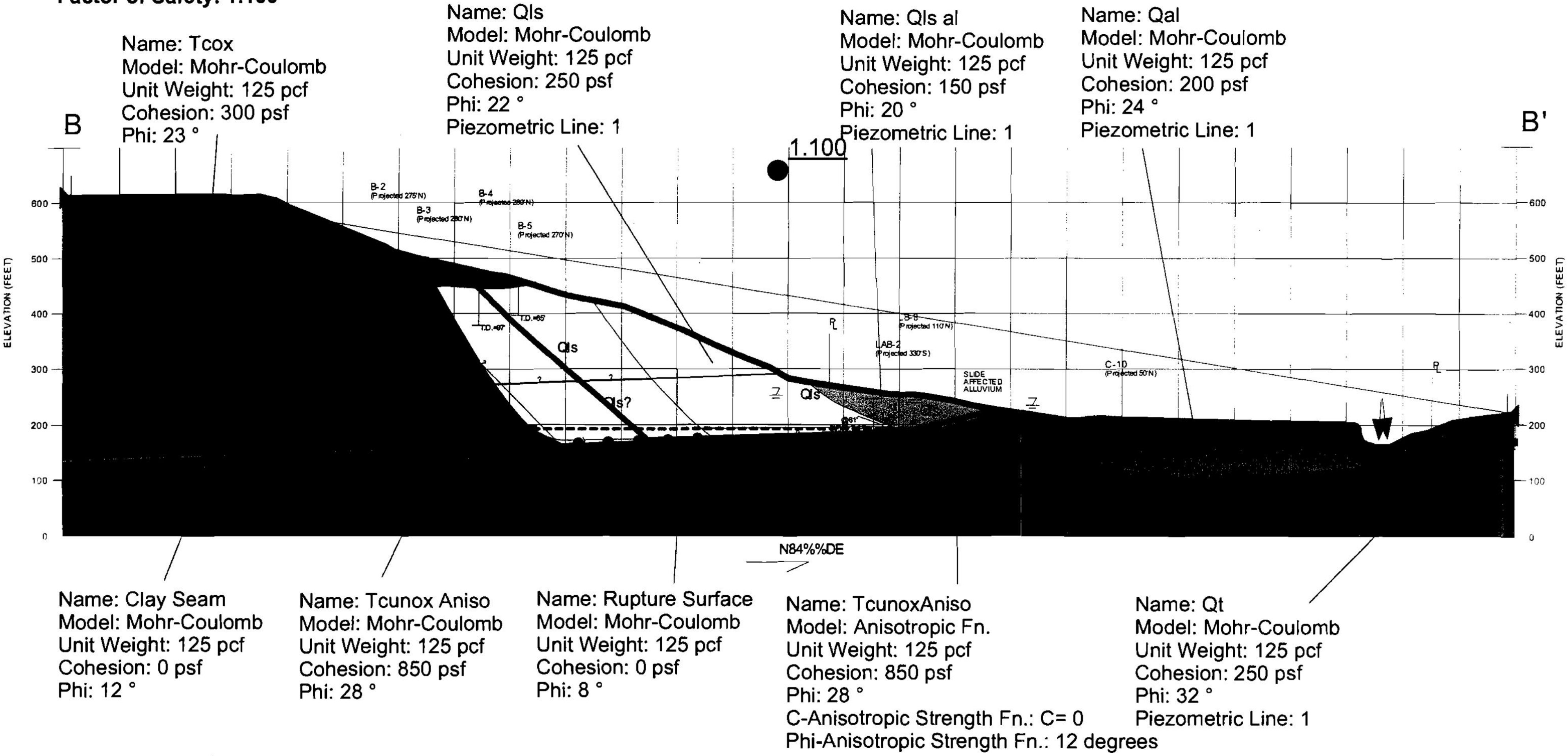
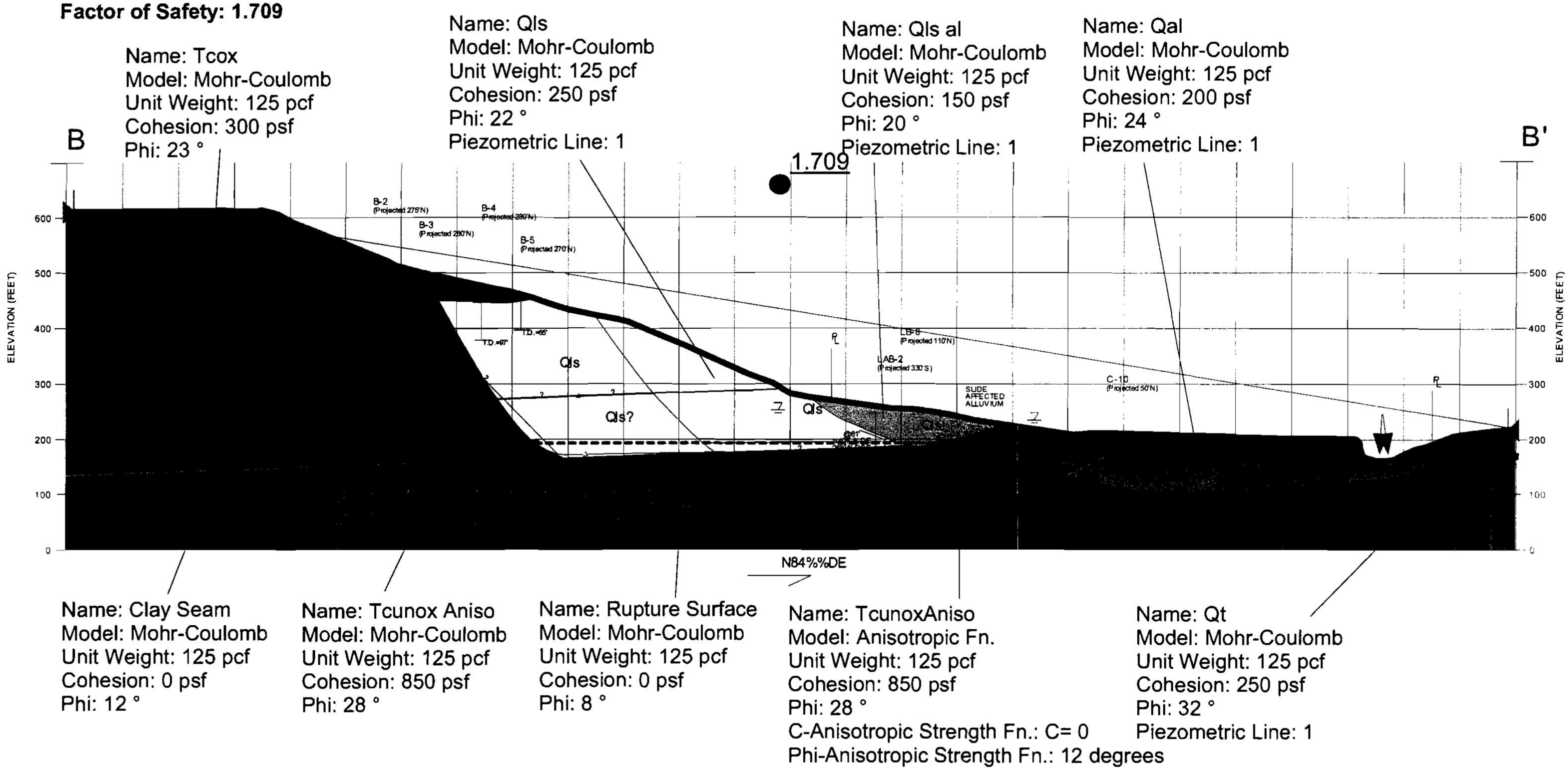
DOCKETED	
Docket Number:	24-OPT-02
Project Title:	Compass Energy Storage Project
TN #:	264556
Document Title:	Geotechnical Evaluation Report (TN #255561-6) References Part 2_9 of 9
Description:	N/A
Filer:	Erin Phillips
Organization:	Dudek
Submitter Role:	Applicant Consultant
Submission Date:	7/2/2025 6:58:42 PM
Docketed Date:	7/2/2025

Title: Continuing Life Rancho San Juan Capistrano
File Name: Section B-B' Middle R Peak.gsz
Date: 1/14/2009
Method: Spencer
Factor of Safety: 1.100



Title: Continuing Life Rancho San Juan Capistrano
File Name: Section B-B' Deep.gsz
Date: 1/14/2009
Method: Spencer
Factor of Safety: 1.709



SECTION C-C'

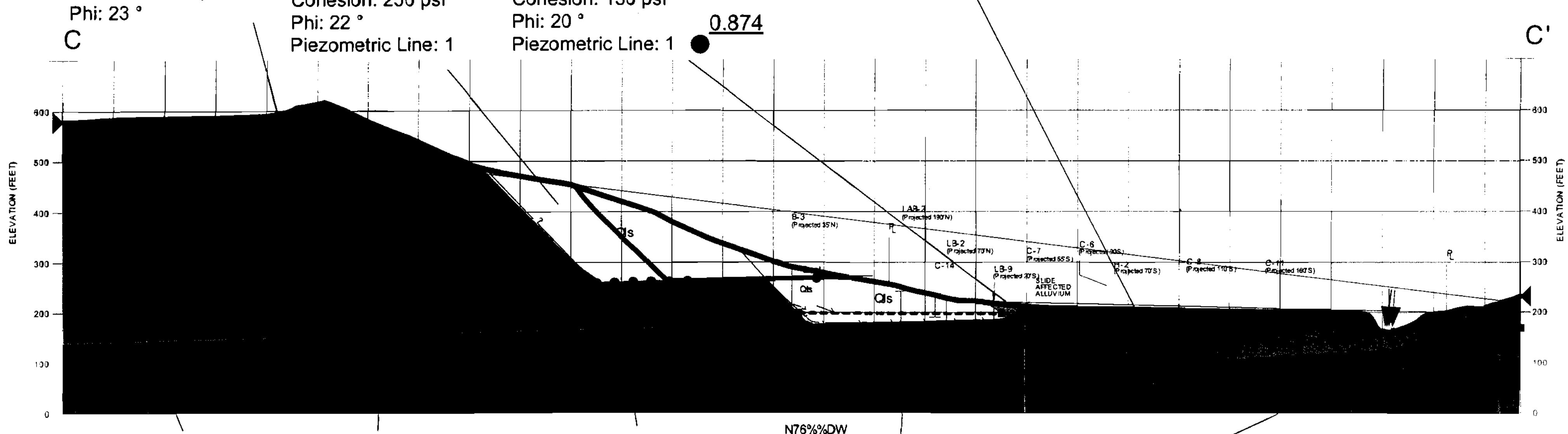
Title: Continuing Life Rancho San Juan Capistrano
File Name: Section C-C' Upper.gsz
Date: 1/13/2009
Method: Spencer
Factor of Safety: 0.874

Name: Tc
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 300 psf
Phi: 23 °

Name: Qls
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 250 psf
Phi: 22 °
Piezometric Line: 1

Name: Qls al
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 150 psf
Phi: 20 °
Piezometric Line: 1

Name: Qal
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 200 psf
Phi: 24 °
Piezometric Line: 1



Name: Tcunox Aniso
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 850 psf
Phi: 28 °

Name: Clay Seam
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi: 12 °

Name: Rupture Surface
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi: 8 °

Name: Tcunox
Model: Anisotropic Fn.
Unit Weight: 125 pcf
Cohesion: 850 psf
Phi: 28 °
C-Anisotropic Strength Fn.: C = 0
Phi-Anisotropic Strength Fn.: 12 degrees

Name: Qt
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 250 psf
Phi: 32 °
Piezometric Line: 1

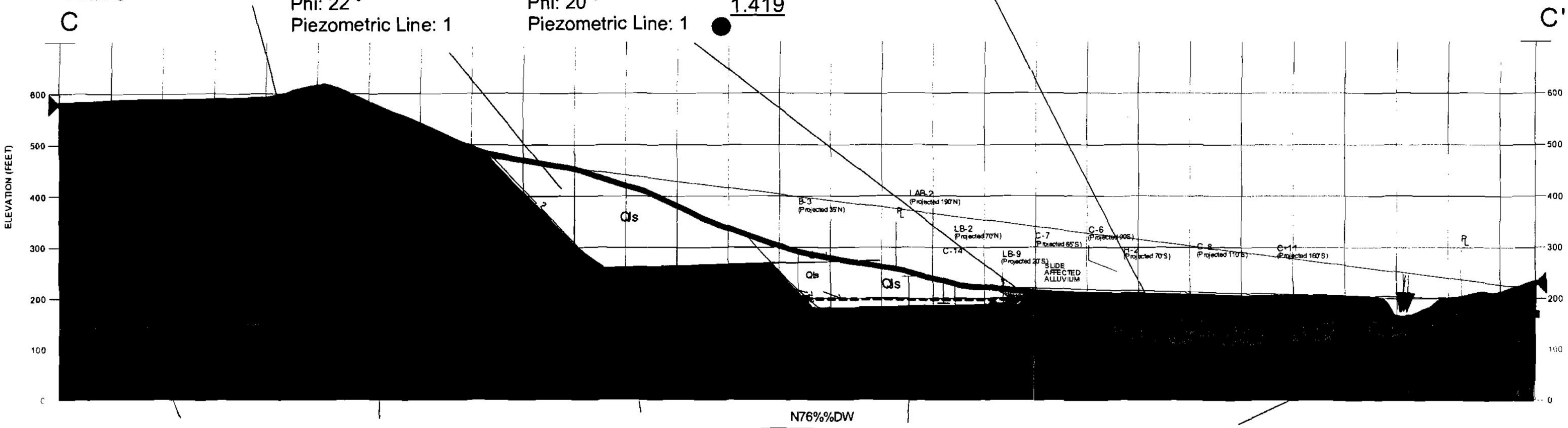
Title: Continuing Life Rancho San Juan Capistrano
File Name: Section C-C' Middle.gsz
Date: 1/13/2009
Method: Spencer
Factor of Safety: 1.419

