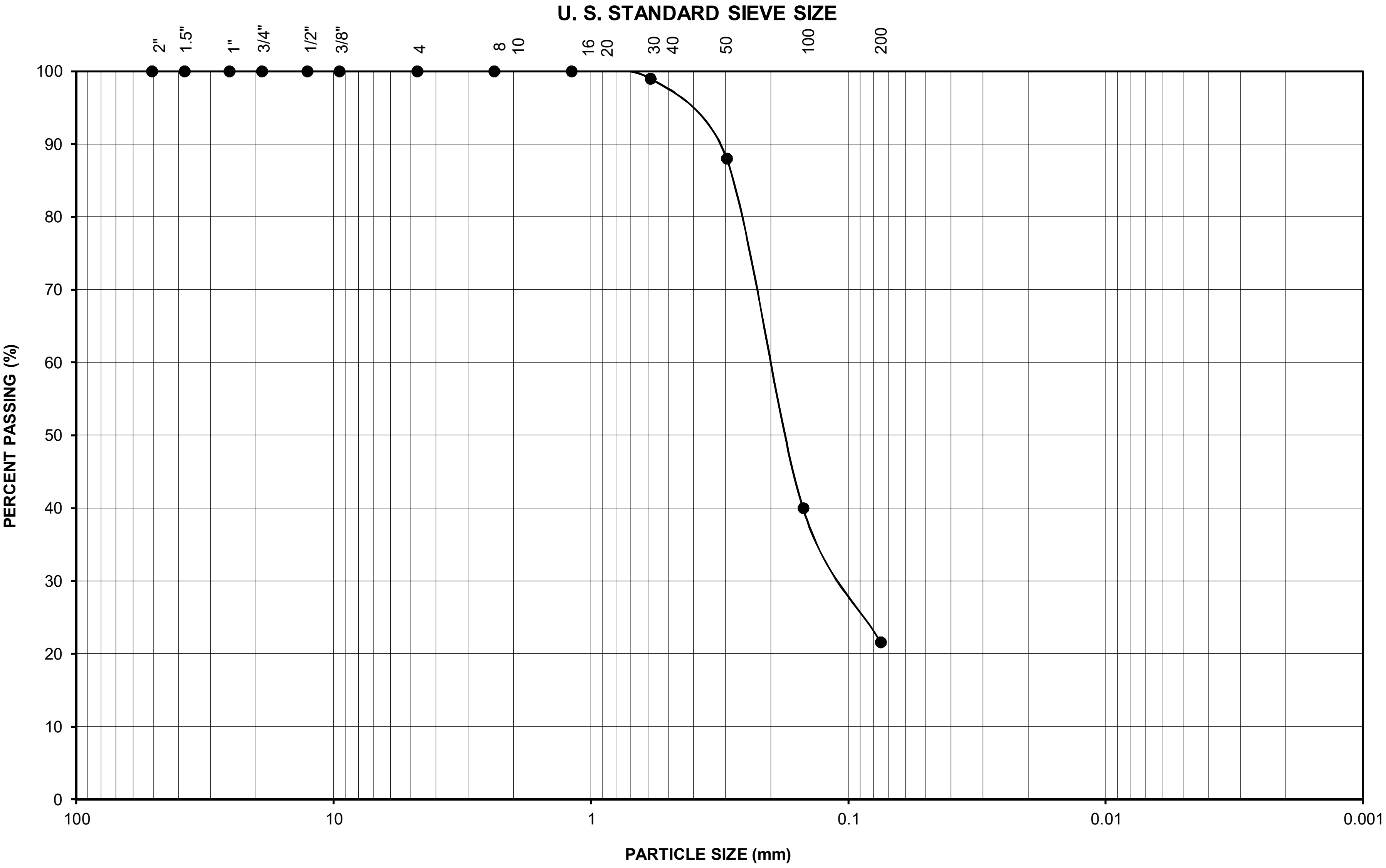
LOCATION	DEPTH (feet)	MAXIUM DRY DENSITY (PCF)	OPTIMUM MOISTURE (%)
B-3	0-5	123	10.7



PARTICLE SIZE ANALYSIS

CTE *INC.* Construction Testing & Engineering, Inc.
1441 Montiel Rd Ste 115, Escondido, CA 92026 Ph (760) 746-4955

Sample Designation	Sample Depth (feet)	Symbol	Liquid Limit (%)	Plasticity Index	Classification
B-3	10	●	0	0	SM
B-3	18.5	■	0	0	SC
CTE JOB NUMBER:		10-15341G		FIGURE:	C-1

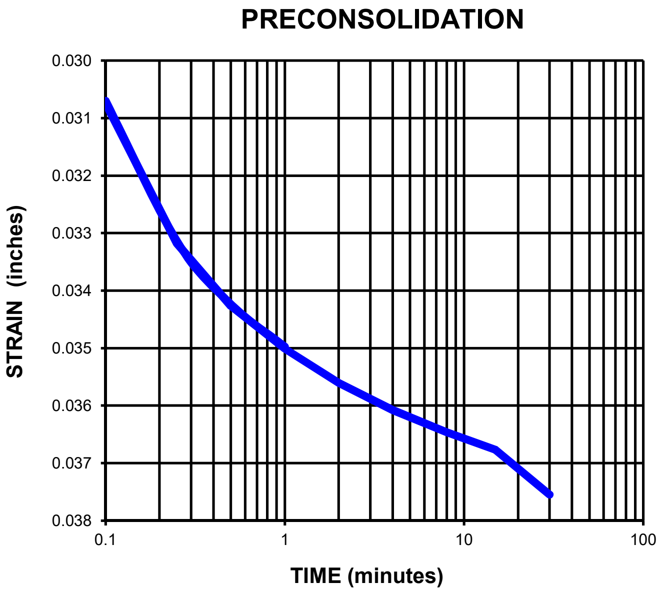


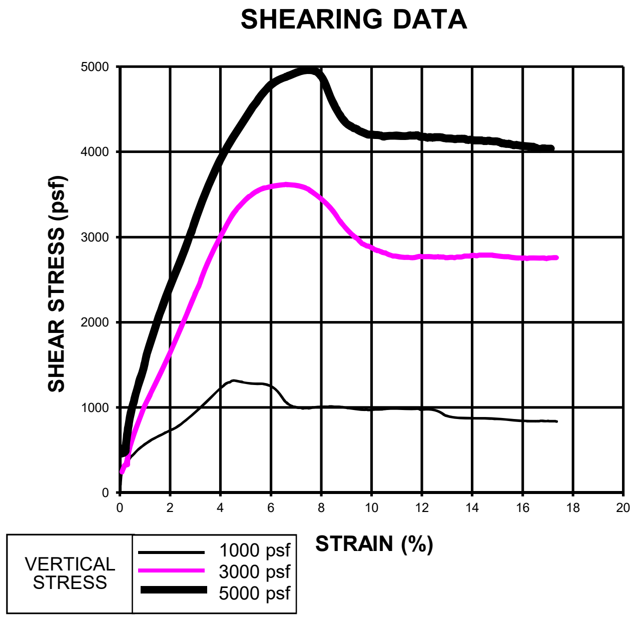
PARTICLE SIZE ANALYSIS

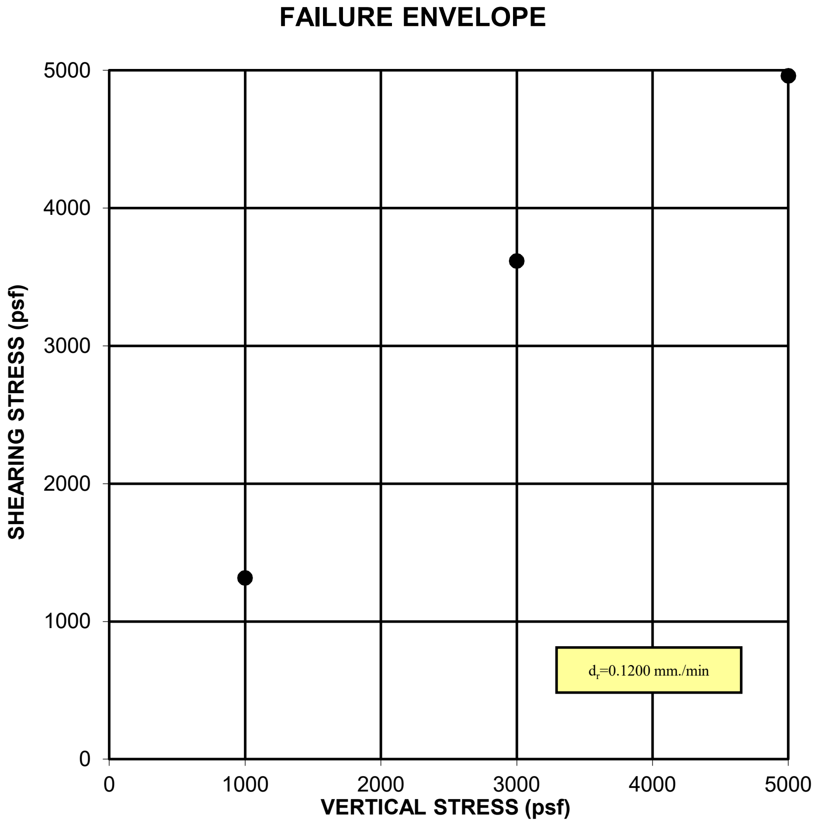
CTE INC. Construction Testing & Engineering, Inc.
1441 Montiel Rd Ste 115, Escondido, CA 92026 Ph (760) 746-4955

Sample Designation	Sample Depth (feet)	Symbol	Liquid Limit (%)	Plasticity Index	Classification
B-4	18.5	●	0	0	SM

CTE JOB NUMBER: 10-15341G FIGURE: C-2


PRECONSOLIDATION


SHEARING DATA


FAILURE ENVELOPE

d_r=0.1200 mm./min

SHEAR STRENGTH TEST - ASTM D3080

Job Name: Tri-City

Project Number: 10-15341G

Lab Number: 30273

Sample Location: B-4 @ 5'

Sample Description: Grayish Brown SC

Sample Date: 12/12/2019

Test Date: 12/16/2019

Tested by: KF

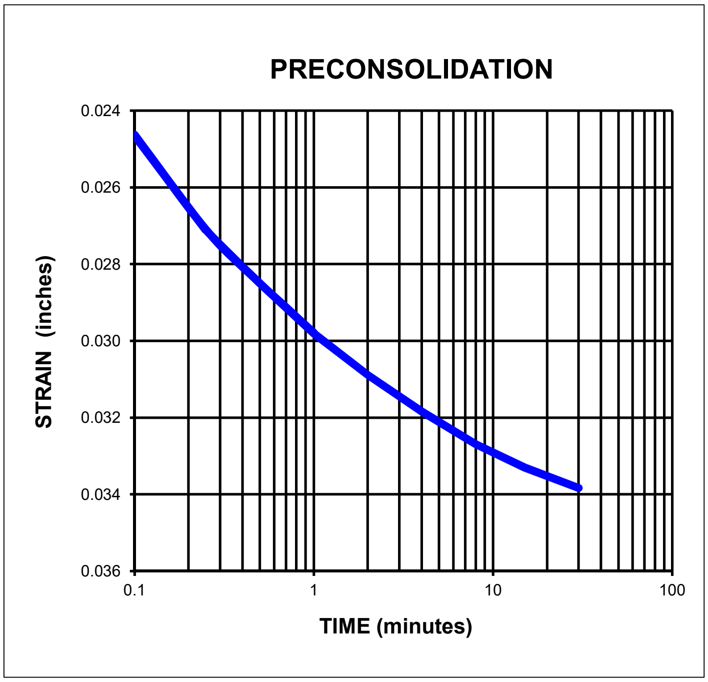
Initial Dry Density (pcf): 117.9

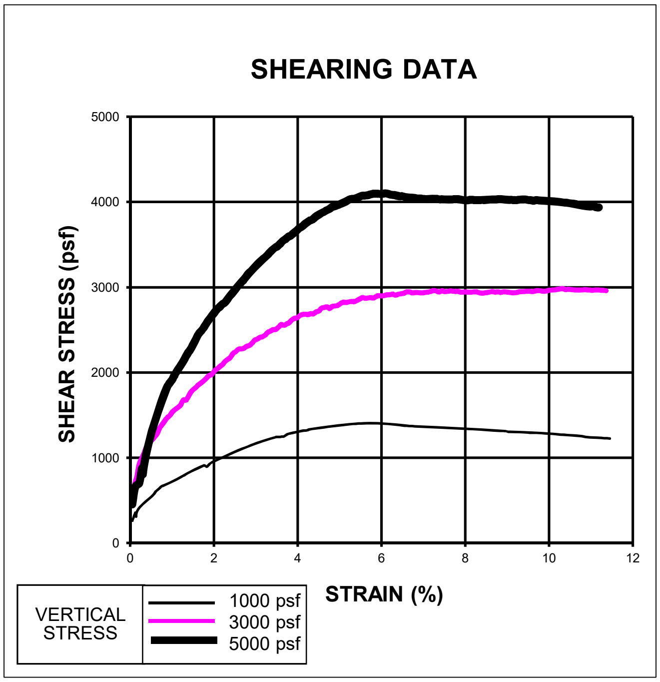
Initial Moisture (%): 6.4

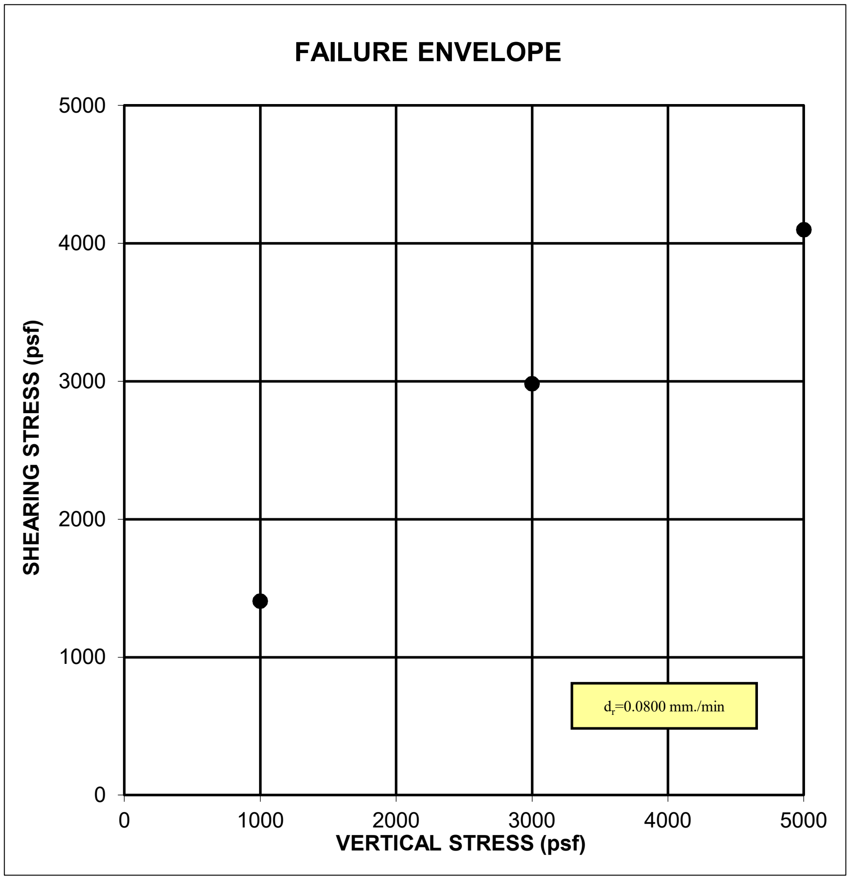
Final Moisture (%): 15.1

Cohesion: 560 psf

Angle Of Friction: 42.3







SHEAR STRENGTH TEST - ASTM D3080

Job Name: Tri-city

Initial Dry Density (pcf): 117.0

Project Number: 10-15341G

Sample Date: 12/12/2019

Initial Moisture (%): 9.3

Lab Number: 30273

Test Date: 12/18/2019

Final Moisture (%): 23.1

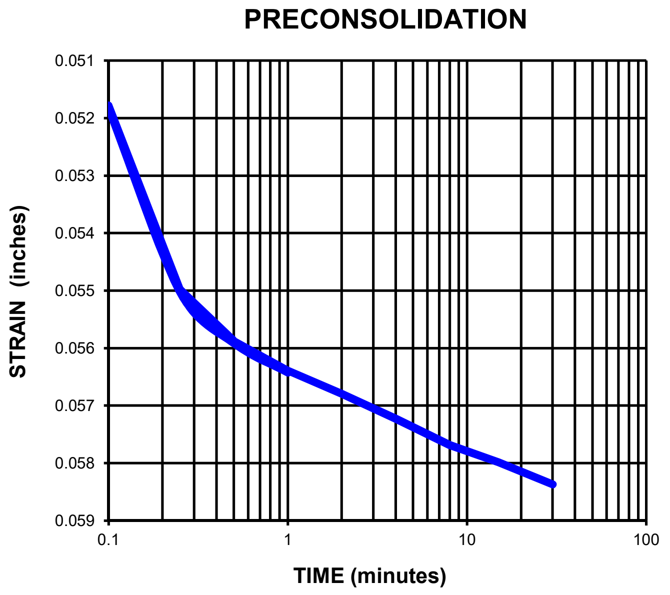
Sample Location: B-4 @ 10'

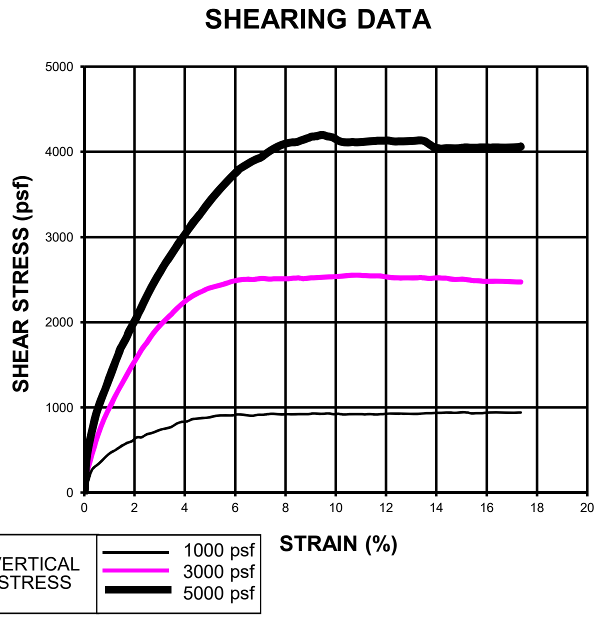
Tested by: KF

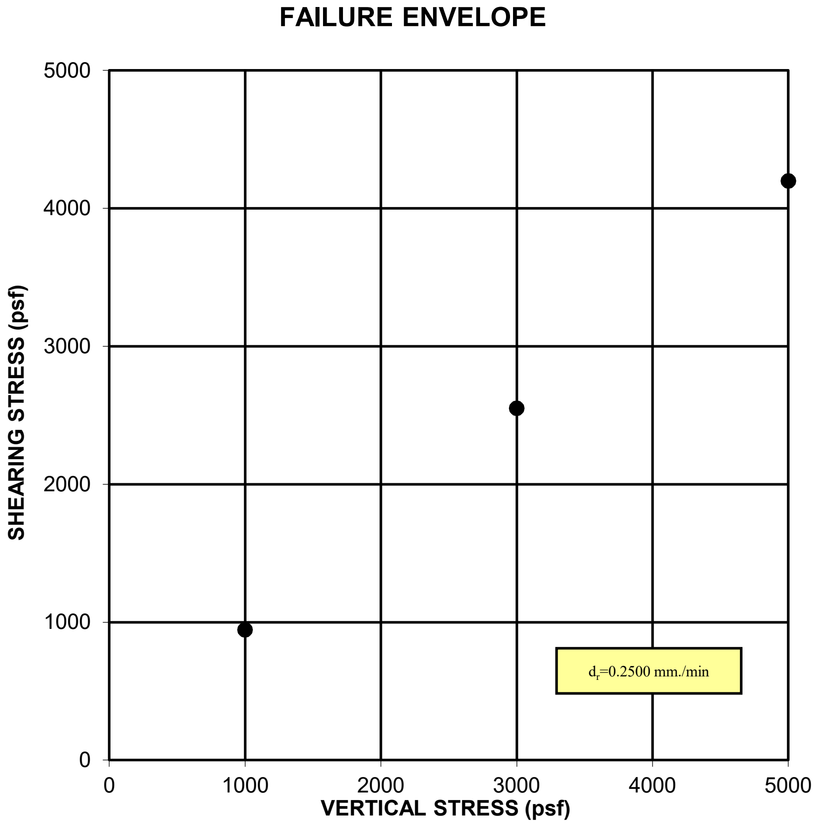
Cohesion: 800 psf

Sample Description: Grayish Brown CL

Angle Of Friction: 34.0







CTE INC.