Name: Tc
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 300 psf
Phi: 23 °

Name: Qls
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 250 psf
Phi: 22 °
Piezometric Line: 1

Name: Qls al
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 150 psf
Phi: 20 °
Piezometric Line: 1

Name: Qal
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 200 psf
Phi: 24 °
Piezometric Line: 1



Name: Tcunox Aniso
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 850 psf
Phi: 28 °

Name: Clay Seam
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi: 12 °

Name: Rupture Surface
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi: 8 °

Name: Tcunox
Model: Anisotropic Fn.
Unit Weight: 125 pcf
Cohesion: 850 psf
Phi: 28 °
C-Anisotropic Strength Fn.: C = 0
Phi-Anisotropic Strength Fn.: 12 degrees

Name: Qt
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 250 psf
Phi: 32 °
Piezometric Line: 1

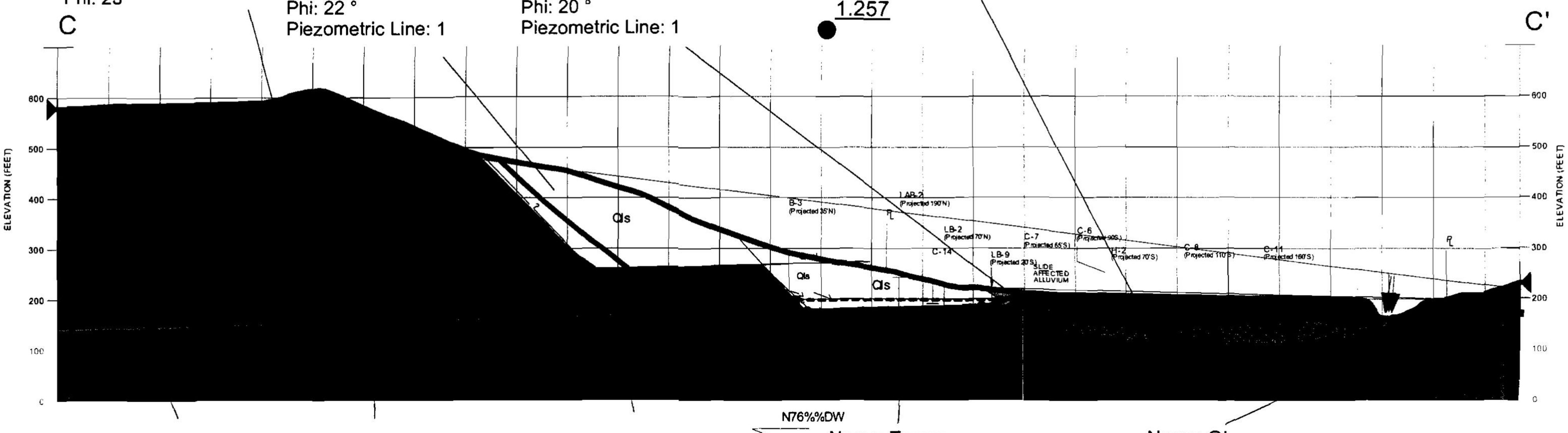
Title: Continuing Life Rancho San Juan Capistrano
File Name: Section C-C' Middle R Peak.gsz
Date: 1/14/2009
Method: Spencer
Factor of Safety: 1.257

Name: Tc
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 300 psf
Phi: 23 °

Name: Qls
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 250 psf
Phi: 22 °
Piezometric Line: 1

Name: Qls al
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 150 psf
Phi: 20 °
Piezometric Line: 1

Name: Qal
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 200 psf
Phi: 24 °
Piezometric Line: 1



Name: Tcunox Aniso
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 850 psf
Phi: 28 °

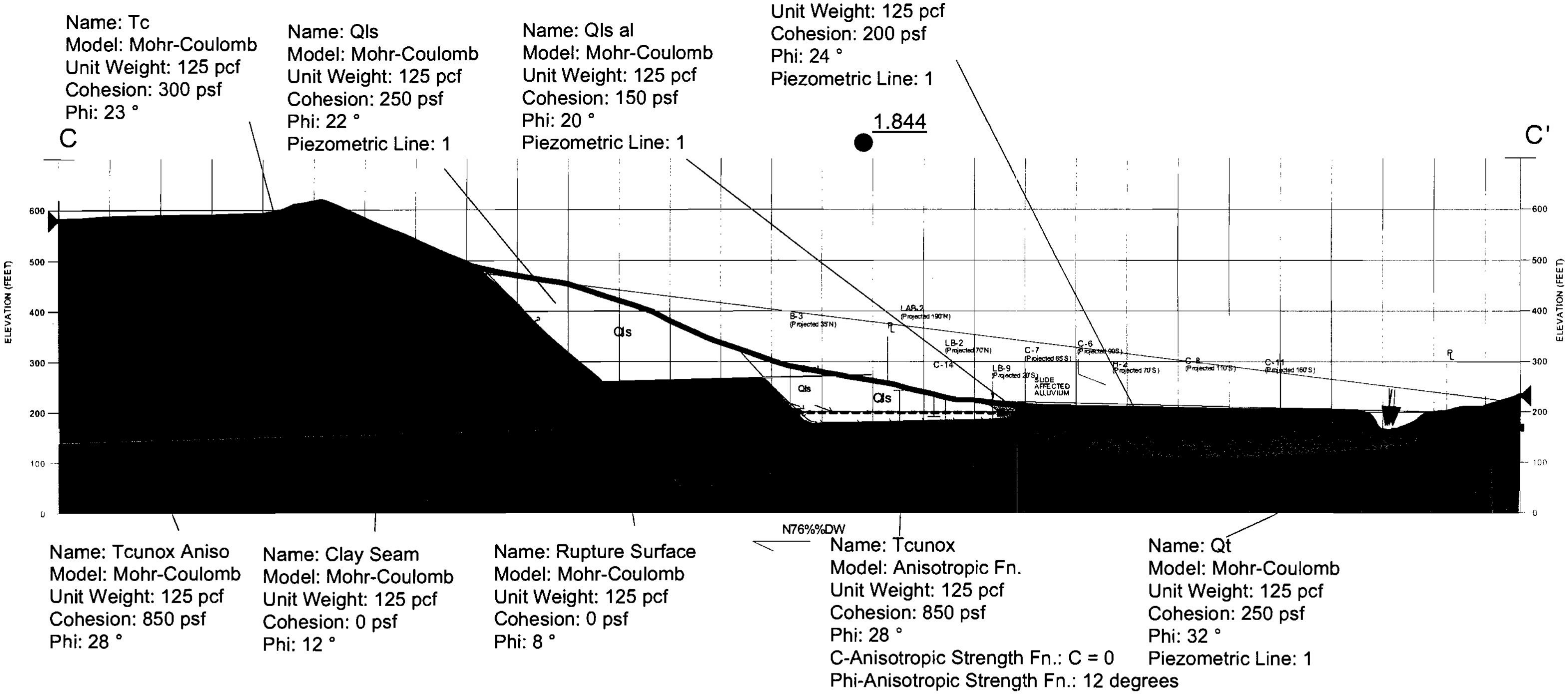
Name: Clay Seam
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi: 12 °

Name: Rupture Surface
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 0 psf
Phi: 8 °

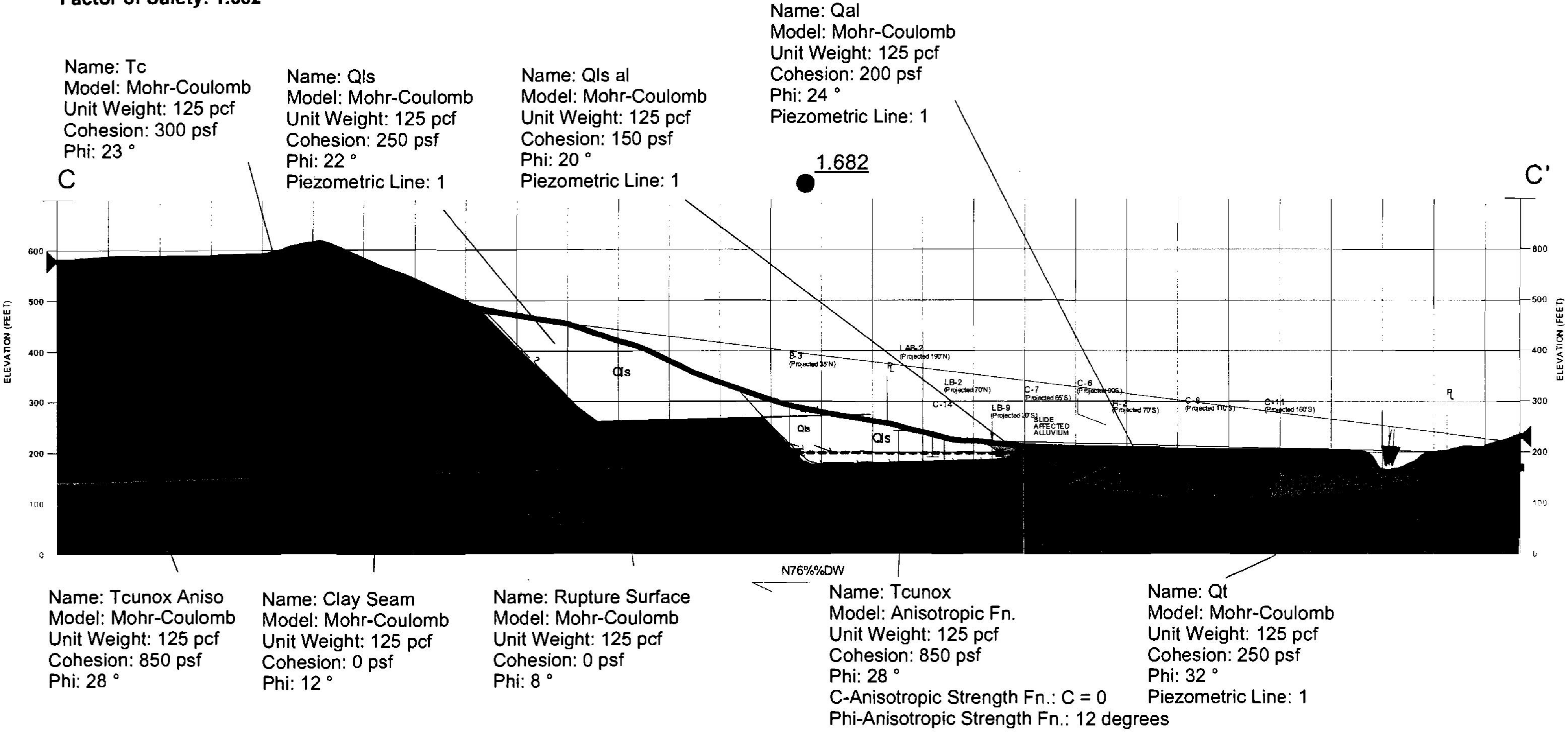
Name: Tcunox
Model: Anisotropic Fn.
Unit Weight: 125 pcf
Cohesion: 850 psf
Phi: 28 °
C-Anisotropic Strength Fn.: C = 0
Phi-Anisotropic Strength Fn.: 12 degrees

Name: Qt
Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion: 250 psf
Phi: 32 °
Piezometric Line: 1

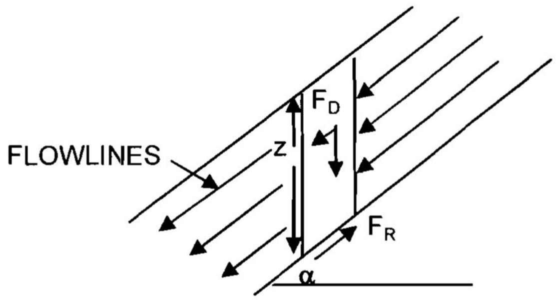
Title: Continuing Life Rancho San Juan Capistrano
File Name: Section C-C' Deep R Peak.gsz
Date: 1/14/2009
Method: Spencer
Factor of Safety: 1.844



Title: Continuing Life Rancho San Juan Capistrano
File Name: Section C-C' Deep.gsz
Date: 1/13/2009
Method: Spencer
Factor of Safety: 1.682



SURFICIAL SLOPE STABILITY ANALYSIS



Project No.:	012383-001
Case:	2H:1V Slope / Compacted Fill @ 90% R.C.

Depth of Saturation (ft), Z	=	**3**
Buoyant Unit Weight of Soil (pcf), γ_b	=	**67.6**
Total Unit Weight of Soil (pcf), γ_t	=	**130**
Slope Angle, α	=	**26.6**
Angle of Internal Friction, ϕ	=	**26**
Cohesion (psf), c	=	**300**

Force Tending To Cause Movement:

$F_D = Z\gamma_t \sin 2\alpha / 2$ = **156.14** lb/ft

Force Tending To Resist Movement:

$F_R = Z\gamma_b \cos^2 \alpha \ \tan\phi + (c)$

= **379.08** lb/ft

F.S. : $$\frac{2Z\gamma_b \cos^2\alpha \ \tan\phi + 2c}{Z\gamma_t \sin 2\alpha}$$

F.S. = 2.43

SURFICIAL STABILITY	Project Name :	Continuing Life Rancho SJC
	Project Number :	012383-001
	Designed/Checked :	DXB/SAC
		Leighton

APPENDIX F

LEIGHTON AND ASSOCIATES, INC.

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

1.0 General

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

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

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

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

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

1.3 The Earthwork Contractor: The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications. The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

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

2.0 Preparation of Areas to be Filled

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

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

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

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

2.2 Processing: Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation: In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

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

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

3.0 Fill Material

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

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

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

4.0 Fill Placement and Compaction

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

4.2 Fill Moisture Conditioning: Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557-07).

4.3 Compaction of Fill: After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557-07). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes: In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557-07.

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

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

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

5.0 Subdrain Installation

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

6.0 Excavation

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

7.0 Trench Backfills

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

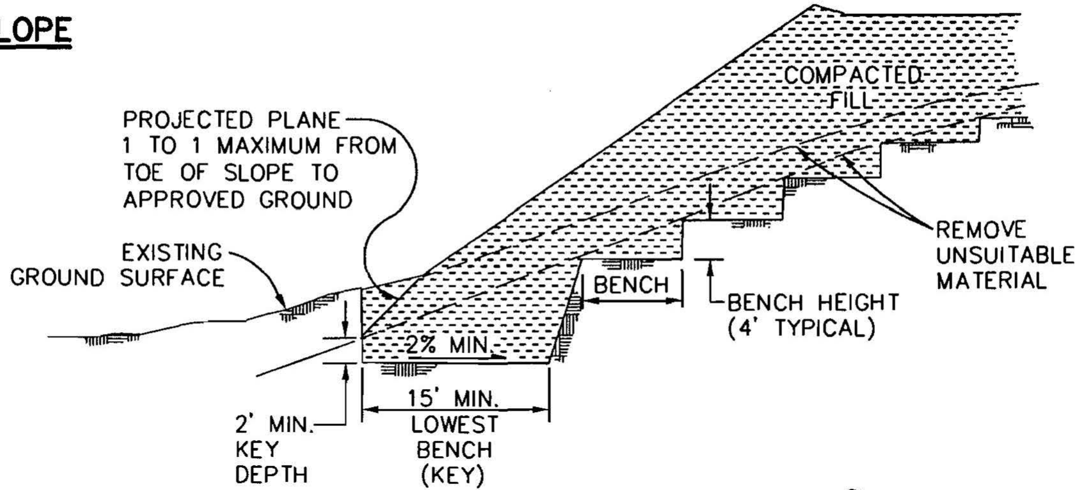
7.2 All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit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

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

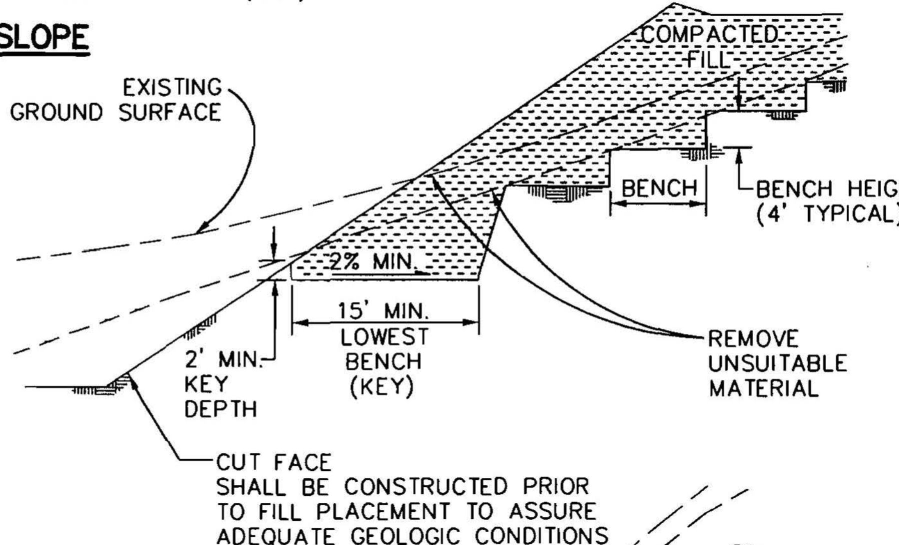
7.4 The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

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

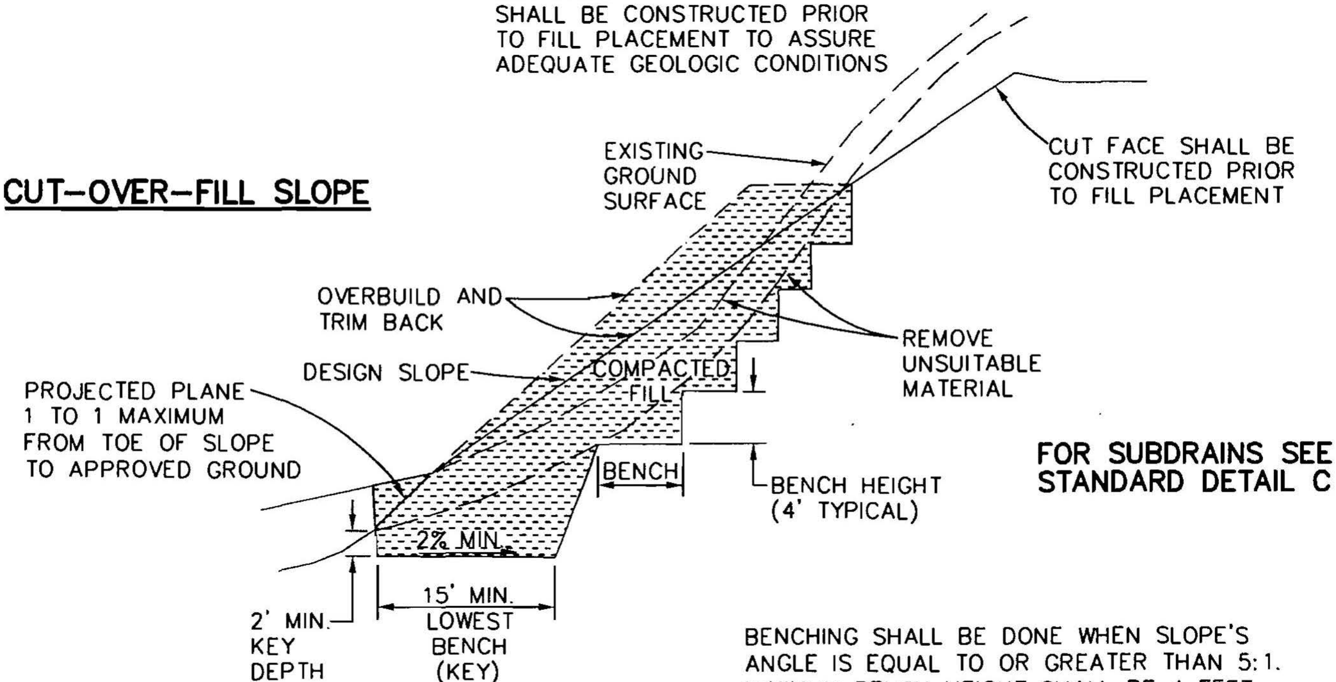
FILL SLOPE



FILL-OVER-CUT SLOPE



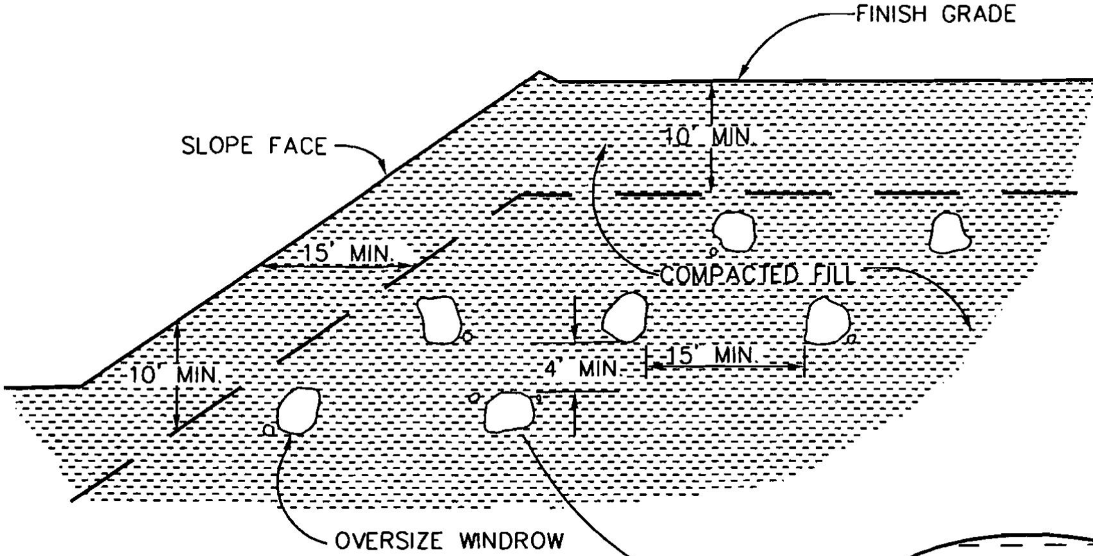
CUT-OVER-FILL SLOPE

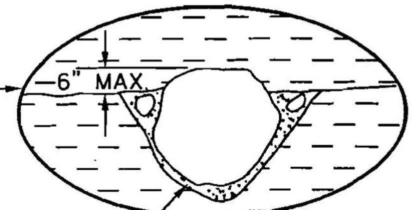


FOR SUBDRAINS SEE STANDARD DETAIL C

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

KEYING AND BENCHING	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS A

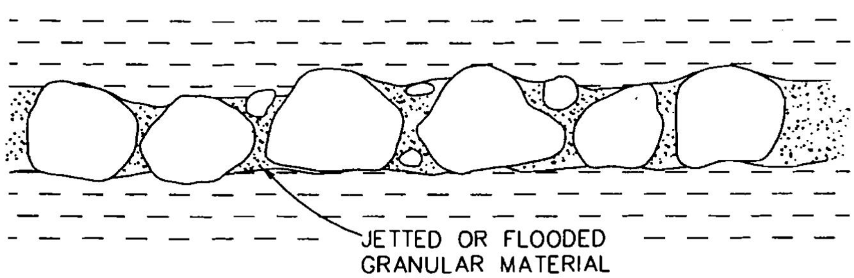




- OVERSIZE ROCK IS LARGER THAN 8 INCHES IN LARGEST DIMENSION.
- EXCAVATE A TRENCH IN THE COMPACTED FILL DEEP ENOUGH TO BURY ALL THE ROCK.
- BACKFILL WITH GRANULAR SOIL JETTED OR FLOODED IN PLACE TO FILL ALL THE VOIDS.
- DO NOT BURY ROCK WITHIN 10 FEET OF FINISH GRADE.
- WINDROW OF BURIED ROCK SHALL BE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

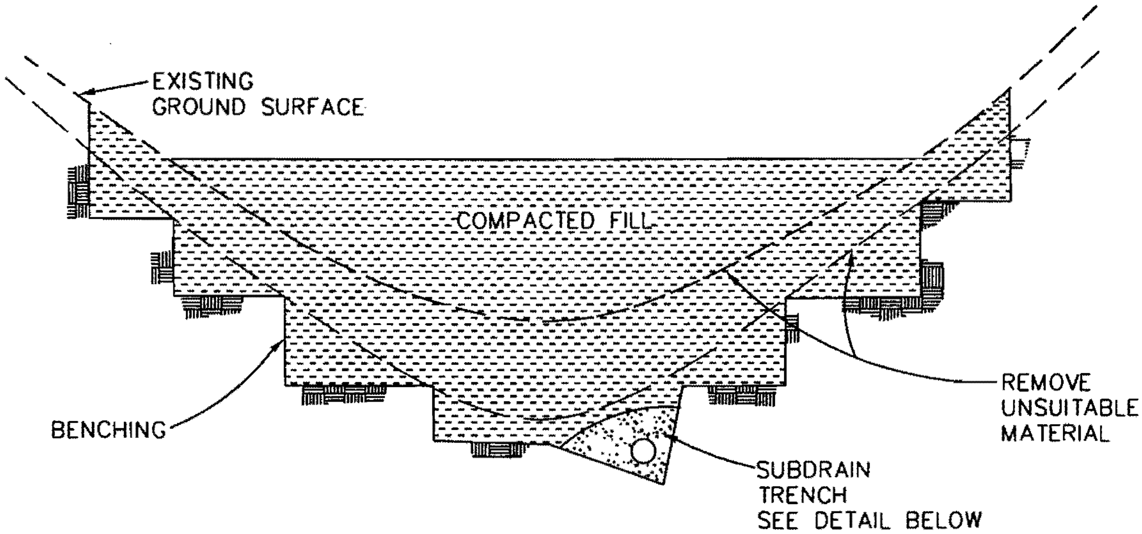
GRANULAR MATERIAL TO BE DENSIFIED IN PLACE BY FLOODING OR JETTING.