SHEAR STRENGTH TEST - ASTM D3080

Job Name:	Tri-city			Initial Dry Density (pcf):	99.3
Project Number:	10-15341G	Sample Date:	12/12/2019	Initial Moisture (%):	9.5
Lab Number:	30273	Test Date:	12/19/2019	Final Moisture (%):	26.4
Sample Location:	B-4 @ 15'	Tested by:	KF	Cohesion:	120 psf
Sample Description:	Grayish Brown SM			Angle Of Friction:	39.1

APPENDIX D

STANDARD SPECIFICATIONS FOR GRADING

Section 1 - General

Construction Testing & Engineering, Inc. presents the following standard recommendations for grading and other associated operations on construction projects. These guidelines should be considered a portion of the project specifications. Recommendations contained in the body of the previously presented soils report shall supersede the recommendations and or requirements as specified herein. The project geotechnical consultant shall interpret disputes arising out of interpretation of the recommendations contained in the soils report or specifications contained herein.

Section 2 - Responsibilities of Project Personnel

The geotechnical consultant should provide observation and testing services sufficient to general conformance with project specifications and standard grading practices. The geotechnical consultant should report any deviations to the client or his authorized representative.

The Client should be chiefly responsible for all aspects of the project. He or his authorized representative has the responsibility of reviewing the findings and recommendations of the geotechnical consultant. He shall authorize or cause to have authorized the Contractor and/or other consultants to perform work and/or provide services. During grading the Client or his authorized representative should remain on-site or should remain reasonably accessible to all concerned parties in order to make decisions necessary to maintain the flow of the project.

The Contractor is responsible for the safety of the project and satisfactory completion of all grading and other associated operations on construction projects, including, but not limited to, earth work in accordance with the project plans, specifications and controlling agency requirements.

Section 3 - Preconstruction Meeting

A preconstruction site meeting should be arranged by the owner and/or client and should include the grading contractor, design engineer, geotechnical consultant, owner's representative and representatives of the appropriate governing authorities.

Section 4 - Site Preparation

The client or contractor should obtain the required approvals from the controlling authorities for the project prior, during and/or after demolition, site preparation and removals, etc. The appropriate approvals should be obtained prior to proceeding with grading operations.

Clearing and grubbing should consist of the removal of vegetation such as brush, grass, woods, stumps, trees, root of trees and otherwise deleterious natural materials from the areas to be graded. Clearing and grubbing should extend to the outside of all proposed excavation and fill areas.

Demolition should include removal of buildings, structures, foundations, reservoirs, utilities (including underground pipelines, septic tanks, leach fields, seepage pits, cisterns, mining shafts, tunnels, etc.) and other man-made surface and subsurface improvements from the areas to be graded. Demolition of utilities should include proper capping and/or rerouting pipelines at the project perimeter and cutoff and capping of wells in accordance with the requirements of the governing authorities and the recommendations of the geotechnical consultant at the time of demolition.

Trees, plants or man-made improvements not planned to be removed or demolished should be protected by the contractor from damage or injury.

Debris generated during clearing, grubbing and/or demolition operations should be wasted from areas to be graded and disposed off-site. Clearing, grubbing and demolition operations should be performed under the observation of the geotechnical consultant.

Section 5 - Site Protection

Protection of the site during the period of grading should be the responsibility of the contractor. Unless other provisions are made in writing and agreed upon among the concerned parties, completion of a portion of the project should not be considered to preclude that portion or adjacent areas from the requirements for site protection until such time as the entire project is complete as identified by the geotechnical consultant, the client and the regulating agencies.

Precautions should be taken during the performance of site clearing, excavations and grading to protect the work site from flooding, ponding or inundation by poor or improper surface drainage. Temporary provisions should be made during the rainy season to adequately direct surface drainage away from and off the work site. Where low areas cannot be avoided, pumps should be kept on hand to continually remove water during periods of rainfall.

Rain related damage should be considered to include, but may not be limited to, erosion, silting, saturation, swelling, structural distress and other adverse conditions as determined by the geotechnical consultant. Soil adversely affected should be classified as unsuitable materials and should be subject to overexcavation and replacement with compacted fill or other remedial grading as recommended by the geotechnical consultant.

The contractor should be responsible for the stability of all temporary excavations. Recommendations by the geotechnical consultant pertaining to temporary excavations (e.g., backcuts) are made in consideration of stability of the completed project and, therefore, should not be considered to preclude the responsibilities of the contractor. Recommendations by the geotechnical consultant should not be considered to preclude requirements that are more restrictive by the regulating agencies. The contractor should provide during periods of extensive rainfall plastic sheeting to prevent unprotected slopes from becoming saturated and unstable. When deemed appropriate by the geotechnical consultant or governing agencies the contractor shall install checkdams, desilting basins, sand bags or other drainage control measures.

In relatively level areas and/or slope areas, where saturated soil and/or erosion gullies exist to depths of greater than 1.0 foot; they should be overexcavated and replaced as compacted fill in accordance with the applicable specifications. Where affected materials exist to depths of 1.0 foot or less below proposed finished grade, remedial grading by moisture conditioning in-place, followed by thorough recompaction in accordance with the applicable grading guidelines herein may be attempted. If the desired results are not achieved, all affected materials should be overexcavated and replaced as compacted fill in accordance with the slope repair recommendations herein. If field conditions dictate, the geotechnical consultant may recommend other slope repair procedures.

Section 6 - Excavations

6.1 Unsuitable Materials

Materials that are unsuitable should be excavated under observation and recommendations of the geotechnical consultant. Unsuitable materials include, but may not be limited to, dry, loose, soft, wet, organic compressible natural soils and fractured, weathered, soft bedrock and nonengineered or otherwise deleterious fill materials.

Material identified by the geotechnical consultant as unsatisfactory due to its moisture conditions should be overexcavated; moisture conditioned as needed, to a uniform at or above optimum moisture condition before placement as compacted fill.

If during the course of grading adverse geotechnical conditions are exposed which were not anticipated in the preliminary soil report as determined by the geotechnical consultant additional exploration, analysis, and treatment of these problems may be recommended.

6.2 Cut Slopes

Unless otherwise recommended by the geotechnical consultant and approved by the regulating agencies, permanent cut slopes should not be steeper than 2:1 (horizontal: vertical).

The geotechnical consultant should observe cut slope excavation and if these excavations expose loose cohesionless, significantly fractured or otherwise unsuitable material, the materials should be overexcavated and replaced with a compacted stabilization fill. If encountered specific cross section details should be obtained from the Geotechnical Consultant.

When extensive cut slopes are excavated or these cut slopes are made in the direction of the prevailing drainage, a non-erodible diversion swale (brow ditch) should be provided at the top of the slope.

6.3 Pad Areas

All lot pad areas, including side yard terrace containing both cut and fill materials, transitions, located less than 3 feet deep should be overexcavated to a depth of 3 feet and replaced with a uniform compacted fill blanket of 3 feet. Actual depth of overexcavation may vary and should be delineated by the geotechnical consultant during grading, especially where deep or drastic transitions are present.

For pad areas created above cut or natural slopes, positive drainage should be established away from the top-of-slope. This may be accomplished utilizing a berm drainage swale and/or an appropriate pad gradient. A gradient in soil areas away from the top-of-slopes of 2 percent or greater is recommended.

Section 7 - Compacted Fill

All fill materials should have fill quality, placement, conditioning and compaction as specified below or as approved by the geotechnical consultant.

7.1 Fill Material Quality

Excavated on-site or import materials which are acceptable to the geotechnical consultant may be utilized as compacted fill, provided trash, vegetation and other deleterious materials are removed prior to placement. All import materials anticipated for use on-site should be sampled tested and approved prior to and placement is in conformance with the requirements outlined.

Rocks 12 inches in maximum and smaller may be utilized within compacted fill provided sufficient fill material is placed and thoroughly compacted over and around all rock to effectively fill rock voids. The amount of rock should not exceed 40 percent by dry weight passing the 3/4-inch sieve. The geotechnical consultant may vary those requirements as field conditions dictate.

Where rocks greater than 12 inches but less than four feet of maximum dimension are generated during grading, or otherwise desired to be placed within an engineered fill, special handling in accordance with the recommendations below. Rocks greater than four feet should be broken down or disposed off-site.

7.2 Placement of Fill

Prior to placement of fill material, the geotechnical consultant should observe and approve the area to receive fill. After observation and approval, the exposed ground surface should be scarified to a depth of 6 to 8 inches. The scarified material should be conditioned (i.e. moisture added or air dried by continued discing) to achieve a moisture content at or slightly above optimum moisture conditions and compacted to a minimum of 90 percent of the maximum density or as otherwise recommended in the soils report or by appropriate government agencies.

Compacted fill should then be placed in thin horizontal lifts not exceeding eight inches in loose thickness prior to compaction. Each lift should be moisture conditioned as needed, thoroughly blended to achieve a consistent moisture content at or slightly above optimum and thoroughly compacted by mechanical methods to a minimum of 90 percent of laboratory maximum dry density. Each lift should be treated in a like manner until the desired finished grades are achieved.

The contractor should have suitable and sufficient mechanical compaction equipment and watering apparatus on the job site to handle the amount of fill being placed in consideration of moisture retention properties of the materials and weather conditions.

When placing fill in horizontal lifts adjacent to areas sloping steeper than 5:1 (horizontal: vertical), horizontal keys and vertical benches should be excavated into the adjacent slope area. Keying and benching should be sufficient to provide at least six-foot wide benches and a minimum of four feet of vertical bench height within the firm natural ground, firm bedrock or engineered compacted fill. No compacted fill should be placed in an area after keying and benching until the geotechnical consultant has reviewed the area. Material generated by the benching operation should be moved sufficiently away from the bench area to allow for the recommended review of the horizontal bench prior to placement of fill.

Within a single fill area where grading procedures dictate two or more separate fills, temporary slopes (false slopes) may be created. When placing fill adjacent to a false slope, benching should be conducted in the same manner as above described. At least a 3-foot vertical bench should be established within the firm core of adjacent approved compacted fill prior to placement of additional fill. Benching should proceed in at least 3-foot vertical increments until the desired finished grades are achieved.

Prior to placement of additional compacted fill following an overnight or other grading delay, the exposed surface or previously compacted fill should be processed by scarification, moisture conditioning as needed to at or slightly above optimum moisture content, thoroughly blended and recompacted to a minimum of 90 percent of laboratory maximum dry density. Where unsuitable materials exist to depths of greater than one foot, the unsuitable materials should be over-excavated.

Following a period of flooding, rainfall or overwatering by other means, no additional fill should be placed until damage assessments have been made and remedial grading performed as described herein.

Rocks 12 inch in maximum dimension and smaller may be utilized in the compacted fill provided the fill is placed and thoroughly compacted over and around all rock. No oversize material should be used within 3 feet of finished pad grade and within 1 foot of other compacted fill areas. Rocks 12 inches up to four feet maximum dimension should be placed below the upper 10 feet of any fill and should not be closer than 15 feet to any slope face. These recommendations could vary as locations of improvements dictate. Where practical, oversized material should not be placed below areas where structures or deep utilities are proposed. Oversized material should be placed in windrows on a clean, overexcavated or unyielding compacted fill or firm natural ground surface. Select native or imported granular soil (S.E. 30 or higher) should be placed and thoroughly flooded over and around all windrowed rock, such that voids are filled. Windrows of oversized material should be staggered so those successive strata of oversized material are not in the same vertical plane.

It may be possible to dispose of individual larger rock as field conditions dictate and as recommended by the geotechnical consultant at the time of placement.

The contractor should assist the geotechnical consultant and/or his representative by digging test pits for removal determinations and/or for testing compacted fill. The contractor should provide this work at no additional cost to the owner or contractor's client.

Fill should be tested by the geotechnical consultant for compliance with the recommended relative compaction and moisture conditions. Field density testing should conform to ASTM Method of Test D 1556-00, D 2922-04. Tests should be conducted at a minimum of approximately two vertical feet or approximately 1,000 to 2,000 cubic yards of fill placed. Actual test intervals may vary as field conditions dictate. Fill found not to be in conformance with the grading recommendations should be removed or otherwise handled as recommended by the geotechnical consultant.

7.3 Fill Slopes

Unless otherwise recommended by the geotechnical consultant and approved by the regulating agencies, permanent fill slopes should not be steeper than 2:1 (horizontal: vertical).

Except as specifically recommended in these grading guidelines compacted fill slopes should be over-built two to five feet and cut back to grade, exposing the firm, compacted fill inner core. The actual amount of overbuilding may vary as field conditions dictate. If the desired results are not achieved, the existing slopes should be overexcavated and reconstructed under the guidelines of the geotechnical consultant. The degree of overbuilding shall be increased until the desired compacted slope surface condition is achieved. Care should be taken by the contractor to provide thorough mechanical compaction to the outer edge of the overbuilt slope surface.

At the discretion of the geotechnical consultant, slope face compaction may be attempted by conventional construction procedures including backrolling. The procedure must create a firmly compacted material throughout the entire depth of the slope face to the surface of the previously compacted firm fill intercore.

During grading operations, care should be taken to extend compactive effort to the outer edge of the slope. Each lift should extend horizontally to the desired finished slope surface or more as needed to ultimately established desired grades. Grade during construction should not be allowed to roll off at the edge of the slope. It may be helpful to elevate slightly the outer edge of the slope. Slough resulting from the placement of individual lifts should not be allowed to drift down over previous lifts. At intervals not exceeding four feet in vertical slope height or the capability of available equipment, whichever is less, fill slopes should be thoroughly dozer trackrolled.

For pad areas above fill slopes, positive drainage should be established away from the top-of-slope. This may be accomplished using a berm and pad gradient of at least two percent.

Section 8 - Trench Backfill

Utility and/or other excavation of trench backfill should, unless otherwise recommended, be compacted by mechanical means. Unless otherwise recommended, the degree of compaction should be a minimum of 90 percent of the laboratory maximum density.

Within slab areas, but outside the influence of foundations, trenches up to one foot wide and two feet deep may be backfilled with sand and consolidated by jetting, flooding or by mechanical means. If on-site materials are utilized, they should be wheel-rolled, tamped or otherwise compacted to a firm condition. For minor interior trenches, density testing may be deleted or spot testing may be elected if deemed necessary, based on review of backfill operations during construction.

If utility contractors indicate that it is undesirable to use compaction equipment in close proximity to a buried conduit, the contractor may elect the utilization of light weight mechanical compaction equipment and/or shading of the conduit with clean, granular material, which should be thoroughly jetted in-place above the conduit, prior to initiating mechanical compaction procedures. Other methods of utility trench compaction may also be appropriate, upon review of the geotechnical consultant at the time of construction.

In cases where clean granular materials are proposed for use in lieu of native materials or where flooding or jetting is proposed, the procedures should be considered subject to review by the geotechnical consultant. Clean granular backfill and/or bedding are not recommended in slope areas.

Section 9 - Drainage

Where deemed appropriate by the geotechnical consultant, canyon subdrain systems should be installed in accordance with CTE’s recommendations during grading.

Typical subdrains for compacted fill buttresses, slope stabilization or sidehill masses, should be installed in accordance with the specifications.

Roof, pad and slope drainage should be directed away from slopes and areas of structures to suitable disposal areas via non-erodible devices (i.e., gutters, downspouts, and concrete swales).

For drainage in extensively landscaped areas near structures, (i.e., within four feet) a minimum of 5 percent gradient away from the structure should be maintained. Pad drainage of at least 2 percent should be maintained over the remainder of the site.

Drainage patterns established at the time of fine grading should be maintained throughout the life of the project. Property owners should be made aware that altering drainage patterns could be detrimental to slope stability and foundation performance.

Section 10 - Slope Maintenance

10.1 - Landscape Plants

To enhance surficial slope stability, slope planting should be accomplished at the completion of grading. Slope planting should consist of deep-rooting vegetation requiring little watering. Plants native to the southern California area and plants relative to native plants are generally desirable. Plants native to other semi-arid and arid areas may also be appropriate. A Landscape Architect should be the best party to consult regarding actual types of plants and planting configuration.

10.2 - Irrigation

Irrigation pipes should be anchored to slope faces, not placed in trenches excavated into slope faces.

Slope irrigation should be minimized. If automatic timing devices are utilized on irrigation systems, provisions should be made for interrupting normal irrigation during periods of rainfall.

10.3 - Repair

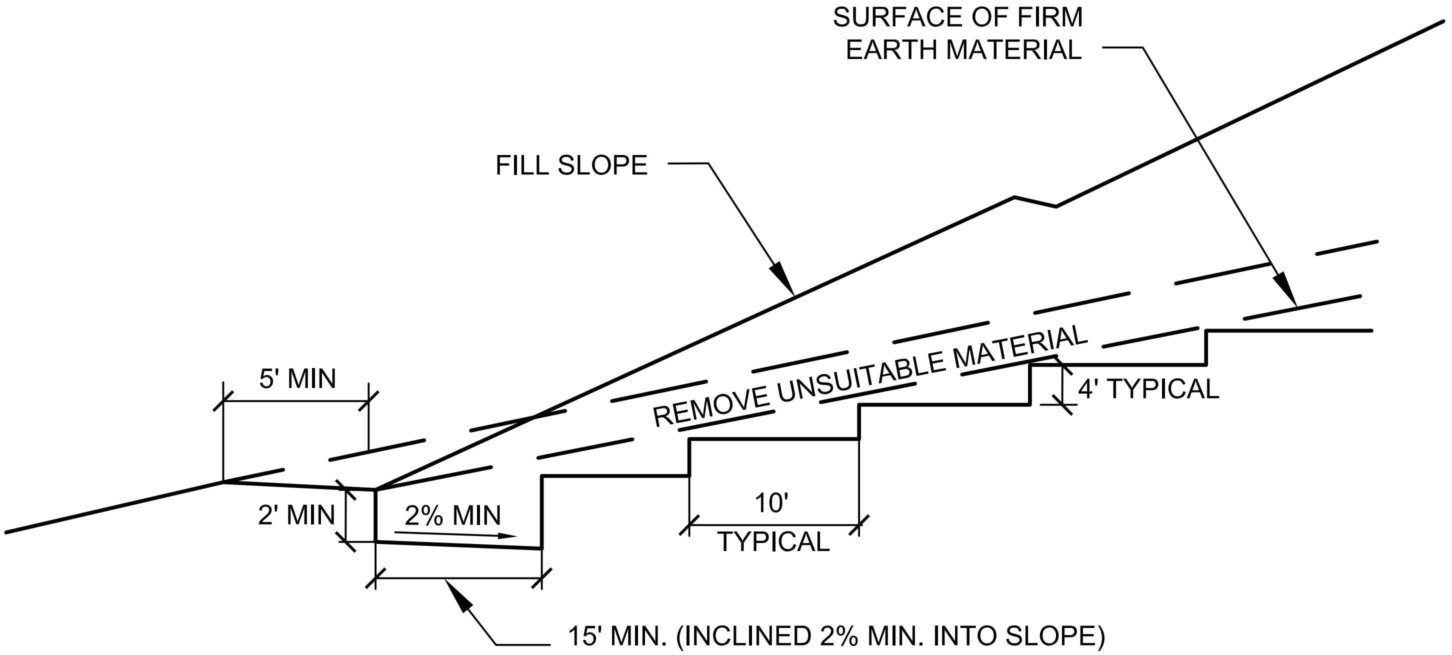
As a precautionary measure, plastic sheeting should be readily available, or kept on hand, to protect all slope areas from saturation by periods of heavy or prolonged rainfall. This measure is strongly recommended, beginning with the period prior to landscape planting.

If slope failures occur, the geotechnical consultant should be contacted for a field review of site conditions and development of recommendations for evaluation and repair.

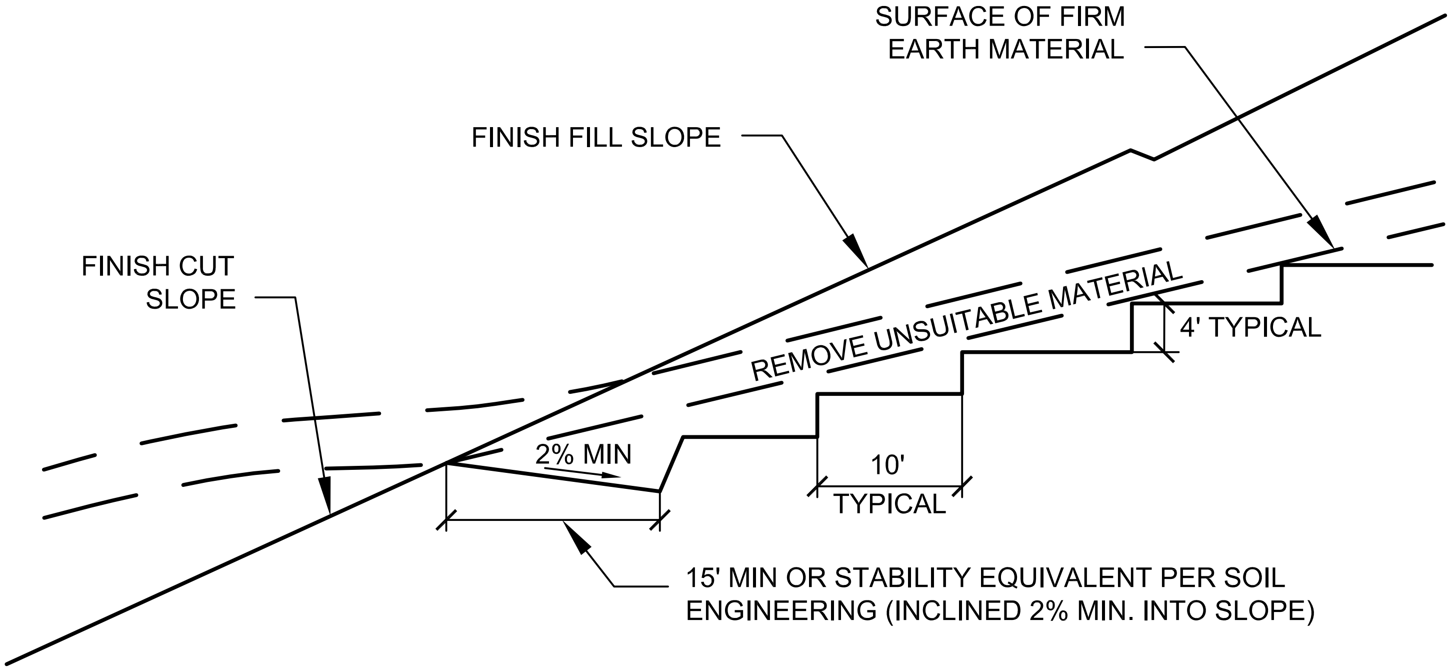
If slope failures occur as a result of exposure to period of heavy rainfall, the failure areas and currently unaffected areas should be covered with plastic sheeting to protect against additional saturation.