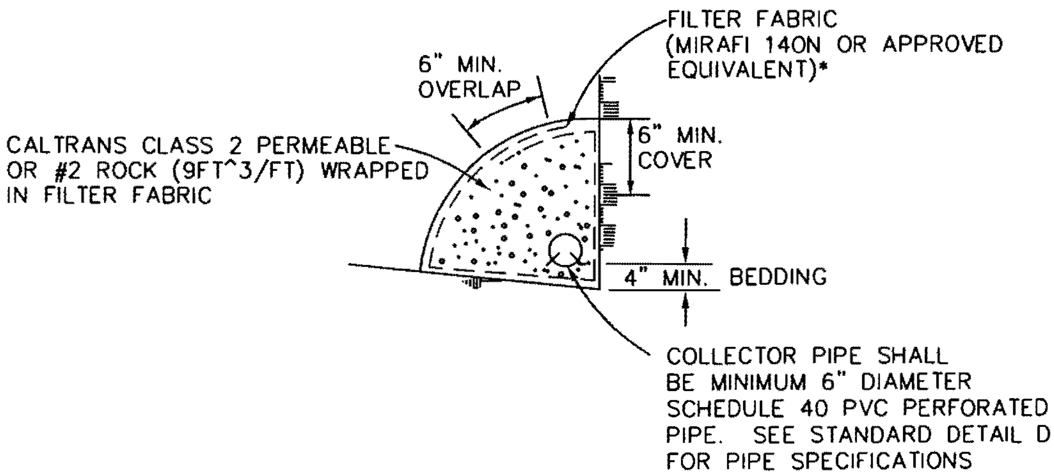
DETAIL



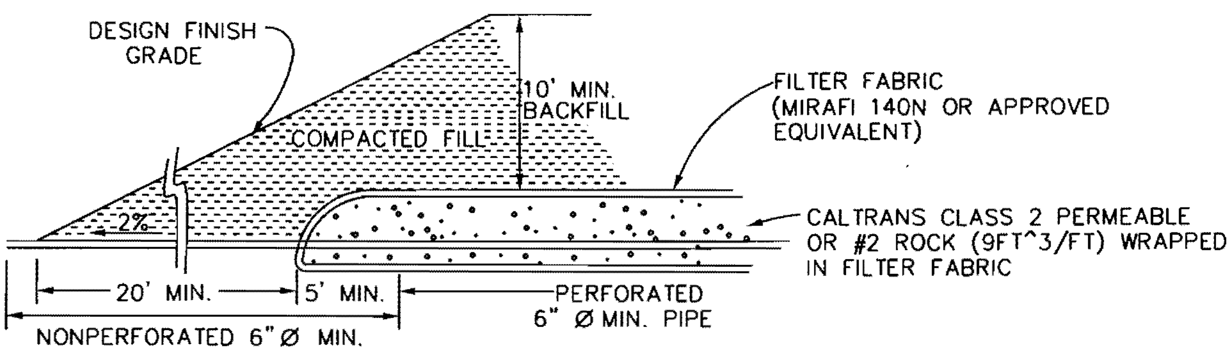
TYPICAL PROFILE ALONG WINDROW

OVERSIZE ROCK DISPOSAL	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS B



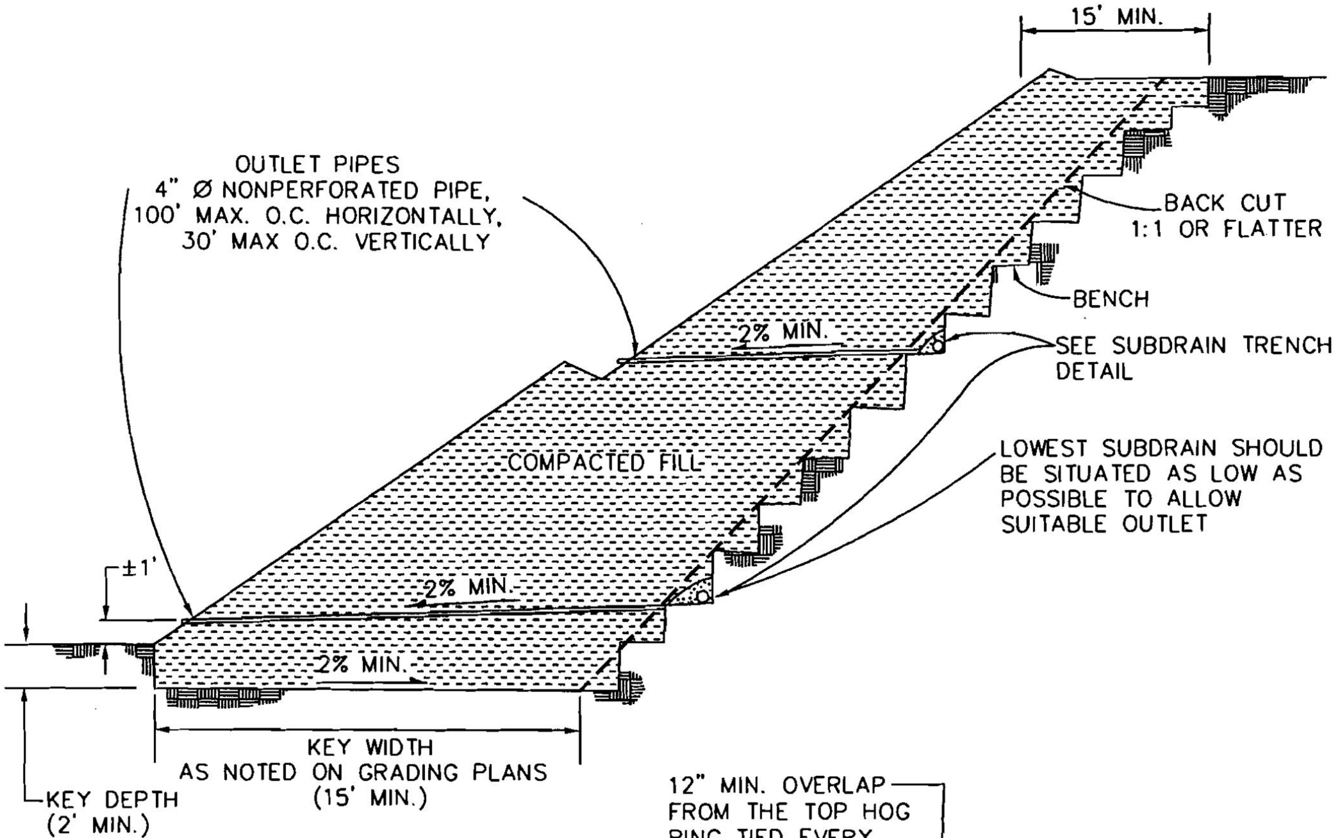


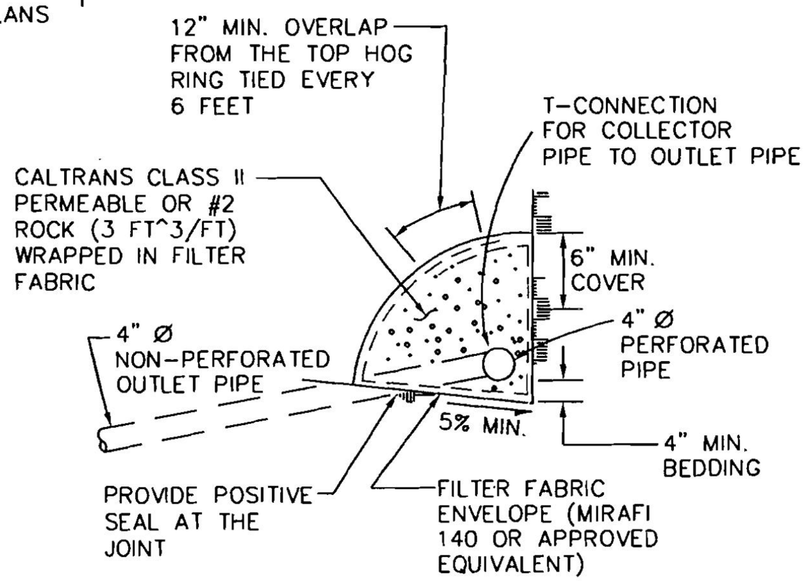
SUBDRAIN DETAIL



DETAIL OF CANYON SUBDRAIN OUTLET

CANYON SUBDRAINS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS C





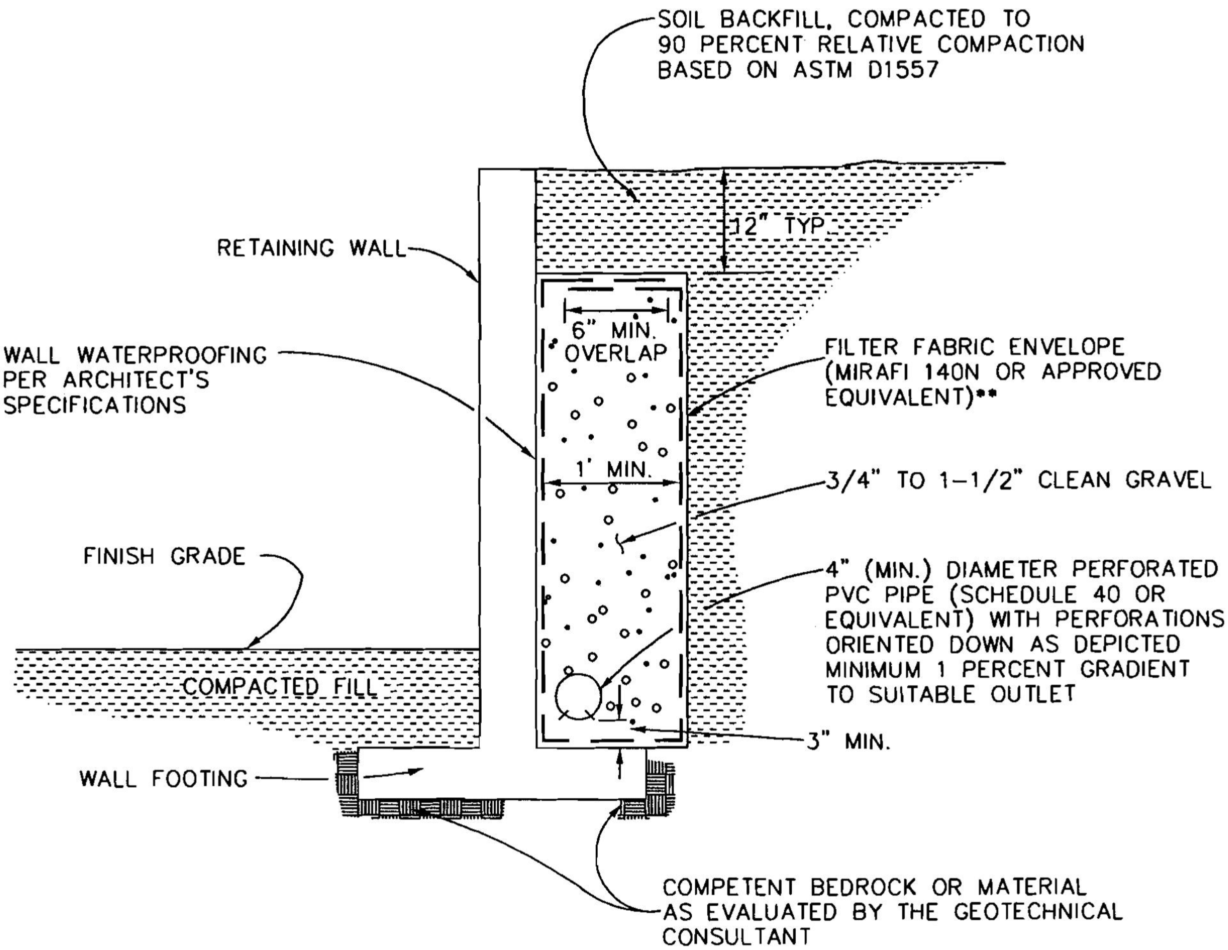
SUBDRAIN TRENCH DETAIL

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

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

All outlet pipe shall be placed in a trench no wide than twice the subdrain pipe. Pipe shall be in soil of SE >/=30 jetted or flooded in place except for the outside 5 feet which shall be native soil backfill.

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



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

RETAINING WALL DRAINAGE DETAIL

GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS E

Geotechnical Engineering Report

Broad Reach Power Compass BESS
San Juan Capistrano, Orange County, CA
November 3, 2021
Terracon Project No. 60215170

Prepared for:
Sargent & Lundy
Chicago, Illinois

Prepared by:
Terracon Consultants, Inc.
Tustin, California

November 3, 2021

Sargent & Lundy
55 E. Monroe
Chicago, Illinois 60603

Attn: Mr. Matthew A. Braet
P: (312) 269-2642
E: Matthew.A.Braet@SargentLundy.com

Re: Geotechnical Engineering Report
Broad Reach Power Compass BESS
San Juan Capistrano, Orange County, CA
Terracon Project No. 60215170

Dear Mr. Braet:

We have completed the Geotechnical Engineering services for the above referenced project. This study was performed in general accordance with Terracon Proposal No. P60215170 dated June 30, 2021. This report presents the findings of the subsurface exploration and provides geotechnical recommendations concerning earthwork and the design and construction of foundations, access roads, and infiltration systems for the proposed project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report or if we may be of further service, please contact us.

Sincerely,
Terracon Consultants, Inc.

Victor V. Nguyen, E.I.T.
Staff Engineer

Scott G. Lawson, P.E., G.E.
Senior Geotechnical Engineer

APR Review by F. Fred Buhamdan, P.E

REGISTERED PROFESSIONAL ENGINEER
SCOTT G. LAWSON
No. GE 2829
Exp. 9/30/23
GEOTECHNICAL
STATE OF CALIFORNIA

REPORT TOPICS

Note: This report was originally delivered in a web-based format. Orange Bold text in the report indicates a referenced section heading. The PDF version also includes hyperlinks which direct the reader to that section and clicking on the *GeoReport* logo will bring you back to this page. For more interactive features, please view your project online at client.terracon.com.

ATTACHMENTS

Geotechnical Engineering Report

Broad Reach Power Compass BESS
San Juan Capistrano, Orange County, CA
Terracon Project No. 60215170
November 3, 2021

INTRODUCTION

This report presents the results of our geotechnical engineering services performed for the proposed Compass Battery Energy Storage System (BESS) facility for Broad Reach Power to be located in San Juan Capistrano, Orange County, California. Approximate coordinates for the center of the site are 33.53205°N, 117.67753°W. The purpose of these services is to provide information and geotechnical engineering recommendations relative to subsurface conditions and construction of the proposed BESS and infiltration systems.

Terracon's geotechnical engineering scope of work for this project included the advancement of 30 test borings to approximate depths ranging between 5 and 101½ feet below existing ground surface (bgs) within and adjacent to the proposed BESS facility footprint.

Maps showing the site and boring locations are shown in the **Site Location** and **Exploration Plan** sections, respectively. The results of the laboratory testing performed on soil samples obtained from the site during the field exploration are included on the boring logs and as separate graphs in the **Exploration Results** section.

SITE CONDITIONS

The following description of site conditions is derived from our site visit in association with the field exploration and our review of publicly available geologic and topographic maps.

Item	Description
Parcel Information	The proposed project consists of design and construction of the new Compass BESS facility to be located in the City of San Juan Capistrano, Orange County, California. Approximate coordinates for the center of the site are 33.53205 °N, 117.67753 °W.
Existing Improvements	Site is mostly undeveloped with native grasses and trees. A garden center with several small buildings is located near the center of the site.
Current Ground Cover	Exposed soils with sparse vegetation, and denser vegetation on the hillside to the west.

Item	Description
Existing Topography (from Google Earth Pro)	Within the footprint of the proposed facility, the eastern half is relatively flat with approximate elevations ranging from 206 to 212 feet above mean sea level (MSL). The western half has a gradual slope down towards the east with approximate elevations ranging from 230 feet to 212 feet MSL. Immediately to the west of the facility footprint are tall slopes ascending several hundred feet. Immediately to the east of the facility footprint is Oso Creek which steeply descends as much as 40 feet to an approximate bottom elevation of 169 feet MSL. Stability assessment of the eastern and western surrounding slopes is not included in our scope of work and is being evaluated by Sargent & Lundy (S&L) based on data provided by Terracon.

PROJECT DESCRIPTION

Item	Description
Proposed Structures	■ 138/34kV transformers supported on mat foundations. ■ Batteries supported on slabs or mat foundations. ■ A 138kV transmission line pole supported by a drilled pier may be included in the project, but the location is currently unknown. ■ A San Diego Gas & Electric (SDG&E) substation will be constructed within the project footprint, with equipment foundations similar to those described above. We understand that a "buried" retaining wall will be constructed near the eastern edge of the project site to limit the impact of erosion occurring along the banks of the adjacent Oso Creek. In addition, we understand based on discussions with S&L that other retaining walls are being considered to achieve final site grades. Details regarding these walls were not available at the time this report was prepared. We understand that geotechnical engineers with S&L will be responsible for the geotechnical design and evaluation of these walls, using geotechnical data and recommendations provided in this report.
Finished Grade Elevation	Finished grade elevation was not provided at the time of preparation of this report.
Grading	Grading plans were not provided at the time of preparation of this report. Based on our understanding of the conceptual site plan and the existing topography of the site, we anticipate that cuts as much as 20 feet and fill on the order of 5 may be required in order to reach final grade elevation.
Infiltration Systems	Based on our communications with S&L, infiltration systems (such as retention basins or bio-swales) are anticipated for on-site stormwater management.
Access Roads	We understand that unpaved roads will be constructed onsite to access BESS areas.

GEOTECHNICAL CHARACTERIZATION

We have developed a general characterization of the subsurface soil and groundwater conditions based upon our review of the data and our understanding of the geologic setting and planned construction.

The geotechnical characterization forms the basis of our geotechnical calculations and evaluation of site preparation and foundation options. As noted in **General Comments**, the characterization is based upon widely spaced exploration points across the site, and variations are likely.

Geologic Conditions

The site is situated on a stream terrace west of and adjacent to Oso Creek in Orange County, California. Oso Creek forms a canyon that dissects a portion of the San Joaquin Hills - a coastal range of southern California. The hills are formed in layered sedimentary formations that include the Capistrano Formation. The Capistrano Formation consists of poorly-consolidated, fossiliferous, marine sandy-siltstone and mudstone. Capistrano beds are susceptible to landsliding as evidenced by landslide deposits mapped west of the site. The stream terrace area of the site is underlain by colluvium derived from adjacent hillsides and alluvial stream deposits of Oso Creek. A regional geologic map is included in the **Site Location** section.

Earthquake faults are not mapped within or projecting toward the site. The buried San Joaquin Hills thrust fault underlies the San Joaquin Hills approximately 5 kilometers northwest of the site. More distant faults include the Newport-Inglewood fault zone and the Elsinore fault zone located approximately 10 kilometers southwest and 29 kilometers northeast of the site, respectively.

Subsurface Conditions

Based on the results of the borings performed for our subsurface exploration, alluvial and colluvial soils were encountered extending to approximate boring depths of 26½ to 76½ feet bgs. Landslide deposits (Qls) were encountered in borings B-4, B-5, B-6, and B-11 extending to approximate depths of 10 to 50 feet bgs. Capistrano Formation (Tc) bedrock was encountered beneath alluvial/colluvial and landslide deposits beginning at approximate boring depths ranging from 26½ to 62 feet bgs. Soil deposits generally consisted of interbedded layers of soft to hard lean clay with variable amounts of sand and gravel, silt with varying amounts of sand, and silty clay with varying amounts of sand and gravel. Interbedded layers of loose to very dense sand with varying amounts of silt and clay were encountered in borings B-2, B-3, B-10, and B-17 from depths of approximately 31½ to 40, 51½ to 65, 51½ to 76½, and 50 to 76½ feet bgs, respectively. Materials of the Capistrano Formation were generally recovered as interbedded layers of very stiff to hard elastic silt with trace sand, lean clay with varying amounts of sand and silt, and silty clay with varying amounts of sand. The following table summarizes the approximate depth to Capistrano formational materials.

Boring ID	Depth to Capistrano Formation Materials (ft, bgs)	Corresponding Elevation at Top of Formation (feet, MSL)	Boring Termination Depth (ft, bgs)
B-4	50	197	91½
B-5	62	192	76½
B-6	40	194	51
B-11	55	185	91½
B-12	40	192	76½
B-13	60	189	76½
B-14	30	194	51½
B-15	60	153	101½
B-19	26½	190½	51½
B-24	35	181	76½
B-25	41½	167½	76½

Conditions encountered at each boring location are indicated on the individual boring logs shown in the **Exploration Results** section and are attached to this report. Stratification boundaries on the boring logs represent the approximate location of changes in native soil types; in situ, the transition between materials may be gradual.

Lab Results

Laboratory tests were conducted on selected soil formational material samples and the test results are presented in the **Exploration Results** section and on the boring logs. Atterberg limits test results indicate that the on-site soils generally have low to medium plasticity. A modified proctor test conducted in accordance with ASTM D1557 indicates that the near surface sandy lean clay soil tested has a maximum dry density of 120.8 pcf and corresponding optimum moisture content of 13.3 percent. Expansion index testing on near surface soils from borings B-1 and B-17 indicate that these soils have expansion indices of 39 and 64 corresponding to low to medium expansion potential as determined by ASTM D4829. Consolidation testing performed on four samples collected from borings within the upper 10 feet indicated negligible to moderate swell. R-value testing conducted on a near surface silty clay soil sample indicated an R-Value of 27.

The following tables summarize the results of direct shear and unconsolidated undrained triaxial testing. Additional information for these tests is provided in the **Exploration Results** section.

Direct Shear Strength Test Results

Boring ID	Depth (ft, bgs)	USCS Material Type	Friction Angle (degree)[1]	Cohesion (psf)[1]
B-10	10	CL	25	300
B-12	10	CL	33	650
B-13	10	CL	32	1,050

1. Values are peak values.

Unconsolidated Undrained Triaxial Test Results

Boring ID	Depth (ft, bgs)	USCS Material Type	Ultimate Undrained Shear Strength (ksf)[1]
B-5	60	CL-ML	4.16
B-13	30	CL	3.87
B-19	45	CL-ML[2]	3.29

1. Unconsolidated undrained conditions.
2. Capistrano Formation material recovered as CL-ML.

Electrical Resistivity Testing

Terracon performed field measurements of soil electrical resistivity for the support of grounding design. Soil resistivity data was obtained along three traverses selected by the client and shown in the **Exploration Plan**. The testing was performed in general accordance with ASTM G57 - Wenner Four Electrode Method. The Wenner arrangement (equal electrode spacing) was used with the 'a'-spacing of 3, 5, 7, 10, 20, 30, 40, 50, 60, 70, 80, 90, 100, 120, 140, and 160 feet. The "a" spacing is generally considered to be the depth of influence of the test. The electrical resistivity test results are presented in **Exploration Results**.

Groundwater Conditions

Groundwater was encountered in 4 of the 30 borings advanced at the site. Boring specific groundwater observations can be found on the logs in the **Exploration Results** section of this report. These observations represent groundwater conditions at the time of the field exploration and may not be indicative of other times, or at other locations.

Boring ID	Depth to Groundwater (feet, bgs)	Groundwater Elevation (feet, MSL)
B-3	70.4	139
B-10	47.3	162
B-17	47.0	161
B-19	47.5	170

Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the borings were performed. Therefore, groundwater levels during construction or at other times in the life of the structures may be higher or lower than the levels indicated on the boring logs. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

In clayey soils with low permeability, the accurate determination of groundwater level may not be possible without long term observation. Long term observation after drilling could not be performed as borings were backfilled immediately upon completion due to safety concerns. Groundwater levels can best be determined by implementation of a groundwater monitoring plan.

Based on review of Plate 1.2 of the Seismic Hazard Zone Report (SHZP) for the San Juan Capistrano 7.5-Minute Quadrangle (CDMG)[1], historic shallow groundwater depth in the vicinity of the project site is reported at approximately 5 feet bgs in the area of the creek on the east side of the site. However, the creek level is approximately 40 feet below the elevation of the site.

SEISMIC CONSIDERATIONS

The 2019 California Building Code (CBC) Seismic Design Parameters have been generated using the SEAOC/OSHPD Seismic Design Maps Tool. This web-based software application calculates seismic design parameters in accordance with ASCE 7-16 and 2019 CBC. The 2019 CBC requires that a site-specific ground motion study be performed in accordance with Section 11.4.8 of ASCE 7-16 for Site Class D sites with a mapped S_1 value greater than or equal 0.2.