In the accompanying Standard Details, appropriate repair procedures are illustrated for superficial slope failures (i.e., occurring typically within the outer one foot to three feet of a slope face).

BENCHING FILL OVER NATURAL

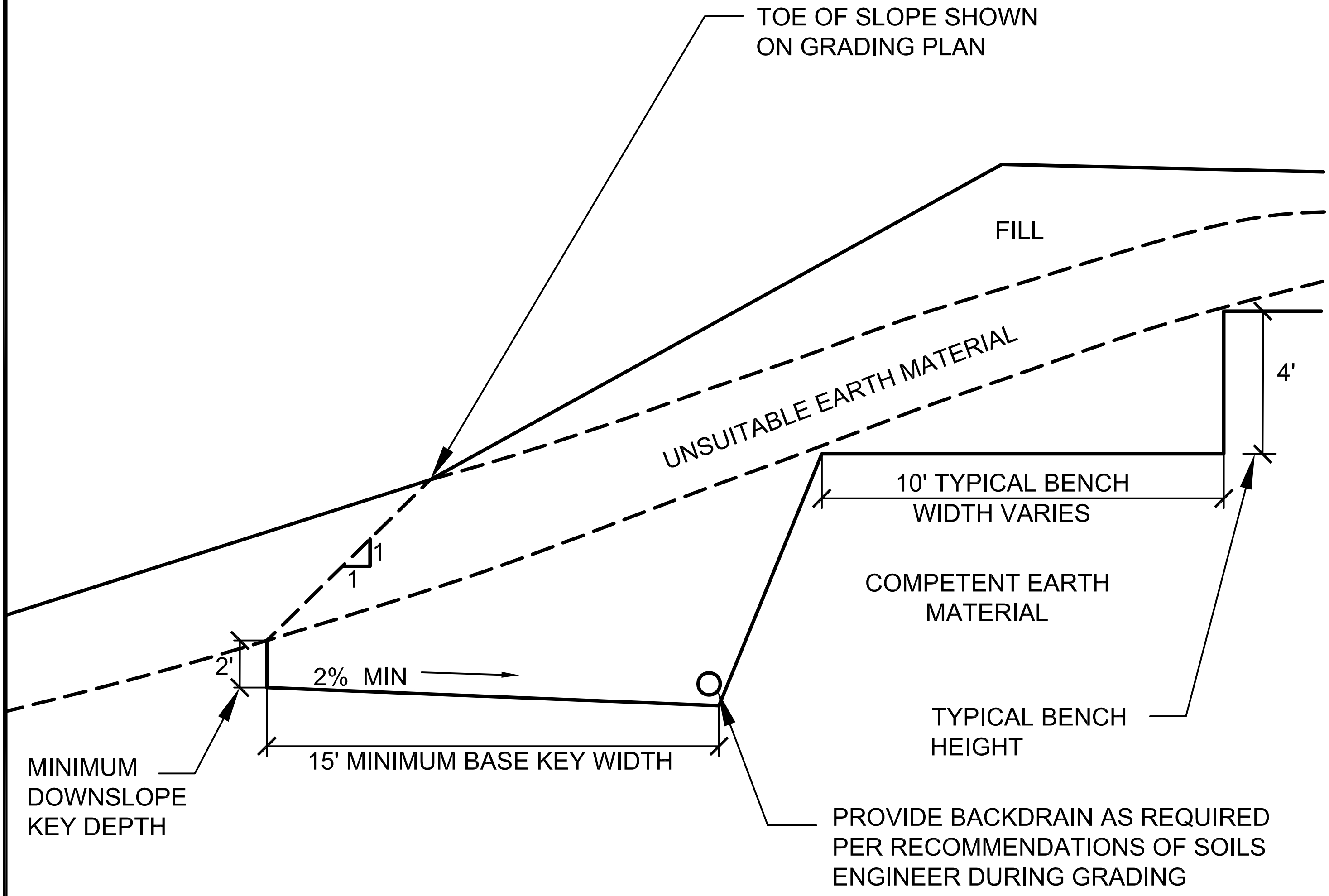


BENCHING FILL OVER CUT



NOT TO SCALE

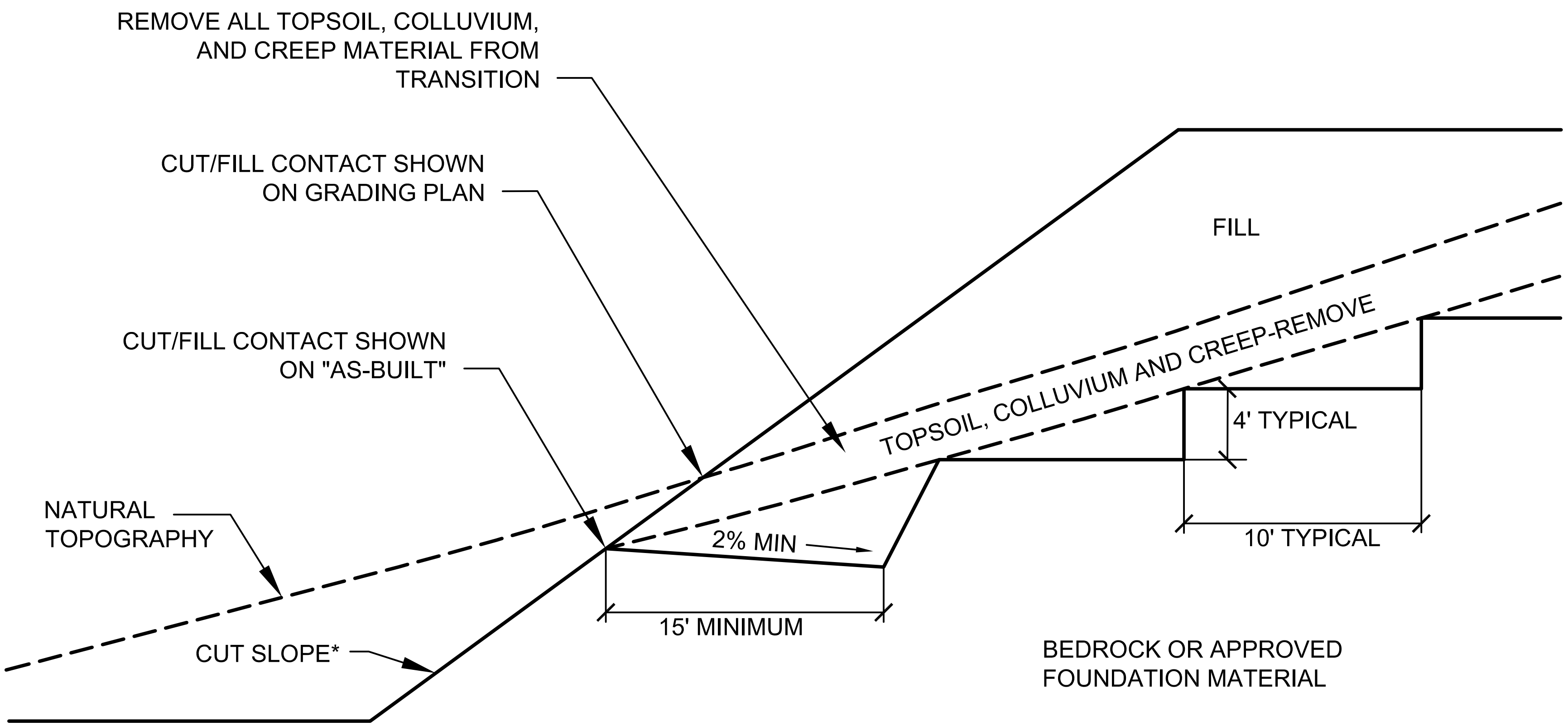
BENCHING FOR COMPACTED FILL DETAIL



WHERE NATURAL SLOPE GRADIENT IS 5:1 OR LESS, BENCHING IS NOT NECESSARY. FILL IS NOT TO BE PLACED ON COMPRESSIBLE OR UNSUITABLE MATERIAL.

NOT TO SCALE

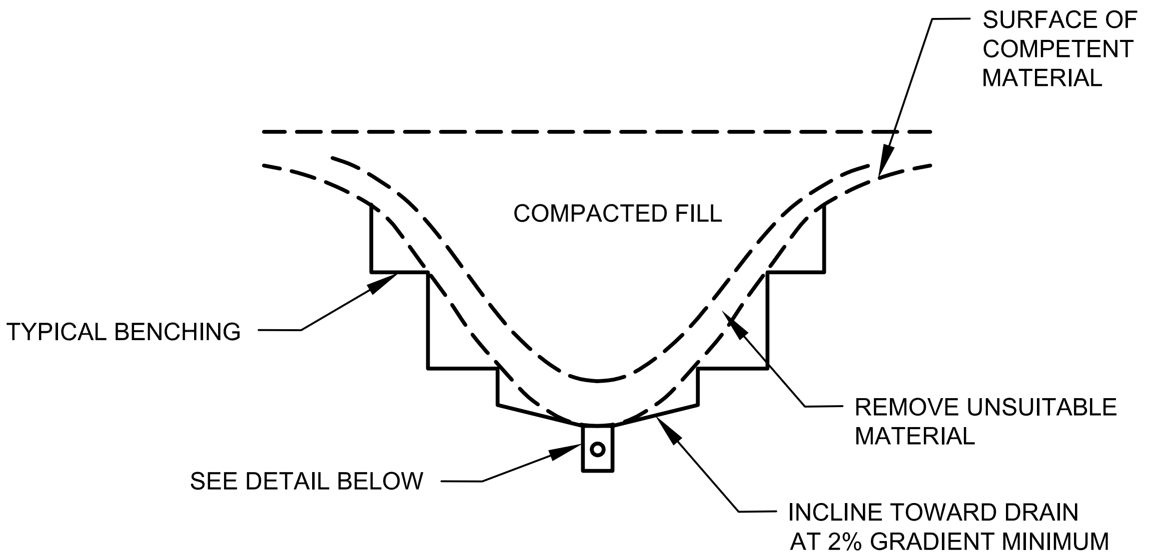
FILL SLOPE ABOVE NATURAL GROUND DETAIL



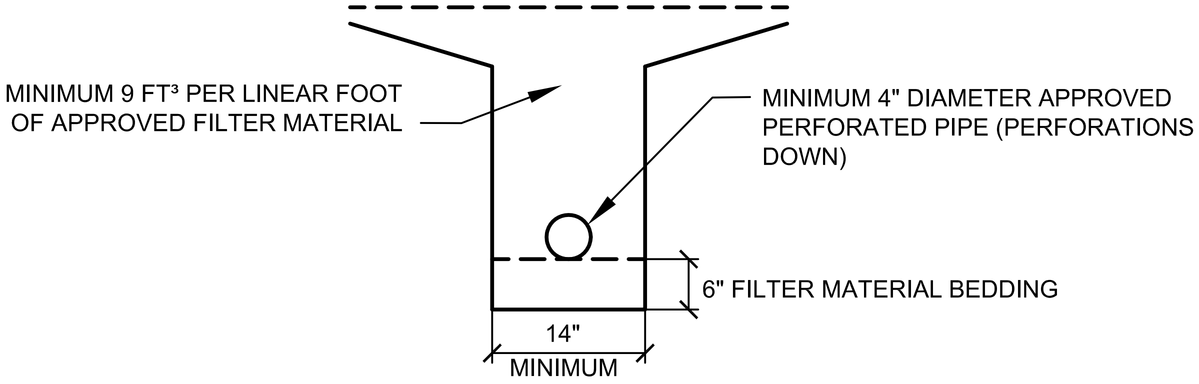
*NOTE: CUT SLOPE PORTION SHOULD BE MADE PRIOR TO PLACEMENT OF FILL

NOT TO SCALE

FILL SLOPE ABOVE CUT SLOPE DETAIL



DETAIL



CALTRANS CLASS 2 PERMEABLE MATERIAL
FILTER MATERIAL TO MEET FOLLOWING
SPECIFICATION OR APPROVED EQUAL:

SIEVE SIZE	PERCENTAGE PASSING
1"	100
¾"	90-100
⅜"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

APPROVED PIPE TO BE SCHEDULE 40 POLY-VINYL-CHLORIDE (P.V.C.) OR APPROVED EQUAL. MINIMUM CRUSH STRENGTH 1000 psi

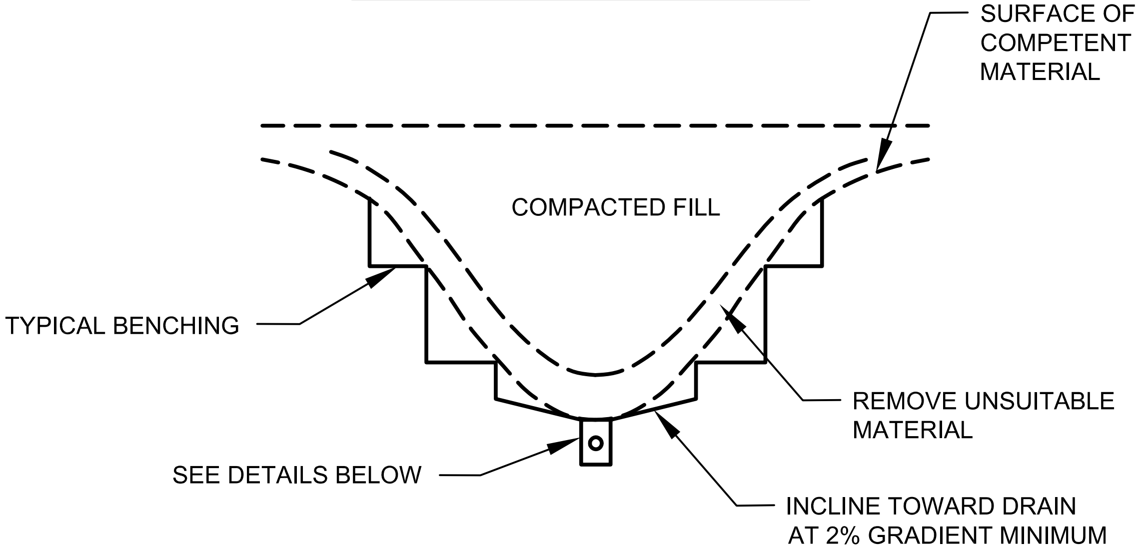
PIPE DIAMETER TO MEET THE FOLLOWING CRITERIA, SUBJECT TO FIELD REVIEW BASED ON ACTUAL GEOTECHNICAL CONDITIONS ENCOUNTERED DURING GRADING

LENGTH OF RUN	PIPE DIAMETER
INITIAL 500'	4"
500' TO 1500'	6"
> 1500'	8"

NOT TO SCALE

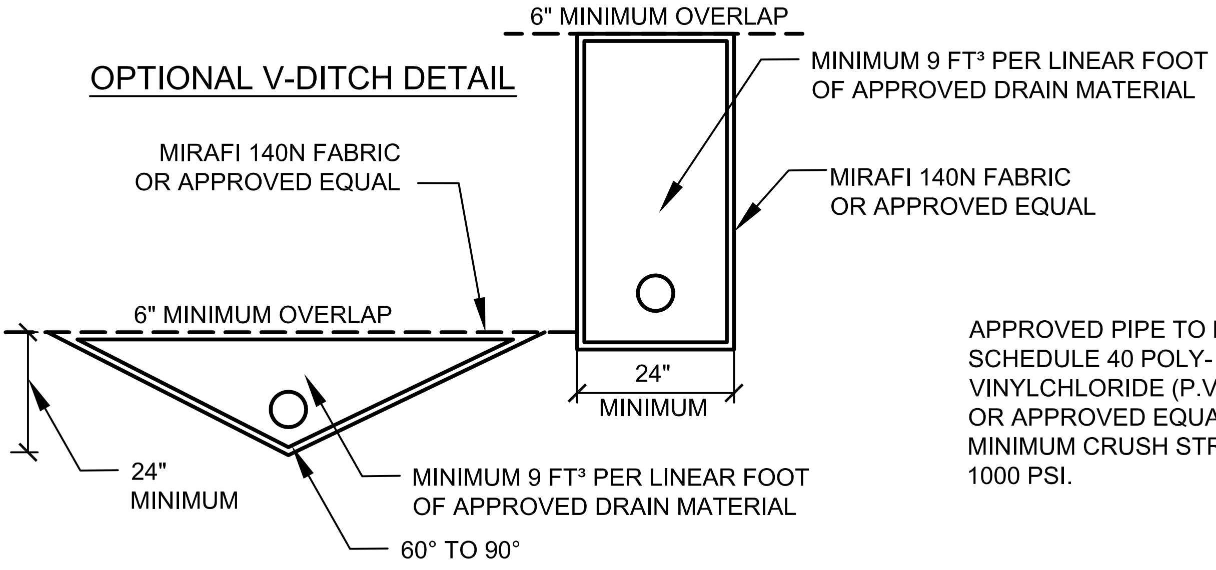
TYPICAL CANYON SUBDRAIN DETAIL

CANYON SUBDRAIN DETAILS



TRENCH DETAILS

OPTIONAL V-DITCH DETAIL



APPROVED PIPE TO BE SCHEDULE 40 POLY-VINYLCHLORIDE (P.V.C.) OR APPROVED EQUAL. MINIMUM CRUSH STRENGTH 1000 PSI.