However, Section 11.4.8 of ASCE 7-16 includes an exception from such analysis for specific structures on Site Class D sites. The commentary for Section 11 of ASCE 7-16 (Page 534 of Section C11 of ASCE 7-16) states that “In general, this exception effectively limits the requirements for site-specific hazard analysis to very tall and or flexible structures at Site Class D sites.” Based on our understanding of the proposed structures, it is our assumption that the

[1] California Department of Conservation Division of Mines and Geology (CDMG), *“Seismic Hazard Zone Report for the San Juan Capistrano 7.5 Minute Quadrangle, Orange County, California”*, 2001.

exception in Section 11.4.8 applies to the proposed structures for this project. However, the structural engineer should verify the applicability of this exception.

Based on this exception, the spectral response accelerations presented below were calculated using the site coefficients (F_a and F_v) from Tables 1613.2.3(1) and 1613.2.3(2) presented in Section 1613 of the 2019 CBC.

Description	Value
2019 California Building Code Site Classification (CBC) [1]	D
Site Latitude (°N)	33.53205
Site Longitude (°W)	117.67753
S_s Spectral Acceleration for a 0.2-Second Period	1.2
S_1 Spectral Acceleration for a 1-Second Period	0.431
F_a Site Coefficient for a 0.2-Second Period	1.02
F_v Site Coefficient for a 1-Second Period	1.872

1. Seismic site classification in general accordance with the *2019 California Building Code*.

Typically, a site-specific ground motion study may generate less conservative coefficients and acceleration values which may reduce construction costs. We recommend consulting with a structural engineer to evaluate the need for such study and its potential impact on construction costs. Terracon should be contacted if a site-specific ground motion study is desired.

Faulting and Estimated Ground Motions

The site is located in southern California, which is a seismically active area. The type and magnitude of seismic hazards affecting the site are dependent on the distance to causative faults, the intensity, and the magnitude of the seismic event. As calculated using the USGS Unified Hazard Tool, the San Joaquin Hills Fault, which is considered to have the most significant effect at the site from a design standpoint has a modelled earthquake magnitude of 7.53 and is located approximately 5 kilometers from the site.

Based on the SEAOC/OSHPD Seismic Design Maps Tool, using the American Society of Civil Engineers (ASCE 7-16) standard, the modified peak ground acceleration (PGA_M) at the project site is expected to be 0.563g. Based on the USGS Unified Hazard Tool, the project site has a deaggregated modal magnitude of 7.69. Furthermore, the site is not located within an Alquist-Priolo Earthquake Fault Zone based on our review of the State Fault Hazard Maps.[2]

[2] https://maps.conservation.ca.gov/cgs/informationwarehouse/regulatorymaps/.

LIQUEFACTION

Liquefaction is a mode of ground failure that results from the generation of high pore water pressures during earthquake ground shaking, causing loss of shear strength. Liquefaction is typically a hazard where loose sandy soils exist below groundwater. The California Geological Survey (CGS) has designated certain areas as potential liquefaction hazard zones. These are areas considered at a risk of liquefaction-related ground failure during a seismic event, based upon mapped surficial deposits and the presence of a relatively shallow water table.

The site is located within a State-designated Seismic Hazard Zone for liquefaction potential. A seismic hazard map is included in the **Site Location** section.

Subsurface soils generally consisted of interbedded layers of soft to hard lean clay, silt with varying amounts of sand, and silty clay. An interbedded layer of loose silty sand was encountered in borings B-2 from a depth of approximately 31½ to 40 feet bgs, respectively. Materials encountered from the Capistrano Formation were generally recovered as interbedded layers of very stiff to hard elastic silt with trace sand, lean clay with varying amounts of sand and silt, and silty clay with varying amounts of sand.

We understand that liquefaction analysis for the project will be performed by Sargent & Lundy. Terracon performed a preliminary liquefaction analysis for the site in general accordance with the DMG Special Publication 117. The liquefaction study utilized the software “LiquefyPro” by CivilTech Software. This analysis was based on soil data from the borings B-2, B-3, B-7, and B-15. A PGA_M of 0.563 g and a modal magnitude of 7.69 for the project site were used. Calculations utilized a depth to groundwater of 45 feet bgs based on review of available data and the depth to groundwater encountered in our borings. Settlement analysis used the Tokimatsu, M-correction method and the fines percentage were corrected for liquefaction using the Stark/Olson method.

Based on calculation results, seismically induced settlement of saturated and unsaturated sands was found to occur in one of the four borings (B-2). Settlement at the location of boring B-2 is estimated to be approximately ½ inch or less. The detailed liquefaction potential analysis results are attached to this report in **Supporting Documents** section of the **Appendix**.

LANDSLIDE

Based on our review of a geologic hazards map designated by the California Geologic Survey, the battery and equipment pads do not appear to be located within a mapped seismically-induced landslide zone. However, landslide deposits are mapped directly west of the proposed pads. Landslide deposits were encountered in boring B-4, B-5, B-6, and B-11 extending to approximate depths of 10 to 50 feet bgs. A seismic hazard map is included in the **Site Location** section.

It is our understanding that stability assessment of the surrounding slopes is excluded from our scope of work and is being evaluated by Sargent & Lundy (S&L) based on data provided by Terracon.

CORROSIVITY

Results of laboratory soluble sulfate, soluble chloride, electrical resistivity, and pH testing are included in the **Exploration Results** section of this report. The values may be used to estimate potential corrosive characteristics of the on-site soils with respect to contact with the various underground materials which will be used for project construction.

Corrosivity Test Results Summary						
Boring	**Sample Depth (ft)**	**USCS Material Type**	**Soluble Sulfate (ppm)**	**Chlorides (ppm)**	**Electrical Resistivity (Ω-cm)**	**pH**
B-1	0-5	CL	50	27	1,867	8.0
B-3	0-5	CL/ML	46	40	2,215	8.0
B-12	0-5	CL	64	23	1,461	8.0
B-25	0-5	CL	91	36	1,313	8.1

Results of soluble sulfate testing indicate samples of the on-site soils tested possess negligible sulfate concentrations when classified in accordance with Table 19.3.1.1 of the ACI Design Manual. Concrete should be designed in accordance with the exposure class S0 provisions of the ACI Design Manual, Section 318, Chapter 19.

STORMWATER MANAGEMENT

Eight (8) in-situ percolation tests were performed from approximate depths of 0 to 5 and 5 to 10 feet bgs. A 2-inch thick layer of gravel was placed in the bottom of each boring after the borings were drilled to investigate the soil profile. A 3-inch diameter perforated pipe was installed on top of the gravel layer in each boring. Gravel was used to backfill between the perforated pipes and the boring sidewall. The borings were then filled with water for a pre-soak period of 24 hours. Testing began after a pre-soak period. At the beginning of the test, the pipes were refilled with water and readings were taken at standardized time intervals. Percolation rates are provided in the following table:

TEST RESULTS				
Test Location (depth, feet bgs)	**Soil Classification**	**Final Measured Percolation Rate (in/hr.)**	**Correlated Infiltration Rate[1] (in/hr.)**	**Water Head (in)**
P-1 (5 – 10)	Silty clay	1.4	0.03	97
P-2 (5 – 10)	Lean clay with sand	1.2	0.04	63
P-3 (0 – 5)	Silty clay with sand	3.1	0.41	14
P-4 (5 – 10)	Silty clay with sand	3.4	0.18	36
P-5 (0 – 5)	Silty clay	5.0	0.63	14
P-6 (5 – 10)	Silty clay	1.2	0.05	44
P-7 (0 – 5)	Silty clay with sand	7.2	0.97	31
P-8 (5 – 10)	Lean clay	1.4	0.04	66

[1]If proposed infiltration system will mainly rely on vertical downward seepage, the correlated infiltration rates should be used. The infiltration rates were correlated using the Porchet method.

It is apparent that percolation rates were relatively higher in near surface soils within the upper 5 feet than the deeper soils. Infiltration within shallow systems will likely create perched water conditions on top on the underlying less permeable soils. Therefore, perched water could move laterally and manifest at the face of the descending slopes east of the site, which may cause scour and ultimately slope failures. We recommend that measures be taken to mitigate this type of occurrence, if onsite infiltration is implemented. In the event infiltration systems onsite will be utilized, the following paragraphs include design and construction considerations.

With time, the bottoms of infiltration systems tend to plug with organics, sediments, and other debris. Long term maintenance will likely be required to remove these deleterious materials to help reduce decreases in actual percolation rates.

The percolation tests were performed with clear water, whereas the storm water will likely not be clear, but may contain organics, fines, and grease/oil. The presence of these deleterious materials will tend to decrease the rate that water percolates from the infiltration systems. Design of the stormwater infiltration systems should account for the presence of these materials and should incorporate structures/devices to remove these deleterious materials. A safety factor should be applied to these measured rates.

Based on the soils encountered in our borings, we expect the percolation rates of the soils could be different than measured in the field due to variations in fines and gravel content. The design elevation and size of the proposed infiltration system should account for this expected variability in infiltration rates.

Infiltration testing should be performed after construction of the infiltration system to verify the design infiltration rates. It should be noted that siltation and vegetation growth along with other factors may affect the infiltration rates of the infiltration areas. The actual infiltration rate may vary from the values reported here. Infiltration systems should be located a minimum of 10 feet from any existing or proposed foundation system.

GEOTECHNICAL OVERVIEW

The site appears suitable for the proposed construction based upon geotechnical conditions encountered in the test borings, provided that the findings and recommendations presented in this report are incorporated into project design and construction.

The site is bounded from the east and west by steep slopes and mapped landslide potential areas. Stability assessment of the eastern and western surrounding slopes is not included in our scope of work and is being evaluated by Sargent & Lundy (S&L) based on the findings of this report.

Expansive soils are present on this site. This report provides recommendations to help mitigate the effects of soil shrinkage and expansion; however, even if these procedures are followed, some movement and at least minor cracking in the structures should be anticipated. The severity of cracking and other cosmetic damage such as uneven floor slabs will probably increase if any modification of the site results in excessive wetting or drying of the expansive soils. Eliminating the risk of movement and cosmetic distress may not be feasible, but it may be possible to further reduce the risk of movement if significantly more expensive measures are used during construction. We would be pleased to discuss other construction alternatives with you upon request.

Based on our explorations, review of geologic maps, and areas designated by the California Geologic Survey, landslide deposits were encountered in multiple borings extending to approximate depths of 10 to 50 feet bgs. Based on the provided outline of the proposed project, the landslide deposits were encountered outside the outline of the proposed structures. In the event additional structures will be constructed west of the outline of the project and within the landslide deposits area, these deposits should be removed and the excavation thoroughly cleaned prior to backfill placement and/or construction.

Batteries, transformers, and associated equipment should be supported on a mat foundation system. Mat foundations should bear on a minimum of 2 foot of engineered fill beneath the bottom of the foundations, or 4 feet below existing grades, whichever is greater. Engineered fill supporting mat foundations should comprise of low volume change materials conforming to the specifications of our ***Fill Materials and Placement*** section of this report.

It is our understanding that a proposed 138kV transmission line pole will be supported on a drilled pier.

Estimated movements described in this report are based on effective drainage for the life of the structures and cannot be relied upon if effective drainage is not maintained. Exposed ground, extending at least 10 feet from the perimeter, should be sloped a minimum of 5% away from the structures to provide positive drainage away from the structures. Grades around the structures should be periodically inspected and adjusted as part of the structure's maintenance program.

Geotechnical engineering recommendations for foundation systems and other earth connected phases of the project are outlined below. The recommendations contained in this report are based upon the results of test borings, laboratory testing, engineering analyses, and our current understanding of the proposed project. The **General Comments** section provides an understanding of the report limitations.

EARTHWORK

The following presents recommendations for site preparation, excavation, subgrade preparation, and placement of engineered fills on the project. The recommendations presented are for the design and construction of foundations and are contingent upon following the recommendations outlined in this section.

Earthwork on the project should be observed and evaluated by Terracon. The evaluation of earthwork should include observation and testing of engineered fill, subgrade preparation, foundation bearing soils, and other geotechnical conditions exposed during the construction of the project.

Site Preparation

Strip and remove existing vegetation, debris, pavements and other deleterious materials from proposed building and roadway areas. Exposed surfaces should be free of mounds and depressions which could prevent uniform compaction. The site should be initially graded to create a relatively level surface to receive fill and provide for a relatively uniform thickness of fill beneath proposed building structures.

Demolition of the existing structures should include complete removal of all foundation systems and remaining underground utilities within the proposed construction area. This should include removal of any loose backfill found adjacent to existing foundations. All materials derived from the demolition of existing structures and pavements should be removed from the site and not be allowed for use as on-site fill, unless processed in accordance with the fill requirements included in this report.

Based on our explorations, review of geologic maps, and areas designated by the California Geologic Survey, landslide deposits were encountered in multiple borings extending to approximate depths of 10 to 50 feet bgs. Based on the provided outline of the proposed project, the landslide deposits were encountered outside the outline of the proposed structures. In the event additional structures will be constructed west of the outline of the project and within the landslide deposits area, these deposits should be removed and the excavation thoroughly cleaned prior to backfill placement and/or construction.

Although no evidence of fills, utilities, or underground facilities such as septic tanks, cesspools, basements, and utilities was observed during the site reconnaissance, such features could be encountered during construction. If unexpected fills, utilities, or underground facilities are encountered, such features should be removed, and the excavation thoroughly cleaned prior to backfill placement and/or construction.

Subgrade Preparation

Strip and remove existing vegetation, debris, and other deleterious materials from proposed foundation areas.

Mat foundations should be supported on engineered fill extending 2 feet beneath the bottom of the foundations, or 4 feet below existing grades, whichever is greater. Engineered fill supporting mat foundations should consist of low volume change materials conforming to the specifications of our **Fill Materials and Placement** section of this report. The lateral extent of the overexcavation should extend a minimum of 2 foot beyond the edge of the foundation. Subsequent to the surface clearing and over-excavation efforts, the exposed subgrade soils which will support engineered fill areas constructed at grade, should be prepared to a minimum depth of 10 inches. Subgrade preparation should generally include scarification, moisture conditioning, and compaction. The moisture content and compaction of subgrade soils should be maintained until construction.

Based upon the subsurface conditions determined from the geotechnical exploration, subgrade soils exposed during construction are anticipated to be relatively workable. However, the workability of the subgrade may be affected by precipitation, repetitive construction traffic or other factors. If unworkable conditions develop, workability may be improved by scarifying and drying.

Excavation

It is anticipated that excavations for the proposed construction can be accomplished with conventional earthmoving equipment.

The bottom of excavations should be thoroughly cleaned of loose soils and disturbed materials prior to backfill placement and/or construction.

We recommend that the walls of the proposed excavations for the trenches be shored or sloped in conformance with OSHA excavation and trench safety standards. Based on the soils encountered onsite, it is our opinion that these soils can be classified as OSHA Type B or C, depending on the materials exposed during grading. If any excavation is extended to a depth of more than 20 feet, it will be necessary to have the side slopes designed by a professional engineer.

Soils from the excavation should not be stockpiled higher than six 6 feet or within ten 10 feet of the edge of an open trench. Construction of open cuts adjacent to existing structures, including underground pipes, is not recommended within a 1½ H:1V plane extending beyond and down from the perimeter of the structure.

It may be necessary for the contractor to retain a geotechnical engineer to monitor the soils exposed in all excavations and provide engineering services for slopes. This will provide an opportunity to monitor the soils encountered and to modify the excavation slopes as necessary. It also offers an opportunity to verify the stability of the excavation slopes during construction.

Individual contractors are responsible for designing and constructing stable, temporary excavations. Excavations should be sloped or shored in the interest of safety following local, and federal regulations, including current OSHA excavation and trench safety standards.

Fill Materials and Placement

All fill materials should be inorganic soils free of vegetation, debris, and fragments larger than three inches in size. Pea gravel or other open-graded materials should not be used as fill or backfill without the prior approval of the geotechnical engineer.

Due to the on-site soil's expansion potential, they are not recommended for use as engineered fill beneath foundations. Such soils may be used as fill materials for the following:

- general site grading
- exterior slabs
- roadway areas

Imported low volume change soils should be used as engineered fill supporting shallow foundations.

Imported soils for use as fill material within proposed structure areas should conform to low volume change materials as indicated in the following specifications:

Gradation	Percent Finer by Weight (ASTM C 136)
3"	100
No. 4 Sieve	50-100
No. 200 Sieve	10-30

- Liquid Limit .... 30 (max)
- Plasticity Index .... 15 (max)
- Maximum Expansion Index* .... 20 (max)

*ASTM D4829

The contractor shall notify the Geotechnical Engineer of import sources sufficiently ahead of their use so that the sources can be observed and approved as to the physical characteristic of the import material. For all import material, the contractor shall also submit current verified reports from a recognized analytical laboratory indicating that the import has a "not applicable" (Class S0) potential for sulfate attack based upon current ACI criteria and is "mildly corrosive" to ferrous metal and copper. The reports shall be accompanied by a written statement from the contractor that the laboratory test results are representative of all import material that will be brought to the job.

Engineered fill should be placed and compacted in horizontal lifts, using equipment and procedures that will produce recommended moisture contents and densities throughout the lift. Fill lifts should not exceed 10 inches loose thickness.

Compaction Requirements

Recommended compaction and moisture content criteria for engineered fill materials are as follows:

Material Type and Location	Per the Modified Proctor Test (ASTM D 1557)		
	Minimum Compaction Requirement	Range of Moisture Contents for Compaction Above Optimum	
		Minimum	Maximum
Approved imported low volume change fill soils:			
Beneath foundations:	90%	-2%	+2%
Utility trenches (structural areas)*:	90%	-2%	+2%
On-site native soils			
Beneath access roads:	95%	+1%	+4%
Utility trenches (Landscape areas):	90%	+1%	+4%
Exterior slabs:	90%	+1%	+4%
Miscellaneous backfill:	90%	+1%	+4%

* Upper 12 inches should be compacted to 95% within structural areas.

Grading and Drainage

Positive drainage should be provided during construction and maintained throughout the life of the development. Infiltration of water into utility trenches or foundation excavations should be prevented during construction. Backfill against foundations and in utility line trenches should be well compacted and free of all construction debris to reduce the possibility of moisture infiltration.

We recommend a minimum horizontal setback distance of 10 feet from the perimeter of any building and the high-water elevation of the nearest storm-water retention basin.

Utility Trenches

It is anticipated that the on-site soils will provide suitable support for underground utilities and piping that may be installed. Any soft and/or unsuitable material encountered at the bottom of excavations should be removed and be replaced with an adequate bedding material. A non-expansive granular material with a sand equivalent greater than 30 should be used for bedding and shading of utilities, unless allowed or specified otherwise by the utility manufacturer.

On-site materials are considered suitable for backfill of utility and pipe trenches from one foot above the top of the pipe to the final ground surface, provided the material is free of organic matter and deleterious substances. Imported low volume change soils should be used for trench backfill in structural areas.

Trench backfill should be mechanically placed and compacted as discussed earlier in this report. Compaction of initial lifts should be accomplished with hand-operated tampers or other lightweight compactors. If trenches are placed beneath footings, the backfill should satisfy the gradation and expansion index requirements of engineered fill discussed in this report. Flooding or jetting for placement and compaction of backfill is not recommended.

Construction Considerations

Upon completion of filling and grading, care should be taken to maintain the subgrade moisture content prior to construction. Construction traffic over the completed subgrade should be avoided to the extent practical. The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. If the subgrade should become desiccated, saturated, or disturbed, the affected material should be removed, or these materials should be scarified, moisture conditioned, and recompacted prior to construction.

On-site clay and silt soils may pump and unstable subgrade conditions could develop during general construction operations, particularly if the soils are wetted and/or subjected to repetitive construction traffic. The use of light construction equipment would aid in reducing subgrade disturbance. The use of remotely operated equipment, such as a backhoe, would be beneficial to perform cuts and reduce subgrade disturbance.

Should unstable subgrade conditions develop stabilization measures will need to be employed. Stabilization measures may include placement of aggregate base and multi-axial geogrid. Use of lime, fly ash, kiln dust or cement could also be considered as a stabilization technique. Laboratory evaluation is recommended to determine the effect of chemical stabilization on subgrade soils prior to construction.

We recommend that the earthwork portion of this project be completed during extended periods of dry weather if possible. If earthwork is completed during the wet season (typically November through April) it may be necessary to take extra precautionary measures to protect subgrade soils. Wet season earthwork operations may require additional mitigative measures beyond that which would be expected during the drier summer and fall months. This could include diversion of surface runoff around exposed soils and draining of ponded water on the site. Once subgrades are established, it may be necessary to protect the exposed subgrade soils from construction traffic.

Construction Observation and Testing

The exposed subgrade and each lift of compacted fill should be tested, evaluated, and reworked, as necessary, until approved by the geotechnical engineer's representative prior to placement of additional lifts. We recommend that each lift of fill be tested for density and moisture content at a frequency of one test for every 2,500 square feet of compacted fill in the structural areas. We recommend one density and moisture content test for every 50 linear feet of compacted utility trench backfill. This testing frequency criteria may be adjusted during construction as allowed by the geotechnical engineer of record.

Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean Terracon is assuming responsibility for construction site safety, or the contractor's activities; such responsibility shall neither be implied nor inferred.

The geotechnical engineer should be retained during the construction phase of the project to observe earthwork and to perform necessary tests and observations during subgrade preparation; proof-rolling; placement and compaction of controlled compacted fills; backfilling of excavations to the completed subgrade.

FOUNDATIONS

The proposed batteries, transformers, and associated equipment may be supported on mat foundations bearing on engineered fill. Recommendations for foundations for the proposed structures and related structural elements are presented in the following paragraphs.

If the site has been prepared in accordance with the requirements noted in **Earthwork**, the following design parameters are applicable for shallow foundations.

Mat Foundation Design Recommendations

DESCRIPTION	RECOMENDATION
Foundation Type	Mat foundations
Bearing Material[3]	A minimum 2-foot of engineered fill beneath the bottom of the foundations, or 4 feet below existing grades, whichever is greater. Low volume change materials should be used as engineered fill for support of proposed mat.
Allowable Bearing Pressure[1,7]	1-inch settlement ■ 1,800 psf for mat foundation (Up to 10 feet wide) ■ 1,000 psf for mat foundation (Up to 20 feet wide) ■ 800 psf for mat foundation (Up to 30 feet wide) 2-inch settlement ■ 4,000 psf for mat foundation (Up to 10 feet wide) ■ 2,700 psf for mat foundation (Up to 20 feet wide) ■ 1,900 psf for mat foundation (Up to 30 feet wide)
Minimum Foundation Width	2 feet
Ultimate Coefficient of Sliding Friction [4]	0.30
Minimum Embedment Depth Below Finished Grade	12 inches
Estimated Total Settlement from Structural Loads [2]	See **Allowable Bearing Pressure**
Estimated Differential Settlement[2,6]	½ of total settlement

1. The maximum net allowable bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation. An appropriate factor of safety has been applied.
2. Unsuitable or loose/soft, dry, and low-density soils should be removed and replaced per the recommendations presented in the **Earthwork**.
3. Use of passive earth pressures require the sides of the excavation for the spread footing foundation to be nearly vertical and the concrete placed neat against these vertical faces or that the footing forms be removed and compacted structural fill be placed against the vertical footing face.
4. Can be used to compute sliding resistance where foundations are placed on suitable soil/materials. Should be neglected for foundations subject to net uplift conditions.
5. For sloping ground, maintain depth below the lowest adjacent exterior grade within 5 horizontal feet of the structure. The designer should select an appropriate factor of safety during design.
6. Differential settlements are as measured over a span of 40 feet.
7. Maximum width is based on settlement analysis with allowable settlement of 1 and 2 inches.

Settlement calculations were performed utilizing Westergaard and Hough's methods[5] to estimate the static settlement for various foundation widths with an allowable settlement of 1 and 2 inches.

[5] FHWA Geotechnical Engineering Circular No. 6 – Shallow Foundations, FHWA-SA-02-054.

Since there are several factors that will control the design of mat foundations besides vertical load, Terracon should be consulted when the final foundation depth and width are determined to assist the structural designer in the evaluation of anticipated settlement.

For structural design of mat foundations, a modulus of subgrade reaction (Kv_1) of 120 pounds per cubic inch (pci) may be used. Other details including treatment of loose foundation soils, superstructure reinforcement and observation of foundation excavations as outlined in the Earthwork section of this report are applicable for the design and construction of a mat foundation at the site.

The subgrade modulus (K_v) for the mat is affected by the size of the mat foundation and would vary according the following equation:

$$K_v = K_{v1}/B$$

Where: K_v is the modulus for the size footing being analyzed
B is the width of the mat foundation.

Shallow Foundation Design Considerations

Finished grade is defined as the lowest adjacent grade within five feet of the foundation for perimeter (or exterior) footings.

The allowable foundation bearing pressure applies to dead loads plus design live load conditions. The design bearing pressure may be increased by one-third when considering total loads that include wind or seismic conditions. The weight of the foundation concrete below grade may be neglected in dead load computations.

Foundations should be reinforced as necessary to reduce the potential for distress caused by differential foundation movement. The use of joints at openings or other discontinuities in masonry walls is recommended.

Foundation excavations should be observed by the geotechnical engineer. If the soil conditions encountered differ significantly from those presented in this report, supplemental recommendations will be required.

DEEP FOUNDATIONS

The proposed transmission tower can be supported on drilled shafts. The location of the proposed transmission tower was not available at the time of preparation of this report. Based on correspondence with S&L, we understand that the transmission tower may be situated within the central area of the site. Design recommendations for foundations for the proposed structures and related structural elements are presented in the following paragraphs.