DRAIN MATERIAL TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUAL:

SIEVE SIZE	PERCENTAGE PASSING
1 ½"	88-100
1"	5-40
¾"	0-17
⅜"	0-7
NO. 200	0-3

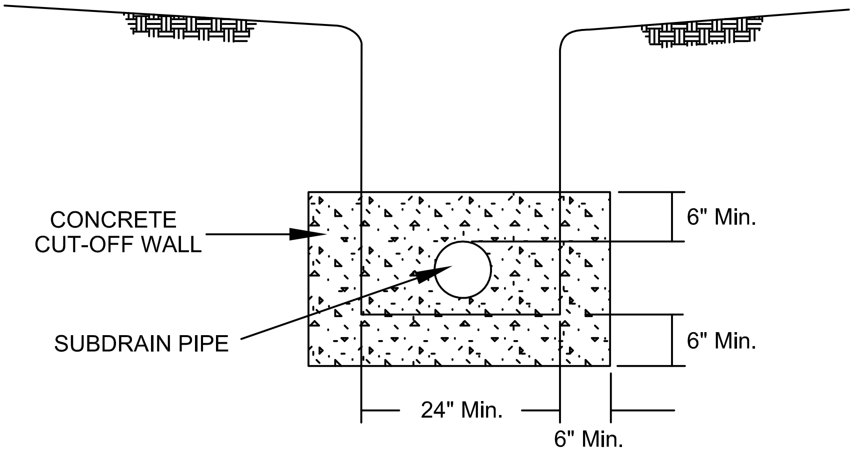
PIPE DIAMETER TO MEET THE FOLLOWING CRITERIA, SUBJECT TO FIELD REVIEW BASED ON ACTUAL GEOTECHNICAL CONDITIONS ENCOUNTERED DURING GRADING

LENGTH OF RUN	PIPE DIAMETER
INITIAL 500'	4"
500' TO 1500'	6"
> 1500'	8"

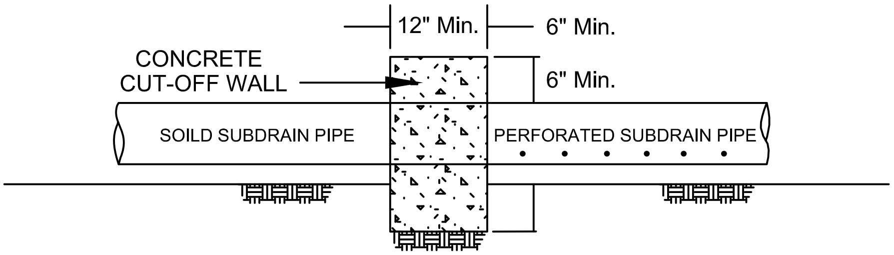
NOT TO SCALE

GEOFABRIC SUBDRAIN

FRONT VIEW



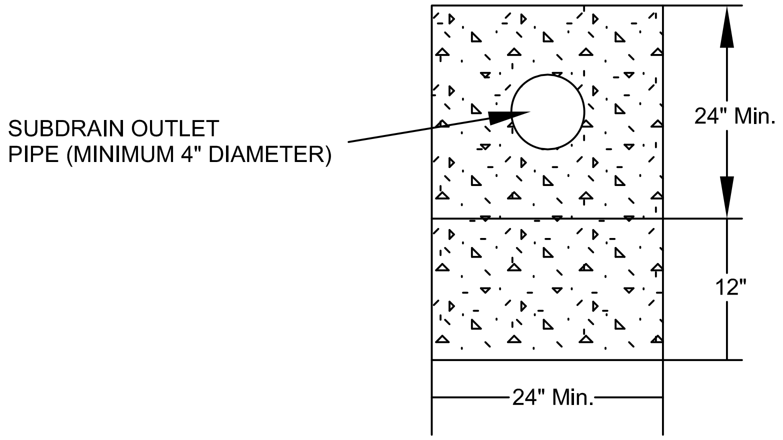
SIDE VIEW



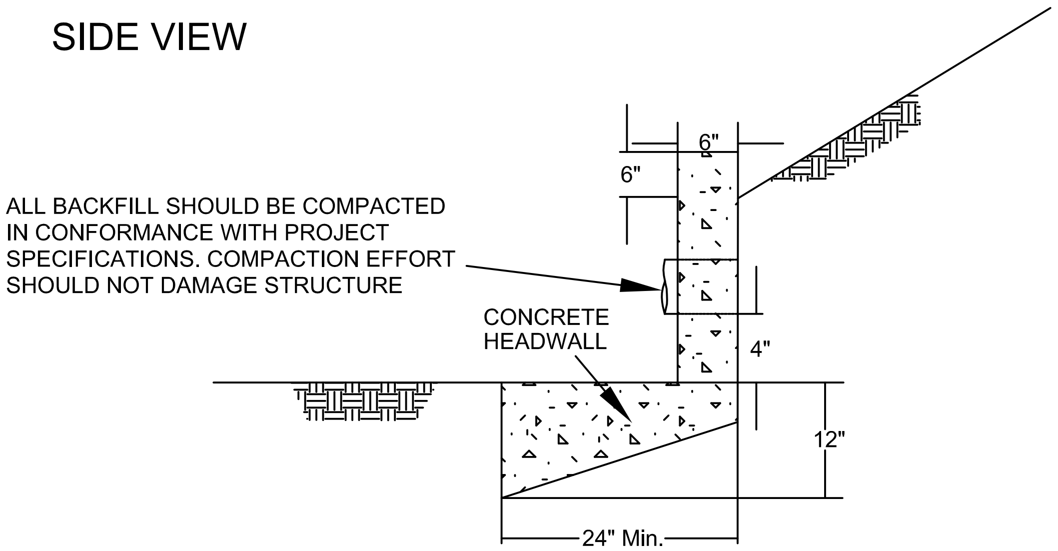
NOT TO SCALE

RECOMMENDED SUBDRAIN CUT-OFF WALL

FRONT VIEW



SIDE VIEW



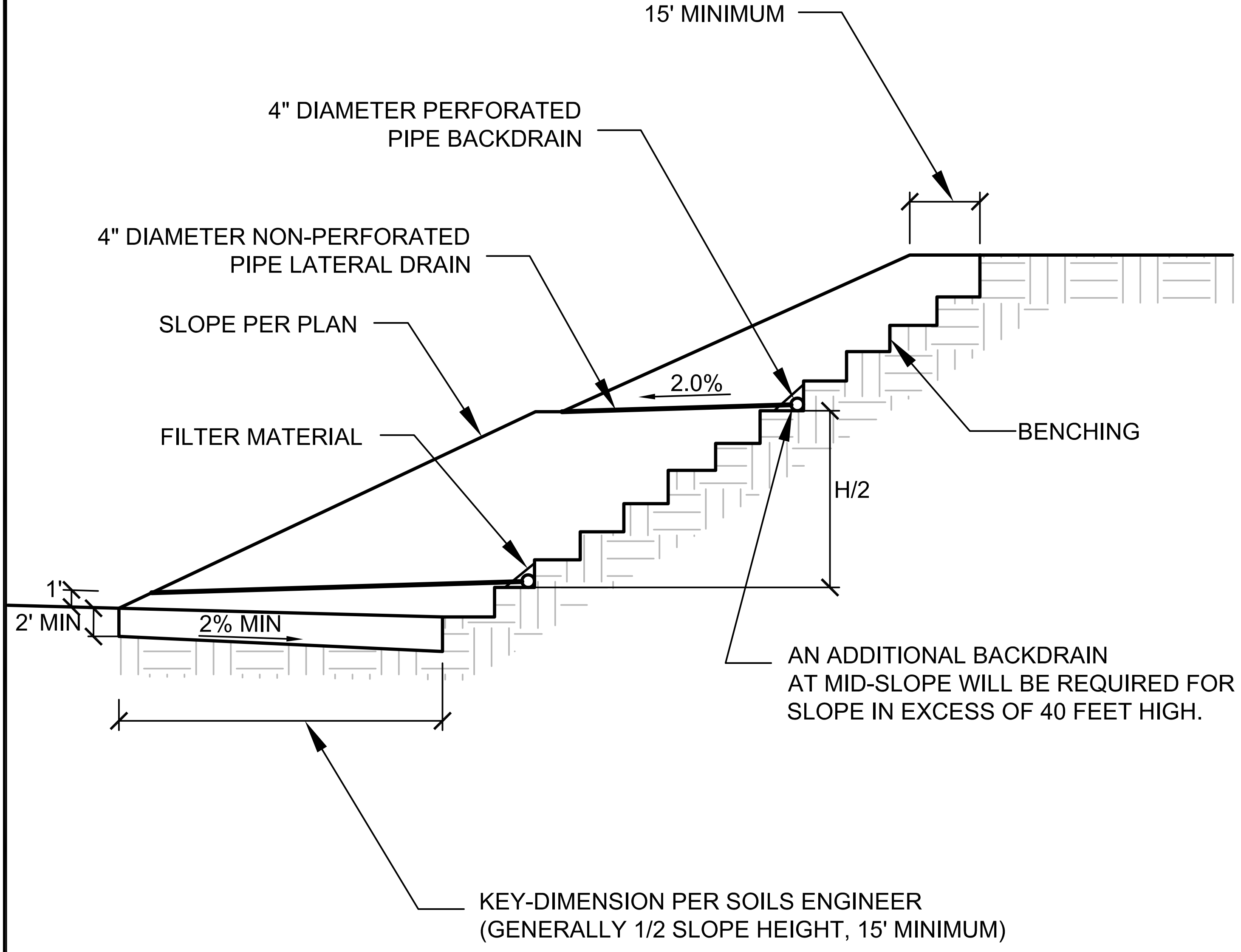
NOTE: HEADWALL SHOULD OUTLET AT TOE OF SLOPE OR INTO CONTROLLED SURFACE DRAINAGE DEVICE

ALL DISCHARGE SHOULD BE CONTROLLED

THIS DETAIL IS A MINIMUM DESIGN AND MAY BE MODIFIED DEPENDING UPON ENCOUNTERED CONDITIONS AND LOCAL REQUIREMENTS

NOT TO SCALE

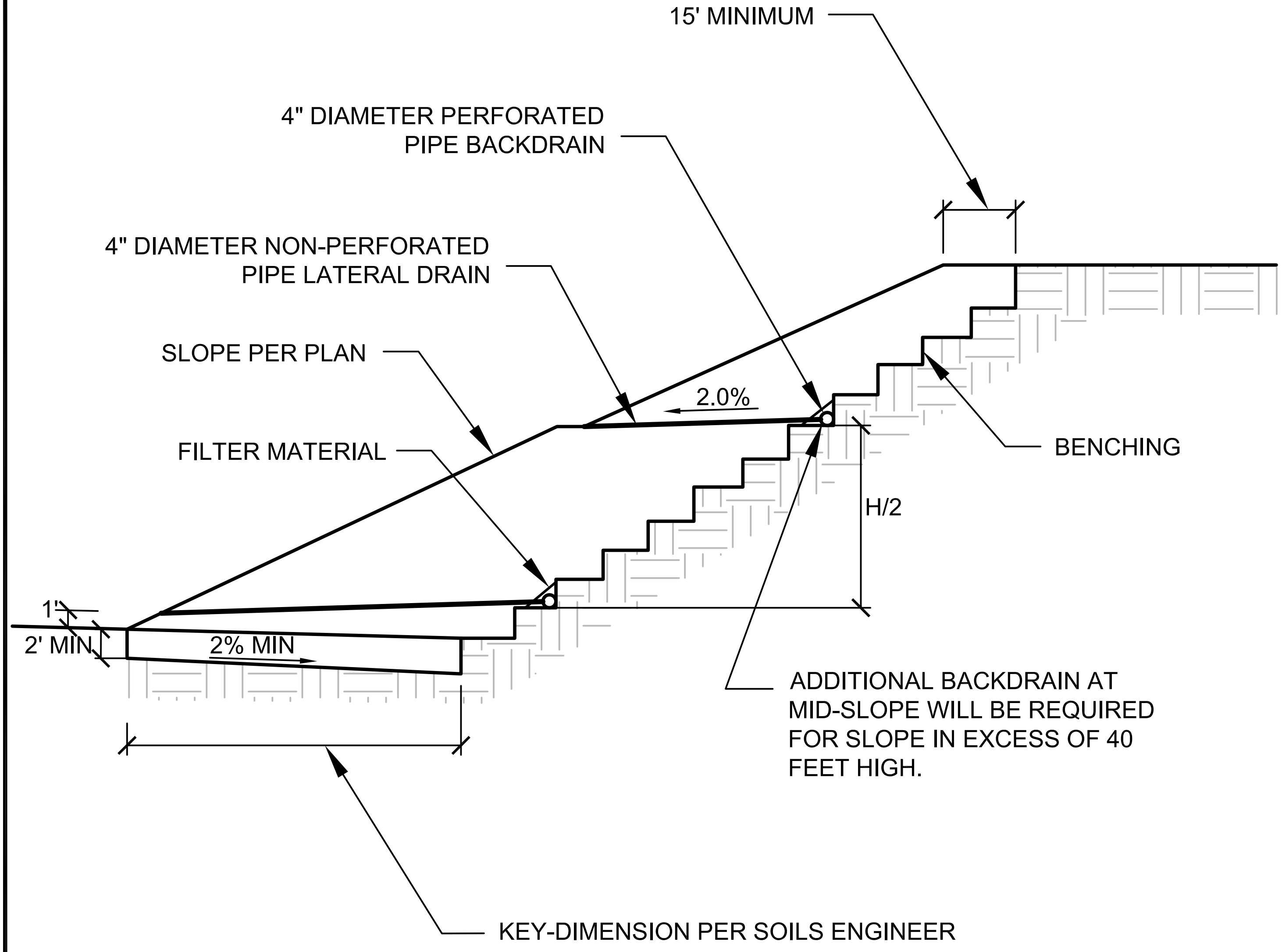
TYPICAL SUBDRAIN OUTLET HEADWALL DETAIL



DIMENSIONS ARE MINIMUM RECOMMENDED

NOT TO SCALE

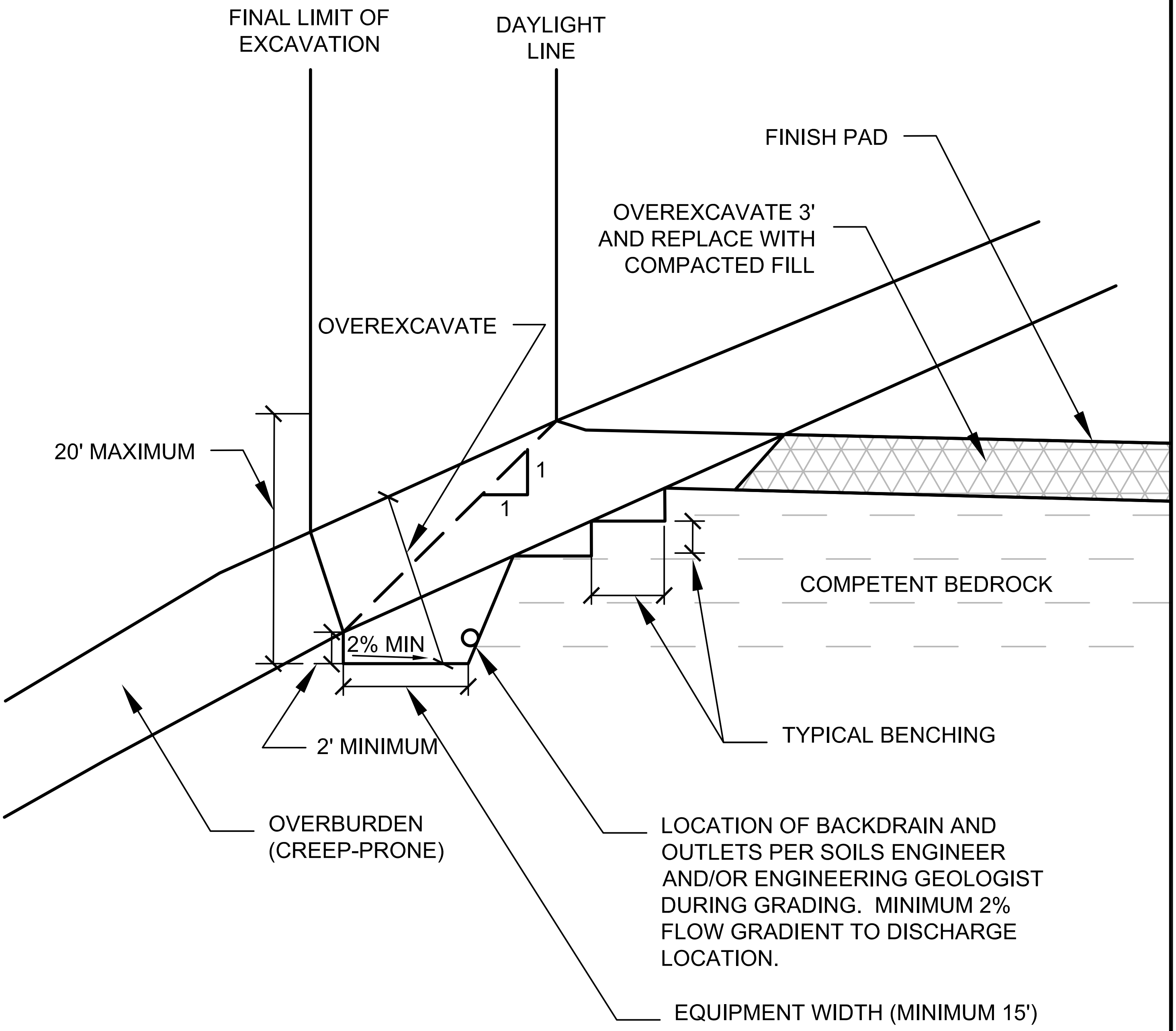
TYPICAL SLOPE STABILIZATION FILL DETAIL



DIMENSIONS ARE MINIMUM RECOMMENDED

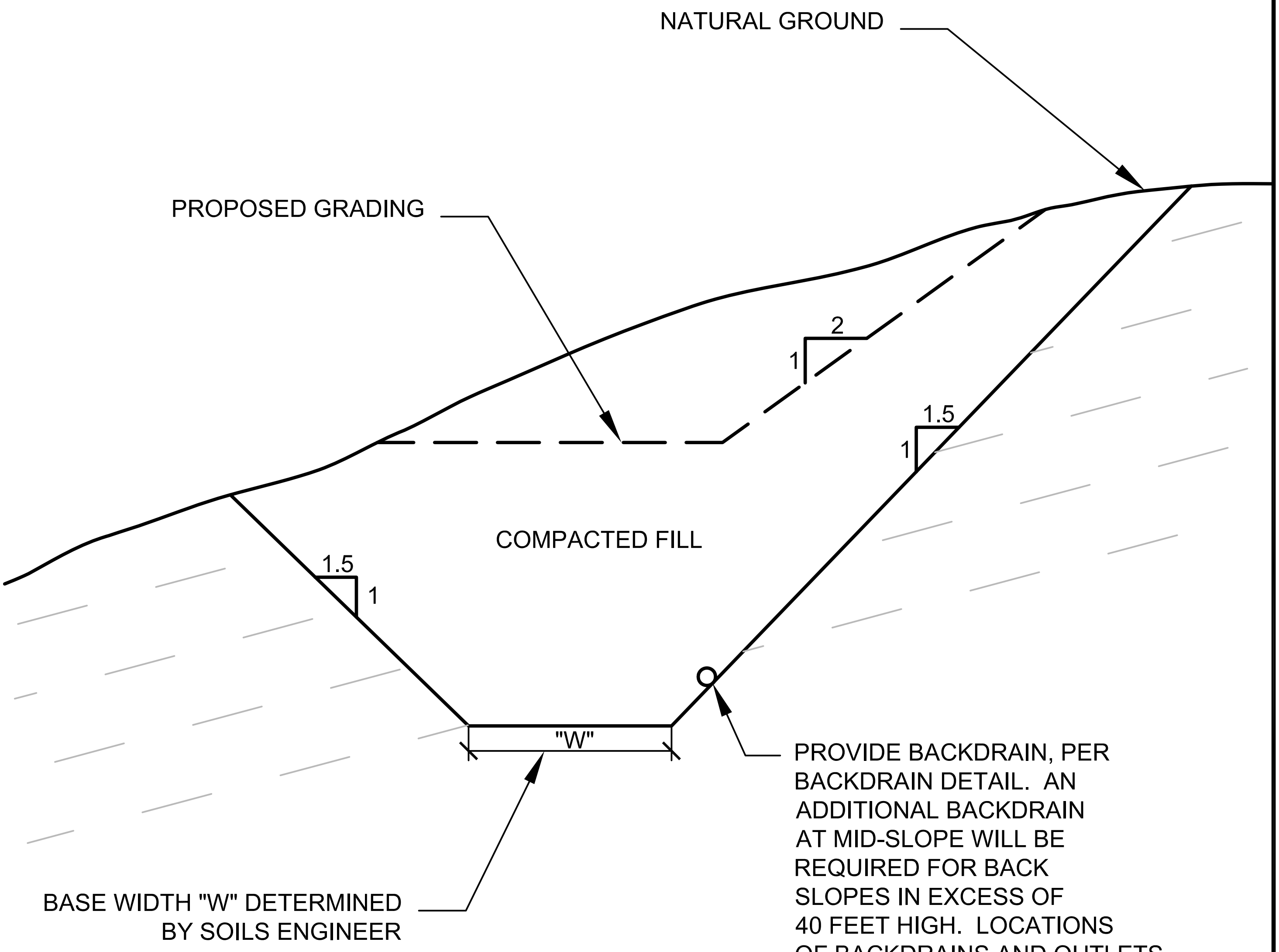
NOT TO SCALE

TYPICAL BUTTRESS FILL DETAIL



NOT TO SCALE

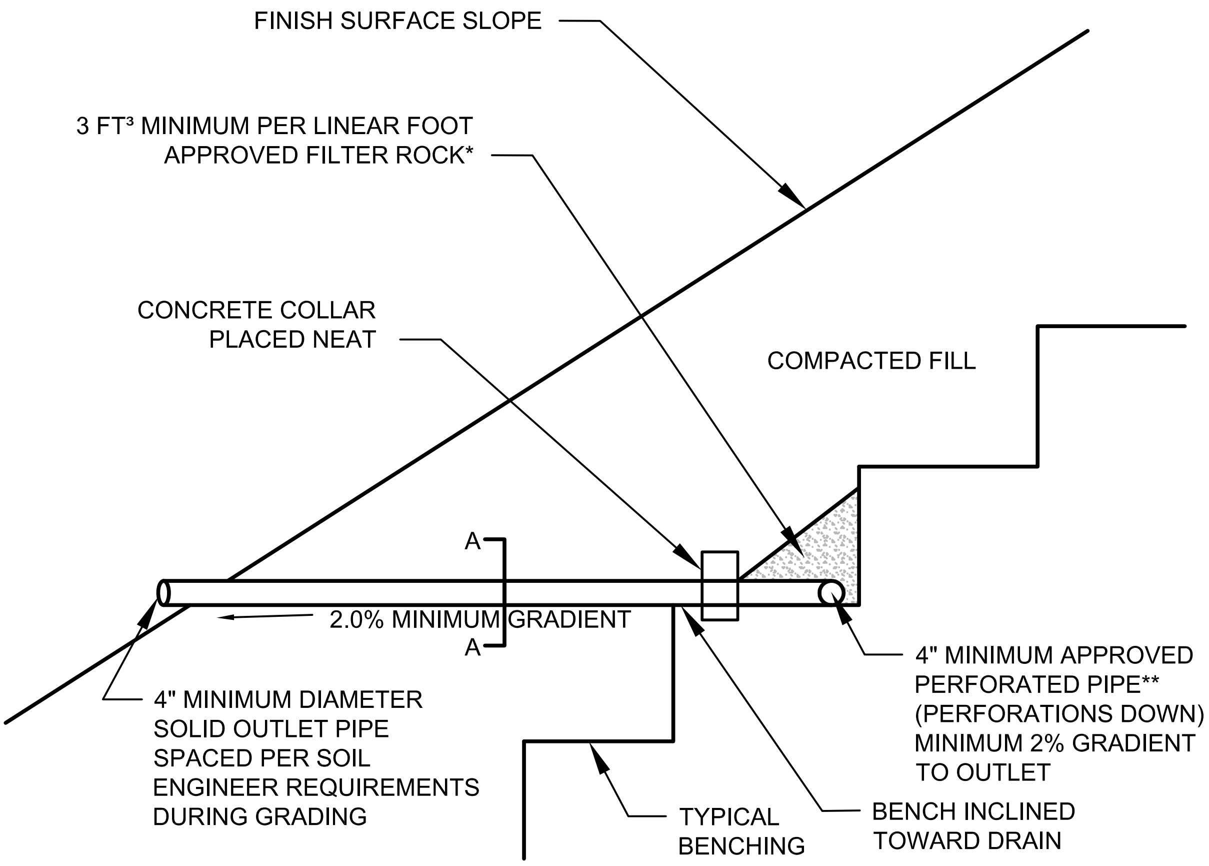
DAYLIGHT SHEAR KEY DETAIL



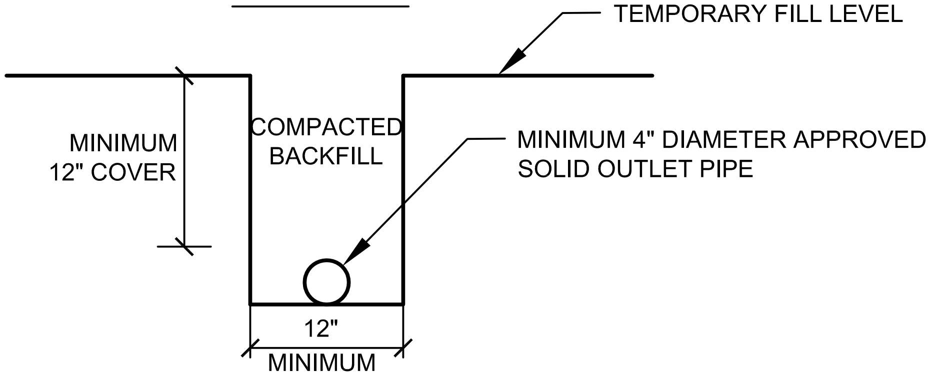
PROVIDE BACKDRAIN, PER BACKDRAIN DETAIL. AN ADDITIONAL BACKDRAIN AT MID-SLOPE WILL BE REQUIRED FOR BACK SLOPES IN EXCESS OF 40 FEET HIGH. LOCATIONS OF BACKDRAINS AND OUTLETS PER SOILS ENGINEER AND/OR ENGINEERING GEOLOGIST DURING GRADING. MINIMUM 2% FLOW GRADIENT TO DISCHARGE LOCATION.

NOT TO SCALE

TYPICAL SHEAR KEY DETAIL



DETAIL A-A



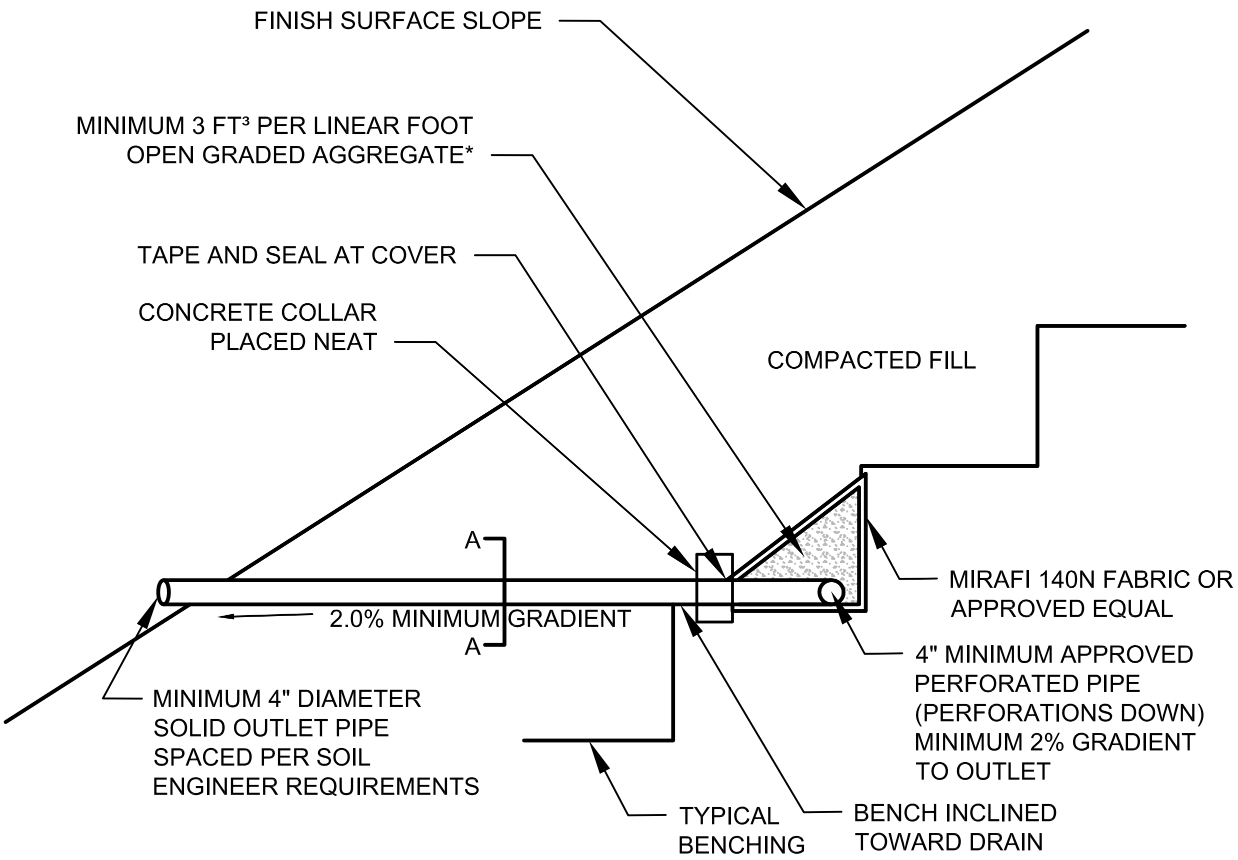
**APPROVED PIPE TYPE:
SCHEDULE 40 POLYVINYL CHLORIDE
(P.V.C.) OR APPROVED EQUAL.
MINIMUM CRUSH STRENGTH 1000 PSI

*FILTER ROCK TO MEET FOLLOWING
SPECIFICATIONS OR APPROVED EQUAL:

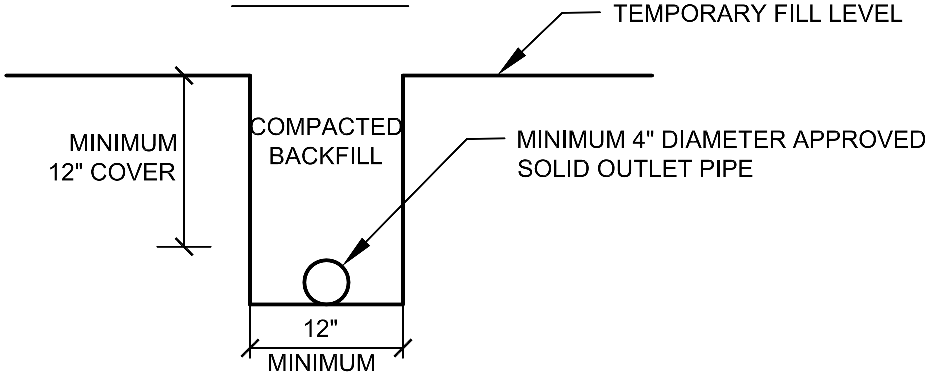
SIEVE SIZE	PERCENTAGE PASSING
1"	100
¾"	90-100
⅜"	40-100
NO. 4	25-40
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

NOT TO SCALE

TYPICAL BACKDRAIN DETAIL



DETAIL A-A

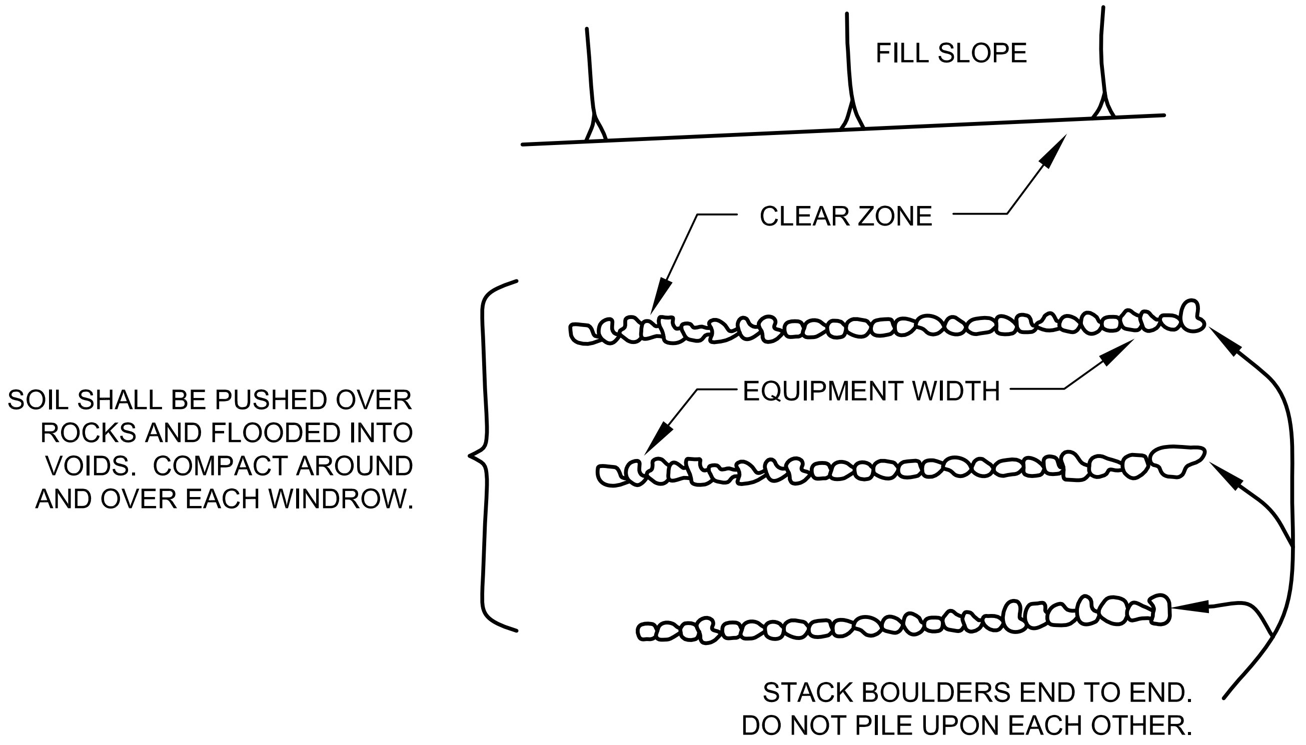


*NOTE: AGGREGATE TO MEET FOLLOWING SPECIFICATIONS OR APPROVED EQUAL:

SIEVE SIZE	PERCENTAGE PASSING
1 ½"	100
1"	5-40
¾"	0-17
⅜"	0-7
NO. 200	0-3

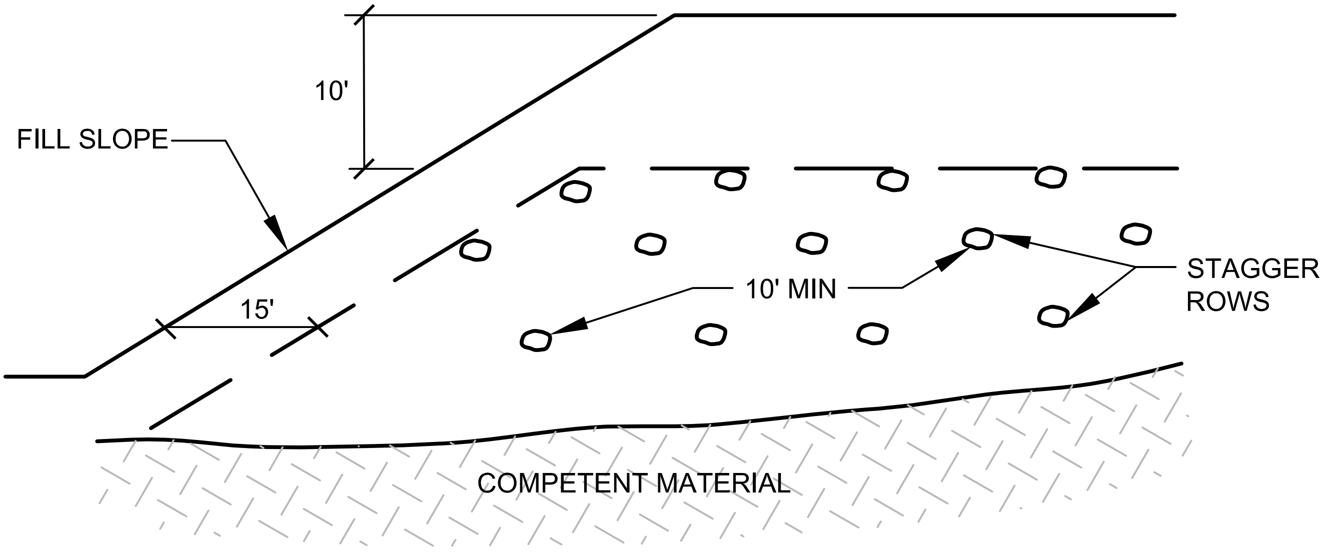
NOT TO SCALE

BACKDRAIN DETAIL (GEOFRABIC)



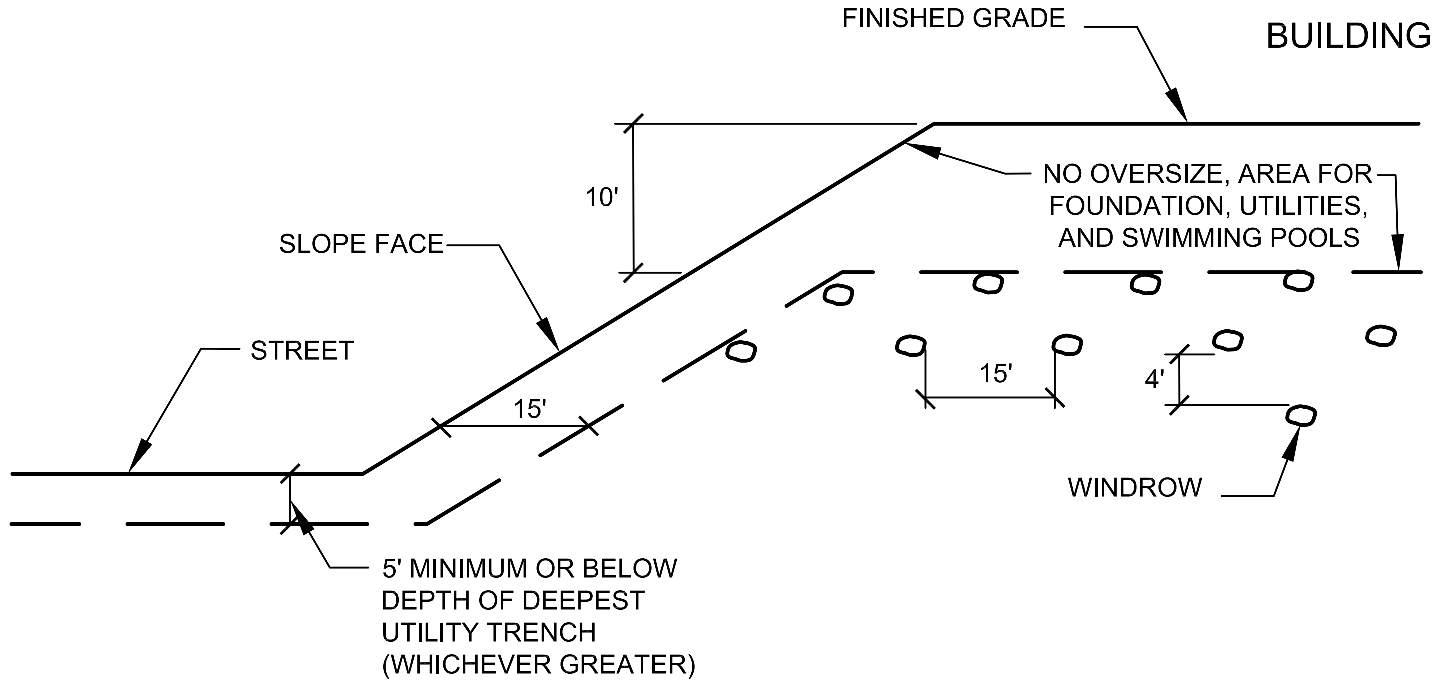
SOIL SHALL BE PUSHED OVER ROCKS AND FLOODED INTO VOIDS. COMPACT AROUND AND OVER EACH WINDROW.

STACK BOULDERS END TO END. DO NOT PILE UPON EACH OTHER.

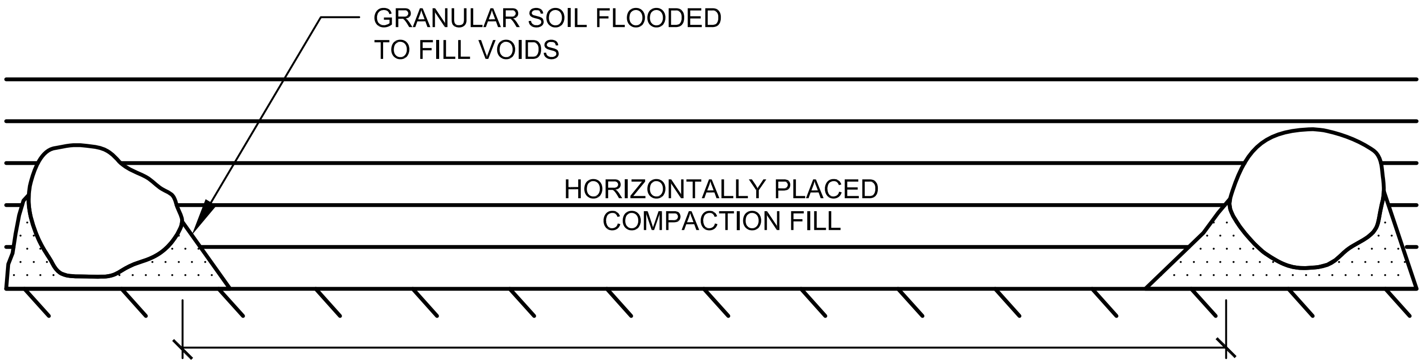


NOT TO SCALE

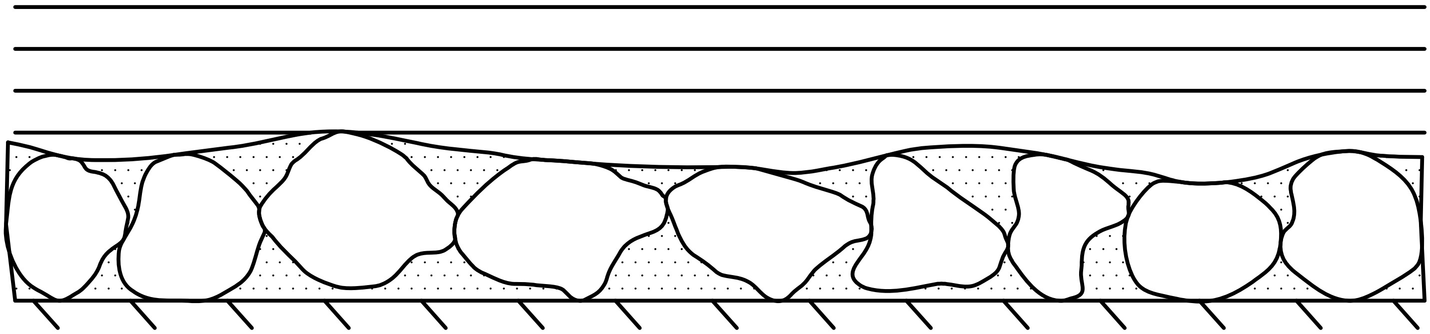
ROCK DISPOSAL DETAIL



TYPICAL WINDROW DETAIL (EDGE VIEW)



PROFILE VIEW

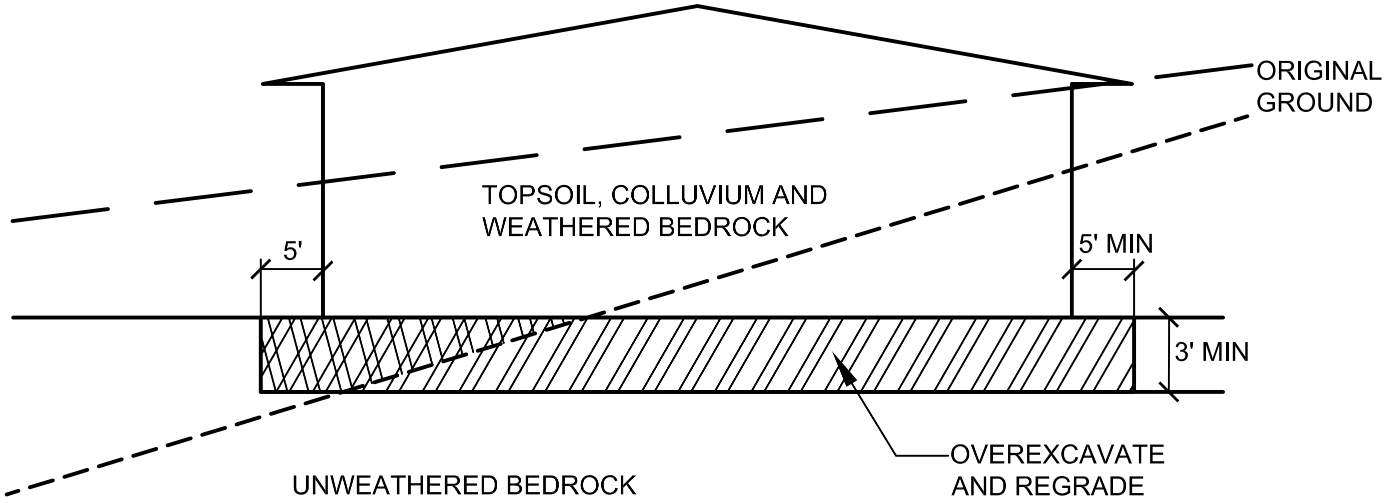


NOT TO SCALE

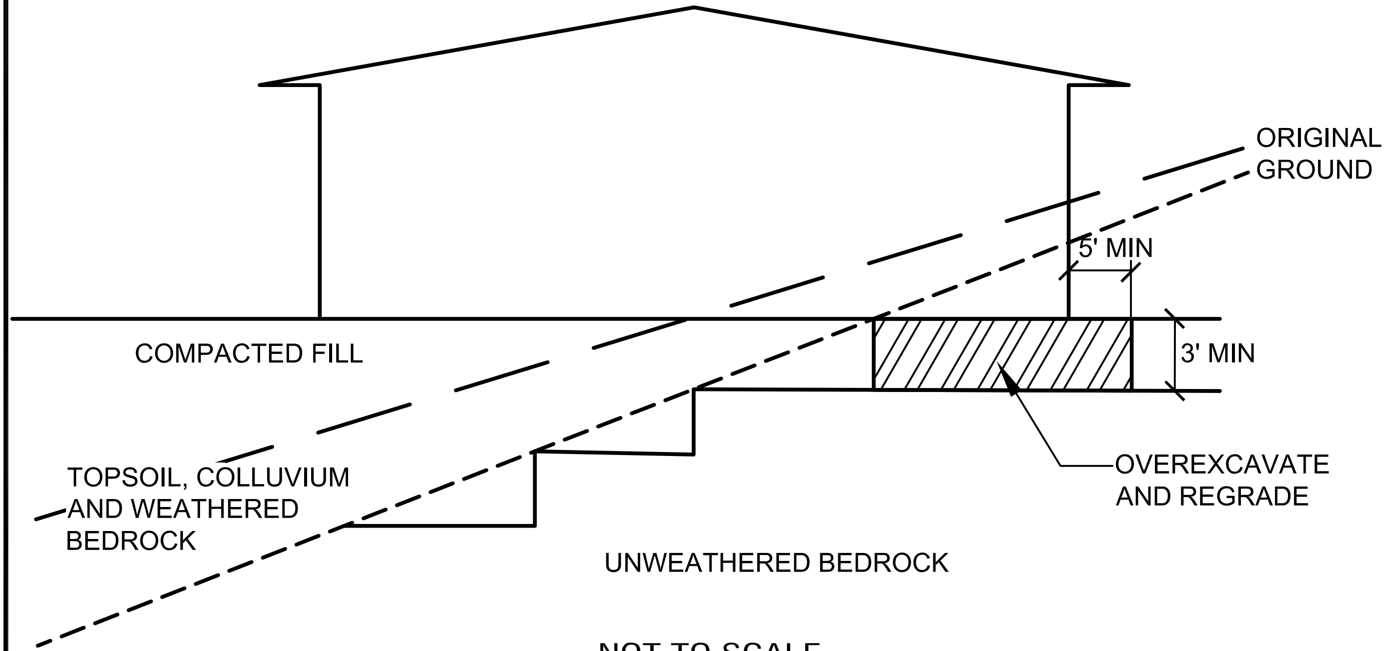
ROCK DISPOSAL DETAIL

GENERAL GRADING RECOMMENDATIONS

CUT LOT



CUT/FILL LOT (TRANSITION)



NOT TO SCALE

TRANSITION LOT DETAIL

APPENDIX E

PERCOLATION TO INFILTRATION CALCULATIONS AND FIELD DATA

TABLE 4.2
PERCOLATION TEST DATA

P-1 **Total Depth:** **43 inches**

Time	Test Interval Time	Test Refill	Water Level Initial/Start	Water Level End/Final	Incremental Water Level Change	Percolation Rate	Percolation Rate
	(minutes)	Depth /Inches	Depth /Inches	Depth /Inches	(inches)	inches/minutes	inches/hour
8:00 AM	Initial	None	34.00	initial	-		
8:30 AM	30	NO	34.00	34.69	0.69	0.02	1.38
9:00 AM	30	NO	34.69	34.75	0.06	0.00	0.13
9:30 AM	30	NO	34.75	34.81	0.06	0.00	0.13
10:00 AM	30	NO	34.81	34.88	0.06	0.00	0.13
10:30 AM	30	33.875	34.88	35.06	0.19	0.01	0.38
11:00 AM	30	NO	33.88	34.06	0.19	0.01	0.38
11:30 AM	30	NO	34.06	34.13	0.06	0.00	0.13
12:00 PM	30	NO	34.13	34.19	0.06	0.00	0.13

P-2 **Total Depth:** **60 inches**

Time	Test Interval Time	Test Refill	Water Level Initial/Start	Water Level End/Final	Incremental Water Level Change	Percolation Rate	Percolation Rate
	(minutes)	Depth /Inches	Depth /Inches	Depth /Inches	(inches)	inches/minutes	inches/hour
8:02 AM	Initial	None	52.00	initial	-		
8:30 AM	30	NO	52.00	52.25	0.25	0.0083333	0.500
9:02 AM	30	NO	52.25	52.88	0.625	0.0208333	1.250
9:32 AM	30	51.75	52.88	53.25	0.375	0.0125	0.750
10:02 AM	30	NO	51.75	52.25	0.5	0.0166667	1.000
10:32 AM	30	NO	52.25	52.75	0.5	0.0166667	1.000
11:02 AM	30	51.875	52.75	53.00	0.25	0.0083333	0.500
11:32 AM	30	NO	51.88	52.13	0.25	0.0083333	0.500
12:02 PM	30	NO	52.13	52.56	0.438	0.015	0.875

Percolation Rate Conversion P-1					Percolation Rate Conversion P-2				
				Inches					Inches
Time Interval,			Δt =	30	Time Interval,			Δt =	30
Final Depth of Water,			D_f =	34.19	Final Depth of Water,			D_f =	52.56
Test Hole Radius,			r =	3	Test Hole Radius,			r =	3
Initial Depth to Water,			D_0 =	34.13	Initial Depth to Water,			D_0 =	52.13
Total Depth of Test Hole,			D_T =	43	Total Depth of Test Hole,			D_T =	60
H_o =	8.875	in			H_o =	7.875	in		
H_f =	8.8125	in			H_f =	7.4375	in		
ΔH = ΔD =	0.0625	in			ΔH = ΔD =	0.4375	in		
H_{avg} =	8.84375	in			H_{avg} =	7.65625	in		
I_t =	0.018	in/hr			I_t =	0.143	in/hr		

TABLE 4.2.1

RESULTS OF PERCOLATION TESTING WITH FACTOR OF SAFETY APPLIED

Test Location	Test Depth		Soil Type*	Percolation Rate (inches per hour)	Infiltration Rate (inches per hour)	Infiltration Rate with FOS of 2 Applied (inches per hour)
	(inches)	Case	(USCS Classification)			
P-1	43	III	Stockpile	0.125	0.018	0.009
P-2	60	III	Qppf	0.875	0.143	0.072

APPENDIX F

I-8 WORKSHEET

Worksheet I-8 : Categorization of Infiltration Feasibility Condition

<table>
<tr><th colspan="2">Categorization of Infiltration Feasibility Condition</th><th colspan="2">Worksheet I-8</th></tr>
<tr><td colspan="4">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?</td></tr>
<tr><td>Criteria</td><td>Screening Question</td><td>Yes</td><td>No</td></tr>
<tr><td>1</td><td>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.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis: The NRCS soils across the site are all Type D soils with very high surface runoff. The site soils are consistent with the NRCS mapped soil types based on site explorations and percolation testing. Four soil types were present in the area of the proposed development, stockpile, Quaternary Previously Placed Fill, Residual Soil, and Tertiary Santiago Formation.
Two percolation tests were completed, with one test in stockpile material and the other in Previously Placed Fill. The calculated infiltration rates (with an applied factor of safety of two) ranged from 0.009 to 0.072 inch 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>2</td><td>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.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis: Due to the minimal permeability of the geologic units encountered at the site, surface water would likely migrate laterally or mound locally. This could result in the infiltrating water discharging onto the adjacent descending slope and migrating off site. After water begins discharging onto the slope face, the material will likely erode resulting in an oversteepened unstable slope that would be susceptible to failure. Long-term soil softening due to water and weathering would also be expected to contribute to increased slope instability.
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
</table>

<table>
<tr><th colspan="4">Worksheet I-8 Page 2 of 4</th></tr>
<tr><td>Criteria</td><td>Screening Question</td><td>Yes</td><td>No</td></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: According to Geotracker, the nearest known "Open" LUST cleanup site is over 3,000 feet away from the site.
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: Surface water is not located within the site vicinity, therefore infiltrating site water is not anticipated to have an impact.
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>No Full</td></tr>
</table>

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by City Engineer to substantiate findings.

<table>
<tr><th colspan="4">Worksheet I-8 Page 3 of 4</th></tr>
<tr><td colspan="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?</td></tr>
<tr><td>Criteria</td><td>Screening Question</td><td>Yes</td><td>No</td></tr>
<tr><td>5</td><td>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.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis: The site soils are generally not anticipated to allow infiltration of at any appreciable rate.
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>6</td><td>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.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis: Due to the minimal permeability of the geologic units encountered at the site, surface water would likely migrate laterally or mound locally. This could result in the infiltrating water discharging onto the adjacent descending slope and migrating off site. After water begins discharging onto the slope face, the material will likely erode resulting in an oversteepened unstable slope that would be susceptible to failure. Long-term soil softening due to water and weathering would also be expected to contribute to increased slope instability.
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>
</table>

<table>
<tr><th colspan="4">Worksheet 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: According to Geotracker, the nearest known "Open" LUST cleanup site is over 3,000 feet away from the site.

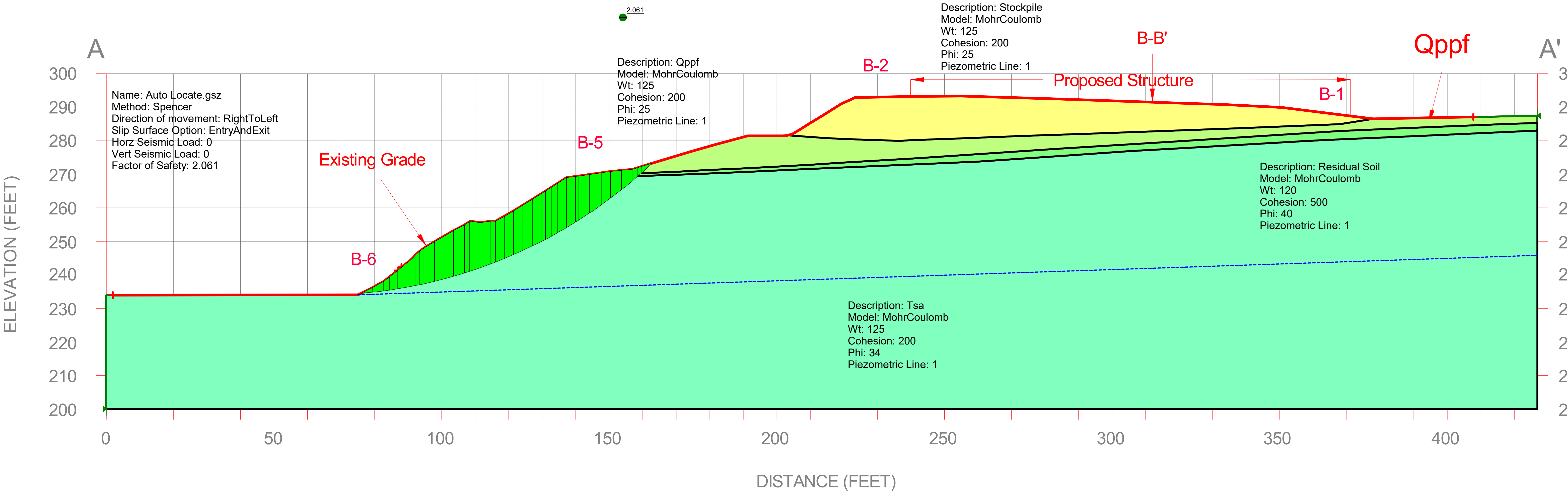
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</td></tr>
<tr><td>8</td><td>Can infiltration be allowed without violating 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: Surface water is not located within the site vicinity, therefore infiltrating site water is not anticipated to have an impact.

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 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration.
If any answer from row 5-8 is no, then infiltration of any volume is considered to be infeasible within the drainage area. The feasibility screening category is No Infiltration.</td><td>No Inf.</td></tr>
</table>

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by City Engineer to substantiate findings

APPENDIX G

SLOPE STABILITY ANALYSIS



Name: Auto Locate.gsz
Method: Spencer
Direction of movement: RightToLeft
Slip Surface Option: EntryAndExit
Horz Seismic Load: 0
Vert Seismic Load: 0
Factor of Safety: 2.061

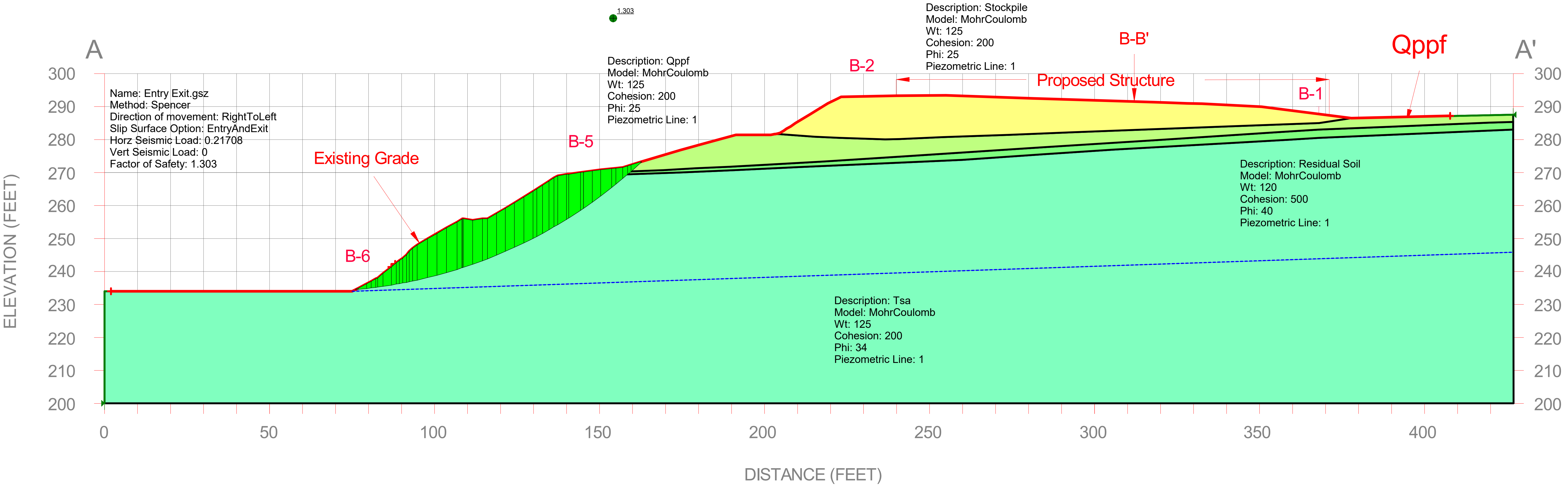
Description: Qppf
Model: MohrCoulomb
Wt: 125
Cohesion: 200
Phi: 25
Piezometric Line: 1

Description: Stockpile
Model: MohrCoulomb
Wt: 125
Cohesion: 200
Phi: 25
Piezometric Line: 1

Description: Residual Soil
Model: MohrCoulomb
Wt: 120
Cohesion: 500
Phi: 40
Piezometric Line: 1

Description: Tsa
Model: MohrCoulomb
Wt: 125
Cohesion: 200
Phi: 34
Piezometric Line: 1

CROSS SECTION A-A'



CROSS SECTION A-A'

3131 Camino Del Rio North., Suite 1080 San Diego, CA 92108 619.521.8500 kpff.com

MEMO

DATE: 2020-11-19

TO: CTE, Inc

FROM: KPFF

RE: Tri-City Medical Center - Proposed Psychiatric Health Facility

This memo is generated in response to a conference call held on 10-18-2020. As mentioned during this call changes to the proposed design of the facility have occurred that require review/coordination with CTE and were applicable and updated addendum letter to the Geotechnical Report dated January 6, 2020. This memo serves to highlight those changes.

- **Foundation Deepening with CLSM:** The project anticipates existing soils will be over-excavated and recompacted. Per the current Geotechnical Report, section 6.7.1, it notes that foundations should embed a minimum of 30-inches below lowest adjacent subgrade elevation and also that it is anticipated that all footings will be founded entirely in properly recompacted fill materials. It also notes that CLSM may be required to extend the foundation into deeper soils. Please confirm intent and use of CLSM is as anticipated and whether or not the "subgrade" is meant to be native soils. Structural detail 12/S0500 has been provided as part of the foundation design sheets for reference and confirmation our understanding.

- **Foundation Setback:** The project proposed building includes deepened foundations at the northwest corner, southwest and southeast corners to accommodate a 10' foundation setback. The current Geotech report states 12' is required but was reduced as part of follow-up coordination after the report was publish. Structural detail 15/S0500 has been provided as part of the foundation design sheets for reference and confirmation.

- **Existing Slopes and Fill Conditions:** The project boundary has been extended south to allow for the embankment of spoil materials. This results in work outside of the project limit in the current geotechnical report. Slopes are currently limited to 2:1 max in this area. Please review and provide comment or confirmation. Please also note if current criteria can be increased.

- **Segmental Block Retaining Wall:** A wall manufacturer is anticipated to provide a gravity block wall in multiple locations of the site. We expect they will require design level inputs for global and local stability such as: bearing capacities, the friction slope angle of onsite materials and anticipated groundwater conditions. Please provide.

- **Slope Benching and Drainage:** Current slopes are indicated as being greater than 5:1 and require keys and benches for embankments. Please confirm intent to drain and slope these areas has been met. We also note the existing slope at the newly proposed southern area is roughly at or below 5:1 up to the property line and the slope increases drastically outside of the property boundary.

CTE INC.
Construction Testing & Engineering, Inc.
Inspection | Testing | Geotechnical | Environmental & Construction Engineering | Civil Engineering | Surveying

August 19, 2020 CTE Job No. 10-15341G

County of San Diego
Attention: Mr. David Dobson
10920 Via Frontera #300
San Diego, CA 92127 Via Email: David.Dobson@ sdcounty.ca.gov

Subject: Response to RFI and Meeting Note Questions
Proposed Tri-City Medical Center Improvements
4002 Vista Way
Oceanside, California

Reference: Geotechnical Investigation
Proposed Psychiatric Health Facility
4002 Vista Way, Oceanside, California
Prepared by CTE, Dated January 6, 2020
CTE Job No.: 10-15341G

Grading and Drainage Plans for:
Tri-City Medical Center Behavioral Health Facility
Prepared by kppf
Project No. 006240.00

Mr. Dobson:

As requested, Construction Testing & Engineering, Inc. (CTE) provides the following responses to the Turner Engineering Group technical review and meeting note questions.

TURNER ENGINEERING GROUP QUESTIONS:

Question No. 47: Study a reduction in fill depth from 8'-0" at building footprint to 3'-0" below footings. (page 9/22)

Response to Question No. 47: The overexcavation recommendation of 1/2 maximum depth of fill beneath the proposed structure may be revised to 1/3 maximum depth of fill beneath the proposed structure to reduce overexcavation of the cut portion.

Question No. 48: Recommend that an additional set of slope stability evaluations be performed with the final grading configuration of the site. (page 6/22)

Response to Question No. 48: See Figure 2 and the attached slope stability analysis.

Question No. 49: Although the residual soil layer appears to be relatively thin, TEG requests a clarification if the "dense native underlying soils" described on page 14 refer to the residual soils, and are therefore not anticipated to be subject to significant settlement when subjected to additional fill and building loads. (pg 7/22)

Response to Question No. 49: It is possible for portions of the residual soil to be medium dense to dense or very stiff and, therefore, not require removal during grading. This determination should be made by a CTE geotechnical representative based on actual exposed conditions during grading. If the residual soil is found to have adequate density, as evaluated by probing or other methods, and those approved portions of the residual soil are left in place, significant settlement of this material is not anticipated.

Question No. 53: TEG concurs with the recommendations for fill, however, TEG suggests an upper limit be provided for moisture content: TEG suggests 4 percent above optimum moisture content for general fill and 5 percent above optimum moisture content for clayey fill. (pg 10/22)

Response to Question No. 53: The suggested upper limits appear to be reasonable.

Question No. 50: Although the residual soil layer appears to be relatively thin, TEG requests a clarification if the "dense native underlying soils" described on page 14 refer to the residual soils, and are therefore not anticipated to be subject to significant settlement when subjected to additional fill and building loads. (pg 7/22).

Response to Question No. 50: See response to question 49.

Question No. 51: Recommend an increase of temporary slope heights from 10 to 20 feet. (page 9/22)

Response to Question No. 51: The temporary slope heights may be increased from 10 to 20 feet.

Question No. 52: Recommend that the slab thicknesses be decreased from 5.0" to 4.0" thick.

Response to Question No. 52: Concrete slabs should be designed based on the anticipated loading, but CTE has recommended a slab minimum thickness of 5.0 inches based on the anticipated site conditions and in order to help reduce cracking. Given proper subgrade preparation, proper curing techniques, adequate installation observation, and properly designed reinforcement, thinner concrete slab-on-grade sections may perform adequately for foot-traffic only areas. Although reduced slab-on-grade sections may perform adequately, some additional random and generally non-structural cracking of the slab-on-grade could occur due to the nominal slab thickness. Slabs subjected to heavier loads such as storage/racking systems and/or traffic will require thicker slab sections and increased reinforcement, which should be provided by the structural designer.

Question No. 54: Recommend a revision of Risk Category from III to II resulting in a reduction of seismic forces by 33% and potential elimination of some moment frames. (page 11/22)

Response to Question No. 54: This item should be addressed by the project structural designer of record.

MEETING NOTE QUESTIONS:

Question a. Existing Fill

i: Geotechnical Engineer to confirm that existing fill (uncertified stockpile fill) is acceptable for use and updated report is required.

Response:

i: The stockpile fill material is considered to be suitable for reuse as compacted fill for the proposed project, subject to approval by the geotechnical representative during grading.

Question b. Slope Stability Evaluations

i: Geotechnical Engineer to confirm and provide recommendation for the following TEG statement: 1. TEG suggests that an additional set of slope stability evaluations be performed with the final grading configuration at the site. TEG suggests that bedding be included within the evaluation unless the rock is found to be massive throughout the mass. The potential impact of a fault on slope stability should also be included unless the fault is known to not represent a plane of weakness and/or is shown to not extend onto the site. In addition, TEG suggests that the existing swale(s) that will be left in place be evaluated for continued suitability, and repairs (if necessary) and a maintenance program be recommended for existing or new swales on the slopes.

ii. For the purpose of slope stability evaluation, Geotechnical Engineer to provide a discussion of bedding in the Santiago Formation.

iii. Although the residual soil layer appears to be relatively thin, Geotechnical Engineer to provide clarification if the “dense native underlying soils” described on page 14 refer to the residual soils, and are therefore not anticipated to be subject to significant settlement when subjected to additional fill and building loads.

Response:

i: Requested slope stability analysis is attached. Based on observation of the geologic units during the subsurface investigation, the units were generally found to be massive and were modeled in that manner for the slope stability analysis. In addition, no known faults are mapped at, or project toward, the proposed improvement area and evidence of faulting was not observed during the investigation.

ii. The Santiago Formation observed throughout the proposed improvement area was generally found to be massive.

iii. See response to question 49 above.

Question c. Foundation/Floor Slab Recommendations:

i. Slab thickness possible reduction: Geotechnical Engineer and Structural Engineers to provide recommendations on slab thickness.

ii. Storm Water Facility: 1. TEG notes that the infiltration rates into the bedrock are very low and being in proximity to a slope with potential surface instability, TEG recommends that the storm water facilities used on site do not allow infiltration of storm water into the soil/rock underlying the site. Geotechnical and Civil Engineers to review and provide recommendation.

Response:

i. See response to question 52.

ii. The TEG recommendation to avoid infiltration on or adjacent to site slopes is consistent with CTE's findings presented on the I-8 worksheet submitted with the referenced investigation report that indicates the site is not suitable for infiltration. CTE recommends that all retention basins be lined with an impermeable liner to eliminate the possibility of infiltrating water impacting site slopes.

Question d. Grading:

i. With roughly a 16-foot maximum depth of fill, the geotechnical recommendations would require a minimum 8 foot fill depth throughout the building footprint in order to meet the requirements that the minimum fill depth be at least one-half of the maximum fill depth. In the opinion of TEG, although the requirement for achieving at least 2 feet of fill beneath footings should be met, nevertheless, it appears excessive to require 8 feet of fill. As an alternative, it is suggested a minimum fill depth be increased, perhaps to 3 feet below footings, which would be a lesser depth of excavation. Geotechnical and Structural Engineers to review and provide recommendation.

ii. TEG concurs with the recommendations for slope inclination. However, TEG suggests that it might be necessary for a temporary slope with a height greater than 10 feet be constructed; therefore. TEG suggests that the height of slopes for which temporary slope recommendations are provided be increased from 10 to 20 feet. Geotechnical Engineer to review and provide recommendation.

Response:

i. See response to question 41.

ii. See response to question 51.

We appreciate the opportunity to be of service on this project. Should you have any questions or need further information please do not hesitate to contact this office.

Respectfully submitted,
CONSTRUCTION TESTING & ENGINEERING, INC.

DANIEL T. MATH
No.2665
EXP.12/31/20

Dan T. Math, GE #2665
Principal Engineer

AARON J. BEEBY
No.2603
EXP.3/31/22

Aaron J. Beeby, CEG #2603
Senior Geologist

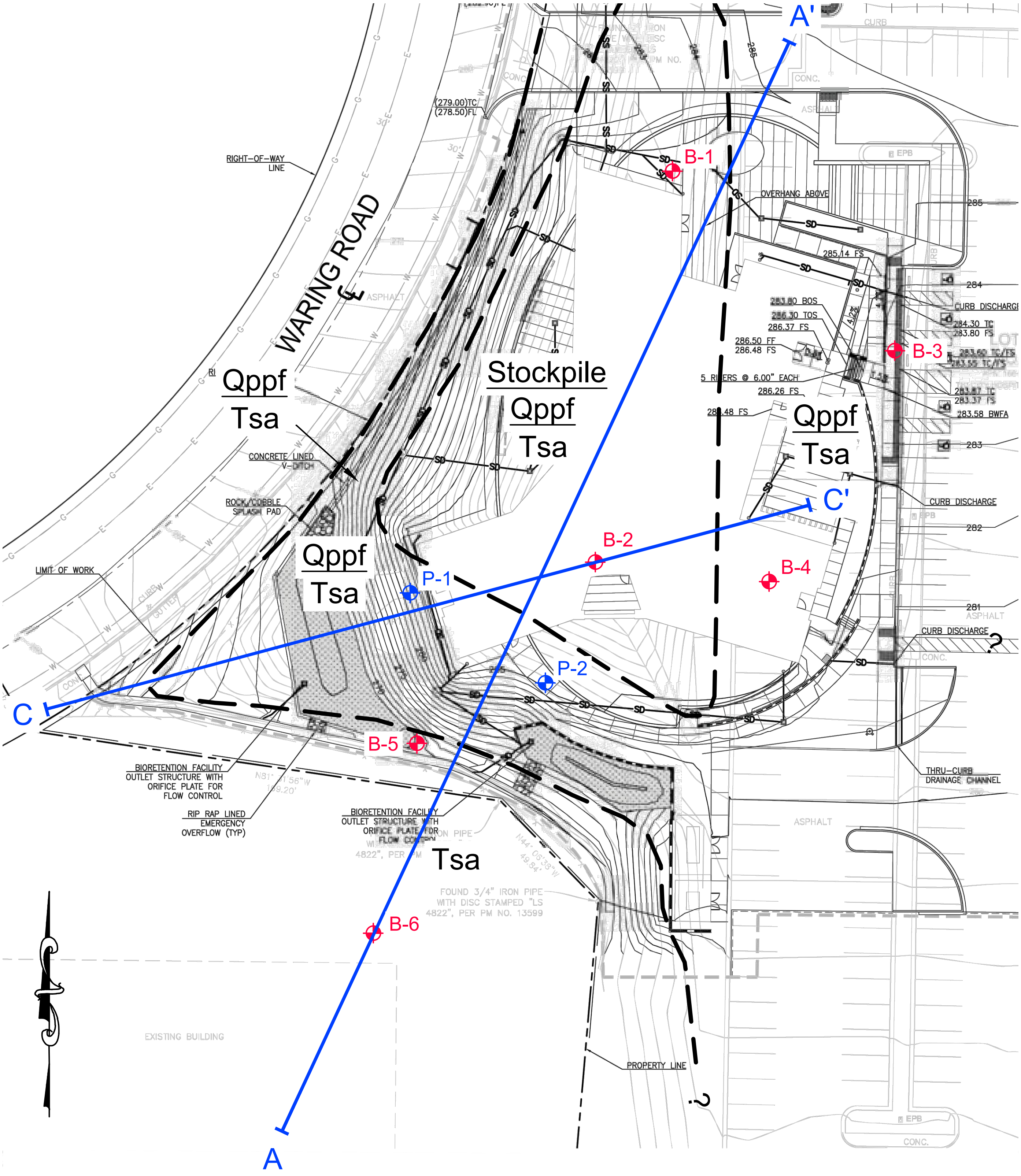
AJB/JFL/DTM:ach

FIGURE

Figure 2	Exploration Location Map
Figure 2A	Cross Sections A-A’ and C-C’

ATTACHMENT:

Attachment 1	Slope Stability Analysis for Cross Section A-A’
Attachment 2	Slope Stability Analysis for Cross Section C-C’



50' 0 25' 50'

LEGEND

B-6 ⊕	Approximate Boring Location
P-2 ⊕	Approximte Perc Test Location
Qppf Tsa	Quaternary Previously Placed Fill over Tertiary Santiago Formation
— — —	Approximate Geologic Contact
● ● ●	Approximate Buried Geologic Contact
C⊢⊣C'	Cross Section

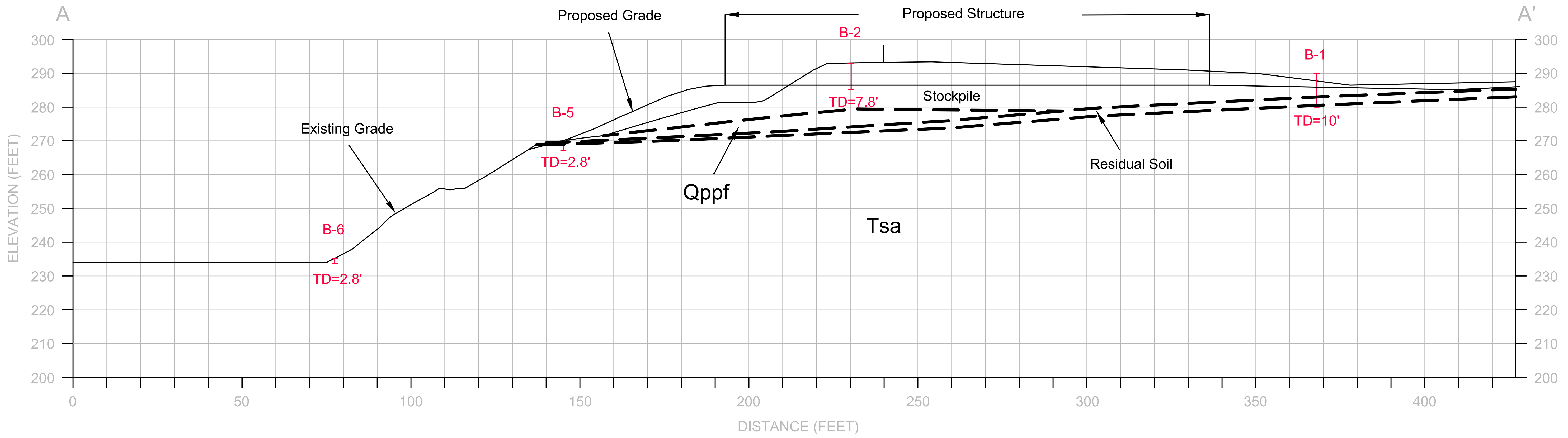
CTE INC. Construction Testing & Engineering, Inc.
1441 Montiel Rd Ste 115, Escondido, CA 92026 Ph (760) 746-4955

GEOLOGIC/EXPLORATION LOCATION MAP

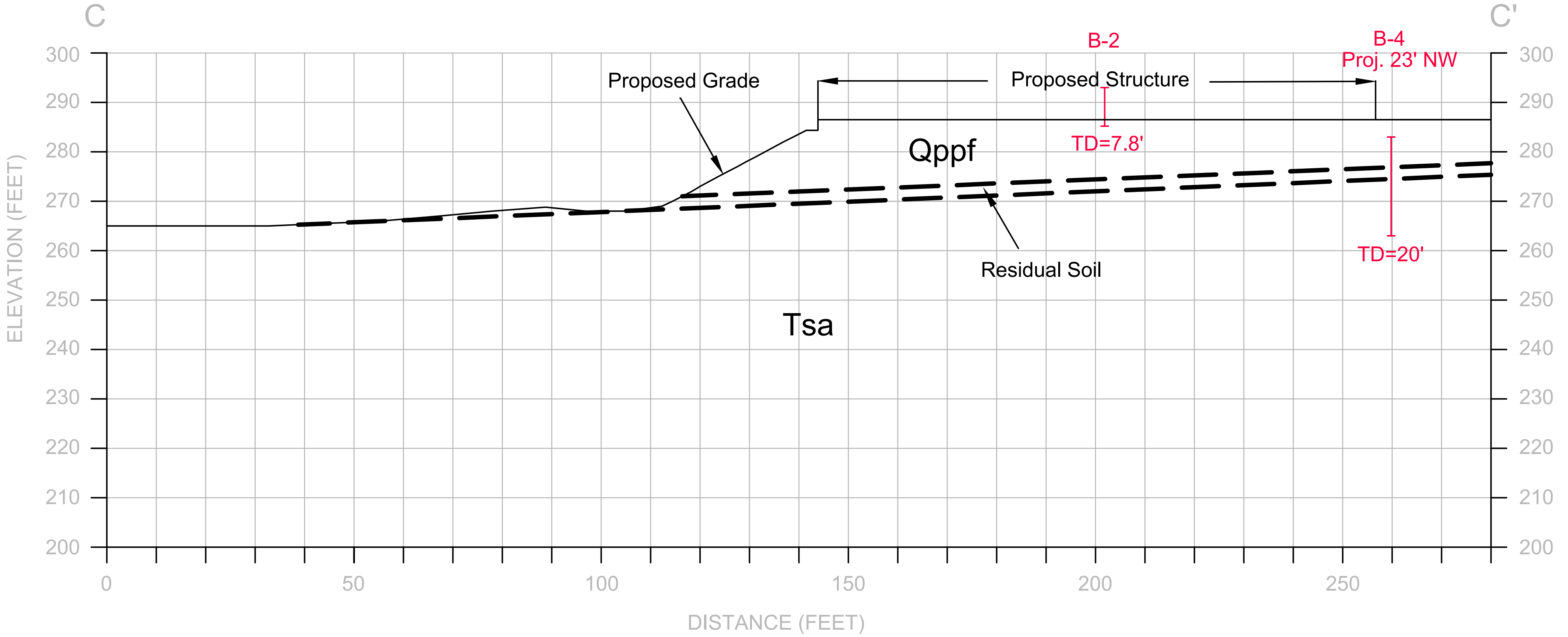
PROPOSED TCMC PSYCHIATRIC HEALTH FACILITY
4002 VISTA WAY
OCEANSIDE, CALIFORNIA

SCALE: 1"=50'	DATE: 8/20
CTE JOB NO.: 10-15341G	FIGURE: 2

S:\Projects\10-15341G (Tri City)\Response to Comments\Figure 2.dwg



CROSS SECTION A-A'



CROSS SECTION C-C'

LEGEND

Qppf	QUATERNARY PREVIOUSLY PLACED FILL
Tsa	TERTIARY SANTIAGO FORMATION
– – –	APPROXIMATE GEOLOGIC CONTACT

S:\Projects\10-15341G (Tri City)\Response to Comments\Figure 2A.dwg

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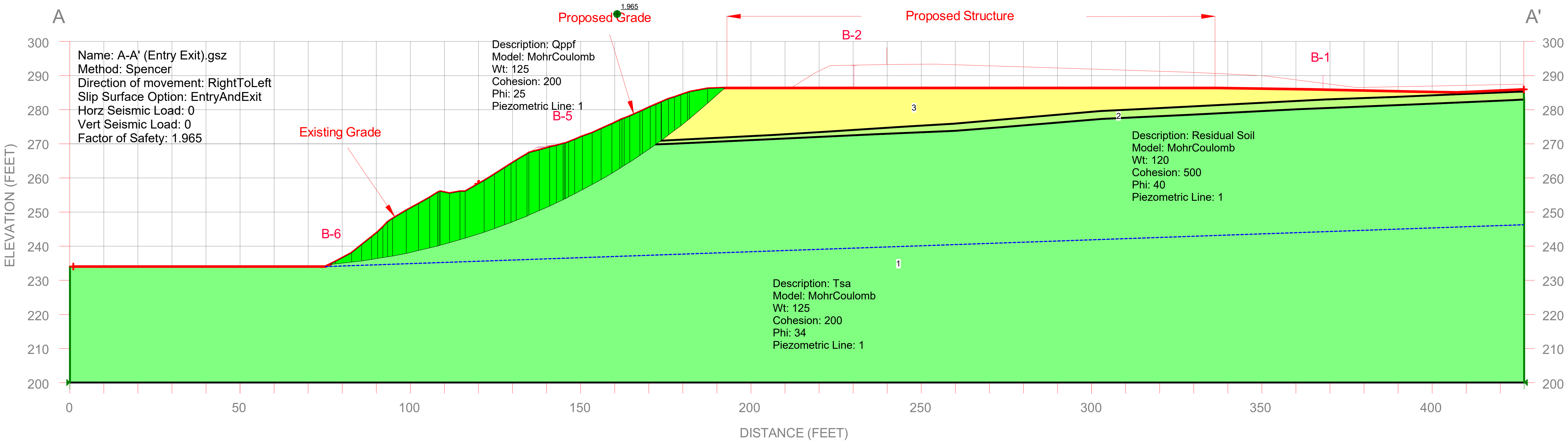
CROSS SECTIONS A-A' and C-C'

PROPOSED TCMC PSYCHIATRIC HEALTH CENTER
4002 VISTA WAY
OCEANSIDE, CALIFORNIA

CTE JOB NO: 10-15341G
SCALE: 1" = 30'
DATE: 8/20
FIGURE: 2A

ATTACHMENT 1

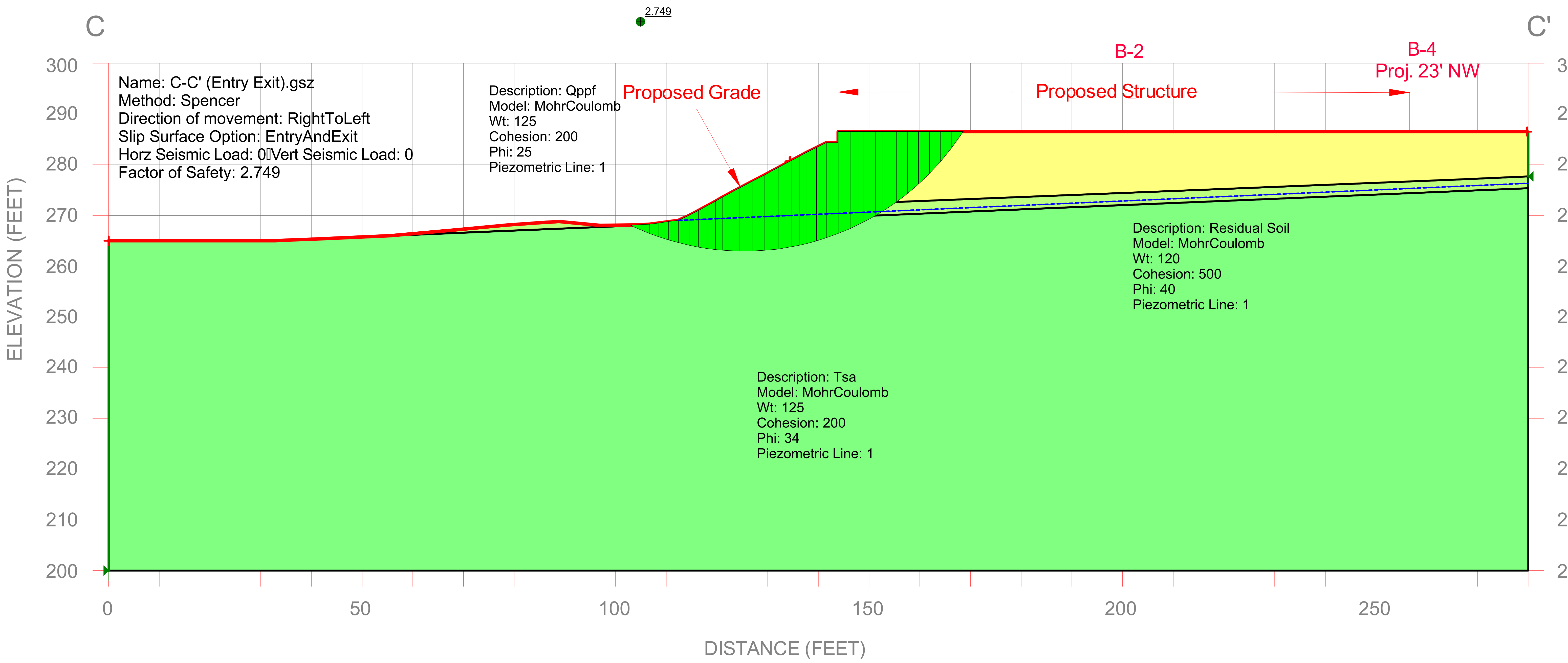
Slope Stability Analysis for Cross Section A-A’



CROSS SECTION A-A'

ATTACHMENT 2

Slope Stability Analysis for Cross Section C-C’



CROSS SECTION C-C'

CTE INC.
Construction Testing & Engineering, Inc.
Inspection | Testing | Geotechnical | Environmental & Construction Engineering | Civil Engineering | Surveying

November 20, 2020 CTE Job No. 10-15341G

County of San Diego
Attention: Mr. David Dobson
10920 Via Frontera #300
San Diego, California 92127
Telephone: (858) 694-3926 Via Email: David.Dobson@sdcounty.gov

Subject: Update to Geotechnical Investigation and Plan Review
Proposed Psychiatric Health Facility
4002 Vista Way
Oceanside, California

References: At end of document

Mr. Dobson:

As requested, Construction Testing & Engineering, Inc. (CTE) provides the following update to the Geotechnical Investigation report referenced below. Additionally, CTE has reviewed the geotechnical aspects of the current referenced plans for substantial conformance with the project geotechnical recommendations.

CTE has reviewed the referenced geotechnical documents and project plans for the proposed improvements and has evaluated site conditions reported therein. Based on our review, the referenced geotechnical report is in general compliance with common geotechnical engineering practices and the referenced plans do not appear to conflict with the recommendations provided. Therefore, recommendations provided in the referenced report are considered applicable for the proposed development except as specifically modified or updated herein. Updated geotechnical recommendations for the proposed segmental block retaining walls and evaluation of the southern slope are also provided herein. CTE reserves the right to further modify the geotechnical recommendations based on actual conditions exposed during construction.

1.0 SEGMENTAL WALL PERAMETERS AND RECOMMENDATIONS

The following recommendations could require revision based on the actual type of Segmental Block Wall(s) that are selected at the site.

STRENGTH AND UNIT WEIGHT

Provided in the table below are preliminary soil parameters that are anticipated to be suitable for design of the proposed walls. We also reserve the right to perform additional testing to confirm that on-site soil processing is consistent throughout the duration of the project. Suitability and soil properties of any proposed imported soils should also be verified by this office.

Soil Zone	Internal Angle of Friction, ϕ (degrees)	Apparent Cohesion, c (psf)	Moist Soil Unit Weight, γ (pcf)
Reinforced Zone (If Applicable, Import Would Likely be Required)	32	0	130
Retained Soil (Fill)	25	100	125
Retained Soil (Dense Native)	35	200	130
Foundation (Fill)	25	100	125
Foundation (Rock)	35	200	130

Based on the anticipated conditions, the above parameters are anticipated to be suitable for analysis of the wall system.

WALL DRAINAGE

In all cases, the walls should include a foundation drain in accordance with the wall manufacturer's recommendations. If on-site soils proposed for use as backfill in the reinforced zone are anticipated to have low permeability rates "chimney-type" back-drains may also be required based on our observations and recommendations during construction. The project civil engineer shall determine if a concrete swale is needed along the top of all segmental walls.

STRUCTURE SETBACK

Foundations for structures should be setback from walls a minimum of 10 feet or 1.25 times the height of the wall, whichever is greater.

2.0 SLOPE STABILITY EVALUATION & RECOMMENDATIONS

In the southern portion of the site an approximately 40-foot high 2:1 (horizontal: vertical) slope descends to the south. According to mapping by Tan (1995), this portion of the site is located in area 3-1, which is described as "Generally Susceptible" to landsliding. However, Kennedy and Tan (2008) do not indicate the presence of mapped landslides at the subject site. In addition, onsite field observations did not indicate the presence of deep gross instabilities. Based on previous field investigation findings, the potential for deep seated landslides at the subject site is considered to be low.

The final input and output data from the limited evaluation of slope stability are attached. For the analysis, the existing slope was modeled based on proposed grades presented on the grading plan provided by Kppf and Google Earth for the remaining lower portion of the slope with geologic conditions based on previous limited studies of the area. Based on laboratory direct shear testing performed for the original investigation, the Tertiary Santiago Formation yielded soil strength values of phi = 34.0^o to 39.1^o and cohesion values = 120 psf to 800 psf. Based on previous laboratory direct shear testing, the residual soil yielded soil strength values of phi = 42.3^o and cohesion = 560 psf. Direst shear testing of the Previously Placed Fill has not been performed. To be conservative, Previously Placed Fill values of phi = 25.0^o and cohesion = 200 psf, residual soil values of phi =

40.0^{o} and cohesion = 500 psf, and Santiago Formation values of phi = 34.0^{o} and cohesion = 200 psf were utilized for the analysis. Based on analysis performed utilizing SLOPE/W, the existing slope is anticipated to exhibit a global static factor of safety in excess of 1.5. However, it is anticipated that surficial soils will continue to erode and may develop shallow slumps and failures on the slope face.

3.0 LIMITATIONS

The update recommendations herein are based on our review of the currently available design information, previous geotechnical investigations, and recent site observations. The anticipated conditions should be verified in the field during construction. Variations may exist and conditions not observed or described in the geotechnical reports may be encountered during construction. If conditions different from those described in the project geotechnical reports are encountered, this office should be notified and additional recommendations, if required, will be provided. The recommendations herein have been developed in order to reduce the potential adverse effects of soil expansion and settlement. However, even with the design and construction precautions, some post-construction movement and associated distress may occur.

CTE appreciates this opportunity to be of service on this project. If you have any questions regarding this report, please do not hesitate to contact the undersigned.

Respectfully submitted,

CONSTRUCTION TESTING & ENGINEERING, INC.

Dan T. Math, GE #2665
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
DANIEL T. MATH
No.2665
EXP.12/31/20
GEOTECHNICAL
STATE OF CALIFORNIA

Aaron J. Beeby, CEG #2603
Senior Engineering Geologist

PROFESSIONAL GEOLOGIST
AARON J. BEEBY
No.2603
CERTIFIED ENGINEERING GEOLOGIST
EXP.3/31/22
STATE OF CALIFORNIA

AJB/DTM/JFL:ach

ATTACHMENTS

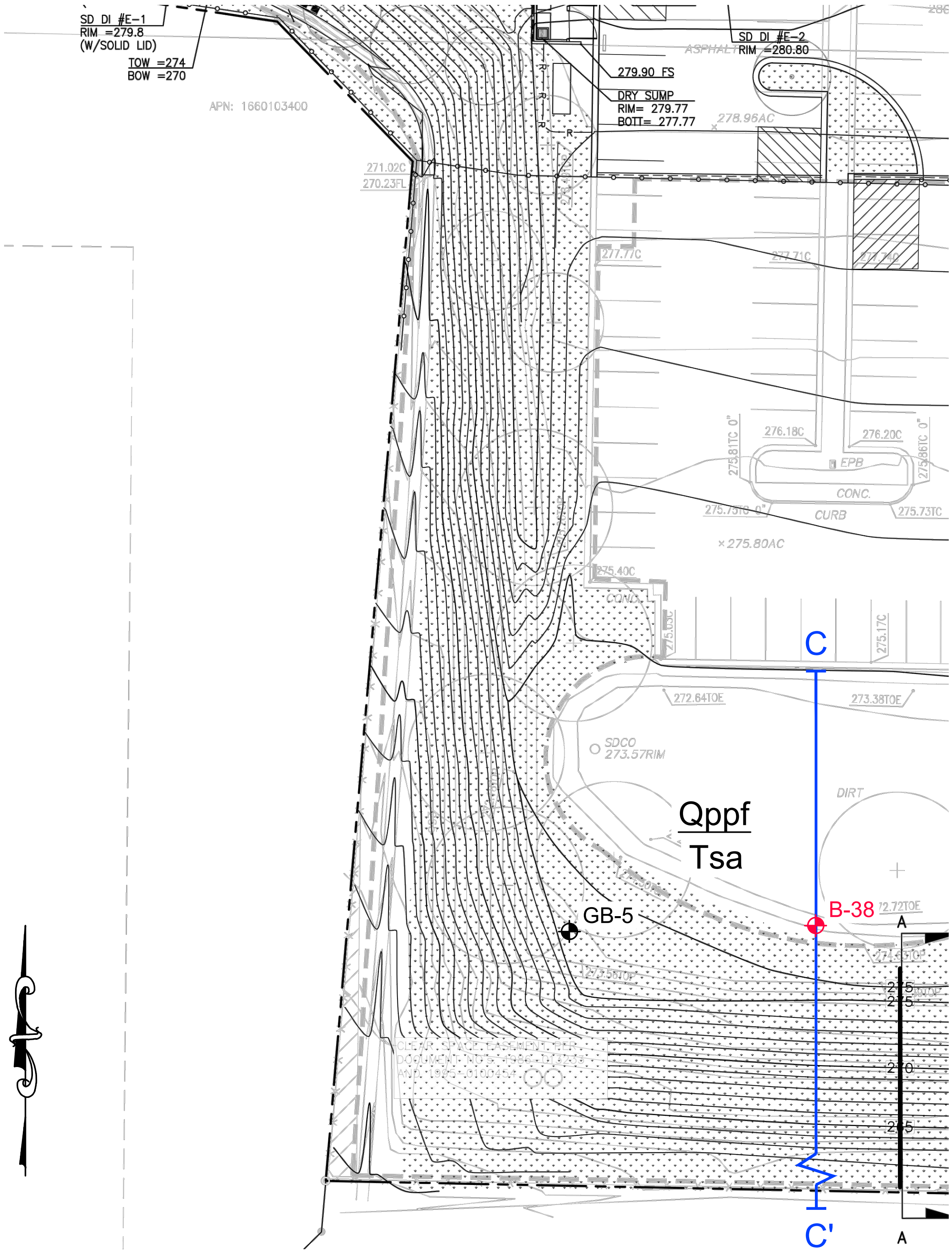
Figure 2: Geologic/Exploration Location Map
Figure 2C: Cross Section C-C’
C-C’

REFERENCES

Response to RFI and Meeting Note Questions
Proposed Tri-City Medical Center Improvements
4002 Vista Way
Oceanside, California
Job No. 10-15341G, Dated August 19, 2020

Preliminary Soils Investigation
Proposed Psychiatric Health Facility
4002 Vista Way
Oceanside, California
Job No. 10-15341G, Dated January 6, 2020

Grading Plan (9 Sheets)
Tri-City Medical Center Psychiatric Health Facility
4002 Vista Way
Oceanside, California
Project No. 006240.00, Dated September 25, 2020



LEGEND

B-38 ⊕	Approximate Boring Location (CTE 2016)
GB-5 ⊕	Approximate Boring Location (Geotechnical Professionals 2013)
Qppf Tsa	Quaternary Previously Placed Fill over Tertiary Santiago Formation
C⊢⊣C'	Cross Section C-C'

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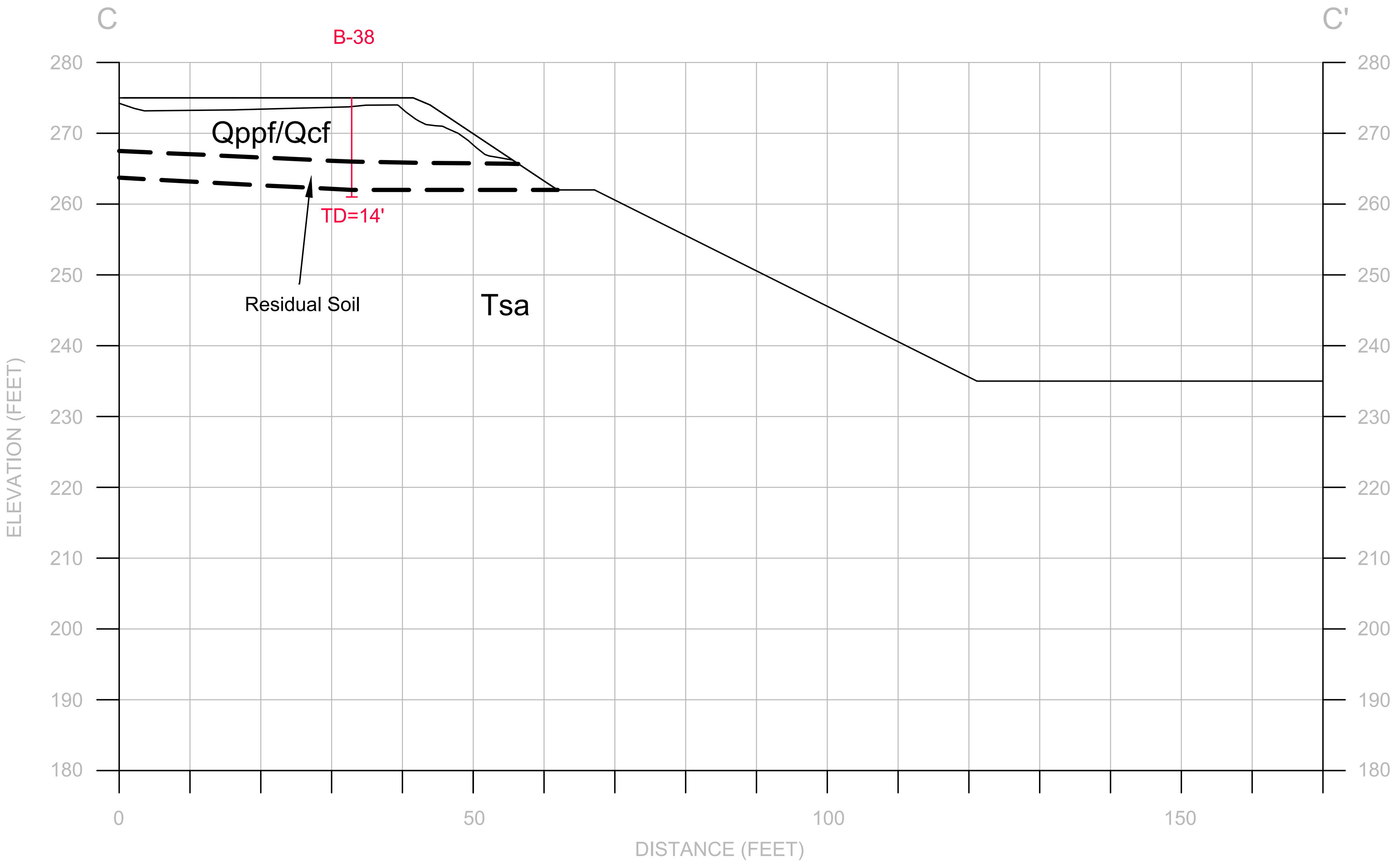
Construction Testing & Engineering, Inc.
1441 Montiel Rd Ste 115, Escondido, CA 92026 Ph (760) 746-4955

GEOLOGIC/EXPLORATION LOCATION MAP
PROPOSED TCMC PSYCHIATRIC HEALTH FACILITY
4002 VISTA WAY
OCEANSIDE, CALIFORNIA

SCALE: 1"=20'	DATE: 11/20
CTE JOB NO.: 10-15341G	FIGURE: 2

S:\Projects\10-15341G (Tri City)\Slope Stability (11-20) Southern Slope\Figure 2 (South Slope).dwg



CROSS SECTION C-C'

LEGEND

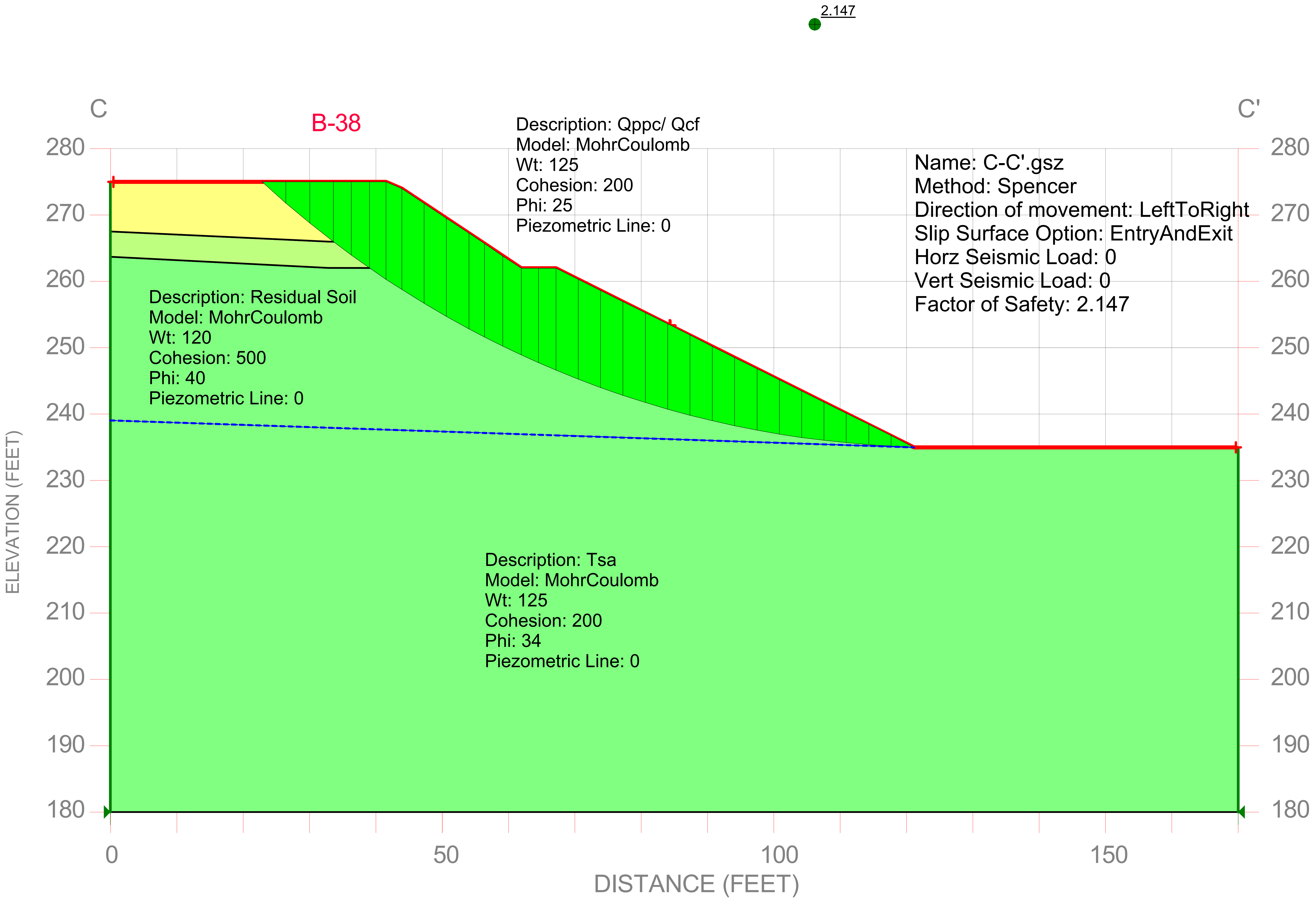
Qppf/ Qcf	QUATERNARY PREVIOUSLY PLACED/ COMPACTED FILL
Tsa	TERTIARY SANTIAGO FORMATION
- - -	APPROXIMATE GEOLOGIC CONTACT

CTE INC. Construction Testing & Engineering, Inc.
1441 Montiel Rd Ste 115, Escondido, CA 92026 Ph (760) 746-4955

CROSS SECTION C-C'
PROPOSED TCMC PSYCHIATRIC HEALTH FACILITY
4002 VISTA WAY
OCEANSIDE, CALIFORNIA

CTE JOB NO: 10-15341G
SCALE: 1" = 20'
DATE: 11/20
FIGURE: 2C

S:\Projects\10-15341G (Tri City)\Slope Stability (11-20) Southern Slope\Figure 2C (Cross Section C-C').dwg



CROSS SECTION C-C'