Total required embedment of the drilled shafts should be determined by the structural engineer based on structural loading and parameters provided in this report.

Drilled Shaft Axial Capacity

The allowable axial capacity for a range of drilled shafts diameters was evaluated and is presented in the graph provided in the **Supporting Documents** section of this report. The allowable total downward (compressive) capacity is based on a factor of safety of 2.5 for side resistance and 3.0 for end bearing. The analysis considered the depth to top of shaft to be 2 feet below existing ground surface. The depth below ground surface indicated in the graphs is referenced from the existing ground surface at the site at the time of the field exploration. The capacity presented is based on a minimum shaft spacing of 3 shaft diameters. Allowable tension capacity may be taken as 60 percent of the allowable compressive capacity, plus the weight of the shaft. Tensile reinforcement should extend to the bottom of shafts subjected to uplift loading.

Drilled Shaft Lateral Capacity

The required depths of shaft embedment should also be determined for design axial loads, lateral loads, and overturning moments to determine the most critical design condition. To support the designer, parameters for use in MFAD software have been tabulated and are presented in the following table.

MFAD 5.0 Recommended Engineering Properties of Soils				
Top Depth Bottom Depth	**Effective Unit Weight (pcf)**	**MFAD Soil Group**	**Cohesion (psf)**	**MFAD Modulus of deformation (ksi)**
2 10	115	CLAY	1,000	0.65
10 30	120	CLAY	1,000	0.65
30 45	120	CLAY	2,000	1.30
45 50	63	CLAY	3,000	1.95

It should be noted that the load capacities provided herein are based on the stresses induced in the supporting soils. The structural capacity of the shafts should be checked to assure that they can safely accommodate the combined stresses induced by axial and lateral forces. Furthermore, the response of the drilled shaft foundations to lateral loads is dependent upon the soil/structure interaction as well as the shaft's actual diameter, length, stiffness and "fixity" (fixed or free-head condition).

Lateral and axial capacity of soils within the upper 2 feet should be neglected due to utilities and disturbance around piers. We recommend that Terracon review the final drilled shaft design to verify that sufficient embedment is achieved.

Drilled Shaft Construction Considerations

All shafts should be reinforced full-depth for the applied axial, lateral and uplift stresses imposed. If multiple shafts are proposed at the transmission tower location, special sequencing of drilled shaft construction should be specified when the center to center spacing between adjacent shafts is less than three diameters. A minimum of 24 hours should be allowed between placement of concrete and initiation of drilling in shafts less than five diameters (center to center spacing) apart from each other.

Drilling to design depths should be possible with conventional single flight power augers. Formation of mushrooms or enlargements at the tops of shafts should be avoided during shaft drilling. If mushrooms develop at the tops of the shafts during drilling, sono-tubes should be placed at the shaft tops to help isolate the shafts.

Groundwater was encountered in some of the exploratory borings. Therefore, seepage or groundwater may be encountered during drilling for the shafts. To control groundwater seepage, the use of temporary steel casing and/or slurry drilling procedures may be required for construction of the drilled shaft foundations. The drilled shaft contractor and foundation design engineer should be informed of these risks.

If shafts are constructed below the groundwater level, the "Wet" shafts should be constructed by slurry displacement techniques. In this process, the shaft excavation is filled with approved polymer-based slurry to counter-balance the hydraulic forces below the water level and stabilize the wall of the shaft. Concrete would then be placed using a tremie extending to within 6 inches of the shaft base of the slurry-filled excavation. The tremie remains inserted several feet into the fresh concrete as it displaces the slurry upward and until placement is complete. The slurry should have a sand content no greater than 1% at the time concrete placement commences. The maximum unit weight of the slurry should be established in consultation with Terracon.

For drilled shaft depths above the depth of groundwater, temporary steel casing may be required to properly drill and clean shafts prior to concrete placement. If disturbed soils are present at the bottom of the drilled shafts, the sloughed materials must be removed, and bottom should be cleaned.

If casing is used for foundation construction, it should be withdrawn in a slow continuous manner maintaining a sufficient head of concrete to prevent caving or the creation of voids in pier concrete. Foundation concrete should have a relatively high fluidity when placed in cased pier holes or through a tremie. Foundation concrete with slump in the range of 6 to 8 inches is recommended when temporary casing is utilized.

Foundation concrete should be placed immediately after completion of drilling and cleaning. If foundation concrete cannot be placed in dry conditions, a tremie should be used for concrete placement. Due to potential sloughing and raveling, foundation concrete quantities may exceed calculated geometric volumes.

Free-fall concrete placement in drilled shafts will only be acceptable if provisions are taken to avoid striking the concrete on the sides of the hole or reinforcing steel. The use of a bottom-dump hopper, or an elephant's trunk discharging near the bottom of the hole where concrete segregation will be minimized, is recommended.

Drilled shaft bearing surfaces must be cleaned prior to concrete placement. A representative of the geotechnical engineer should observe the bearing surface and foundation shaft configuration. If the subsurface soil conditions encountered differ significantly from those presented in this report, supplemental recommendations may be required.

We recommend that all drilled shaft installations be observed on a full-time basis by an experienced geotechnical engineer in order to evaluate that the soils encountered are consistent with the recommended design parameters.

The contractor should check for gas and/or oxygen deficiency prior to any workers entering the excavation for observation and manual cleanup. All necessary monitoring and safety precautions as required by OSHA, State or local codes should be strictly enforced.

LATERAL EARTH PRESSURES

Design Parameters

For engineered fill comprised of on-site soils or imported low volume change materials (required behind retaining walls) above any free water surface, recommended equivalent fluid pressures for unrestrained foundation elements are:

ITEM	VALUE[a, b]
Active Case	38 psf/ft[c]
Passive Case	375 psf/ft
At-Rest Case	58 psf/ft
Surcharge Pressure	0.32 x (Surcharge)
Coefficient of Friction	0.35

[a]Note: The values are based on low volume change engineered fill materials used as backfill behind retaining walls.
[b]Note: Uniform, horizontal backfill, compacted to at least 90% of the ASTM D 1557 maximum dry density, rendering a maximum unit weight of 120 pcf.
[c]Note: Earth pressures should be increased by 40 percent for a slope of 2H:1V behind walls and should be increased by 120 percent for a slope of 1.5H:1V.

The lateral earth pressures herein do not include any factor of safety and are not applicable for submerged soils/hydrostatic loading. Additional recommendations may be necessary if such conditions are to be included in the design.

Total lateral earth pressures acting on the wall, where it is retaining greater than 6 feet, during a seismic event should include the active static forces and a dynamic increment. The active dynamic increment should be applied to unrestrained walls as a resultant force acting at 0.6H height from the base of the wall. Such increment should be added to the static earth pressures. The dynamic lateral earth resultant force (for a 0.56g peak ground acceleration estimated based on the current 2019 California Building Code) is $13H^2$ (in units of pounds per linear foot (plf), where H (in units of feet) is the height of the soil behind the wall.

Adequate drainage should be provided behind the retaining walls to collect water from irrigation, landscaping, surface runoff, or other sources, to achieve a free-draining backfill condition. The wall back drain should consist of Class 2 permeable materials[3] that are placed behind the entire wall height to within 18 inches of ground surface at the top of the wall. As a minimum, the width of Class 2 permeable materials behind the wall should be two feet. As an alternative, drainage panels/mats may be used in lieu of the Class 2 permeable materials. Water collected by the back drain should be directed to an appropriate outlet, such as perforated pipes or weep holes, for disposal.

Fill against foundation and retaining walls should be compacted to densities specified in the Earthwork section of this report. Compaction of each lift adjacent to walls should be accomplished with hand-operated tampers or other lightweight compactors.

[3] In accordance with the requirements and specifications of the State of California Department of Transportation.

The design of any shoring system should consider surcharge loads imposed by the existing structures and vehicular loads in the vicinity of the shoring. In general, surcharge loads should be considered where they are located within a horizontal distance behind the shoring equal to the height of the shoring.

Surcharge loads acting at the top of the shoring should be applied to the shoring over the backfill as a uniform pressure over the entire shoring height and should be added to the static earth pressures. Surcharge stresses due to point loads, line loads, and those of limited extent, such as compaction equipment, should be evaluated using elastic theory.

The design of the shored excavation should be performed by an engineer knowledgeable and experienced with the on-site soil conditions. The contractor should be aware that slope height, slope inclination or excavation depths should in no case exceed those specified in local, state or federal safety regulations, e.g. OSHA Health and Safety Standards for Excavation, 29 CFR Part 1926, or successor regulations. Such regulations are strictly enforced and, if not followed, the owner or the contractor could be liable for substantial penalties.

ACCESS ROADWAYS

Compacted Native Soils Access Road Design Recommendations

Based upon the soil conditions encountered in the test borings, the use of on-site soils for construction of on-site roads is considered acceptable. Without the use of asphalt concrete or other hardened material to surface the roadways, there is an increased potential for erosion and deep rutting of the roadway to occur, however, post construction traffic is anticipated to only consist of pickup trucks for operations and maintenance personnel. Therefore, construction of the un-surfaced native roadways should consist of a minimum of 10-inches of compacted on-site soils.

It is our understanding that proposed compacted native roadway grades will match adjacent existing grades so that the existing natural drainage patterns are generally unchanged. The un-surfaced roads are expected to function with periodic maintenance.

Aggregate Surface Roadway Design Recommendations

Aggregate surface roadway design was conducted in general accordance with the Army Corps of Engineers (ACOE) Technical Manual TM-5-822, Design of Aggregate Surface Roads and Airfields (1990). The design was based on Category III, traffic containing as much as 15% trucks, but with not more than 1% of the total traffic composed of trucks having three or more axles (Group 3 vehicles), and Road Class G (Under 70 vehicles per day). This assumed traffic loading is for the operations of the proposed facility but not for construction traffic. Based on the Category and Road Class, a Design Index of 1 was utilized, along with a correlated CBR of 5. Terracon should be contacted if significant changes in traffic loads or in the characteristics described are anticipated.

As a minimum, the aggregate surface course should have a minimum thickness of 6 inches and should be constructed over a minimum of 10 inches of scarified, moisture conditioned, and compacted native soils to 90% of the maximum dry density using ASTM D 1557. The recommended thicknesses should be measured after full compaction. The width of the roadway should extend a minimum distance of 1 foot on each side of the desired surface width.

It is our understanding that aggregate surfaced roads and parking areas will be utilized during the construction of this project. Aggregate materials should conform to the specifications of Class II aggregate base in accordance with the requirements and specifications of the State of California Department of Transportation (Caltrans), or other approved local governing specifications.

Positive drainage should be provided during construction and maintained throughout the life of the roadways. Proposed roadway design should maintain the integrity of the road and eliminate ponding.

Roadway Design and Construction Considerations

Regardless of the design, un-surfaced roadways will display varying levels of wear and deterioration. We recommend an implementation of a site inspection program at a frequency of at least once per year to verify the adequacy of the roadways. Preventative measures should be applied as needed for erosion control and re-grading. An initial site inspection should be completed approximately three months following construction.

Preventative maintenance should be planned and provided for through an on-going management program to enhance future roadway performance. Preventative maintenance activities are intended to slow the rate of deterioration, and to preserve the roadway investment.

Surfacing materials should not be placed when the surface is wet. Surface drainage should be provided away from the edge of roadways to reduce lateral moisture transmission into the subgrade.

If rut depths become excessive as construction work progresses, re-grading and re-compaction should be performed as necessary. Care should be taken to reduce or eliminate trafficking of the unpaved access road when the subgrade is wet as this will result in accelerated rutting conditions. Scarification, moisture treatment as necessary, and re-compaction of the roadways will likely be necessary as the roadways deteriorate.

Materials and construction of roadways for the project should be in accordance with the requirements and specifications of the California Department of Transportation or the applicable local governing body.

GENERAL COMMENTS

Our analysis and opinions are based upon our understanding of the project, the geotechnical conditions in the area, and the data obtained from our site exploration. Natural variations will occur between exploration point locations or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. Terracon should be retained as the Geotechnical Engineer, where noted in this report, to provide observation and testing services during pertinent construction phases. If variations appear, we can provide further evaluation and supplemental recommendations. If variations are noted in the absence of our observation and testing services on-site, we should be immediately notified so that we can provide evaluation and supplemental recommendations.

Our Scope of Services does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

Our services and any correspondence or collaboration through this system are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and are accomplished in accordance with generally accepted geotechnical engineering practices with no third-party beneficiaries intended. The findings and recommendations presented in this report were prepared in a manner consistent with the standards of care and skill ordinarily exercised by members of its profession completing similar studies and practicing under similar conditions in the geographic vicinity and at the time these services have been performed. Any third-party access to services or correspondence is solely for information purposes to support the services provided by Terracon to our client. Reliance upon the services and any work product is limited to our client and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly impact excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety, and cost estimating including, excavation support, and dewatering requirements/design are the responsibility of others. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing.