Appendix D

Updated Geotechnical/Geologic Investigation Report (April 15, 2019)

PARTNER

UPDATED GEOTECHNICAL / GEOLOGIC INVESTIGATION REPORT

Encompass Health Hospital Site
517 Shinohara Lane
Chula Vista, California 91911

April 15, 2019
Partner Project Number: 17-199602.7

Prepared for:

Encompass Health
9001 Liberty Parkway
Encinitas, California 92024

Engineers who understand your business

PARTNER

April 15, 2019

Kellye Rohrabaugh
Encompass Health
9001 Liberty Parkway

Subject: **Updated Geotechnical and Geologic Investigation Report**
Encompass Health Hospital Site
512 Shinohara Lane
Chula Vista, California 91911
Partner Project No. 17-199602.7

Dear Kellye Rohrabaugh:

Partner Assessment Corporation (Partner) presents the following updated geotechnical/geologic investigation report based on our general experience with construction practices and geologic/geotechnical conditions on this and other sites. This report is in accordance with the proposal (#199602) dated 7/6/2018, approved by Kellye Rohrabaug of Encompass Health and also was later revised based on proposal (#199602) dated 12/17/2018, approved by John Tschudin of Encompass Health.

The descriptions and findings of our geotechnical report are presented for your use in this electronic format, for your use as shown in the hyperlinked outline below. To return to this page after clicking a hyperlink, hold “alt” and press the “left arrow key” on your keyboard.

We appreciate the opportunity to be of service during this phase of the work.

Sincerely,

Matthew Marcus, PE
Principal Engineer

Razi Quraishi, PG, CEG
Certified Engineering Geologist

Francisca Chan, EIT
Project Engineer

1. GEOTECHNICAL/GEOLOGIC EXECUTIVE SUMMARY

Geologic Zones and Site Hazards:

According to the report*: Regionally the site is located in Peninsular Ranges Geomorphic Province. The Peninsular Ranges Province is traversed by a group of sub-parallel faults and fault zones trending roughly northwest. Several of these faults are major active faults (Rose Canyon, Elsinore, San Jacinto and Newport – Inglewood). Undivided sediments/sedimentary rocks and San Diego Formation occurs within the regional area of the site. The subject property is currently vacant and undeveloped since 1904. Substantial grading, drainage improvements and hydro-seed applications occurred on the northern slopes in 2007. Surficial geology consists of topsoil and artificial fill, overlying residual weathered bedrock (San Diego Formation). The site is in an area where the seismic hazard potential was not evaluated by the State of California, and the historic groundwater levels were not provided by the California Department of Conservation. Based on our evaluation the slopes on the site are stable with regards to landsliding and slope stability. Given the seismic activity in the region we anticipate low to moderate ground shaking during the project life. No other geologic hazards are known or suspected on the project.

Excavation Conditions:

According to the report*: We anticipate extensive grading will be needed on the site to establish the finished grades for the new buildings. We anticipate cut slopes on the order of 20 feet or more on the north end of the property. The stability of the slopes during and after construction have been evaluated and will call for special considerations during construction. In general, the borings encountered soil that would be excavatable using conventional construction equipment in good working condition. However, hard digging conditions may be encountered on the norther portion of the site. Loose fill soils and native sandy soils may be prone to caving during excavation. Groundwater was not encountered during drilling; however, groundwater levels can fluctuate over time.

Foundation/Slab Support:

According to the report*: The upper 1 to 6 feet of soil encountered in our explorations consisted of artificial fill material, debris and plant material. Some debris and deleterious inclusions (paper bags, household garbage, etc.) were noted in the fill. Where present in new building or fill embankment areas, the fill and other deleterious/organic materials should be completely removed to expose clean, competent native soil. Spread foundations should be considered for the new hospital building. The foundations can be supported on engineered fill and/or competent, clean native soil compacted in-place, as described in the report. Slab-on-grade areas should be supported on non-expansive engineered fill extending to competent native soils that are approved by the engineer.

Mass Grading and Soil Reuse:

According to the report*: Site soils are generally expected to be usable as engineered fill on the site, after stripping/grubbing of organic material and disposal of trash, topsoil and debris. The native soil encountered had a relatively low in-place density. As such, we anticipate that volume loss of cut materials will occur after moisture conditioning and compaction, on the order of 15% to 25%. New fills of up to 20 feet in height to be placed on existing slopes should be benched and keyed per CBC requirements. It is recommended to use non-expansive structural fill that is free of deleterious materials, and is properly moisture conditioned and compacted to 95% of the modified proctor (ASTM D 1557) is recommended.

Pavement Design: According to the report*:

Roadway Type	***Subgrade Preparation***	***Pavement Section***
Parking Area Light Duty (TI=4)	Compacted Subgrade	3-in asphalt & 6-in aggregate base
Parking Area Heavy Duty (TI=7)	Compacted Subgrade	6-in concrete & 4-in aggregate base

This summary in no way replaces or overrides the detailed sections of the report*

2. REPORT OVERVIEW & LIMITATIONS

2.1 Report Overview

To develop this report, Partner accessed existing information and obtained site specific data from our exploration program. Partner also used standard industry practices and our experience on previous projects to perform engineering analysis and provide recommendations for construction along with construction considerations to guide the methods of site development. The opinions on the cover letter of this report do not constitute engineering recommendations, and are only general, based on our recent anecdotal experiences and not statistical analysis. Section 1.0, Executive Geotechnical Summary, compiles data from each of the report sections, while each of sections in the report presents a detailed description of our work. The detailed descriptions in Sections 4, 5, 6, 7 and 8 and Appendix A to address slope stability findings and Appendix D constitute our engineering recommendations for the project, and they supersede the Executive Geotechnical Summary.

The report overview, including a description of the planned construction and a list of references, as well as an explanation of the report limitations is provided in Section 2.0. The findings of Partner's geologic review are included in Sections 4.0 and 5.0, Geologic Conditions and Hazards. The descriptions of our methods of exploration and testing, as well as our findings are included in Section 7.0. In addition, logs of our trench excavations are included in Appendix A, Boring Logs are included in Appendix B, and geotechnical laboratory testing is included in Appendix C of the report. Site Location and Site Investigation Plan are included as Figures 1 and 2 in the report.

2.2 Assumed Construction

Partner's understanding of the planned construction was based on information provided by the project team. The proposed site plan is included as Figure 2 to this report. Partner's assumptions regarding the new construction are presented in the below table.

Property Data	
Property Use:	Encompass Health Hospital Site
Building footprint/height	One story above grade, roughly 130,000 sf
Land Acreage (Ac):	Approx. 9.6 Ac, APN 644-040-01-00
Number of Buildings:	1
Expected Cuts and Fills	Unknown
Type of Construction:	Unknown, assumed slab-on-grade with metal framing
Foundations Type	Unknown, assumed shallow foundations
Anticipated Loads	2,000 to 3,000 psf
Traffic Loading	Parking lot and loading dock
Site Information Sources:	APD Consultants, Conceptual Project Plans, 3/7/2019.

2.3 References

The following references were used to generate this report:

California Building Code IBC 2009 and ASCE 7-10

California Geological Survey, Note 36, *California Geomorphic Provinces*, 2002.

California Geological Survey Topographic Map 2015, 7.5 Minute series, *Imperial Beach, CA*, accessed via internet, accessed 1/24/18

Federal Emergency Management Agency, FEMA Flood Map Service Center, accessed 1/24/18

Federal Highway Administration, Rock Slope Engineering, 1979

Google Earth Pro (Online), accessed 1/24/18

Geologic Map of the San Diego Quadrangle, Regional Geologic Map No. 3, 1: Kennedy and Tan, 2008.

Geotechnical Engineering Portable Handbook, Robert W. Day, 2000

Historic Aerials by NETR Online, accessed 1/24/18

Naval Facilities Engineering Command, NAVFAC DM 7.1-.3, Design Manual, Soil Mechanics and Foundations, May 1982, April 1983.

Partner Engineering and Science, Inc., Phase 1 Environmental Assessment Report, *Industrial Land, 517 Shinohara Lane, Chula Vista*, *California,* dated February 1, 2018.

Partner Engineering and Science, Inc., Preliminary Geotechnical Report, *Industrial Land, 517 Shinohara Lane, Chula Vista*, *California,* dated January 16, 2018.

Willian A. Steen & Associates, Otay Valley Industrial Park (Phase 1), As Built, 517 Shinohara Lane, San Diego, CA, dated 10-31-07.

United States Geological Survey, Lower 48 States 2014 Seismic Hazard Map, accessed online 1/24/18

United States Geologic Survey, Earthquake Hazards Program (Online), accessed 1/24/18

2.4 Limitations

The conclusions, recommendations, and opinions in this report are based upon soil samples and data obtained in widely spaced locations that were accessible at the time of exploration, and collected based on project information available at that time. Our findings are subject to field confirmation that the samples we obtained were representative of site conditions. If conditions on the site are different than what was encountered in our borings, the report recommendations should be reviewed by our office, and new recommendations should be provided based on the new information and possible additional exploration if needed. It should be noted that geotechnical subsurface evaluations are not capable of predicting all subsurface conditions, and that our evaluation was performed to industry standards at the time of the study, no other warranty or guarantee is made.

Likewise, our document review and geologic research study made a good-faith effort to review readily available documents that we could access and were aware of at the time, as listed in this letter. We are not able to guarantee that we have discovered, observed, and reviewed all relevant site documents and conditions. If new documents or studies are available following the completion of the report, the recommendations herein should be reviewed by our office, and new recommendations should be provided based on the new information and possible additional exploration if needed.

This report is intended for the use of the client in its entirety for the proposed project as described in the text. Information from this report is not to be used for other projects or for other sites. All of the report must be reviewed and applied to the project or else the report recommendations may no longer apply. If pertinent changes are made in the project plans or conditions are encountered during construction that appear to be different than indicated by this report, please contact this office for review. Significant variations may necessitate a re-evaluation of the recommendations presented in this report. The findings in this report are valid for one year from the date of the report. This report has been completed under specific Terms and Conditions relating to scope, relying parties, limitations of liability, indemnification, dispute resolution, and other factors relevant to any reliance on this report. Any parties relying on this report do so having accepted Partner's standard Terms and Conditions, a copy of which can be found at http: / www.partneresi.com/terms-and-conditions.php

If parties other than Partner are engaged to provide construction geotechnical services, they must be notified that they will be required to assume complete responsibility for the geotechnical phase of the project by concurring with the findings and recommendations in this report or providing alternate recommendations.

3. SITE LOCATION AND PROJECT INFORMATION

3.1 Site Location and Project Information

The planned construction will be situated on a currently undeveloped parcel in Chula Vista, California. The immediately surrounding properties consist of light industrial buildings and residential buildings. Figure 2 presents the project site and the locations of our site exploration. Based on our review of available documents, the site has had the following previous uses:

Historical Use Information		
Period/Date	**Source**	**Description/Use**
1904-1995	Aerial Photographs, Topographic Maps, City Directories, Onsite Observations	Undeveloped Land
1995-Present	Aerial Photographs, Topographic Maps, City Directories, Onsite Observations	Some site improvements: grading, drainage, hydroseeding

4. GEOLOGIC FINDINGS

This section presents the results of a geologic review performed by Partner, for a proposed new construction on site. The general location of the project is shown on Figure 1.

4.1 Regional Geology

Regionally the site is located in Peninsular Ranges Geomorphic Province. The Peninsular Ranges Province is traversed by a group of sub-parallel faults and fault zones trending roughly northwest. Several of these faults are major active faults (Rose Canyon, Elsinore, San Jacinto and Newport – Inglewood). Undivided sediments/sedimentary rocks and San Diego Formation occurs within the regional area of the site. The province varies in width from approximately 30 to 100 miles. The western portion of the province, which includes the project area, consists generally of dissected coastal plain underlain by upper Cretaceous, Tertiary rocks and Quaternary sediments, very old Pleistocene marine and non-marine terrace deposits an bedrock of early Pleistocene and late Pliocene of San Diego Formation.

The Regional Geologic Maps are included in Figures 3 and 4.

Summary of Geologic Data		
Parameter	**Value**	**Source**
Geomorphic Zone	Peninsular Ranges	CGS, Geology of California
Site Ground Elevation Range	140 to 255 feet above MSL	USGS and Site Topographic Survey
Flood Elevation	Zone X (Minimal Flood Hazard)	FEMA
Seismic Hazard Zone	Low to Moderate	USGS and CGS
Geologic Hazards	Low Density Sandy Silty Soils	CGS/ Lab Results
Surface Cover	Artificial Fill/San Diego Formation	Geotechnical/Geologic Investigation
Site Modifications	Previously graded; seed soil type	Google Earth
Surficial Geology	Artificial Fill (AF)/San Diego Formation (Tsdss)	USGS, California Geologic Survey, Geologic Map of San Diego Quadrangle, Site Geologic Mapping
Depth to Residual Soils/ Weathered San Diego Formation	1.5 to 6.0 feet (Approximately)	Boring Logs/ Trenches/ Site Geologic Mapping
Approximate Groundwater Depth	45 to 85 feet	Partner ESA

4.2 Site Engineering Geology and Subsurface Conditions

The site geology and subsurface conditions have been summarized in this section from available geologic data, geologic mapping (Figure 5) and previous subsurface investigations consisting of exploratory six soil borings performed on January 25, 2018 (B-1,B-2, B-3, B-4 ,B-5 and B-6) and four exploratory trenches (TP-1,TP-2,TP-3 and TP-4) are shown at location in Figure 2. Additional borings were performed on February 12, 2019 (B-7, B-10 , B-12, B-13, B-14, B-15) and also continuous core borings on March 15, 2019 (B8-A, B11-A, B16-A).

Trench logs are provided in Appendix A. The soil boring and continuous core logs are provided in Appendix B. The subject property is located approximately at elevation 145 feet to 250 feet above MSL, in an area of sloping topographic relief sloping generally to the south and south east.

Generalized geologic cross sections A-A' and B-B' and C-C' are included in Figure 6, 7 and 8 respectively. Top soil was observed on the scattered areas of the site in varying thickness from 0.5 feet to 2.5 feet. The site is mapped to be underlain by artificial fill (AF) varying in thickness from approximately 1.0 feet to 6.0 feet. The fill generally consists of orangish brown fine to coarse sand, some silt and clay, fine to coarse gravel and cobbles.

Artificial Fill (AF) is underlain by bedrock of early Pleistocene and late Pliocene San Diego Formation (Tsdss). San Diego Formation (silty sandstone) consists of yellowish brown to whitish gray, slightly micaceous, silty fine sand (unified soil classification symbol "SM"), or slightly micaceous, medium dense to dense, moderately weathered grey fine sand, little silt ("SP-SM"). Exploratory trenches indicated the San Diego formation is poorly bedded. The San Diego Formation exhibits low angle, faint bedding dips approximately 4 to 5 degrees towards southwest and strikes approximately N 20 to 25 degrees northwest. The strikes and dips generally co-relates with the regional dip.

4.3 Groundwater and Caving

No active surface ground water seeps or springs were observed at the project site. Subsurface water was not encountered during our field exploration to maximum excavated/drilled depth of 50 feet below existing grade. Trench walls were stable during and after excavation.

However, based on data on an adjacent site, groundwater is approximated around 40-85 feet below ground surface. Seasonal and long-term fluctuations in the groundwater may occur as a result in variations in subsurface conditions, rainfall, run-off conditions and other factors. Therefore, variations from our observations may occur.

4.4 Slope Stability Analysis

Regional Geologic and Site Engineering Geologic Maps (Figures 4 and 5) and Seismic Hazards Map (Figure 9) indicated the site is not located in the landslide area. Site Geologic mapping indicated the residual soils/San Diego Formation slopes are stable. In addition, Partner performed global slope stability analysis of four site cross-sections which had planned retaining walls of 6 feet or higher at the base of soil slopes. The slopes were evaluated for global stability (circular failure) using Bishop and Janbu methods, and soil parameters determined from direct shear testing of relatively "undisturbed" site soils obtained during drilling in a California modified split-spoon sampler. The parameters used were a cohesion of 100 psf and friction angle of 30 degrees. The slope stability cross sections are shown in Appendix D, and the output of the Slide 2d Software models are shown in Appendix E.

Factors of safety in three of the sections were 1.5 or greater with normally sized and embedded foundations. Cross-section H-H', located on the north side of the project includes a roughly 40-ft high cut slope with a 13-ft high retaining wall at its base. This section did not have a 1.5 factor of safety with normally sized and embedded foundation. As such, we recommend that the retaining wall in this location have a cantilevered foundation embedded 4 feet below grade, and that extends 7.5 feet from the centerline of the wall, where wall heights are higher than 6 feet.

In addition, seismic stability analysis was performed on the slopes, based on a maximum horizontal acceleration of 0.375 g for soft rock (site class C) conditions. Based on the information in California SP 117, the K_{eq} factor was 0.5 x .375 for an M 7 earthquake event. As such, a K_{eq} factor of 0.19 was used for the site. The minimum factor of safety determined by this method was 1.06, which is acceptable per California SP 117.

Slope stability analysis at the northern slopes (Location STA #1, Figure 5) indicates the slopes are stable with a calculated factor of safety of 2.58 which is greater than the normally accepted minimum for stable slopes. Slope stability analysis was also conducted at the western areas (Location STA #2 and STA #3, Figure 5) indicated the disturbing forces tending to cause the block to slide down becomes negative. The bedding angle is greater than the slope angle. The bedding dips beneath the slope and the slopes are stable. Slope soil properties and Factor of Safety calculation are included in Appendix A.

All slopes will be subjected to surficial erosion. Therefore, slopes should be protected from surface runoff by means of top of the slopes compacted earth berms.

It is recommended that the slopes should be properly maintained in future by some of these methods: cleaning and removing loose debris, minor grading, controlling surface water, revegetation and by constructing benches. Over- watering and subsequent saturation of slope surface should be avoided.

4.5 Faulting and Seismicity

The subject site is in San Diego County of Southern California. Like the rest of Southern California, it is in a seismically active region. This region is located near the active margin between the North American and Pacific tectonic plates. The seismicity is due to movement along the regional active faults such as the San Andreas, Rose Canyon, Newport-Inglewood, Elsinore and San Jacinto.

According to the State Mining and Geology Board, an active fault is defined which has had surface displacement within the Holocene Epoch (roughly within the last 11,000 years). The State Mining and Geology Board define a potentially active fault as a fault which has been active during the Quaternary Period (roughly within the last 1.6 Million years). Historic and Holocene age faults are considered active, Late Quaternary and Quaternary age faults are considered potentially active, and pre-Quaternary age faults are considered inactive.

The above definitions are used in delineating Earthquake Fault Zones as mandated by the Alquist-Priolo Geologic Hazard Zones Act of 1972 and as subsequently revised in 1994 (Hart, 1997) as the Alquist-Priolo Geologic Hazard Zoning Act and Earthquake Fault Zones. The Act regulates development and construction of buildings intended for human occupancy to mitigate the hazards of surface fault rupture. It defines areas where ground rupture is likely to occur during future earthquakes. Where such zones are designated, a geologic study must be conducted to determine the locations of all active fault lines in the zone before any construction is allowed and to determine whether building setbacks should be established, and no building may be constructed on the fault lines.

Our review of geologic literature pertaining to the site area indicates that there are active faults within the regional area (Rose Canyon Fault, Elsinore Fault, San Jacinto Fault and Newport-Inglewood Fault. The nearest active zone is Rose Canyon Fault Zone located in 6.7 miles west of the project site.

Rose Canyon Fault Zone Parameters	
Length:	55 to 70 (km)
Fault Type:	Right Lateral/Strike Slip
Slip rate:	1.5 mm/ year
Dip:	90 degrees

Based on the 2010 California Fault Activity Map (Jennings and Bryant 2010, Figure 9), active faults are not mapped on the site. Quaternary La Nacion Fault Zone is located approximately 0.3 miles east from the project site. Geologic mapping by Partner indicated structural continuity across the site, further suggesting the absence of active faults in the area explored.

No evidence of active or potentially active faulting was observed or encountered in any of our excavations/trenches on the site. It should be noted that the Southern California region is an area of moderate to high seismic risk and it is not considered feasible to render structures fully resistant to seismic related hazard. The minimum seismic design should comply with the 2013 California Building Code (CBC) and ASCE 7-10 using the seismic parameters recommended in Section 6.0 of this report.

5. SECONDARY SEISMIC HAZARDS

This section presents the results of a geologic review performed by Partner, for a proposed new construction on site. The general location of the project is shown on Figure 1.

5.1 Surface/Subsurface Fault Rupture

Surface fault rupture resulting from the movement of nearby major faults is not known with certainty but is considered low. However, due to the known active and potentially active faults in the region, low to moderate ground shaking should be expected during the life of the proposed structures.

5.2 Liquefaction

Liquefaction is defined as a seismic phenomenon in which loose or soft, saturated, fine-grained soil mass suffers a substantial reduction in its shear strength when subjected to high-intensity ground shaking and exhibits a liquid-like behavior.

During earthquakes, excess pore water pressures may develop in saturated soil deposits as a result of induced cyclic shear stresses. Effects of liquefaction can include sand boils, settlement and bearing capacity failures. Liquefaction occurs when these ground conditions exist: 1) Shallow groundwater; 2) Low density, fine, clean sandy soils; and 3) High-intensity ground motion. Shallow ground water and saturated, clean, sandy soils are not present at the project site.

Published data from California Geological Survey - Seismic Hazards Zone Map, indicates that the project site is not located in an area identified as having a potential for soil liquefaction. The potential for site liquefaction is negligible (see Figure 9).

5.3 Seismically Induced Landslide

According to the published data from California Geological Survey "State of California Seismic Hazard Zones Official Map, the site is not within a landslide zone (see Figure 9).

6. SEISMIC / DESIGN PARAMETERS

When reviewing the 2010 California Building Code, IBC 2009 and ASCE 7-10 the following seismic data should be incorporated into the design.

6.1 Seismic Design Parameters

Latitude: 32.597463 N (Degrees)

Longitude: -117.031415 W (Degrees)

MCE: 2% Probability of Exceedance in 50 Years

Seismic Item	***Value***	***Seismic Item***	***Value***
Site Classification	D	Seismic Design Category	D
Fa (site coefficient)	1.043	Fv (site coefficient)	1.461
Ss (spectral response at 0.2 seconds)	0.892g	S_1 (spectral response at 1.0 second)	0.339g
S_{MS} (maximum considered earthquake spectral acceleration)	0.931g	S_{M1} (maximum considered earthquake acceleration)	0.496 g
S_{DS} (design spectral acceleration)	0.621g	S_{D1} (design spectral acceleration)	0.330g
PGA Max (ASCE '10)	0.375g	67% PGA (ASCE '10)	0.251g

Source: 2010 and 2016 CBC (IBC 2016/ ASCE 7-10) and USGS Seismic Hazards Design Maps.

The Structural Consultant should review the above parameters and the 2010 California Building Code (IBC 2009/ASCE 7-10) to evaluate the seismic design.

7. GEOTECHNICAL EXPLORATION & LABORATORY RESULTS

Our evaluation of soils on the site included field exploration and laboratory testing. The field exploration and laboratory testing programs are briefly described below. Data reports from the field exploration and laboratory testing are provided in Appendix B and Appendix C, respectively.

7.1 Soil/ Continuous Core Borings

The first soil boring program was conducted on January 25, 2018. Six (6) borings were advanced by the use of a track-mounted drill using solid flight auger drilling techniques. The borings were made to depths of 5 to 15 feet below ground surface. Boring B-5 encountered hard drilling material and then was terminated due to damage to the drill rig.

The second soil boring program was conducted on February 12, 2019. The approximate locations of the exploratory borings are shown on Figure 2. Six (6) borings were advanced by the use of a track-mounted drill using solid flight auger drilling techniques. The borings were made to depths of 16.5 feet below ground surface.

Three (3) continuous soil cores were performed on the site to depths of 40 to 50 feet for geologic mapping on March 15, 2019. The geologic data and stratigraphic evaluation from these borings are included in the boring Appendix B. Logs of subsurface conditions encountered in the borings were prepared in the field by a representative of Partner Engineering. Soil samples consisting of relatively undisturbed brass ring samples and Standard Penetration Tests (SPT) samples were collected at approximately 2.5 and 5-foot depth intervals and were returned to the laboratory for testing. The SPTs were performed in accordance with ASTM D 1586. Typed boring logs were prepared from the field logs and are presented in Appendix A. Continuous corings were also conducted on three borings for stratigraphic evaluation.

A summary table description is provided below:

Summary of Geologic Straiographic Data		
Strata	**Depth to Bottom of Layer (bgs*)**	**Description**
Surface Cover	0-1 feet	Grass/ Dirt
Fill Material	Up to 6 feet	Silty sand with gravel and cobbles
San Diego Formation	16+ feet	Silty sandstone, fine silty sand
Groundwater	NA	Not encountered
Bedrock (Very Hard)	NA	Not encountered

7.2 Trenches

The trenches were excavated during July 26 to July 27, 2018. Four (4) trenches were excavated using Backhoe Komatsu, PC 390 LC. The trenches were excavated to depths of 14 feet in the slopes of the parcel. The approximate locations of the trenches are shown on Figure 2.

Logs of subsurface conditions encountered in the trenches were prepared by our Certified Engineering Geologist. Soil Bag samples were taken at TP-1 at approximately 5.5 and 11.0-foot depth interval and were returned to the laboratory for testing. Test pits were backfilled on completion. Typed trench logs were prepared from the field logs and are presented in Appendix A.

7.3 Geotechnical Laboratory Evaluation

Soil samples were submitted to a certified testing laboratory, Hamilton & Associates. Results are attached in Appendix C. Tests performed included in-place moisture and density, sieve analysis, Atterberg and direct shear tests. We have reviewed the results from Hamilton & Associates and are in agreement with the results. The results of laboratory analyses are presented in the boring logs and in Appendix C.

8. PRELIMINARY GEOTECHNICAL RECOMMENDATIONS

The following discussion of findings for the site is based on the assumed construction, geologic review, results of the field exploration, and laboratory testing programs. The recommendations of this report are contingent upon adherence to Appendix D of this report, General Geotechnical Design and Construction Considerations. For additional details on the below recommendations, please see Appendix D.

8.1 Geotechnical Recommendations

- The proposed construction is generally feasible from a geotechnical perspective provided the recommendations and assumptions of this report are followed.

Geologic/General Site Considerations

- Regionally the site is located in Peninsular Ranges Geomorphic Province. The Peninsular Ranges Province is traversed by a group of sub-parallel faults and fault zones trending roughly northwest. Several of these faults are major active faults (Rose Canyon, Elsinore, San Jacinto and Newport – Inglewood). Undivided sediments/sedimentary rocks and San Diego Formation occurs within the regional area of the site. The subject property is currently vacant and undeveloped since 1904, there was substantial grading, drainage improvements and hydro-seed applications on the northern slopes in 2007. The site is in an area where the seismic hazard potential was not evaluated, and the historic groundwater levels were not provided by the California Department of Conservation. Partner conducted geologic and seismic investigations in July – August 2018. Partner's evaluation indicated the hazards of landslide and liquefaction are not present at the project site. No other hazards are known. Due to the proximity to residential homes, additional regulations for construction noise and setbacks should be carefully reviewed during the planning stages.

Excavation Considerations

- We anticipate extensive grading will be needed on the site to establish the finished grades for the new buildings. We anticipate site excavations can be made using conventional construction equipment in good working condition; However, given the quantity of cuts on the site, particularly on the north side of the property, hard excavation may be encountered in some of the deeper cuts. Groundwater was not encountered during drilling; however, groundwater levels can fluctuate over time. Loose fill soils and native sandy soils/San Diego Formation may be prone to caving during excavation. Excavations should be sloped or shored per OSHA requirements.
- On the north side of the property, cuts of up to 20 feet are anticipated. Laying back of cuts up to 20 feet can be done on a temporary basis per OSHA with the consideration of type C, sandy soils at a 1.5:1 horizontal to vertical slope. Such slopes should be monitored for sloughing or loose material on a daily basis for site safety. Where such slopes exceed 20 feet, a shoring or bracing system should be used. This can consist of a temporary soldier pile and lagging retaining wall. The soldier piles may require pre-drilling and grouting for installation. Spacing and depth calculations for this should be done by a certified contractor, and should comply with California and other local jurisdictional requirements. The design can use soil data from Section 8.2 of this report, and more information is provided in Appendix C under Excavations and Dewatering.

Spread Foundation

- We anticipate that spread foundations are planned for the site structure. We anticipate that spread foundations will be proportioned for bearing capacities ranging from 2,000 to 3,000 pounds per square foot or less. The foundations and slabs should be supported on a layer of in-place native soils that have been evaluated and approved by the engineer and compacted in-place, or bear on controlled fill that has been placed and compacted as a part of mass grading, as described below, in Section 8.2 and Appendix C.

Mass Grading Considerations

- All undocumented fills, debris, grass, roots and other plant materials should be removed from structural areas of the site. In the new fill areas, the cleaned subgrade should be proofrolled and evaluated by the engineer with a loaded water truck (4,000 gallon) or equivalent rubber tired equipment. Soft or unstable areas should be repaired per the direction of the engineer.
- Prior to the placement of new fill, Appendix J of the California building code should be carefully reviewed. Given the native slopes on the site, benching and keying of new fills will be needed as shown in Figure 10. The bulk of the new hospital building will be supported on native material; however, a portion is to bear on deep fills (up to 20 feet) placed over the existing slope. For new fill zones where more than 5 feet of fill will support the new building or parking areas, 95% compaction is required to reduce the potential of differential settlement. It is recommended, that this zone start 5 feet from the edge of building or pavement, and extend at a 1:1 slope to the base of fill. In order to achieve this level of compaction, careful attention to moisture conditioning, lift thickness, and compaction equipment selection will be needed.
- We assume that mass grading will be performed prior to the installation of new retaining walls, and the new fill will be cut back where needed to install retaining wall foundations, and to provide room for retaining wall backfill. However, in some cases, it may make sense to partially grade retaining wall areas, so that cut backs for wall installation do not create steep/unstable slopes (greater than 2:1 horizontal to vertical and/or higher than 20 feet) In the event that walls are in-place during grading operations, grading equipment should be routed to avoid retaining walls. Only lightweight equipment should be used to backfill retaining walls, as described below.

Retaining Wall Considerations

- Most of the site retaining walls are in support of new fills, and as such, can be staged so as to not result in a temporary steep cut-back condition for wall installation. However, the wall on the north of the property, cross-section H-H', will require a relatively large over-cut in the existing soil. Partner performed a slope stability analysis of this as a 1.5:1 horizontal to vertical cut, as shown in Appendix D, and demonstrated a factor of safety of 1.05 for global stability. This excavation should be stable on a temporary basis; however, if used, the slope should be regularly monitored and cleaned of any large rocks or loose soil that could slip. Alternatively, the excavation could be supported by a temporary shoring system, consisting of soldier piles or the permanent wall could be constructed of a soldier pile system. Appendix D contains our slope stability cross sections and results.
- The soil parameters for the design of site retaining walls is provided in Section 8.2. The wall designer should check the wall for sliding, overturning, and internal stability. Partner performed global

stability for the four site walls sections that were over 6 feet in height. Factors of safety in three of the four sections were 1.5 or greater with normally sized and embedded foundations. Cross-section H-H', located on the north side of the project includes a roughly 40-ft high cut slope with a 13-ft high retaining wall at its base. This section did not have a 1.5 factor of safety with normally sized and embedded foundation. As such, we recommend that the retaining wall in this location have a cantilevered foundation embedded 4 feet below grade, and that extends 7.5 feet from the centerline of the wall, where wall heights are higher than 6 feet. Construction should proceed in general accordance with Appendix C, with specific attention to Laterally Loaded Structures.

Soil Reuse Considerations

- Site soils were generally acceptable for use as engineered fill. The vegetation and debris should be stripped from the site and should not be incorporated into fill material. It is recommended to use non-expansive structural fill that is free of deleterious materials, and is properly moisture conditioned and compacted to 90-95% of the modified proctor (ASTM D 1557). For deep fills below the building, and at the pavement subgrade elevation 95% should be used, and 90% may be used in other areas where allowed by the building code.

Concrete Considerations

- Concrete should be corrosion resistant, using Type II/V Portland Cement, and fly ash mixtures of 25 percent cement replacement. We recommend a water/cement ratio of 0.45 or less. Site soil may be corrosive to un-protected metallic elements such as pipes, poles, etc. Concrete exposed to freezing weather in cold climates should be air-entrained.

Site Storm Water Considerations

- The site surficial soils are generally undocumented fill and sandy soil. Surface drainage and landscaping design should be carefully planned to protect the new structures from erosion/undermining, and to maintain the site earthwork and structure subgrades in a relatively consistent moisture condition. Water should not flow towards or pond near to new structures, and high water demand plants should not be planned near to structures.

8.2 Geotechnical Parameters

Based on the findings of our field and laboratory testing, we recommend that design and construction proceed per industry accepted practices and procedures, as described in Appendix D, General Geotechnical Design and Construction Considerations (Considerations).

Subgrade Preparation Parameters – (hyperlink to Construction Considerations)

Subgrade Preparation				
Structure	**Bearing Capacity**	**Embedment Depth**	**Bearing Surface** [a]	**Settlement** [d]
Grade Slabs	k=150 pci [b]	NA	95% Compacted Fill or Native to 90%	<1 inch
Spread Foundations	3,000 [c] psf	30 inches	95% Compacted Fill or Native to 90%	<1 inch
Spread Foundations	2,500 [c] psf	24 inches	95% Compacted Fill or Native to 90%	<1 inch
Spread Foundations	2,000 [c] psf	18 inches	95% Compacted Fill or Native to 90%	<1 inch

[a] Repairs in bearing surface areas should be structural fill per the recommendation of the Earthwork section of Appendix C that is moisture conditioned to within 3 percent below to optimum moisture content and compacted to 95 percent or more of the soil maximum dry density per ASTM D1557. Expansive material should not be located within the upper 3 feet of the soil subgrade.

[b] Subgrade modulus value "k", assuming the grade slab is supported by aggregate layer roughly equal to slab thickness (minimum 4 inches)

[c] Can be increased by 1/3 for temporary loading such as seismic and wind

[d] Differential settlement is expected to be half of total settlement

Paving Structural Sections – (hyperlink to Construction Considerations)

Pavement Sections		
Roadway Type	**Subgrade Preparation [a]**	**Pavement Section**
Parking Area Light Duty (TI=4)	Proofrolled/Compacted Subgrade	3-in asphalt & 6-in aggregate base
Parking Area Heavy Duty (TI=7)	Proofrolled/Compacted Subgrade	4-in asphalt & 9-in aggregate base
Parking Area Heavy Duty (TI=7)	Proofrolled/Compacted Subgrade	6-in concrete & 4-in aggregate base
Special High Traffic Areas	Proofrolled/Compacted Subgrade	8-in concrete

[a] Repairs in proofrolled areas should be structural fill per the recommendation of the Earthwork (hyperlink to Construction Considerations) that is moisture conditioned to within 3 percent below to optimum moisture content and compacted to 95 percent or more of the soil maximum dry density per ASTM D1557.

Laterally Loaded Structures Parameters– (hyperlink to Construction Considerations)

Lateral Earth Pressures				
Soil Type	**Coefficient of Friction (µ)**	**Static Fluid Pressure (pcf)**	**Active Fluid Pressure (pcf)**	**Passive Fluid Pressure (pcf)**
Fill Soil	0.3	50	35	300
Native Soil	0.3	50	35	350

**seismic equations*

Combined effect of static and seismic lateral force:

$P_{AE} = F_1 + F_2$

$F_1 = 1/2*A*H^2$ Resultant acting at a distance of H/3 from base of wall

$F_2 = 3/8*K_h*\gamma*H^2$ Resultant acting at a distance of (0.6*H) from base of wall

Where:

F_1 = Static Force (plf) based on active pressure

F_2 = Seismic Lateral Force (plf) based on seismic pressure

γ = 120 pcf

$K_h = S_{DS}/2.5$

A = Active Pressure (pcf)

H = Height of retained soil (ft)

FIGURES

0 0.5 1 MILE

0 1000 FEET 0 500 1000 METERS

SITE LOCATION MAP

517 Shinohara Lane, Chula Vista, CA 91911

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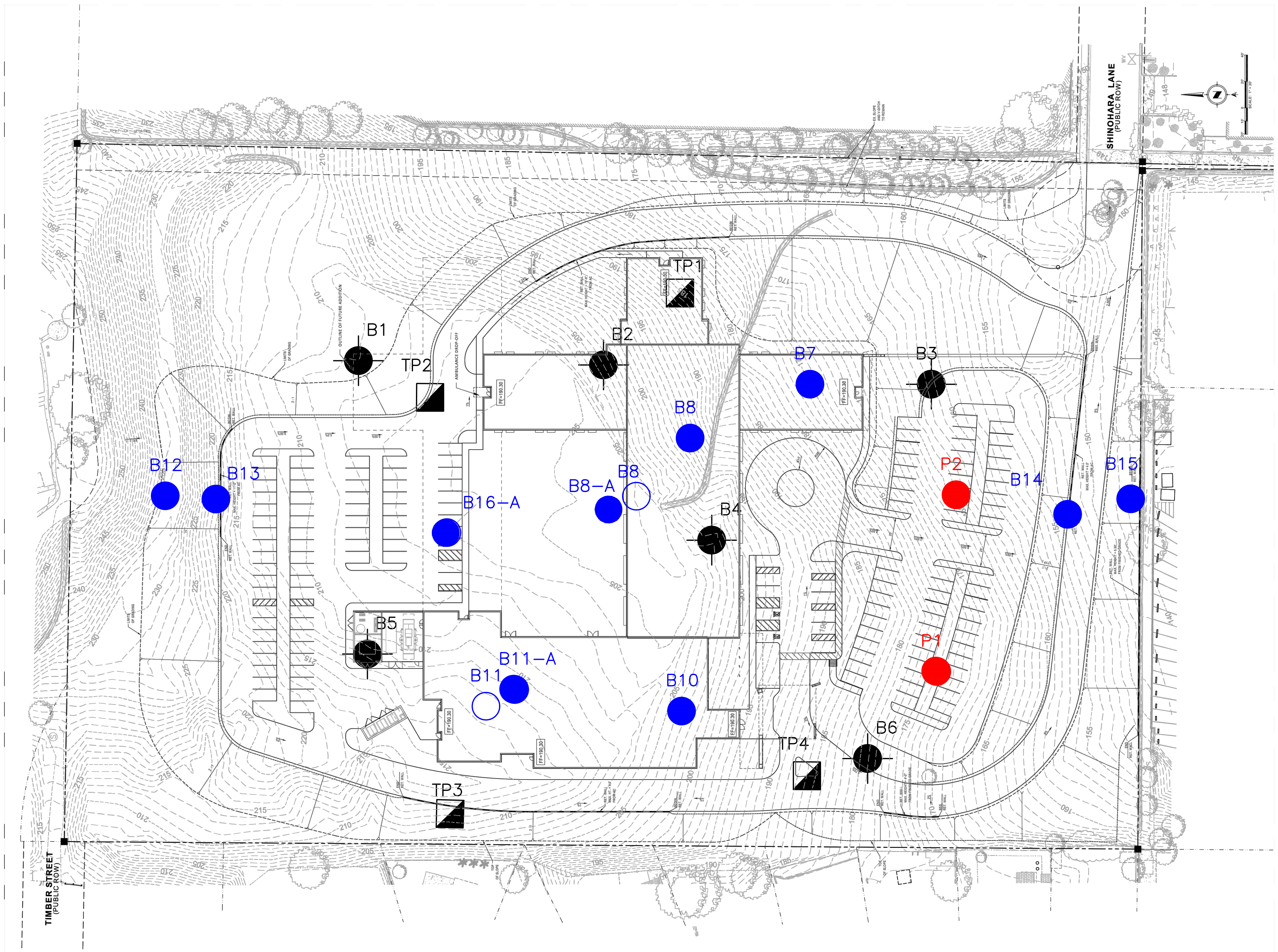
FIGURE 1

Reference: USGS, US Topo, Imperial Quadrangle, California-Los Angeles Co., 7.5 - Minute Series, 2015.



LEGEND:

APPROXIMATE FOOTPRINT OF HOSPITAL BUILDING.

TP1 — TRENCH EXCAVATED DURING JULY 26-27, 2018.

B1 — BORING DRILLED JANUARY 25, 2018.

B1 — BORING DRILLED FEBRUARY 12, 2019.

B11-A — CORE BORING DRILLED MARCH 15, 2019.

B1 — CORE BORING NOT COMPLETED DUE TO MECHANICAL AND SAMPLING DIFFICULTIES.

P1 — PERCOLATION TEST

NOTES:

- BGS = BELOW GROUND SURFACE
- BORING, TRENCHES AND PERCOLATION TEST LOCATIONS ON THIS MAP ARE APPROXIMATE

50' 0' 50' 100'

APPROXIMATE SCALE

TITLE: GEOTECHNICAL/GEOLOGIC INVESTIGATION PLAN

FIGURE:	PREPARED BY:	DATE:	PROJECT NUMBER:
2	FC	MARCH 2019	17-199602.4

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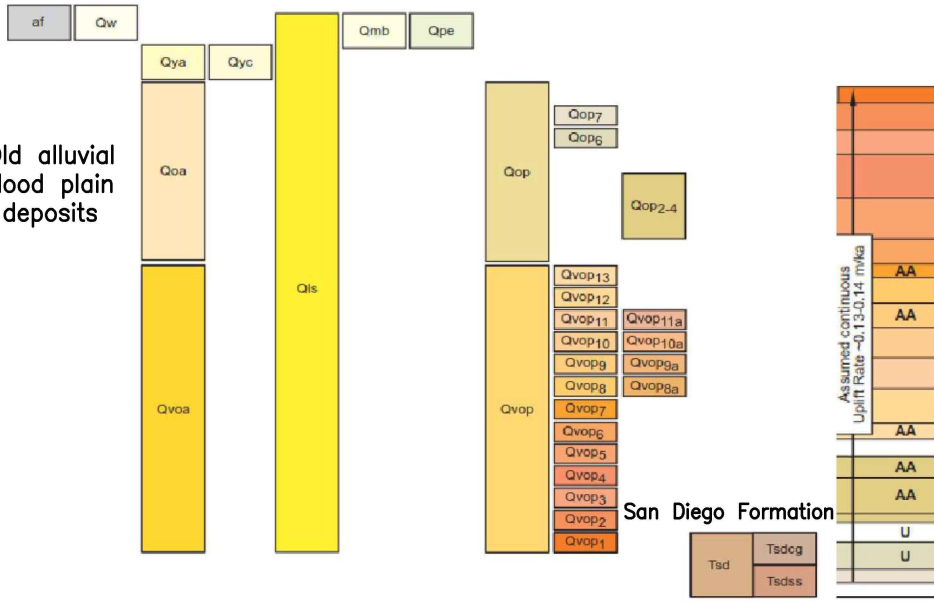
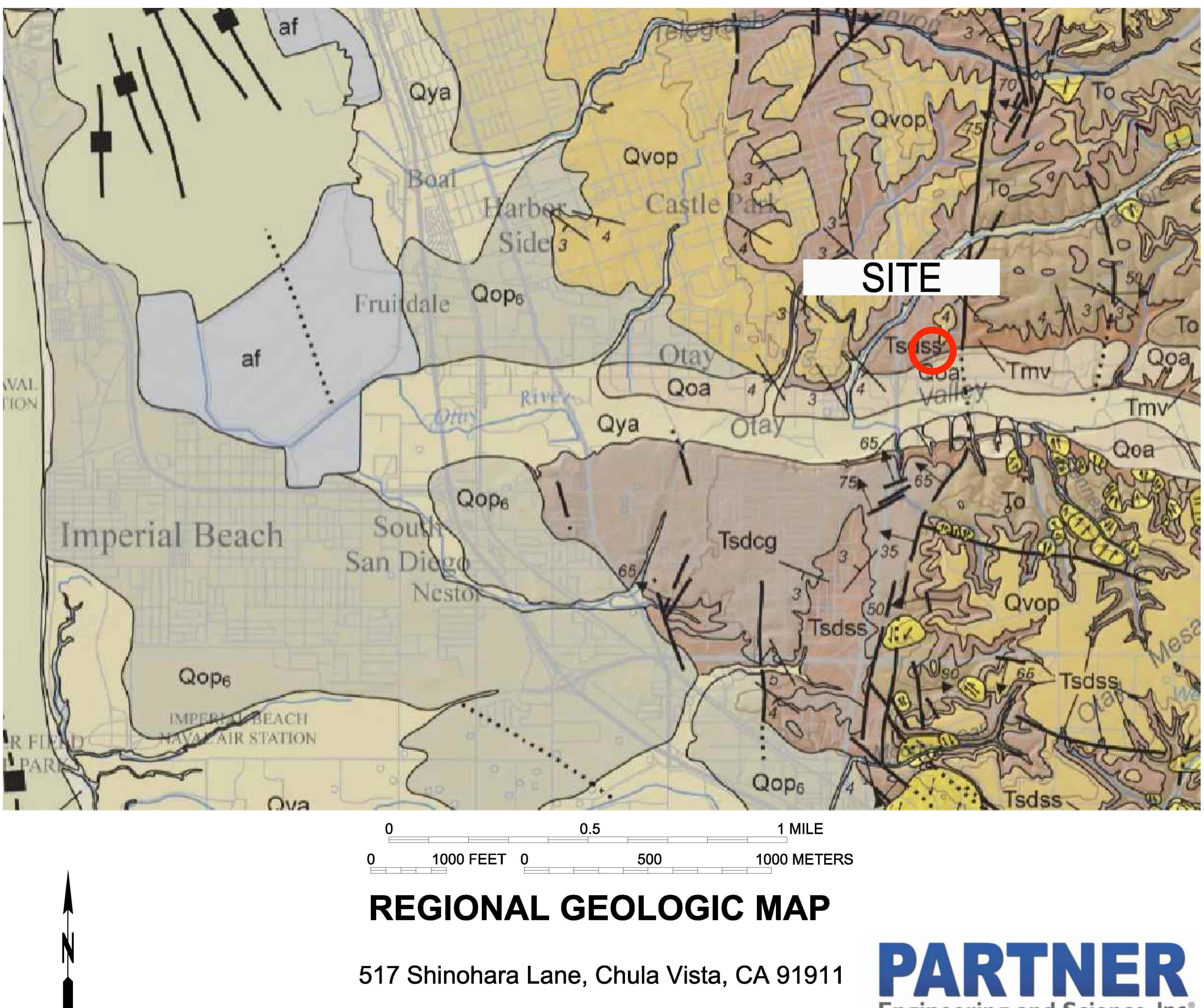
TORRANCE, CALIFORNIA 90501

*Source Drawing from EH Grading Plan, 517 Shinohara Lane, Chula Vista, CA

AERIAL IMAGERY PROVIDED BY GOOGLE AND ITS LICENSORS © 2016



Approximate age in years (ka = 1000 yrs)	Approximate elevation in meters/feet	Terrace name Kern, 1996b	Terrace deposit symbol on map
~1525 ka	203 m/666'	Eagle	Qvop1
~1445 ka	192 m/630'	Flores Hill	Qvop2
~1375 ka	183 m/600'	Alison Canyon	Qvop3
~1290 ka	172 m/564'	Aqueduct	Qvop4
~1160 ka	155 m/509'	Rifle Range	Qvop5
~1050 ka	139 m/456'	Black Mountain	Qvop6
~975 ka	130 m/427'	Mira Mesa	Qvop7
~930 ka	124 m/407'	Tierra Santa	Qvop8
~855 ka	114 m/374'	Linda Vista	Qvop9
~800 ka	105 m/345'	Tecolote	Qvop10
~698 ka	93 m/305'	Clairemont	Qvop11
~630 ka	84 m/276'	Fire Mountain	Qvop12
~510 ka	68 m/223'	San Elijo	Qvop13
~450 ka	62 m/203'	Golf Course	Qop1
~413 ka	55 m/180'	Parry Grove	Qop2
~320 ka	47 m/154'	Guy Fleming	Qop3
~220 ka	33 m/108'	Stuart Mesa	Qop4
~180 ka	29 m/95'	San Clemente	Qop5
~120 ka	22 m/72'	Nestor	Qop6
~80 ka	10 m/33'	Bird Rock	Qop7
~45 ka	6 m/20'	St. Louis	Qop8



REGIONAL GEOLOGIC MAP

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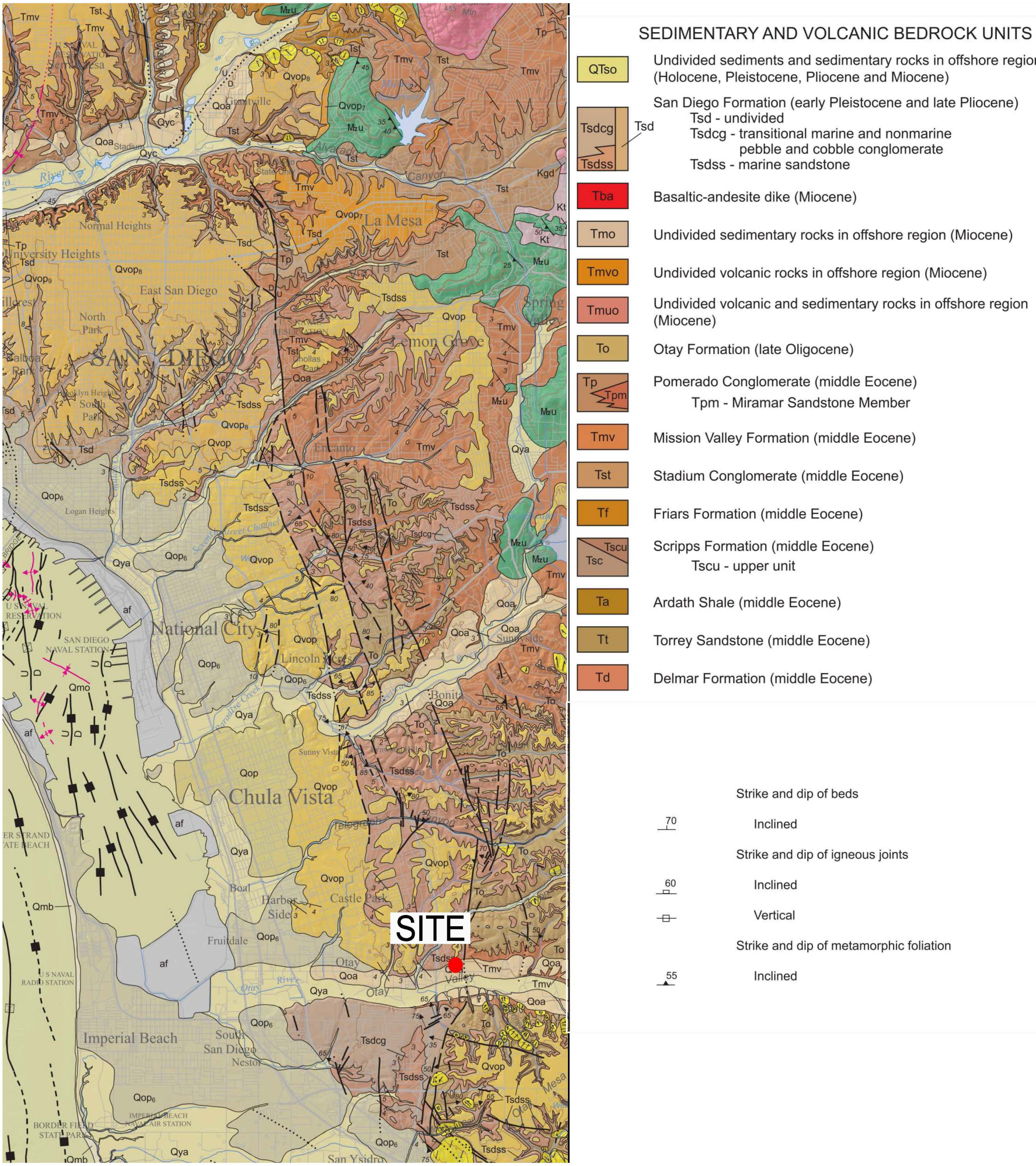
FIGURE 3

Reference: CGS, Geologic Map of San Diego Quadrangle



GEOLOGIC MAP OF SAN DIEGO QUADRANGLE

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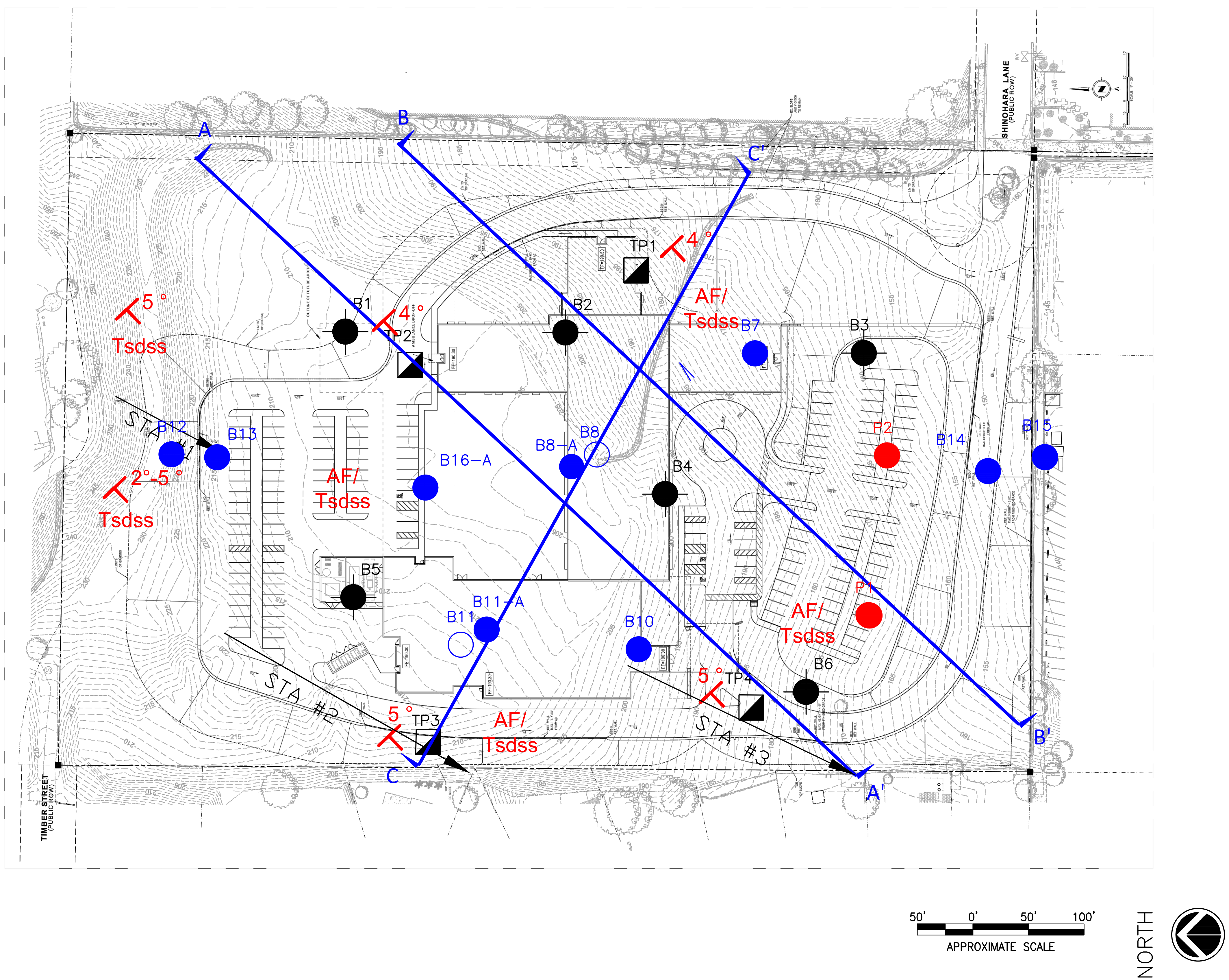
FIGURE 4

Reference: Dibble, and Ehrenspeck, 1992, Geologic Map of the San Diego Quadrangle, San Diego County, California Dept. of Conservation



*Source Drawing from DVELE Architecture Drawings, 752 Skyview Terrace, Ventura, CA, Sheet A1-02, Apr 2018.

LEGEND:

APPROXIMATE FOOTPRINT OF HOSPITAL BUILDING

A A' GEOLOGIC CROSS SECTION

TP1 TRENCH EXCAVATED DURING JULY 26-27, 2018.

B1 BORING DRILLED JANUARY 25, 2018.

B1 BORING DRILLED FEBRUARY 12, 2019 .

B11–A CORE BORING DRILLED MARCH 15, 2019.

B11 CORE BORING NOT COMPLETED DUE TO MECHANICAL AND SAMPLING DIFFICULTIES

P1 PERCOLATION TEST

STRIKE AND DIP OF BEDDING

AF: ARTIFICIAL FILL APPROXIMATELY1 FOOT TO 6 FEET THICK ALONG THE PROJECT SITE

Tsdss: SAN DIEGO FORMATION: SILTY SANDSTONE, GRAYISH WHITE TO YELLOWISH WHITE, FINE, SILTY SAND/ SANDY SILT, MOIST, SLIGHTLY MICACEOUS, MEDIUM DENSE, MODERATELY WEATHERED, EARLY PLEISTOCENE TO LATE PLIOCENE

STA #: LOCALIZED OF SLOPE STABILITY ANALYSIS

NOTES:

- BGS = BELOW GROUND SURFACE
- BORING, TRENCHES AND PERCOLATION TESTS LOCATIONS ON MAP ARE APPROXIMATE
- FOR GEOLOGIC CROSS SECTIONS SEE FIGURES 6, 7 AND 8

TITLE: **SITE ENGINEERING GEOLOGIC MAP GEOLOGIC CROSS SECTION LOCATIONS**

FIGURE:	PREPARED BY:	DATE:	PROJECT NUMBER:
5	FC	MARCH 2019	17–199602.4

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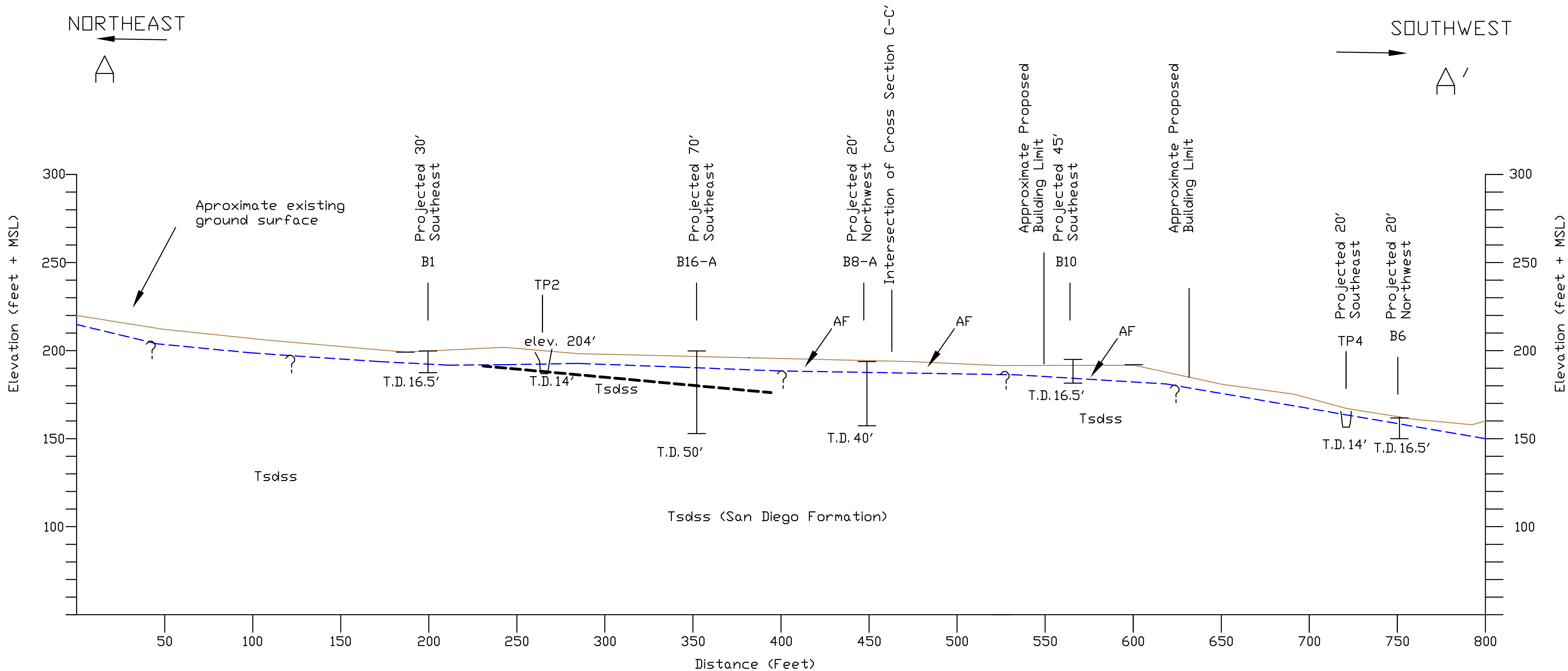

Geologic Units:

AF	Artifical Fill (orange brown, fine to coarse grained sand, some silt and clay, fine to coarse gravel and cobbles, moist, SM-GM*)
Tsdss	San Diego Formation (silty sandstone, grayish white to yellowish white, fine, silty sand/ sandy silt, moist, slightly micaceous, medium dense, moderately weathered, early Pleistocene and late Pliocene)
	Inferred lithologic contact, queried where uncertain
	Bedding 4°-5° degrees towards southwest, faint bedding, strike, N 20° to 25° W

Note: Geologic Cross Section A-A' is shown at location in Figure 5

B1	Partner Hollow Stem Auger Boring (JAN 2018)
	Partner Trench (JUL 2018)
TD	Total Depth of Boring and Trenches (feet)
SM-GM*	Unified Soil Classification symbol field observation

HORIZONTAL SCALE = VERTICAL SCALE

25' 0' 25' 50'

HORIZONTAL SCALE IN FEET

TITLE:

GEOLOGIC CROSS SECTION A-A'

FIGURE:	PREPARED BY:	DATE:	PROJECT NUMBER:
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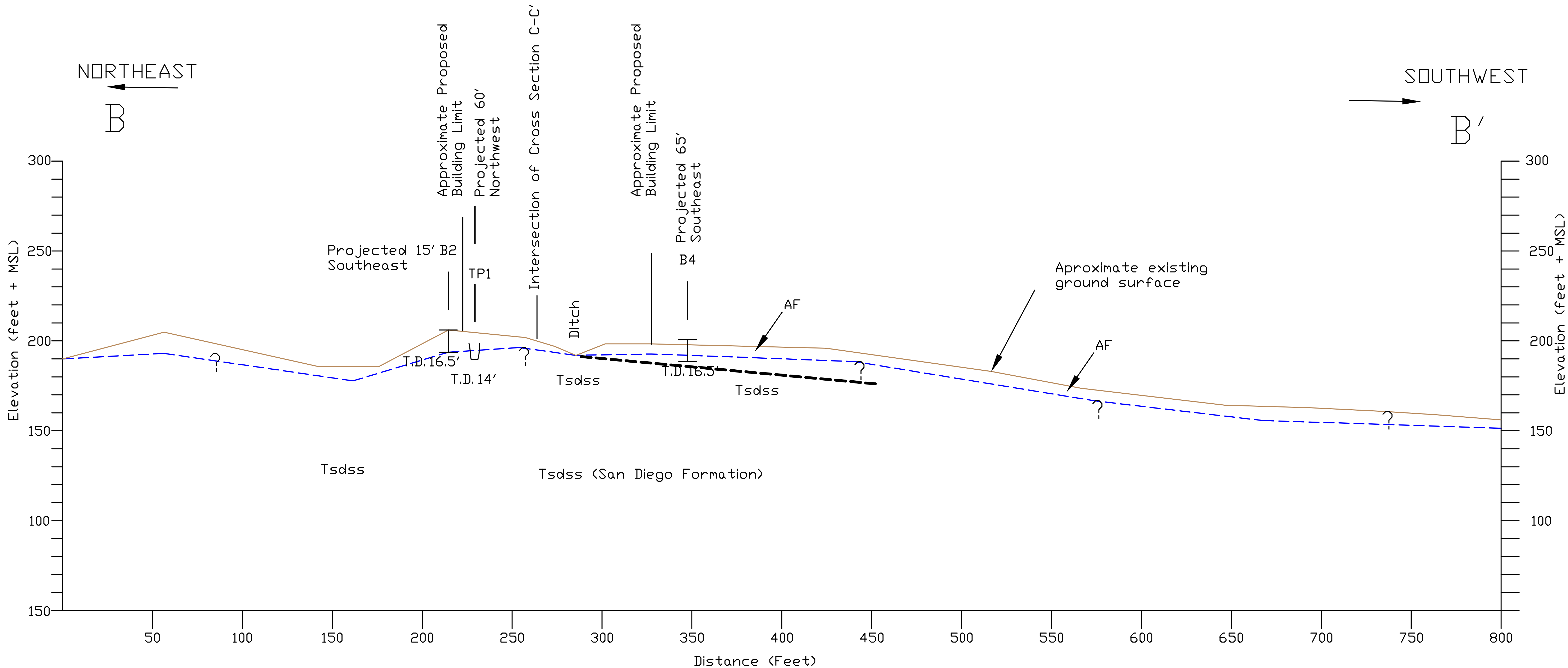

Geologic Units:

AF — Artifical Fill (orange brown, fine to coarse grained sand, some silt and clay, fine to coarse gravel and cobbles, moist, SM-GM*)

Tsdss — San Diego Formation (silty sandstone, grayish white to yellowish white, fine, silty sand/ sandy silt, moist, slightly micaceous, medium dense, moderately weathered, early Pleistocene and late Pliocene)

Inferred lithologic contact, queried where uncertain

Bedding 4°-5° degrees towards southwest, faint bedding, strike, N 20° to 25° W

Note: Geologic Cross Section B-B' is shown at location in Figure 5

B1 — Partner Hollow Stem Auger Boring (JAN 2018)

Partner Trench (JUL 2018)

TD — Total Depth of Boring and Trenches (feet)

SM-GM* Unified Soil Classification symbol field observation

HORIZONTAL SCALE = VERTICAL SCALE

25' 0' 25' 50'

HORIZONTAL SCALE IN FEET

TITLE:

GEOLOGIC CROSS SECTION B-B'

FIGURE:	PREPARED BY:	DATE:	PROJECT NUMBER:
7	FC	APR 2019	17-199602.7

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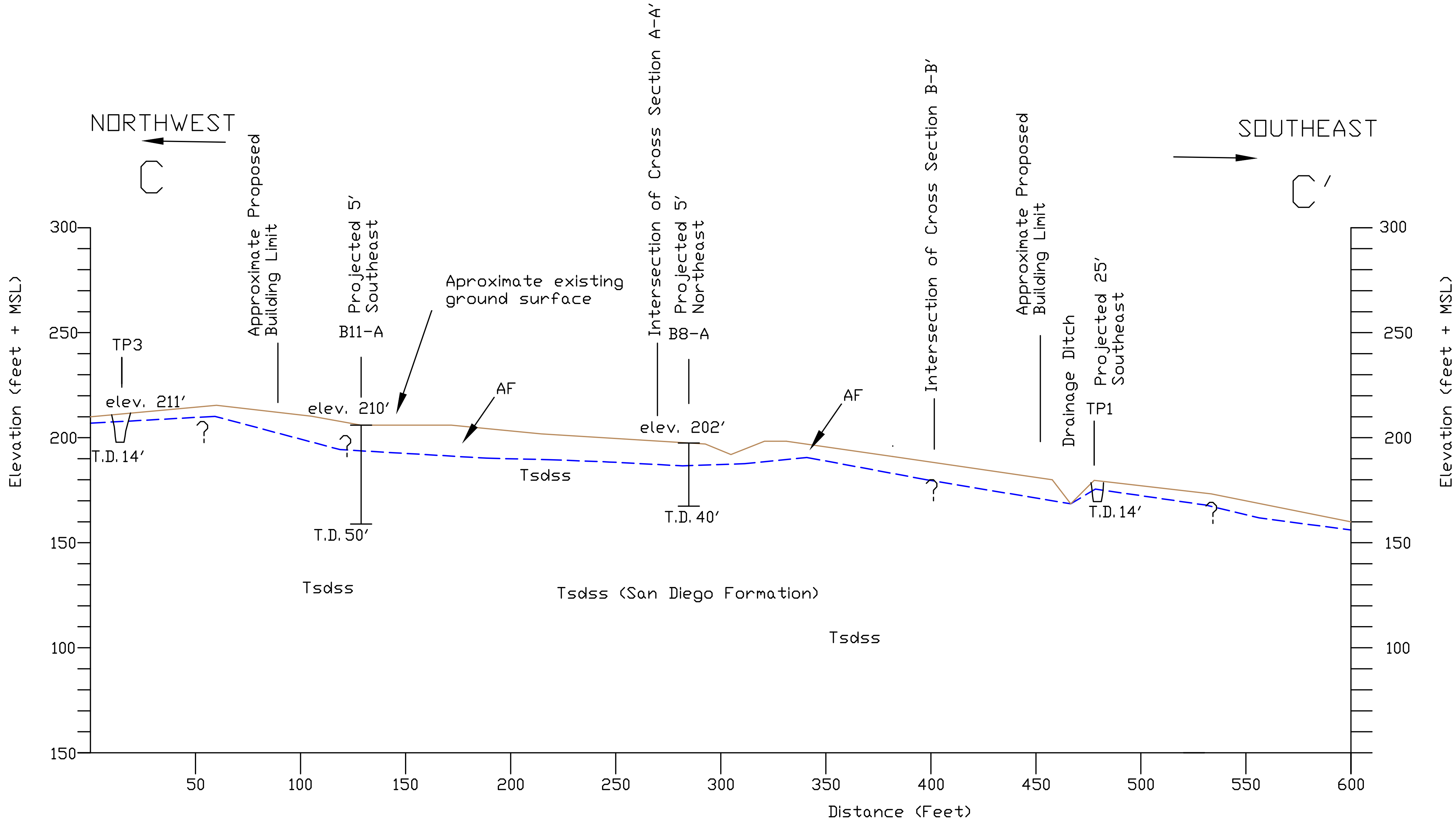

HORIZONTAL SCALE = VERTICAL SCALE

25' 0' 25' 50'

HORIZONTAL SCALE IN FEET

Geologic Units:

AF	Artifical Fill (orange brown, fine to coarse grained sand, some silt and clay, fine to coarse gravel and cobbles, moist, SM-GM*)
Tsdss	San Diego Formation (silty sandstone, grayish white to yellowish white, fine, silty sand/ sandy silt, moist, slightly micaceous, medium dense, moderately weathered, early Pleistocene and late Pliocene)
? (blue dashed line)	Inferred lithologic contact, queried where uncertain

B4	Partner Hollow Stem Auger Boring (JAN 2018)
	Partner Trench (JUL 2018)
TD	Total Depth of Boring and Trenches (feet)
SM-GM*	Unified Soil Classification symbol field observation

Note: Geologic Cross Section C-C' is shown at location in Figure 5

TITLE:

GEOLOGIC CROSS SECTION C-C'

FIGURE:	PREPARED BY:	DATE:	PROJECT NUMBER:
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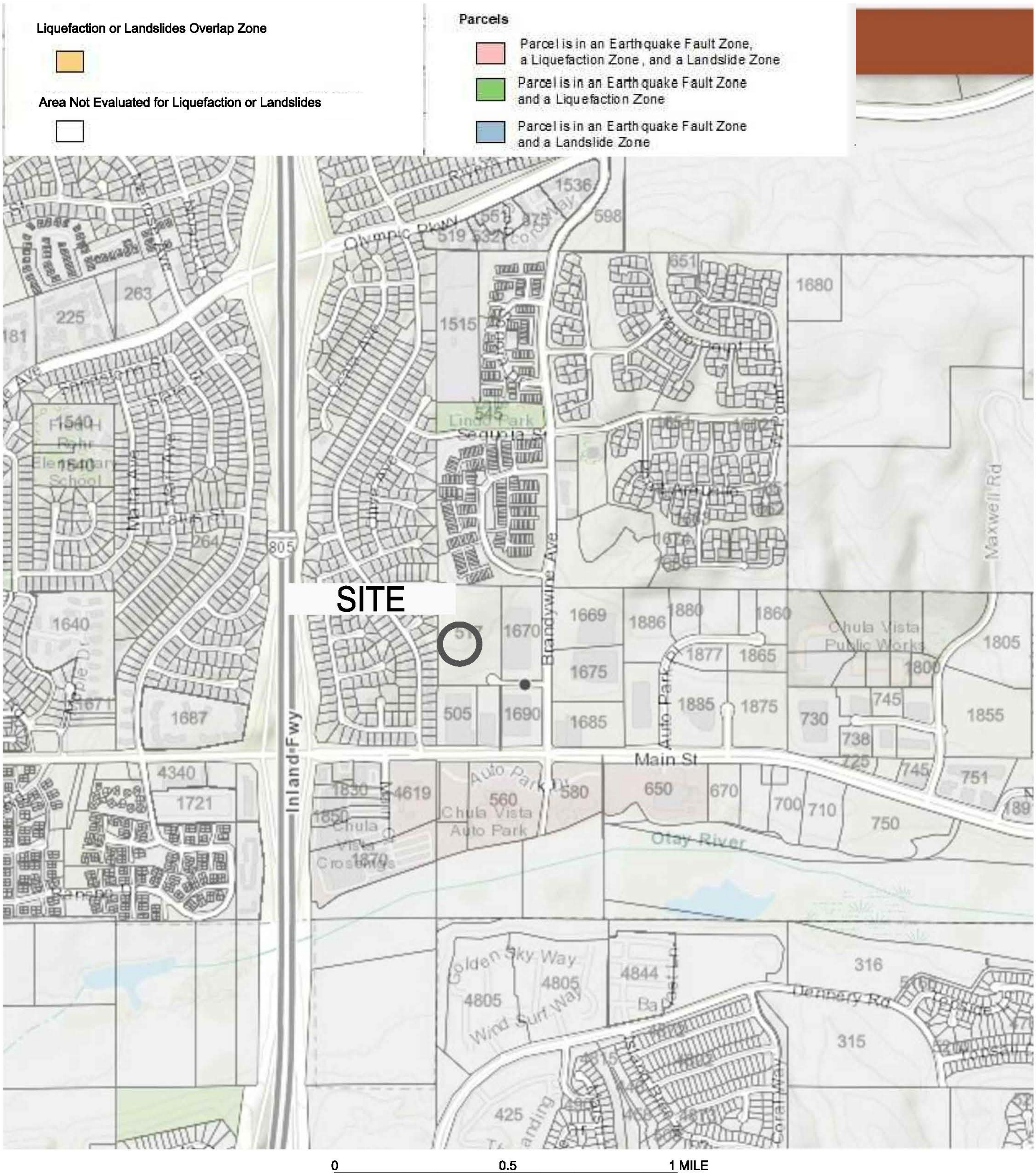

SEISMIC HAZARDS ZONES MAP

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FIGURE 9

Reference: State of California, Earthquakes Zones of Required Investigation, Chula Vista, Bureau of Land Management, ESRI.

SYMBOL EXPLANATION

Fault traces on land are indicated by solid lines where well located, by dashed lines where approximately located or inferred, and by dotted lines where concealed by younger rocks or by lakes or bays. Fault traces are queried where continuation or existence is uncertain. All offshore faults based on seismic reflection profile records are shown as solid lines where well defined, dashed where inferred, queried where uncertain.

FAULT CLASSIFICATION COLOR CODE
(Indicating Recency of Movement)

Fault along which historic (last 200 years) displacement has occurred.

1906 ▶ ◀ 1906
1838 ▷ ◁ 1838

Holocene fault displacement (during past 11,700 years) without historic record.

Late Quaternary fault displacement (during past 700,000 years).

Quaternary fault (age undifferentiated).

Pre-Quaternary fault (older that 1.6 million years) or fault without recognized Quaternary displacement.

ADDITIONAL FAULT SYMBOLS

Bar and ball on downthrown side (relative or apparent).

Arrows along fault indicate relative or apparent direction of lateral movement.

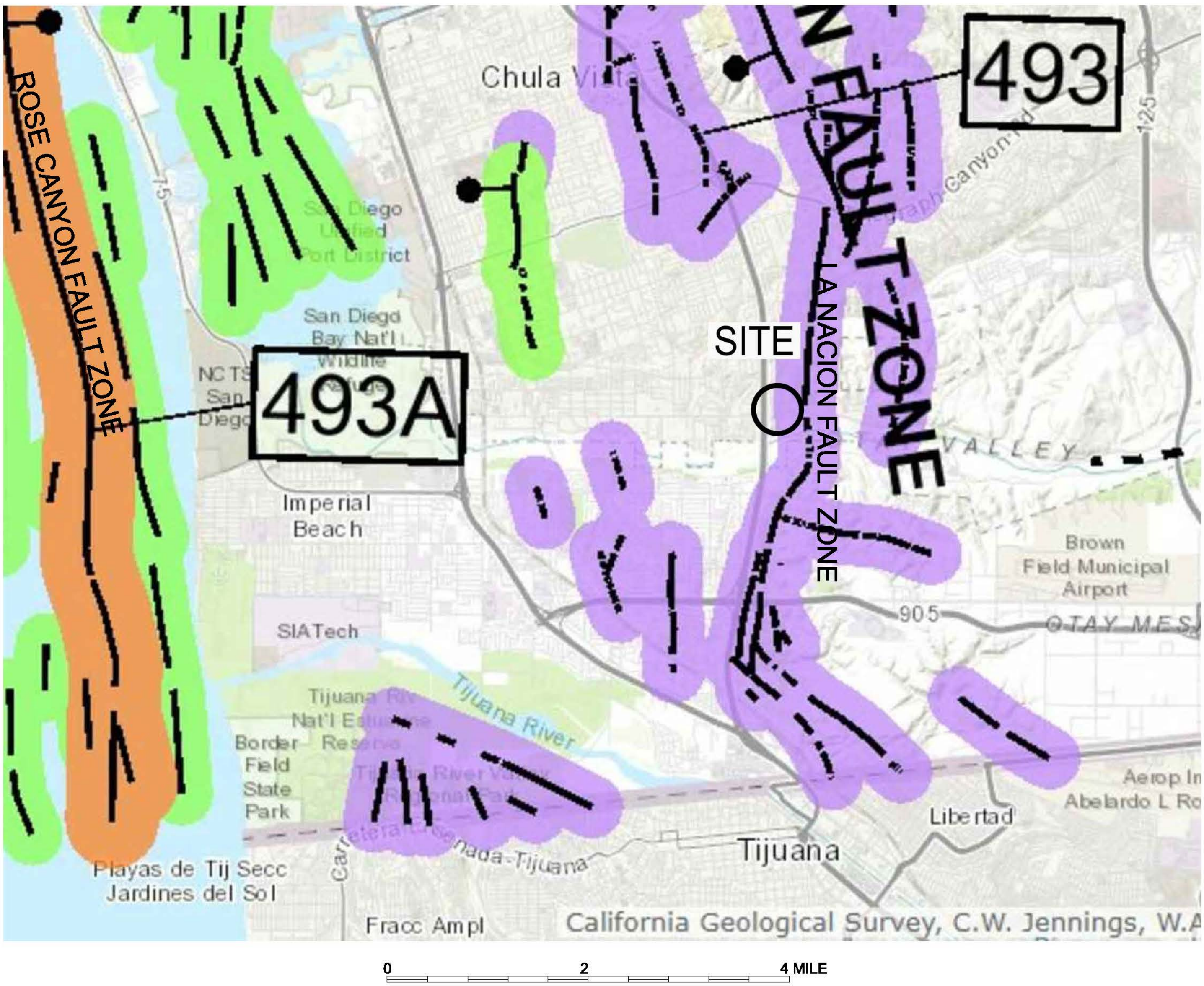
Arrow on fault indicates direction of dip.

Low angle fault (barbs on upper plate).

OTHER SYMBOLS

491

Numbers refer to annotations listed in the appendices of the accompanying report.



CALIFORNIA FAULT ACTIVITY MAP

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FIGURE 10

Reference: CGS, Geologic Data Map 06, Fault Activity Map, 2010, Jennings, California Division of Mines and Geology.

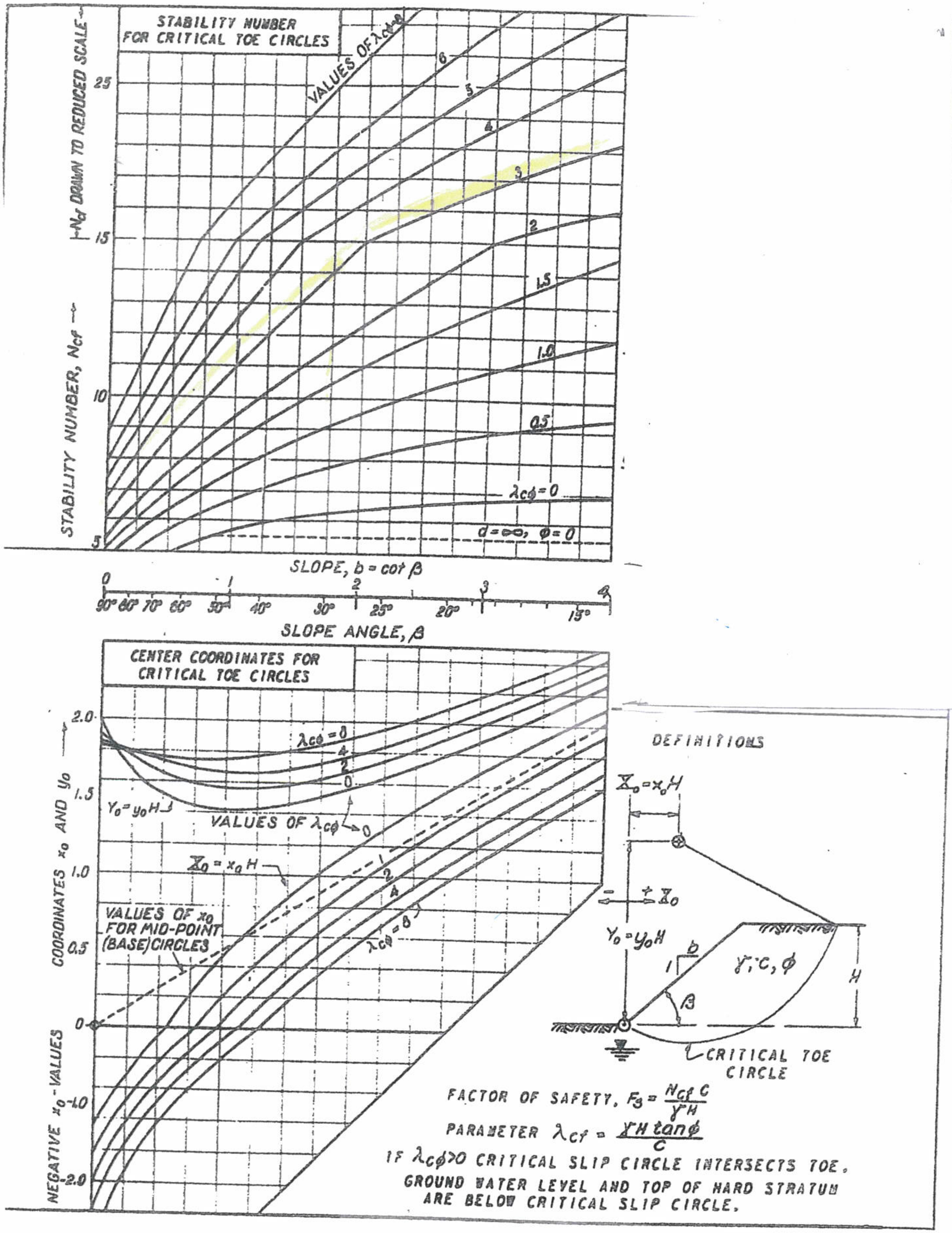
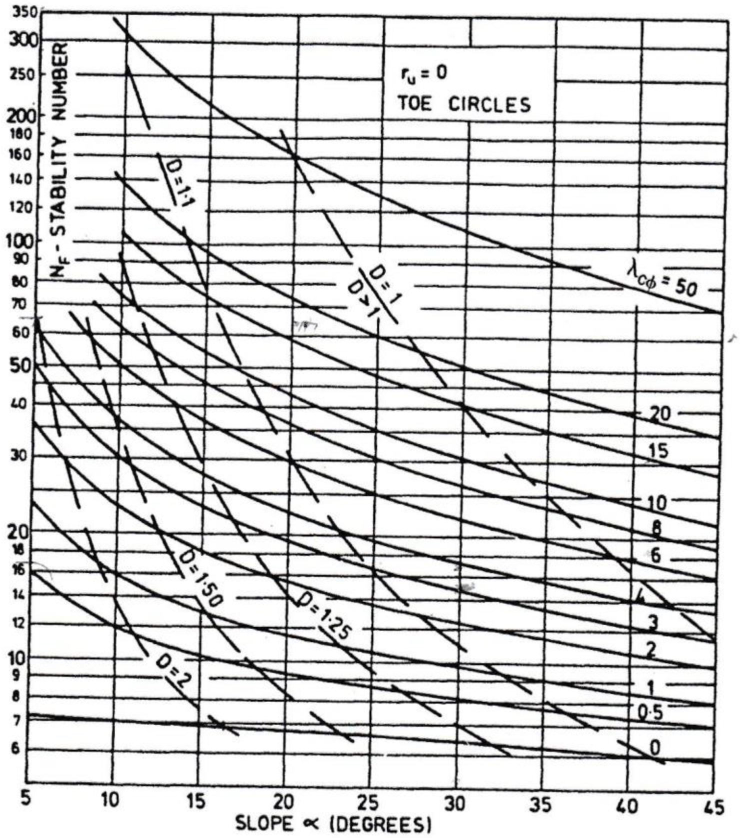


FIGURE: 11

Stability Analysis for Slopes With ϕ and c.



Cousins (1978) chart for failure analysis through the toe of the slope and zero pore water pressures in the slope ($r_u = 0$). See Table 13.13. *(Reprinted with permission of the American Society of Civil Engineers.)*

Figure 12

Reference: Rock Slope Engineering
Federal Highway Administration
FHWA-T8-79-208 – Page A59

Slope Stability Analysis for STA #2 and STA #3 Locations

$W = \frac{1}{2} \gamma H^2 (\cot \alpha - \cot i)$ · · · · · · · · · · · · · (1)

where γ is the unit weight of the rock and H the height of the slope, α = dip of structural discontinuities, i = angle of slope, and W = weight of block

DF = The disturbing force tending to cause the block to slide down

$= W \sin \alpha$

$= \frac{1}{2} \gamma H^2 (\cot \alpha - \cot i) \sin \alpha$ · · · · · · · · · · · · · (2)

RF = The resisting force acting upwards along the α-plane and tending to resist sliding down the plane

$= c_a L + N \tan \varnothing_a$

$= c_a H \cdot \mathrm{cosec}\, \alpha + \frac{1}{2} \gamma H^2 (\cot \alpha - \cot i) \cos \alpha \cdot \tan \varnothing_a$ · · · · · · · · · · · · · (3)

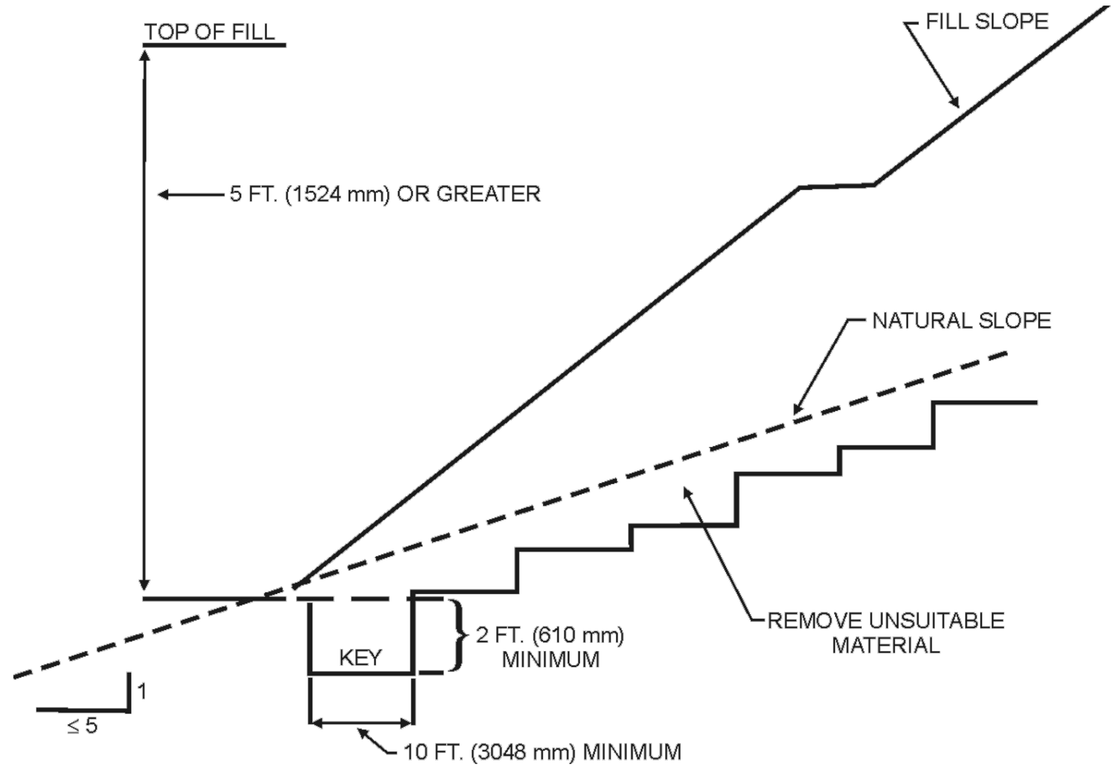
For equilibrium DR = RF · · · · · · · · · · · · · (4)

The factor of safety, F, against sliding down the α-plane is the ratio of the resisting to the disturbing force, i.e.

F = RF/DF · · · · · · · · · · · · · (5)

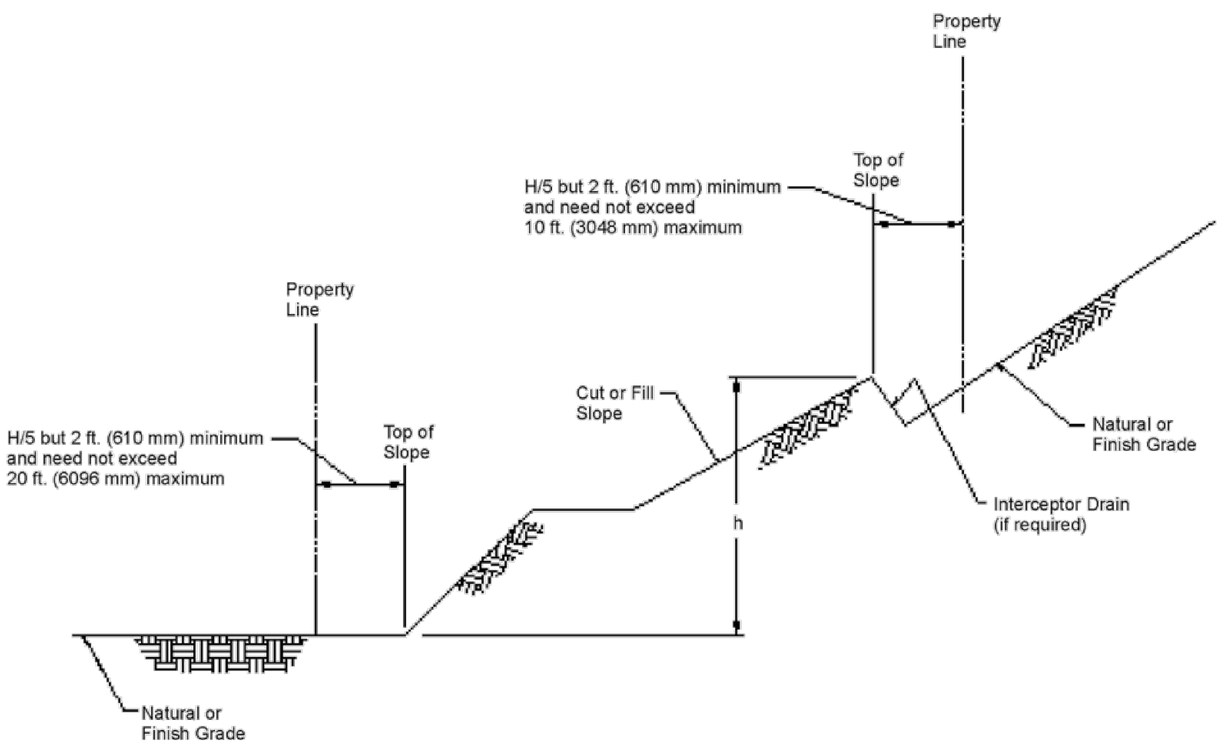
Notes:

(1) For STA #2 and STA #3 locations, $\alpha > i$, the term $(\cot \alpha - \cot i)$ in equation (2) for the disturbing force becomes negative, which is meaningless. The joint dips beneath the slope and the slope is stable.
(2) STA #2 and STA #3 are shown at location on Figure 2 and Figure 5



For SI: 1 foot = 304.8 mm.

FIGURE J107.3
BENCHING DETAILS



BENCHING DETAILS AND DRAINAGE DIMENSIONS

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FIGURE 14

Reference: CBC 2013, Figure J107.3 Benching Detail and J108.1 Drainage Dimensions

Legend to Engineering Geologic Mapping, Rock Boring and Trenches

Igneous and Metamorphic Rock Grain Size Descriptors

Descriptors	Average Crystal Diameter
Very coarse-grained or pegmatic	> 10mm (> 3/8 in)
Coarse-grained	5-10 mm (3/16 – 3/8 in)
Medium-grained	1-5 mm (1/32 – 3/16 in)
Fine-grained	0.1-1 mm (0.04 – 1/32 in)
Aphanitic (cannot be seen with the unaided eye)	< 0.1 mm (<0.04 in)

Bedding, Foliation or Flow Texture Descriptors

Descriptor	Thickness / Spacing
Massive	> 10 ft (> 3 m)
Very thickly (bedded, foliated or banded)	3 – 10 ft (1 – 3 m)
Thickly	1 – 3 ft (300 mm – 1 m)
Moderately	0.3 – 1 ft (100 – 300 mm)
Thinly	0.1 – 0.3 ft (30 – 100 mm)
Very Thinly	0.03 {3/8 in} – 0.1 ft (10 – 30 mm)
Laminated (intensely foliated or banded)	< 0.03 ft {3/8/ in} (< 10 mm)

Weathering Descriptors of Rocks

Descriptor	Symbol	Diagnostic Features
Fresh	FR	No Discoloration, not oxidized
Slightly Weathered	SW	Discoloration or oxidation is limited to surface or, or short distance from, fractures: some feldspar crystals are dull
Moderately Weathered	MW	Discoloration or oxidation extends from fractures, usually throughout: Fe-Mg minerals are "rusty," feldspar crystals are "cloudy."
Highly Weathered	HW	Discoloration or oxidation throughout: all feldspars and Fe-Mg minerals are altered to clay to some extend; or chemical alteration produces in-situ disaggregation. See grain boundary conditions
Completely Weathered	XW	Discoloration or oxidation throughout; but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay

RQD: Rock Quality Designation Criteria

RQD%	Velocity Index	Rock Mass Quality
90 – 100	0.8 – 1.0	Excellent
75 - 90	0.6 – 0.8	Good
50 - 75	0.4 – 0.6	Fair
25 - 50	0.2 – 0.4	Poor
0 - 25	0 – 0.2	Very Poor

Rock Hardness / Strength Descriptors

Descriptor	Symbol	Diagnostic Features
Very Hard	VH	Cannot be scratched with knife or sharp pick.
Hard	H	Can be scratched with knife or sharp pick with difficulty (heavy pressure). Heavy hammer blow required to break specimen.
Moderately Hard	MH	Can be scratched with knife or sharp pick with light or moderate pressure. Core or fragment breaks with moderate hammer blow.
Soft	S	Can be grooved or gouged easily by knife or sharp pick with light pressure, can be scratched with fingernail. Breaks with light to moderate manual pressure.
Very Soft	VS	Can be readily indented, grooved or gouged with fingernail. Breaks with light manual pressure.

% Recovery = $\frac{\text{Total Length of Core Recovered}}{\text{Length of Coring Interval}} \times 100$

R.Q.D.* = $\frac{\text{Sum of Length of all core pieces equal to or greater than 4 inches}}{\text{Length of coring intercal}}$

* No mechanically induced core breaks are included in the computation of the RQD

* R.Q.D. Rock Quality Designation

Fracture Conditions

Symbol	Diagnostic Features
G	Good fit between the fracture sides
P	Poor fit between the facture sides

Weathering Conditions of Fracture Surfaces

Symbol	Diagnostic Features
H	High weathering effects on fracture surface
S	Slight weathering effects on fracture surface
M	Moderate weathering effects on fracture surface

References: 1) Engineering Geology Field Manual, Second Edition, Volume 1, U.S Department of the Interior., Bureau of Reclamation, 1998

2) Naval Facilities Engineering Command, NAVFAC DM-7.1, 7.2, 7.3, Design Manual. Soil Mechanics and Foundations. May 1982, April 1983

Note: At this project location, RQD criteria pertains to very soft to moderately hard rock

APPENDIX A

Global Stability Study

Slope Stability/Trench Logs

Cross Section A-A′, B-B′, C-C′

Slope Stability & Calculations

Global Stability Analyses

Regional Geologic and Site Engineering Geologic Maps (Figures 4 and 5) and Seismic Hazards Map (Figure 8) indicated the site is not located in the landslide area. Site Geologic mapping indicated the slopes are stable. In addition, Partner performed a slope stability analysis using Rocscience software Slide 2D. A summary of our results is shown in the below table.

All slopes will be subjected to surficial erosion. Therefore, slopes should be protected from surface runoff by means of top of the slopes compacted earth berms.

It is recommended that the slopes should be properly maintained in future by some of these methods: cleaning and removing loose debris, minor grading, controlling surface water, revegetation and by constructing benches. Over- watering and subsequent saturation of slope surface should be avoided.

Slope Stability Analysis – Bishop/Janbu (lowest reported)

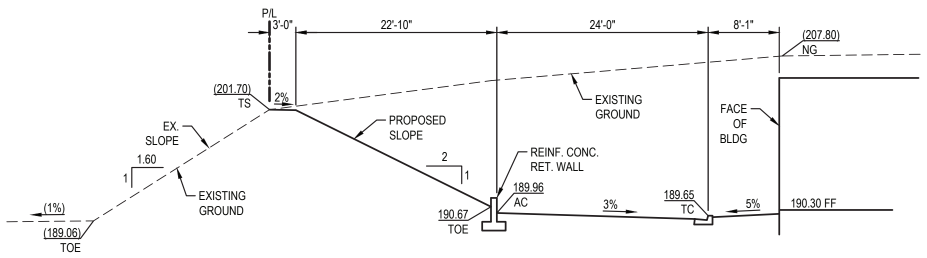
Cross-Section	Slope Height	Slope Angle	Max Retaining Wall Height	Cohesion	Friction Angle	FS Static/Seismic
C-C′	33 feet	2:1 Max	7 feet	100 psf	30 deg	1.8/1.2
D-D′	29 feet	2.2:1 Max	8 feet	100 psf	30 deg	2.2/1.5
G-G′	20 feet	2:1 Max	7 feet	100 psf	30 deg	1.7/1.4
H-H′	45 feet	2:1 Max	14 feet	100 psf	30 deg	***1.37***[a]

[a] ***Factor of safety not sufficient – additional analysis required***

Additional Slope Stability Analysis – Bishop/Janbu (Cross Section H-H′)

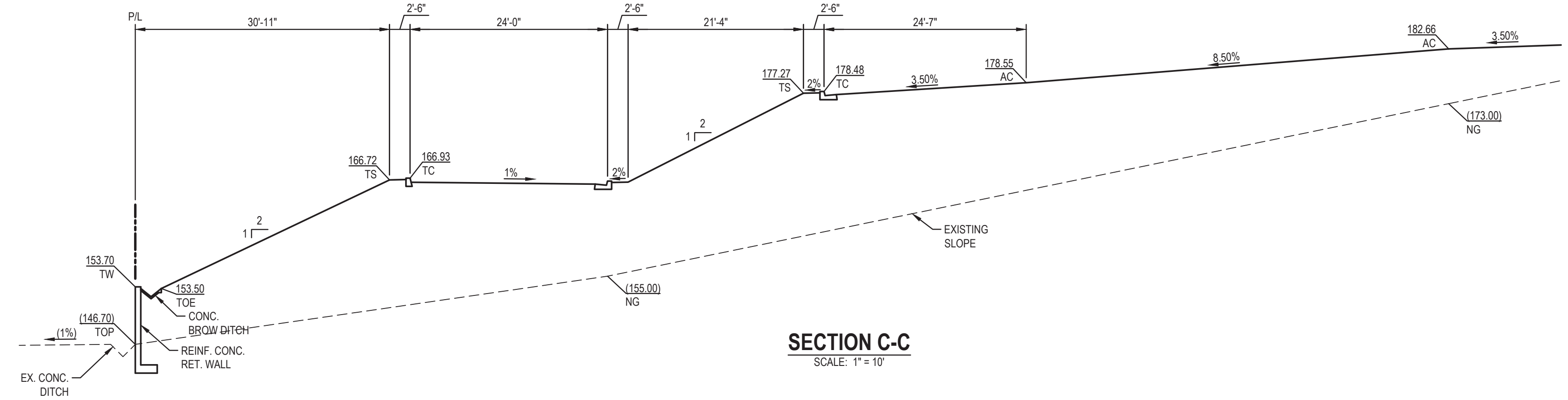
Condition	Slope Height	Slope Angle	Max Retaining Wall Height	Cohesion	Friction Angle	FS Static
Construction Cut	45 feet	1:1 Max	14 feet	100 psf	30 deg	***0.9***[a]
Construction Cut	45 feet	1.5:1 Max	14 feet	100 psf	30 deg	1.05
Foundation 4-ft embedment, 7.5 feet back from wall CL	45 feet	2:1 Max	14 feet	100 psf	30 deg	1.5/1.04

[a] ***Factor of safety not sufficient – additional analysis required***



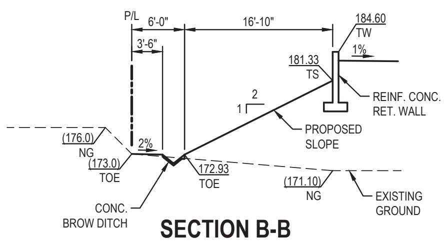
SECTION A-A

SCALE: 1" = 10'



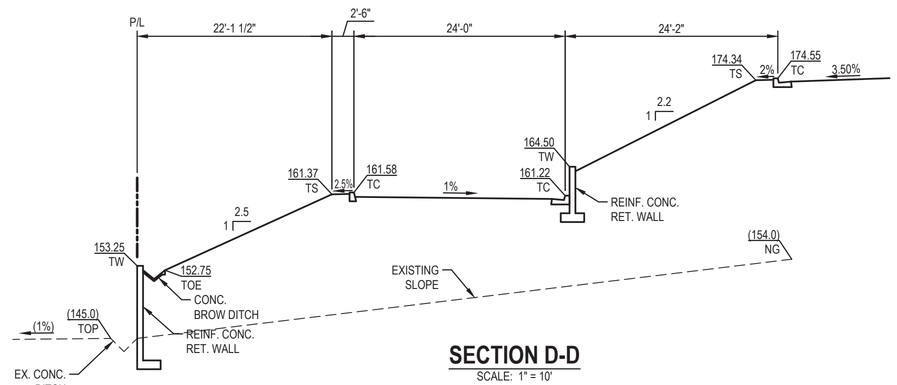
SECTION C-C

SCALE: 1" = 10'



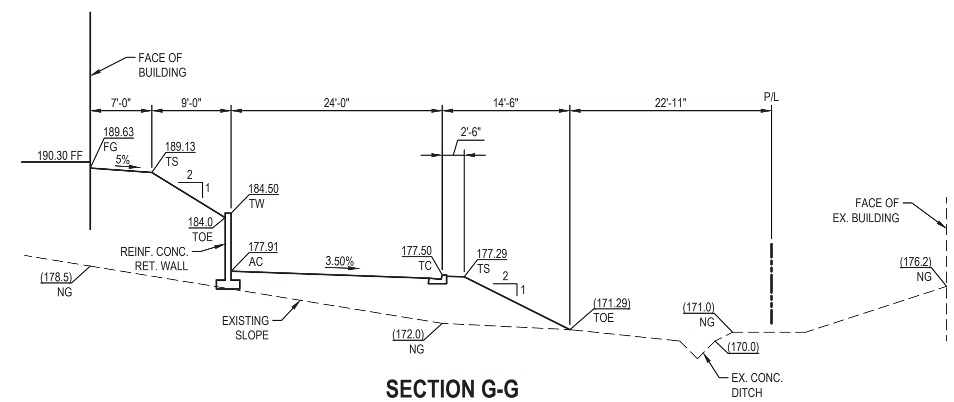
SECTION B-B

SCALE: 1" = 10'



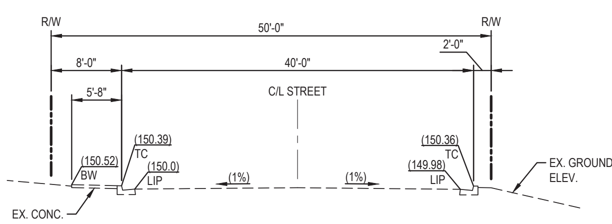
SECTION D-D

SCALE: 1" = 10'



SECTION G-G

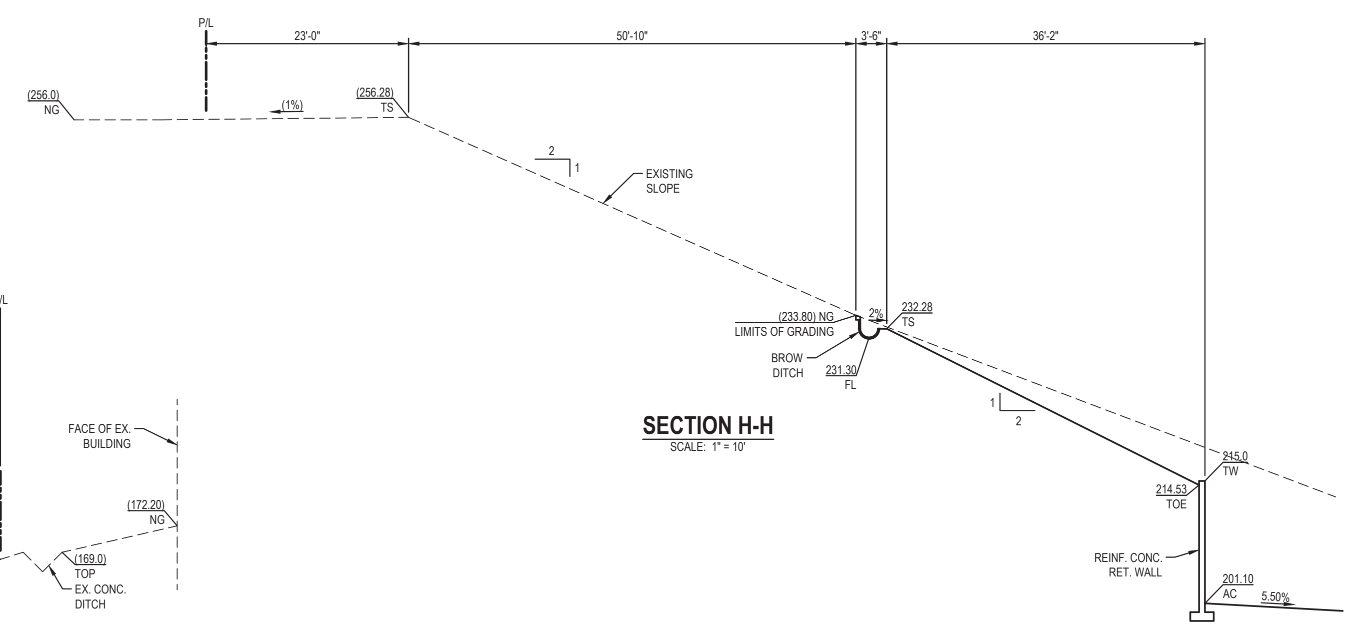
SCALE: 1" = 10'



SECTION E-E

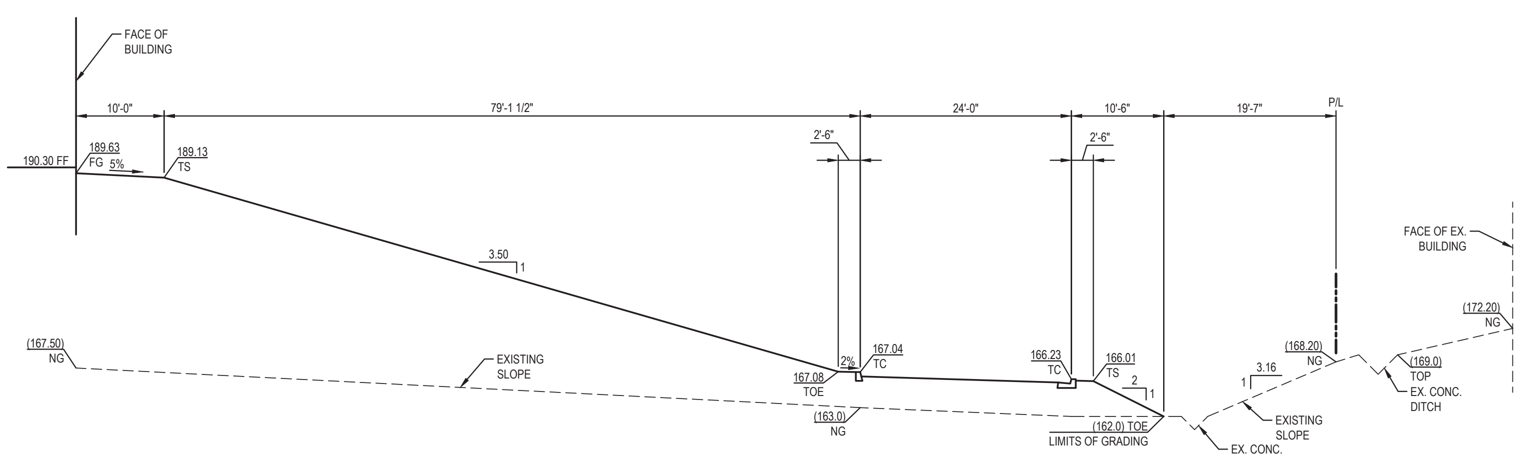
SHINOHARA LANE (PUBLIC STREET)

SCALE: 1" = 10'



SECTION H-H

SCALE: 1" = 10'



SECTION F-F

SCALE: 1" = 10'

BENCH MARK

CITY OF CHULA VISTA SURVEY CONTROL NETWORK BENCHMARK DESIGNATION 1046, BEING A 1-1/2" BRASS DISK ON N SIDE MAIN ST 150' +-E OF OLEANDER AVE ON E END OF CURB INLET. ELEVATION 128.970 FEET (NAVD88).

BASIS OF BEARINGS

BEING THE WESTERLY LINE OF LOT 1 OF SEC 19, T18S, R1W, S.B.M. AS SHOWN ON ROS 21570, COUNTY OF SAN DIEGO, A BEARING OF N00°24'07"E

NO.	DATE	REVISION DESCRIPTION	APP'D	DATE

REGISTERED PROFESSIONAL ENGINEER RABIH F. MAHMOUD 60421 EXPIRES 6/30/20 CIVIL STATE OF CALIFORNIA

APD CONSULTANTS, INC.
PLANNING, ENGINEERING, CONSTRUCTION MANAGEMENT
188 Technology Dr., Suite B, IRVINE, CA 92618
TEL: (949) 336-6336 FAX: (949) 336-6338 www.apdcon.com

RABIH F. MAHMOUD, P.E. R.C.E. NO. 60421 DATE 3/15/2019

SECTIONS AND DETAILS

ENCOMPASS HEALTH CHULA VISTA

517 SHINOHARA LANE, CHULA VISTA, CA 91911

JOB NO. 18033

DATE: 3/15/2019

SHEET 2 OF 5 SHEETS

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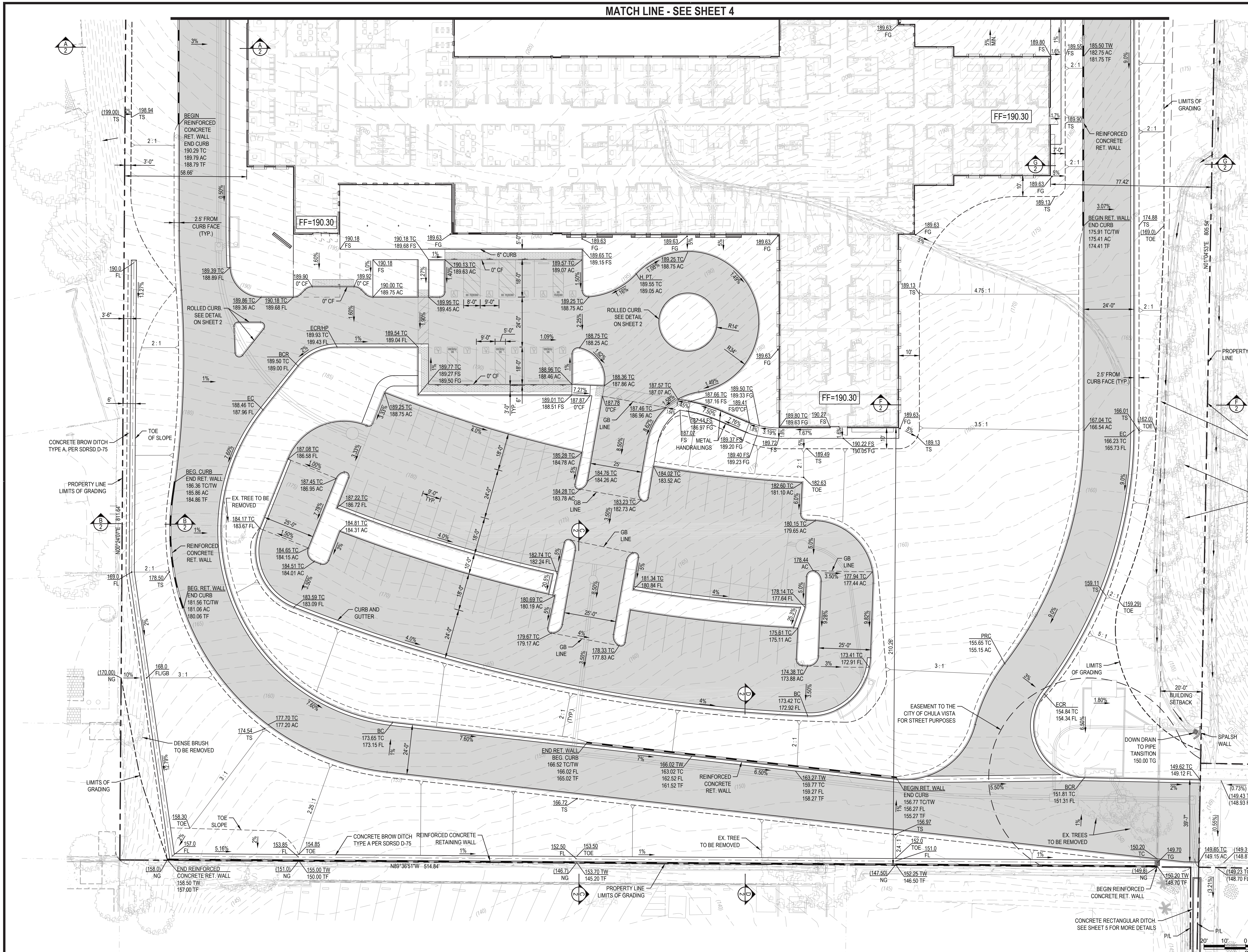

PAVING LEGEND:

PCC PAVEMENT

AC PAVEMENT

ABBREVIATIONS:

AC	ASPHALT CONCRETE
CF	CURB FACE
EX.	EXISTING
FF	FINISHED FLOOR
FG	FINISHED GROUND
FL	FLOW LINE
FS	FINISHED SURFACE
LP	LOW POINT
MAX.	MAXIMUM
MIN.	MINIMUM
P/L	PROPERTY LINE
R/W	RIGHT OF WAY
TB	TOP OF BERM
TC	TOP OF CURB
TG	TOP OF GRATE

SEE SHEET 2 FOR SECTIONS A-A TO H-H

BENCH MARK

CITY OF CHULA VISTA SURVEY CONTROL NETWORK BENCHMARK DESIGNATION 1046, BEING A 1-1/2" BRASS DISK ON N SIDE MAIN ST 150' +-E OF OLEANDER AVE ON E END OF CURB INLET. ELEVATION 128.970 FEET (NAVD88).

BASIS OF BEARINGS

BEING THE WESTERLY LINE OF LOT 1 OF SEC 19, T18S, R1W, S.B.M, AS SHOWN ON ROS 21570, COUNTY OF SAN DIEGO, A BEARING OF N00°24'07"E.

NO.	DATE	REVISION DESCRIPTION	APP'D	DATE
-	-	-		

REGISTERED PROFESSIONAL ENGINEER RABIH F. MAHMOUD 60421 EXPIRES 6/30/20 CIVIL STATE OF CALIFORNIA

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188 Technology Dr., Suite B, IRVINE, CA 92618
TEL: (949) 336-6336 FAX: (949) 336-6338 www.apdcon.com

RABIH F. MAHMOUD, P.E. R.C.E. NO.: 60421 DATE 3/15/2019

PRELIMINARY GRADING AND DRAINAGE PLAN

ENCOMPASS HEALTH CHULA VISTA

517 SHINOHARA LANE, CHULA VISTA, CA 91911

JOB NO. 18033

DATE: 3/15/2019

SHEET 3 OF 5 SHEETS

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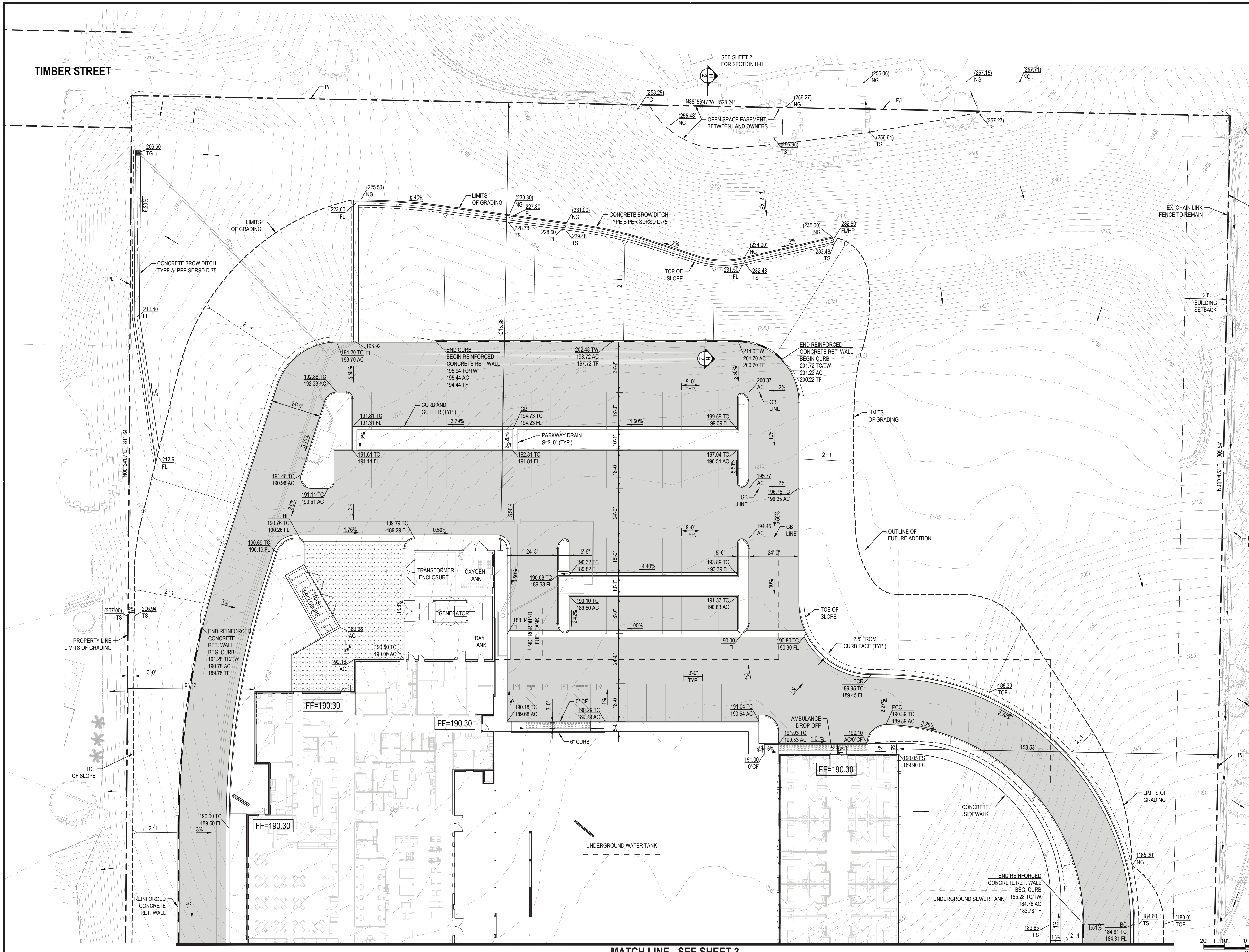

MATCH LINE - SEE SHEET 3

PAVING LEGEND:

PCC PAVEMENT

AC PAVEMENT

ABBREVIATIONS:

AC	ASPHALT CONCRETE
CF	CURB FACE
EX.	EXISTING
FF	FINISHED FLOOR
FG	FINISHED GROUND
FL	FLOW LINE
FS	FINISHED SURFACE
LP	LOW POINT
MAX.	MAXIMUM
MIN.	MINIMUM
P/L	PROPERTY LINE
R/W	RIGHT OF WAY
TB	TOP OF BERM
TC	TOP OF CURB
TG	TOP OF GRATE

SEE SHEET 2 FOR SECTIONS A-A TO H-H

SCALE: 1" = 20'

BENCH MARK

CITY OF CHULA VISTA SURVEY CONTROL NETWORK BENCHMARK DESIGNATION 1046, BEING A 1-1/2" BRASS DISK ON N SIDE MAIN ST 150' +-E OF OLEANDER AVE ON E END OF CURB INLET. ELEVATION 128.970 FEET (NAVD88).

BASIS OF BEARINGS

BEING THE WESTERLY LINE OF LOT 1 OF SEC 19, T18S, R1W, S.B.M, AS SHOWN ON ROS 21570, COUNTY OF SAN DIEGO, A BEARING OF N00°24'07"E.

NO.	DATE	REVISION DESCRIPTION	APP'D	DATE
-	-	-		

REGISTERED PROFESSIONAL ENGINEER ★ RABIH F. MAHMOUD 60421 EXPIRES 6/30/20 CIVIL ★ STATE OF CALIFORNIA

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RABIH F. MAHMOUD, P.E. R.C.E. NO.: 60421 3/15/2019 DATE

PRELIMINARY GRADING AND DRAINAGE PLAN

ENCOMPASS HEALTH CHULA VISTA

517 SHINOHARA LANE, CHULA VISTA, CA 91911

JOB NO. 18033

DATE: 3/15/2019

SHEET 4 OF 5 SHEETS

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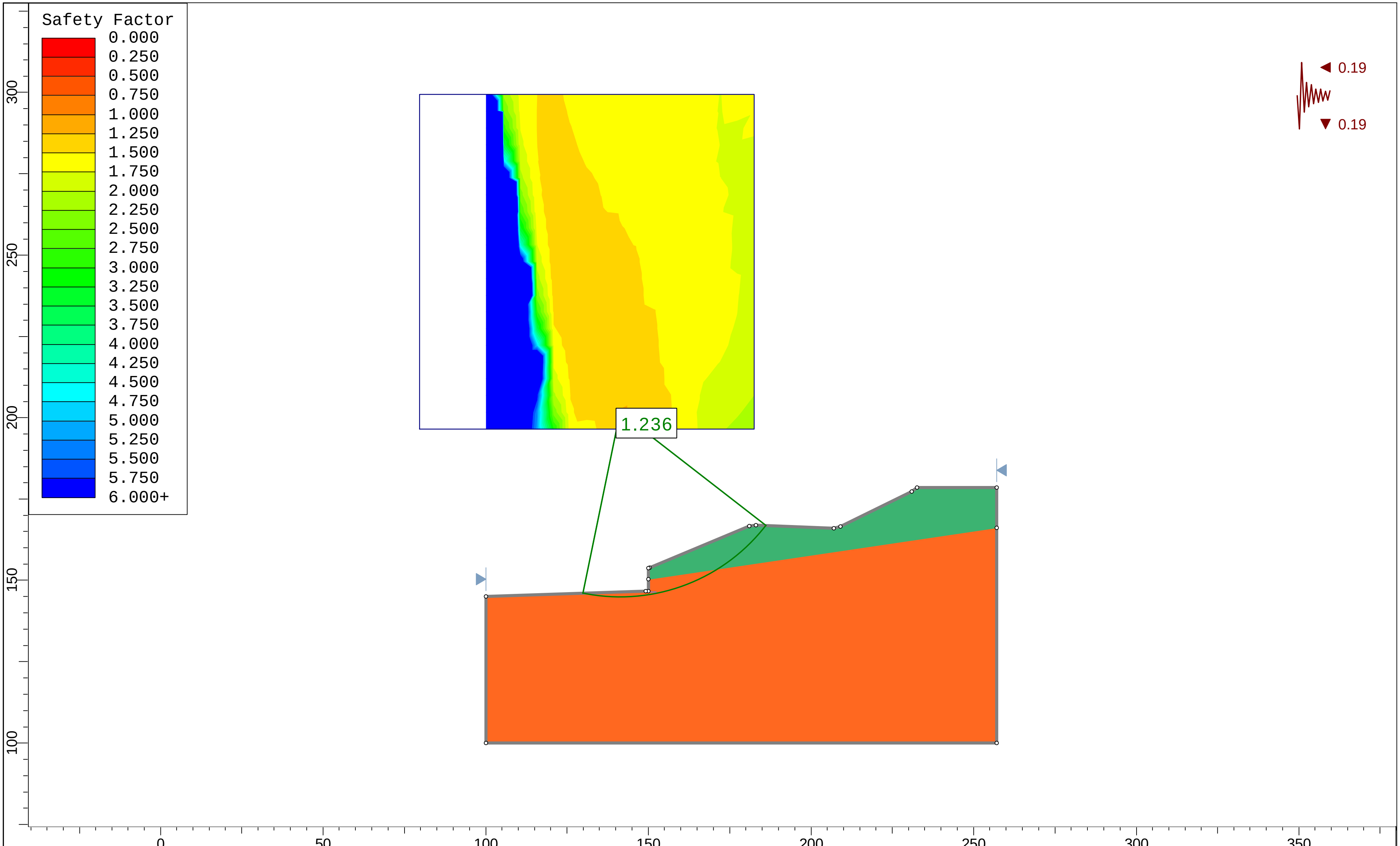

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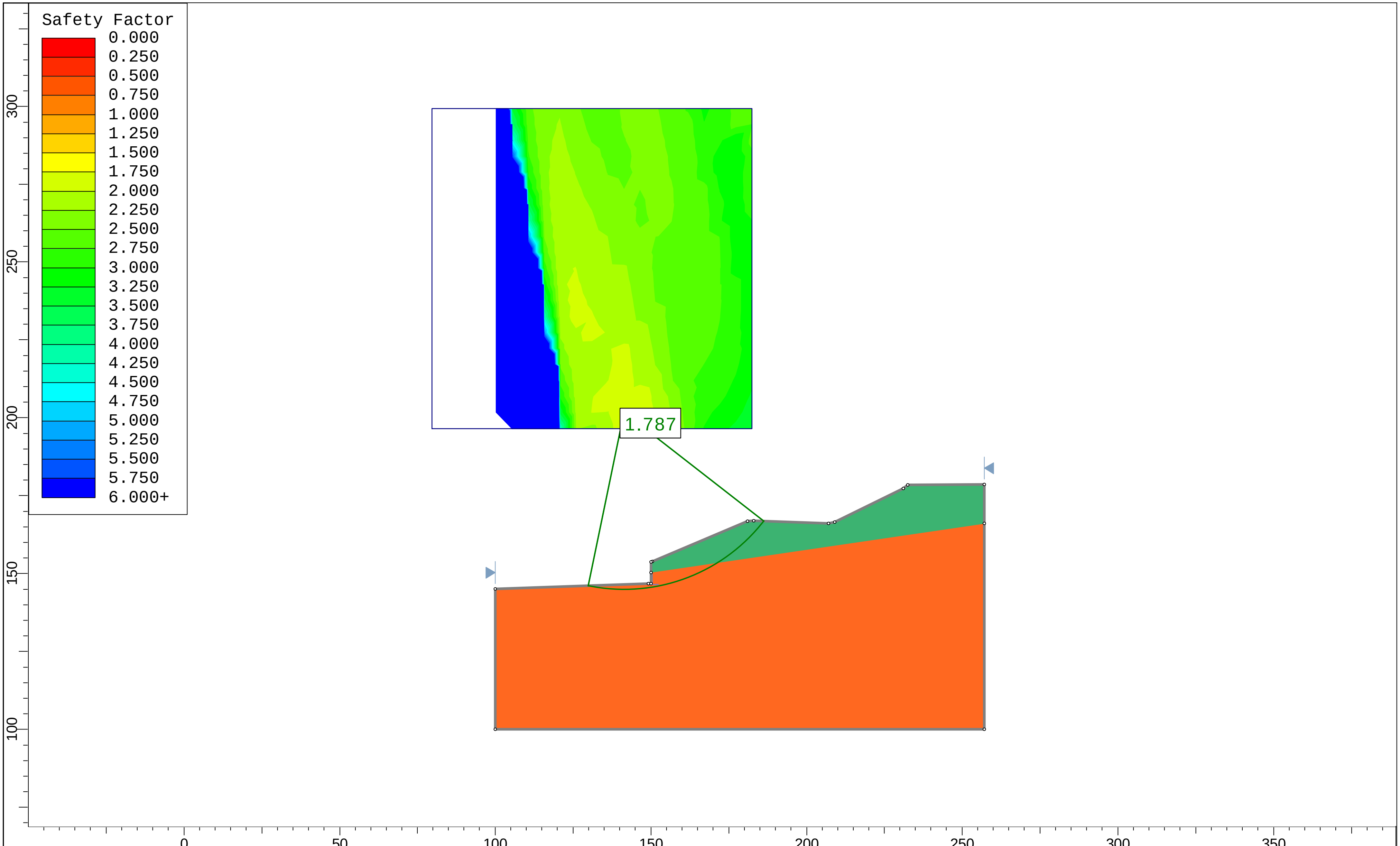

rocscience

SLIDEINTERPRET 8.021

Project	SLIDE - An Interactive Slope Stability Program
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Company	
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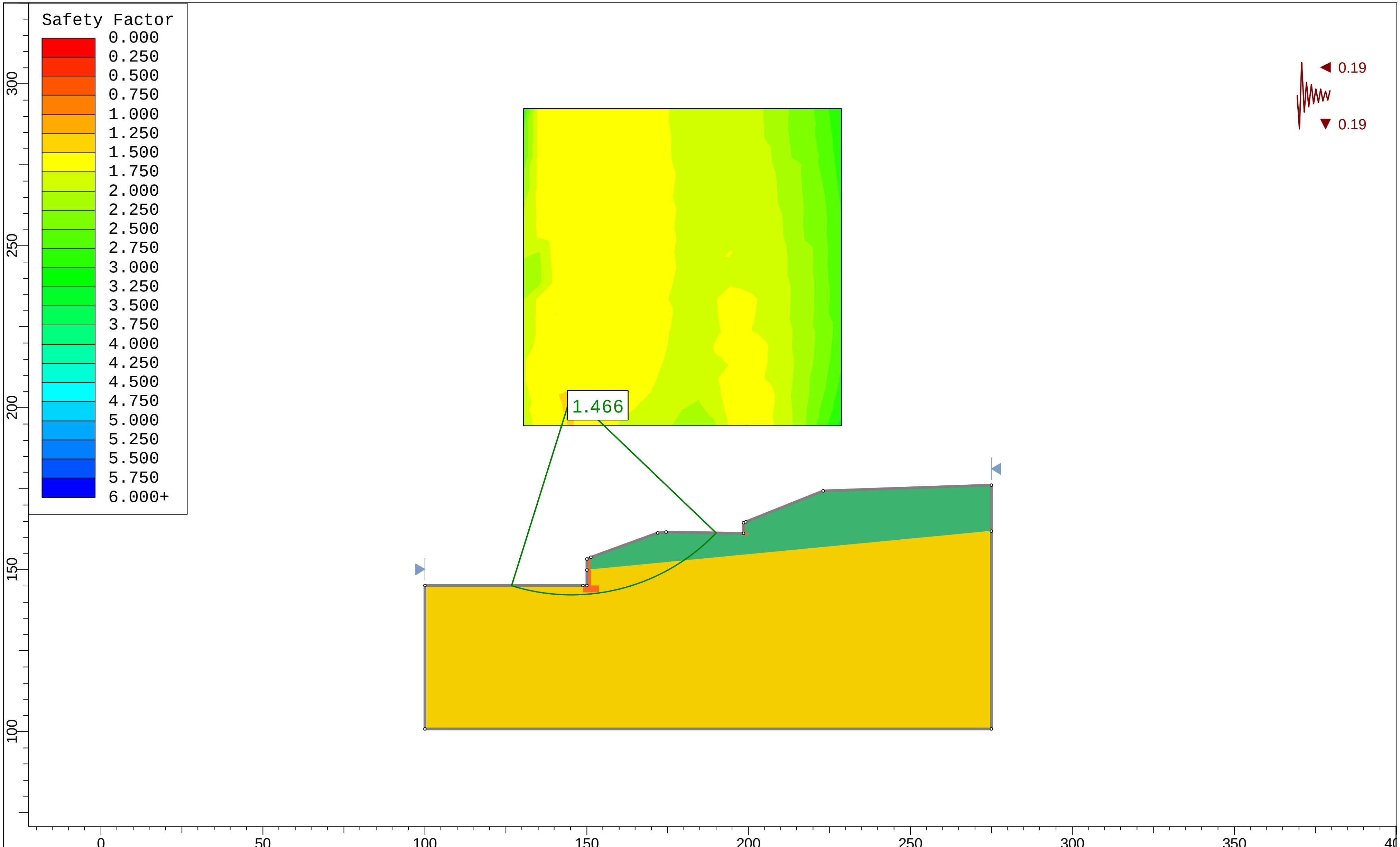

rocscience

SLIDEINTERPRET 8.021

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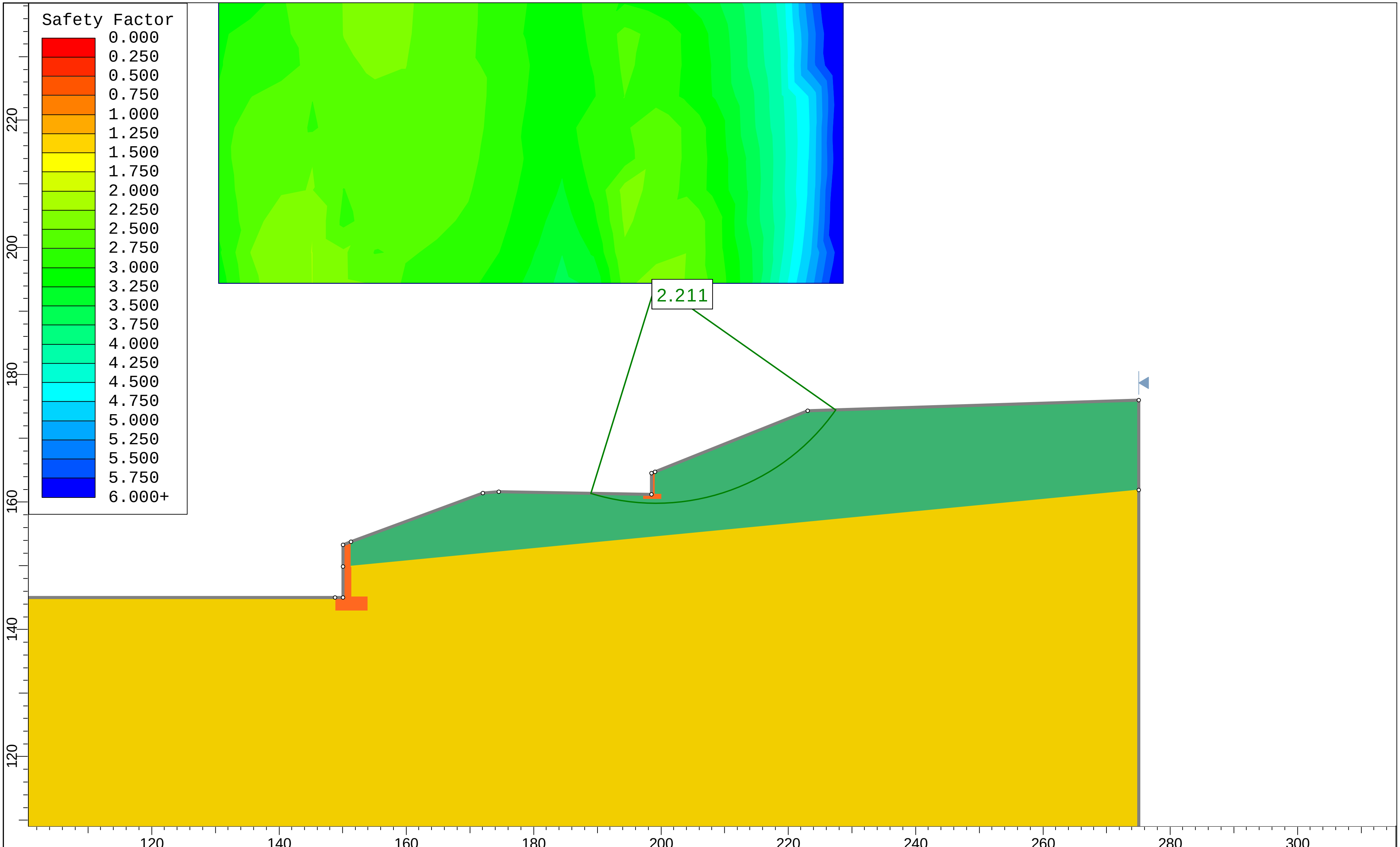

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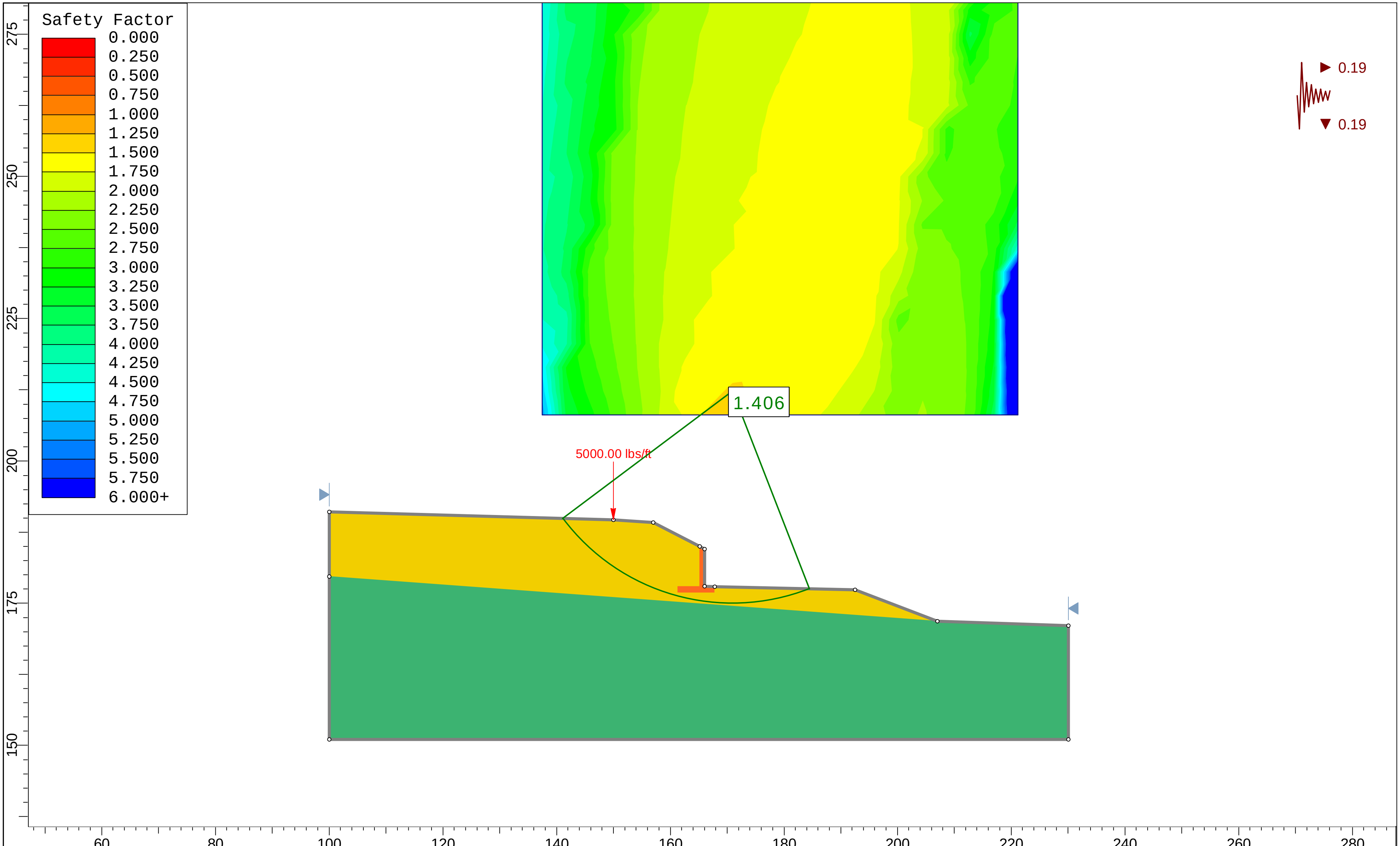

rocscience

SLIDEINTERPRET 8.021

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Analysis Description	Ret Wall Global Stability		
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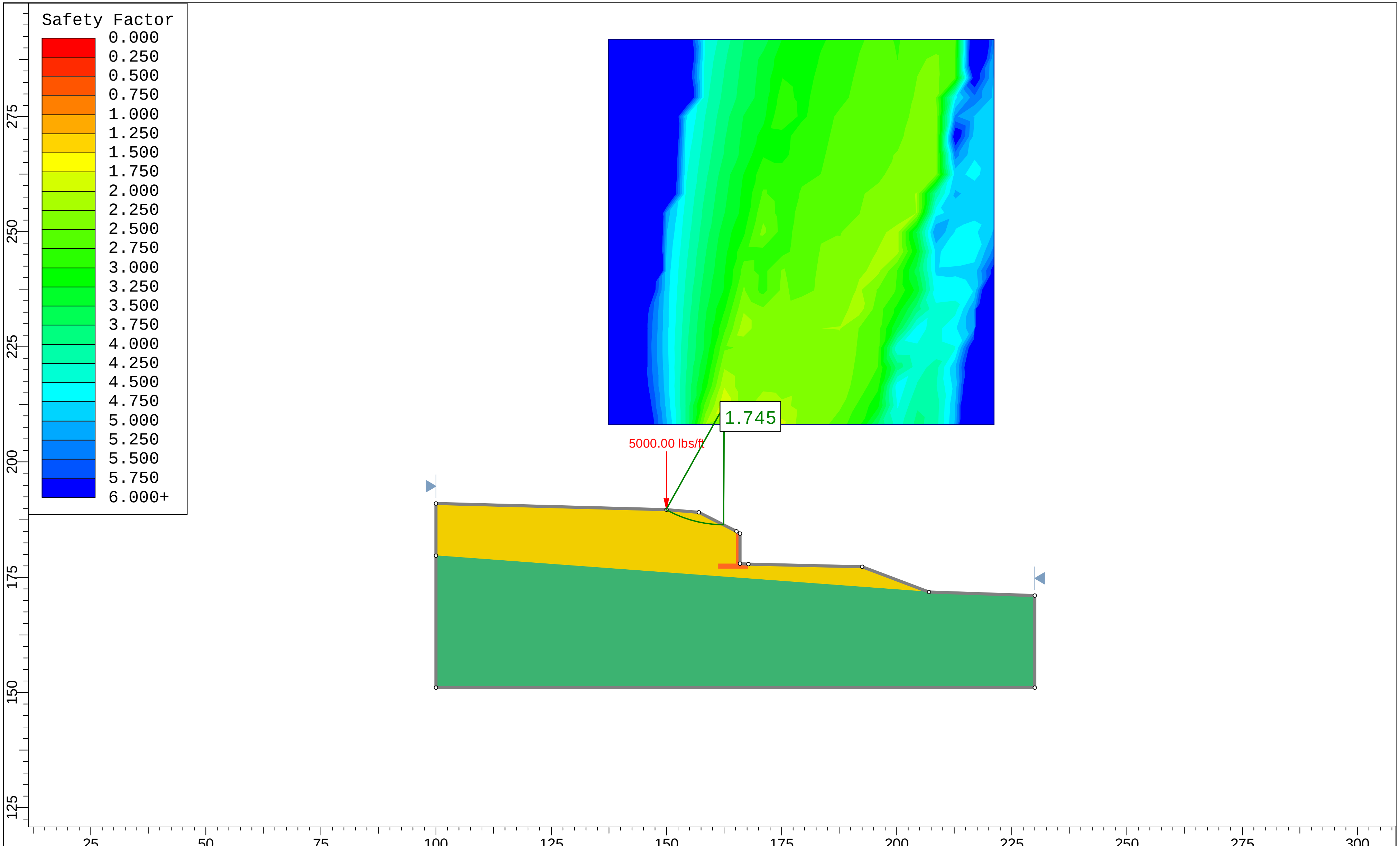
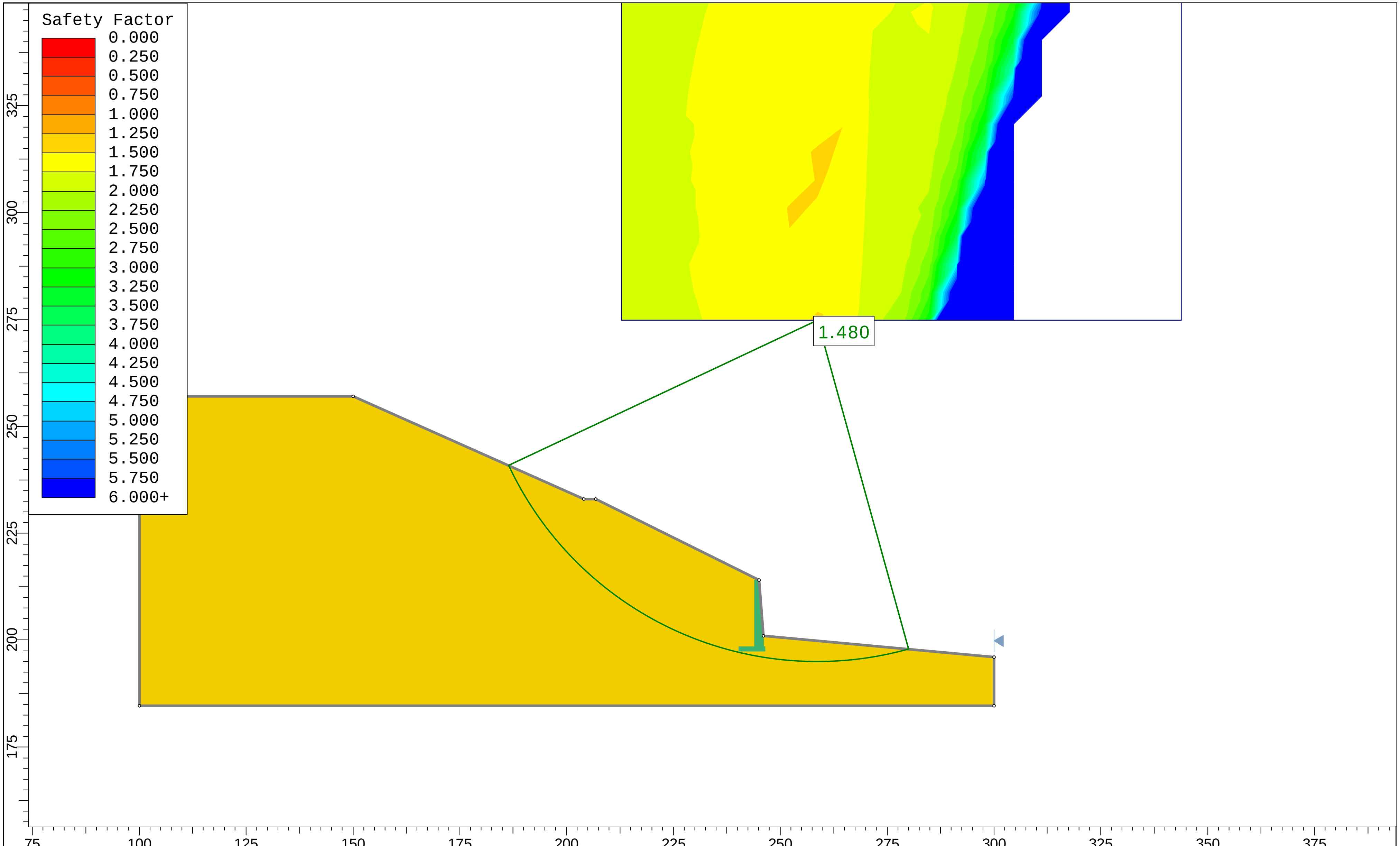


Project	Encompass Hospital		
Analysis Description	Ret Wall Global Stability		
Drawn By: Marcus	Scale: 1:346	Company	Partner Engineering
Date: 3/21/2019		File Name	Slope Analysis Section GG'.slmd

rocscience

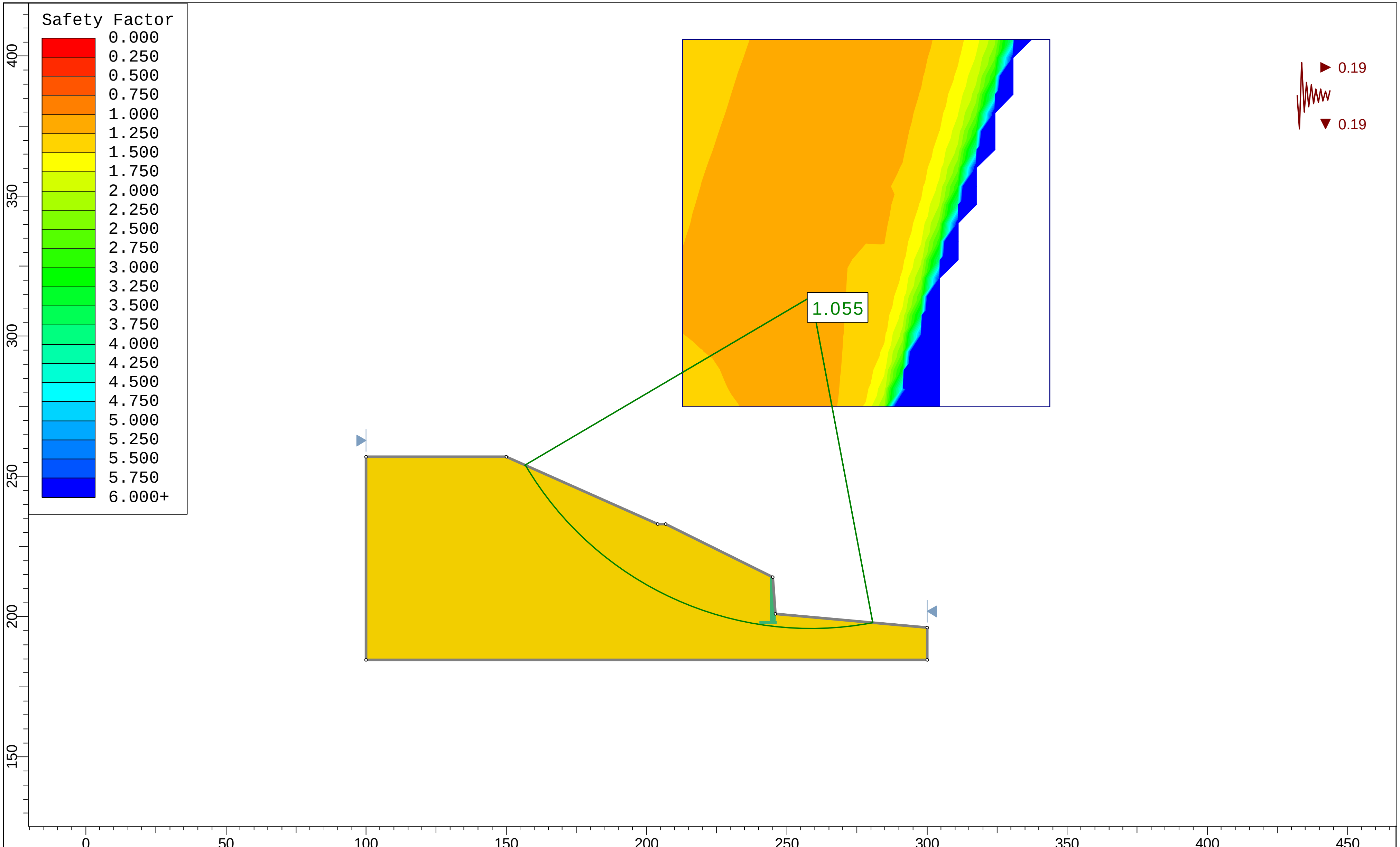
SLIDEINTERPRET 8.021



rocscience

SLIDEINTERPRET 8.021

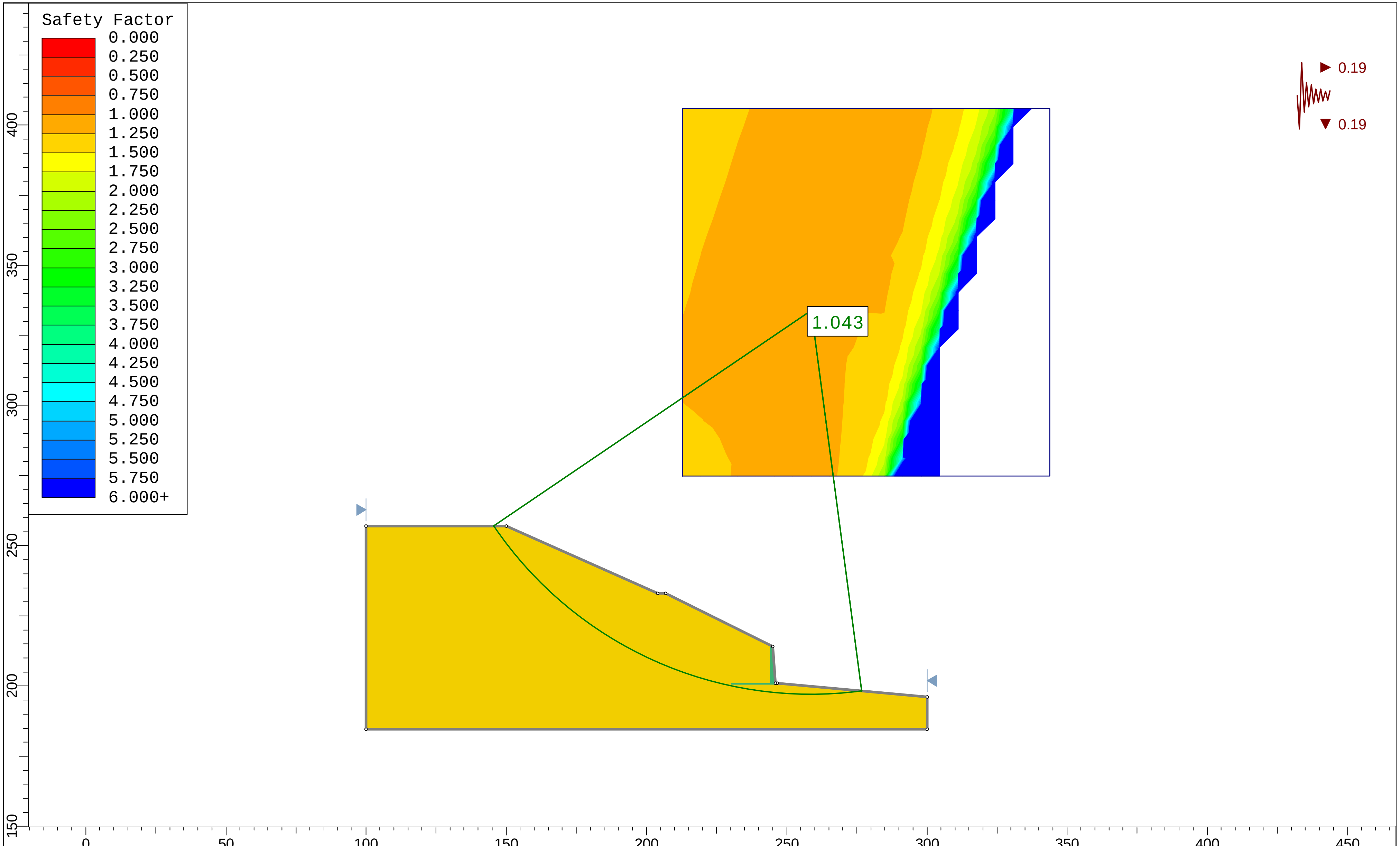
Project	Encompass Hospital		
Analysis Description	Ret Wall Global Stability		
Drawn By Marcus	*Scale* 1:373	*Company*	Partner Engineering
Date 3/21/2019		*File Name*	Slope Analysis Rear Retaining Wall.slmd



rocscience

SLIDEINTERPRET 8.021

Project	Encompass Hospital
Analysis Description	Ret Wall Global Stability
Drawn By	Marcus
Scale	1:568
Company	Partner Engineering
Date	3/21/2019
File Name	Slope Analysis Rear Retaining Wall.slmd



rocscience

SLIDEINTERPRET 8.021

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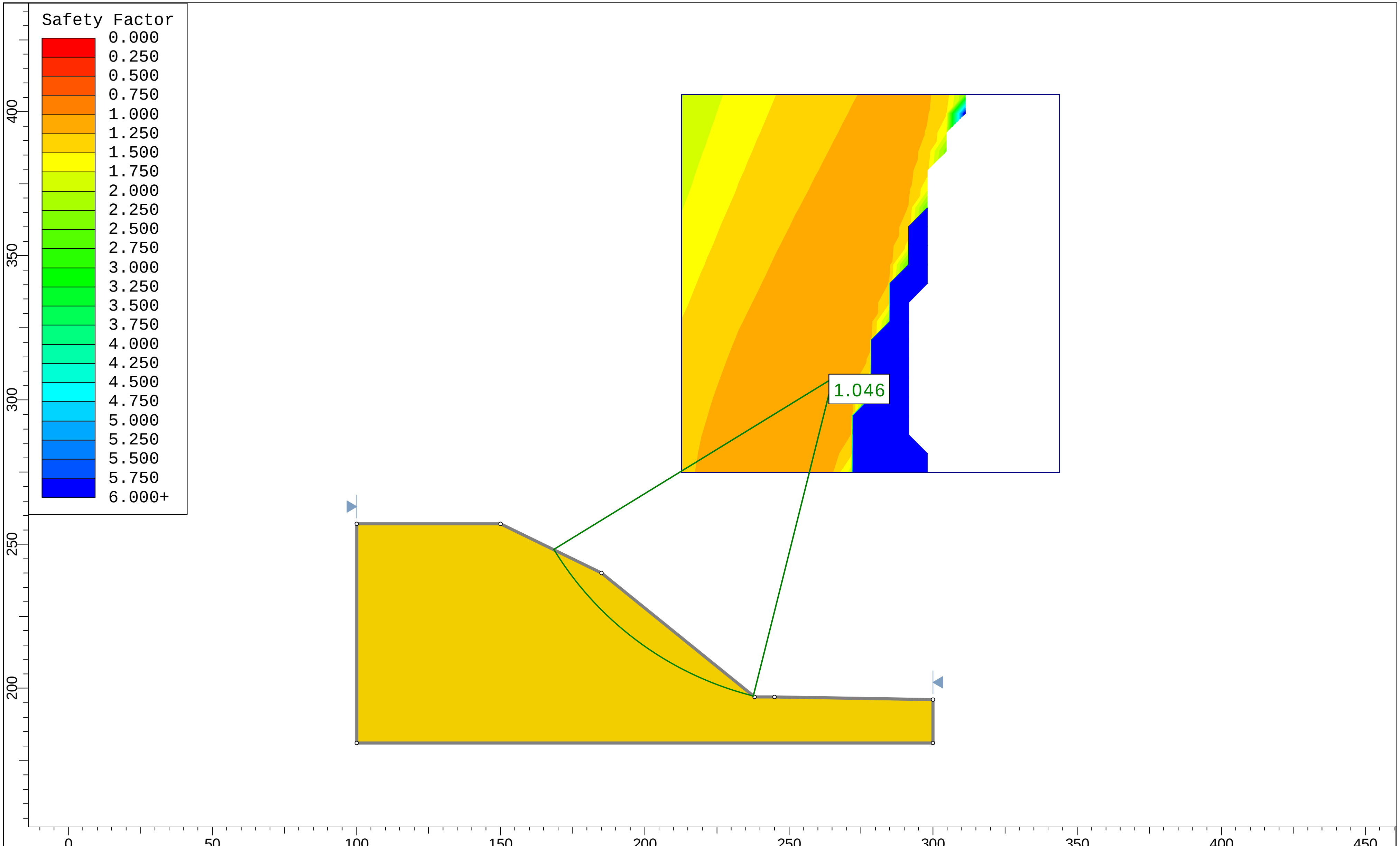
Date 3/12/2019, 12:47:29 PM		*File Name* Slope Analysis Rear Retaining Wall.slmd	



Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description	Temporary Cut HH' for wall		
Drawn By	Marcus	Scale 1:553	Company Partner
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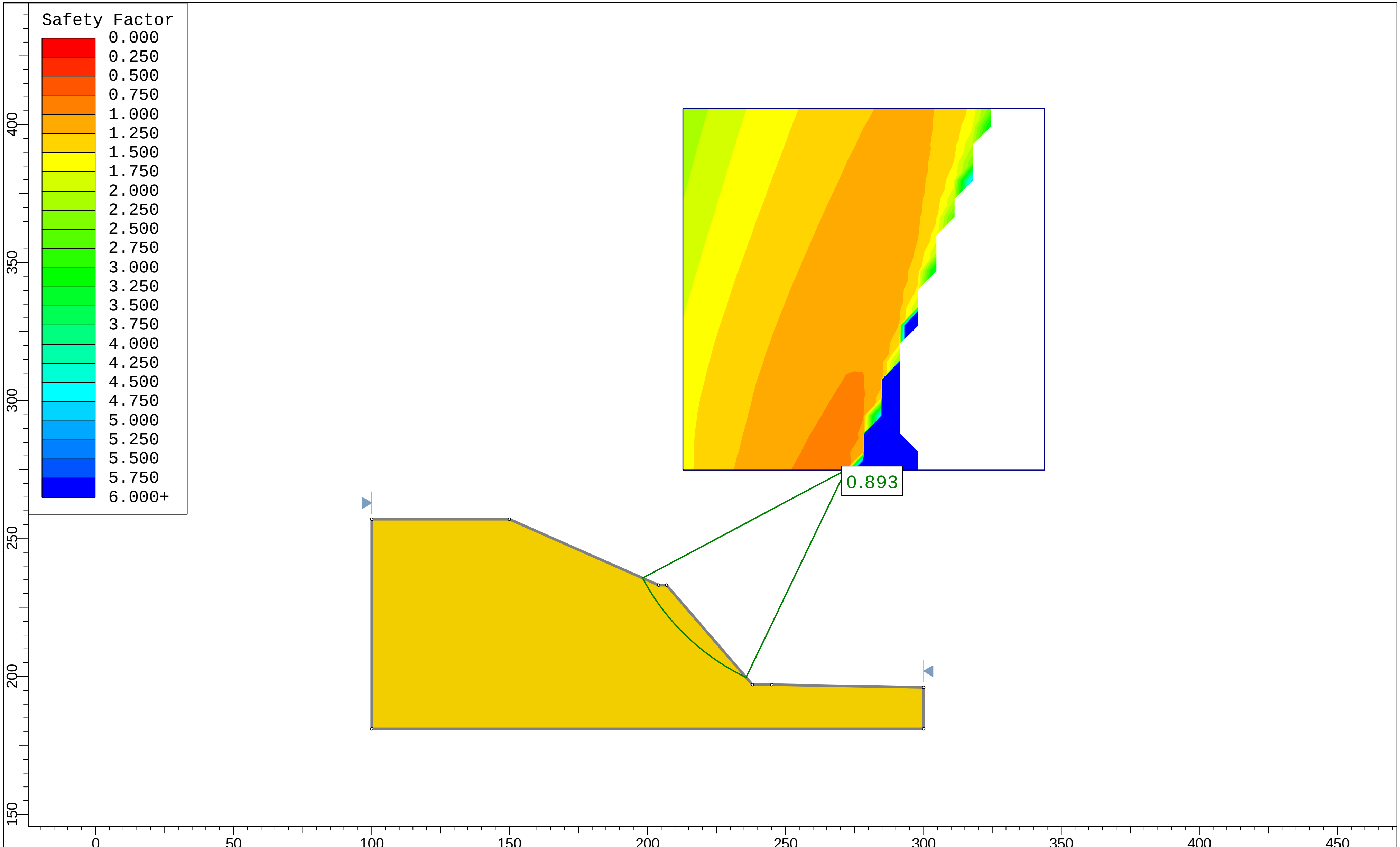
rocscience

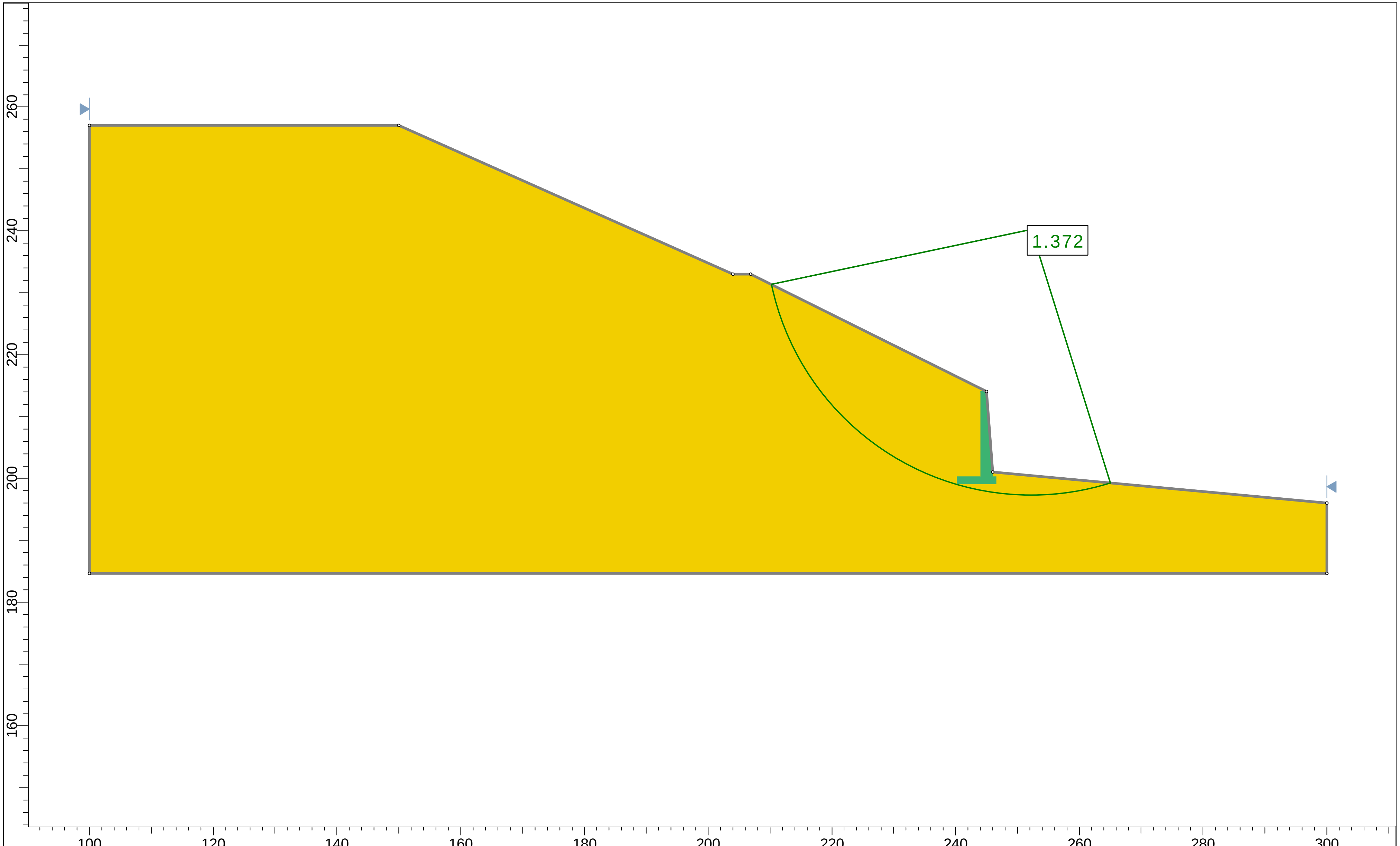
SLIDEINTERPRET 8.021



rocscience

Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description	Temporary Cut HH' for wall		
Drawn By	Marcus	Scale 1:577	Company Partner
Date	3/12/2019, 12:47:29 PM	File Name	Slope Analysis Rear Retaining Wall Cut Condition.slmd

SLIDEINTERPRET 8.021



Project	Encompass Hospital
Analysis Description	Ret Wall Global Stability
Drawn By	Marcus
Scale	1:258
Company	Partner Engineering
Date	3/21/2019
File Name	Slope Analysis Rear Retaining Wall.slmd

rocscience

SLIDEINTERPRET 8.021

Stability Analyses

Regional Geologic and Site Engineering Geologic Maps (Figures 4 and 5) and Seismic Hazards Map (Figure 8) indicated the site is not located in the landslide area. Site Geologic mapping indicated the slopes are stable. Slope stability analysis at the northern slopes (Location STA #1, Figure 5) indicates the slopes are stable with a calculated factor of safety of 2.58 which is greater than the normally accepted minimum for stable slopes. Slope stability analysis was also conducted at the western areas (Location STA #2 and STA #3, Figure 5) indicated the disturbing forces tending to cause the block to slide down becomes negative. The bedding angle is greater than the slope angle. The bedding dips beneath the slope and the slopes are stable. Slope soil properties and Factor of Safety calculation are included in Appendix A.

All slopes will be subjected to surficial erosion. Therefore, slopes should be protected from surface runoff by means of top of the slopes compacted earth berms.

It is recommended that the slopes should be properly maintained in future by some of these methods: cleaning and removing loose debris, minor grading, controlling surface water, revegetation and by constructing benches. Over- watering and subsequent saturation of slope surface should be avoided. We have evaluated the site slopes based on Cousins global stability for weak rock/soil slopes, as well as based on FHWA rock slope sliding analysis from "Rock Slope Engineering" FHWA T8-79-208. We are also providing a Global Stability Analyses, Appendix A

Slope Stability Analysis – Cousins (1978)

Slope ID	Slope Height	Slope Angle	Bedding Angle	Cohesion	Friction Angle	Factor of Safety
STA #1	38 feet	21 degrees	5 degrees	125 psf	35 deg	2.5
STA #2	NA	<3 degrees	4 degrees	125 psf	35 deg	2.5 or more
STA #3	NA	<3 degrees	4 degrees	125 psf	35 deg	2.5 or more

Slope Stability Analysis – FHWA T8-79-208

Slope ID	Slope Height	Slope Angle	Bedding Angle	Cohesion	Friction Angle	Factor of Safety
STA #1	38 feet	21 degrees	5 degrees	125 psf	35 deg	8.3
STA #2	NA	<3 degrees	4 degrees	125 psf	35 deg	ND*
STA #3	NA	<3 degrees	4 degrees	125 psf	35 deg	ND*

Non-Daylighting bedding or fracture planes, cannot be analyzed for block sliding failure

PARTNER

Calculations (location STA #1, Figure 5)

Site Slope Properties

C (cohesion) = 125 psf

γ_d=dry density of soil = 77.3 pcf

m = moisture content = 7.0 %

γ = (saturated density of soil) = γ_d (1 + m) pcf

γ = 77.3 (1.07) =82.71 pcf

B (slope angle) = 21 degrees

Ø = Friction angle = 35 degrees

H = slope height = 38 feet

Cousins (1978) Computation: See Figures 10a and 10b – Reference Charts

$$\lambda c\,Ø = \frac{\gamma\, H \tan Ø}{C} = \frac{82.71 \times 38 \times 0.70}{125}$$

λc Ø = 17.6

From Figure 10a and 10b - Reference Charts

Ncf (Stability Number) = 65

$$\textbf{Factor of Safety},\ FS = \frac{NcfC}{\gamma H} = \frac{65 \times 125}{82.71 \times 38}$$

FS = 2.58 This factor of safety is in excess of the normally accepted minimum for stable slopes.

FHWA T8-79-208 Computation:

Driving Force = Weight x Vector along failure plane

Weight = Density x Area of 2-dimensional sliding wedge with

Area = H^2 (cotangent bedding angle – cotangent of slope angle) = 38^2 (11.4-2.6) = 6,371 sf

Weight = 254 tons

Vector in 5 degree angle 0.09

Driving Force: 44.4 kips

Resisting Force: Cohesion Force + Frictional Force

Cohesion Force = Cohesion x failure plane length = 125 psf x 106 feet = 13.25 kips

Friction Force = Weight x normal vector x tan (friction angle)

Friction Force = 254 tons x 0.99 x tan (35 degrees) = 355.7 kips

Driving Force = 44.4 kips

Resisting Force = 355.7+13.25 kips = 369 kips

Factor of Safety = Resisting Force/Driving Force = 8.3

PARTNER

APPENDIX B

Boring Logs

PARTNER

PARTNER

PROJECT NUMBER	TRENCH NUMBER SHEET 1 OF 1
17-199602.4	TP-1
	TRENCH WALL LOG (A)

PROJECT: Encompass Health
LOCATION: 517 Shinohara Lane, Chula Vista, CA 91911
ELEVATION: 180~; ft MSL
CONTRACTOR: AMG Demolition
DATE EXCAVATED: July 26, 2018
GROUNDWATER LEVEL & DATE: Not encountered
EXCAVATION METHOD: Backhoe: Komatsu PC 390 LC
GEOLOGIST: R. Quraishi
APPROXIMATE DIMENSIONS: LENGTH: 20 ft WIDTH: 15 ft DEPTH: 14 ft
REMARKS: Trench walls stable

LITHOLOGIC DESCRIPTION

(a) 0 to 2.5 ft: Topsoil, blackish brown; fine to coarse sand, some clay & silt, root fragments organic (moist)

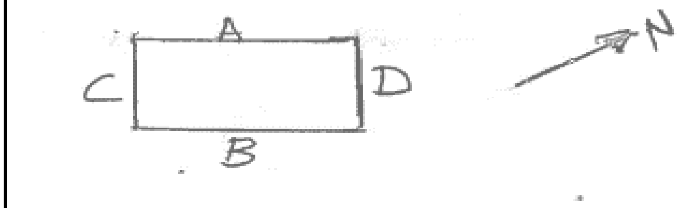
(AF) 2.5 ft to 4 ft: Artificial Fill; orangish brown; fine to coarse sand, some silt & clay, fine to coarse gravel and cobbles, moist (SM* - GM*)

Tsdss (4 to 14 ft) San Diego Formation (silty sandstone): Grayish white to yellowish white, fine, silty sand/sandy silt, moist slightly micaceous, medium dense to dense, moderately weathered to weathered. early Pleistocene and late Pliocene

NOTES

R = Ring Sample
B = Bag Sample
: Approx. limits of excavation
------ : Lithologic contact
?--?--?--? queried where uncertain
* : unified soil classifications symbol

PLAN



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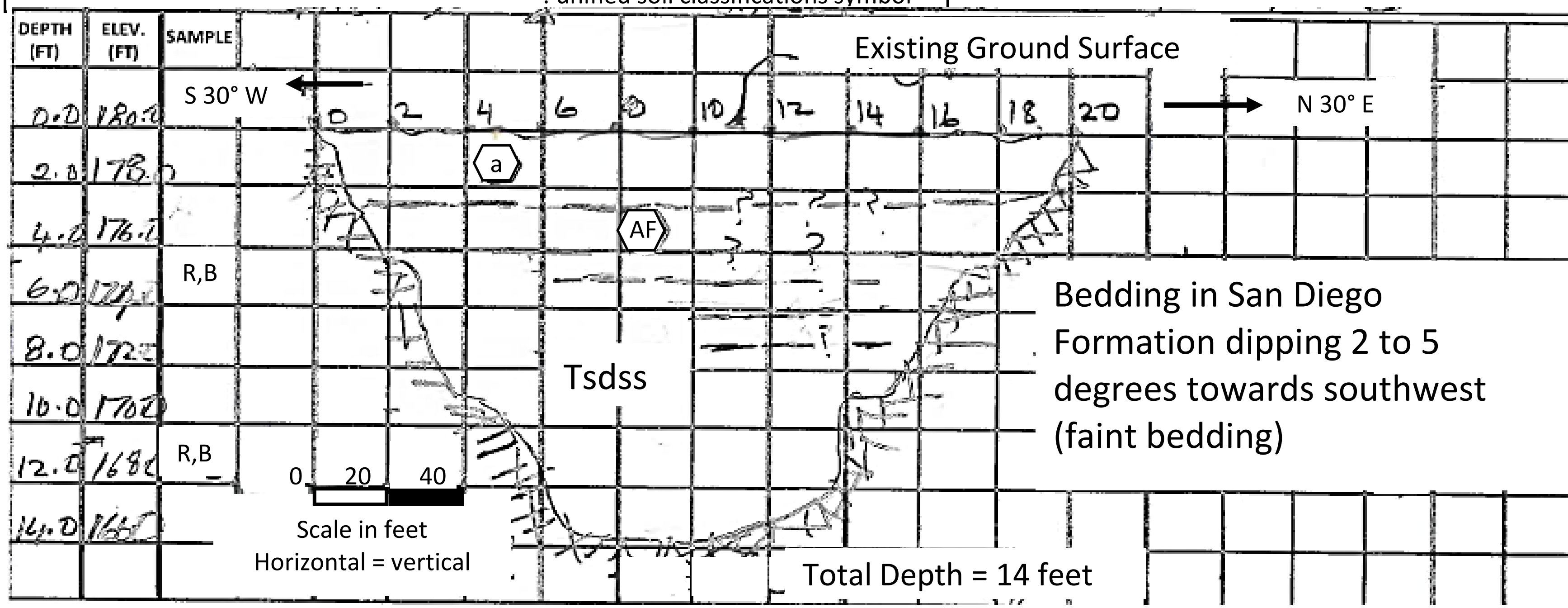


PARTNER

PROJECT NUMBER	TRENCH NUMBER SHEET 1 OF 1
17-199602.4	TP-2
	TRENCH WALL LOG (A)

PROJECT: Encompass Health LOCATION: 517 Shinohara Lane, Chula Vista, CA 91911

ELEVATION: 204~ ft MSL CONTRACTOR: AMG Demolition DATE EXCAVATED: July 26, 2018

GROUNDWATER LEVEL & DATE: Not encountered EXCAVATION METHOD: Backhoe: Komatsu PC 390 LC GEOLOGIST: R. Quraishi

APPROXIMATE DIMENSIONS: LENGTH: 32 ft WIDTH: 9.75 ft DEPTH: 14 ft REMARKS: Trench walls stable

LITHOLOGIC DESCRIPTION

(a) 0 to 0.75 ft: Topsoil; blackish brown; fine to coarse sand, some clay & silt, root fragments organic (moist)

(AF) 0.75 ft to 3.3 ft: Aritifical Fill; orangish brown; fine to coarse sand, some silt & clay, fine to coarse gravel and cobbles (SM* - GM*)

Tsdss (3.3 to 14 ft) San Diego Formation (silty sandstone): Grayish white to yellowish white, fine, silty sand/sandy silt, moist slightly micaceous, medium dense to dense, moderately weathered to weathered. early Pleistocene and late Pliocene

NOTES

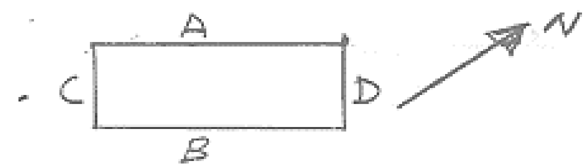
B = Bag Sample

: Approx. limits of excavation

- - - - - - : Lithologic contact

?--?--?--? queried where uncertain

PLAN



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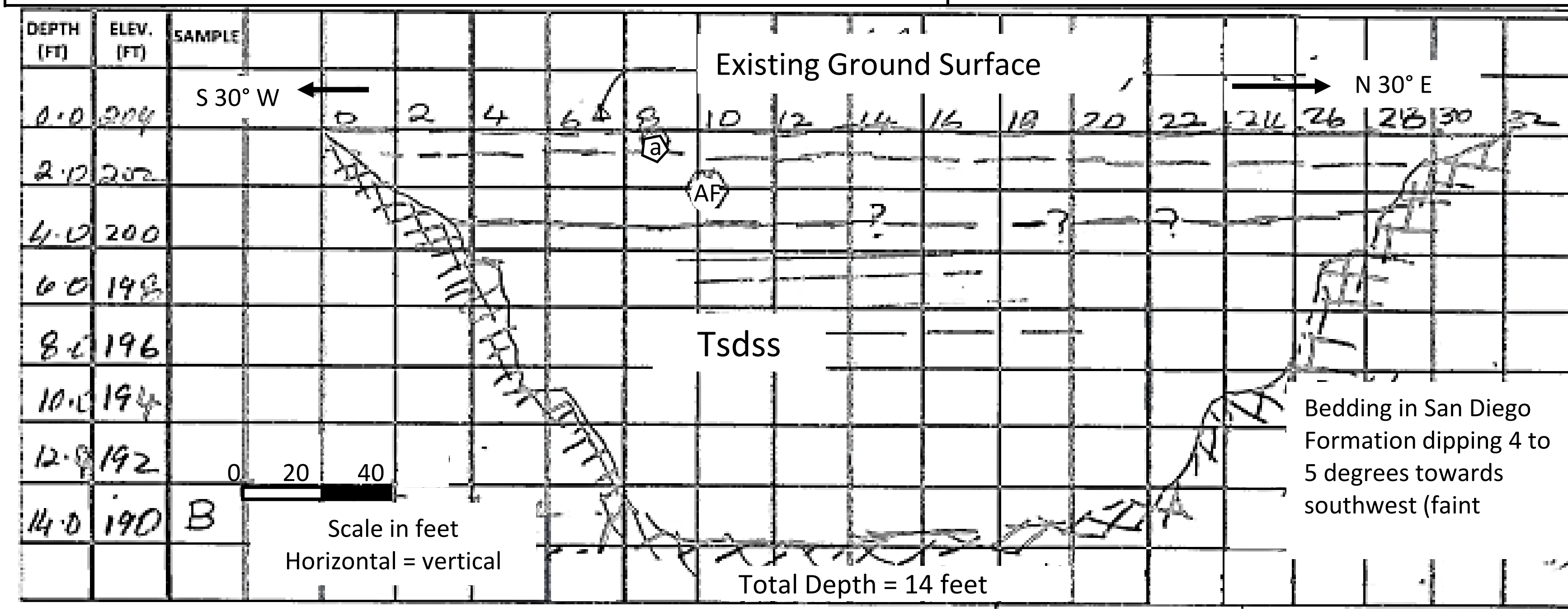


PARTNER

PROJECT NUMBER	TRENCH NUMBER	SHEET 1 OF 1
17-199602.4	TP-3	
	TRENCH WALL LOG (A)	

PROJECT: Encompass Health LOCATION: 517 Shinohara Lane, Chula Vista, CA 91911

ELEVATION: 211~ ft MSL CONTRACTOR: AMG Demolition DATE EXCAVATED: July 27, 2018

GROUNDWATER LEVEL & DATE: Not encountered EXCAVATION METHOD: Backhoe: Komatsu PC 390 LC GEOLOGIST: R. Quraishi

APPROXIMATE DIMENSIONS: LENGTH: 25 ft WIDTH: 21 ft DEPTH: 14 ft REMARKS: Trench walls stable

LITHOLOGIC DESCRIPTION

(a) 0 to 0.5 ft: Topsoil; blackish brown; fine to coarse sand, some clay & silt, root fragments organic (moist)

(AF) 0.5 ft to 1.5 ft: Artificial Fill; orangish brown; fine to coarse sand, some silt & clay, fine to coarse gravel and cobbles (SM* - GM*)

Tsdss (1.5 to 14 ft) San Diego Formation (silty sandstone): Grayish white to yellowish white, fine, silty sand/sandy silt, moist slightly micaceous, medium dense to dense, moderately weathered to weathered. early Pleistocene and late Pliocene

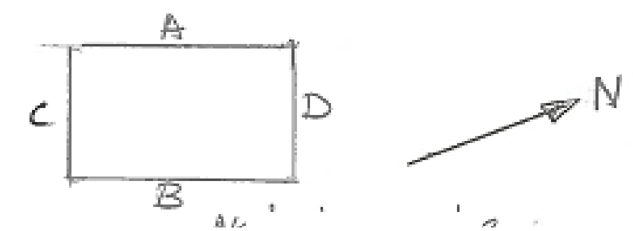
NOTES

: Approx. limits of excavation

- - - - - - : Lithologic contact

?--?--?--? queried where line uncertain

PLAN



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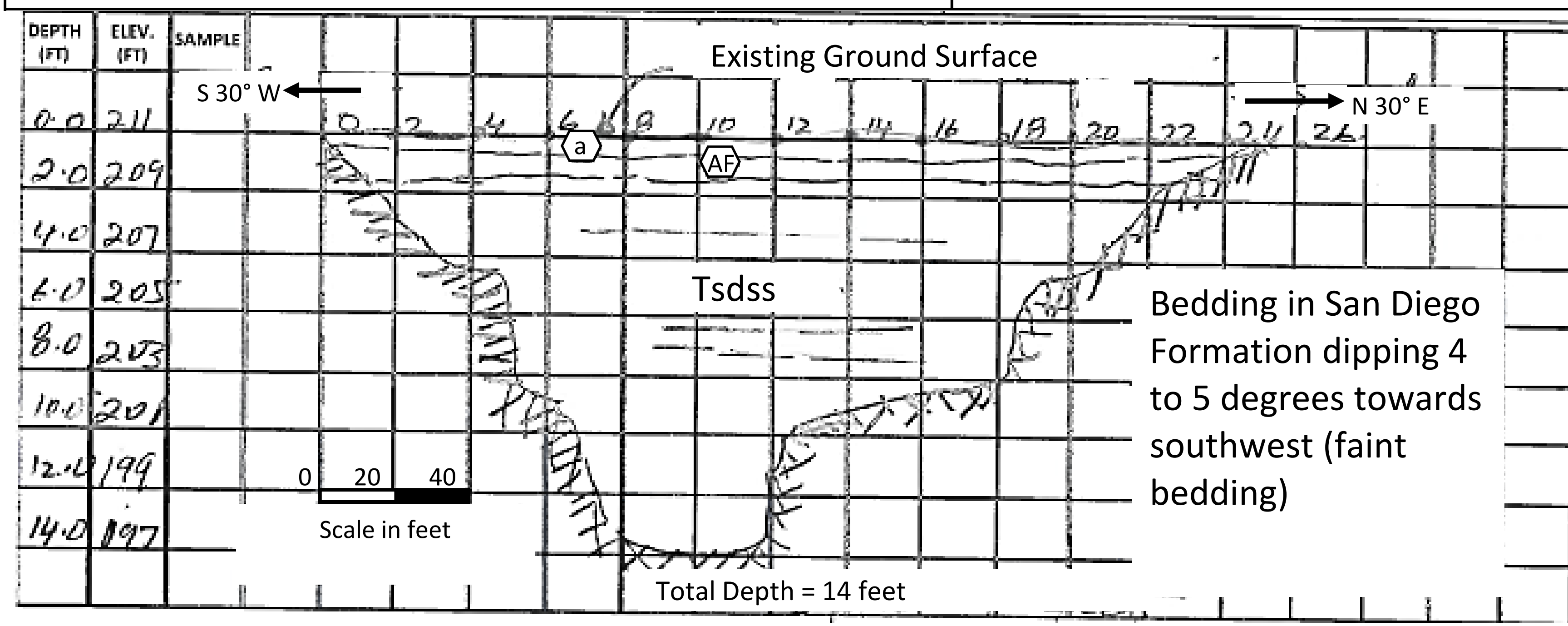


PARTNER

PROJECT NUMBER	TRENCH NUMBER SHEET 1 OF 1
17-199602.4	TP-4

TRENCH WALL LOG (A)

PROJECT: Encompass Health LOCATION: 517 Shinohara Lane, Chula Vista, CA 91911

ELEVATION: 184~ ft MSL CONTRACTOR: AMG Demolition DATE EXCAVATED: July 27, 2018

GROUNDWATER LEVEL & DATE: Not encountered EXCAVATION METHOD: Backhoe: Komatsu PC 390 LC GEOLOGIST: R. Quraishi

APPROXIMATE DIMENSIONS: LENGTH: 21 ft WIDTH: 25 ft DEPTH: 14 ft REMARKS: Trench walls stable

LITHOLOGIC DESCRIPTION

(AF) 0 ft to 3.0 ft: Artificial Fill; orangish brown; fine to coarse sand, some silt & clay, fine to coarse gravel and cobbles (SM* - GM*)

Tsdss (3 to 14 ft) San Diego Formation (silty sandstone): Grayish white to yellowish white, fine, silty sand/sandy silt, moist slightly micaceous, medium dense to dense, moderately weathered to weathered. early Pleistocene and late Pliocene, (SM*), (SM/ML*)

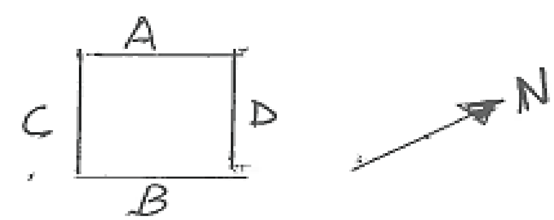
NOTES

* : unified soil

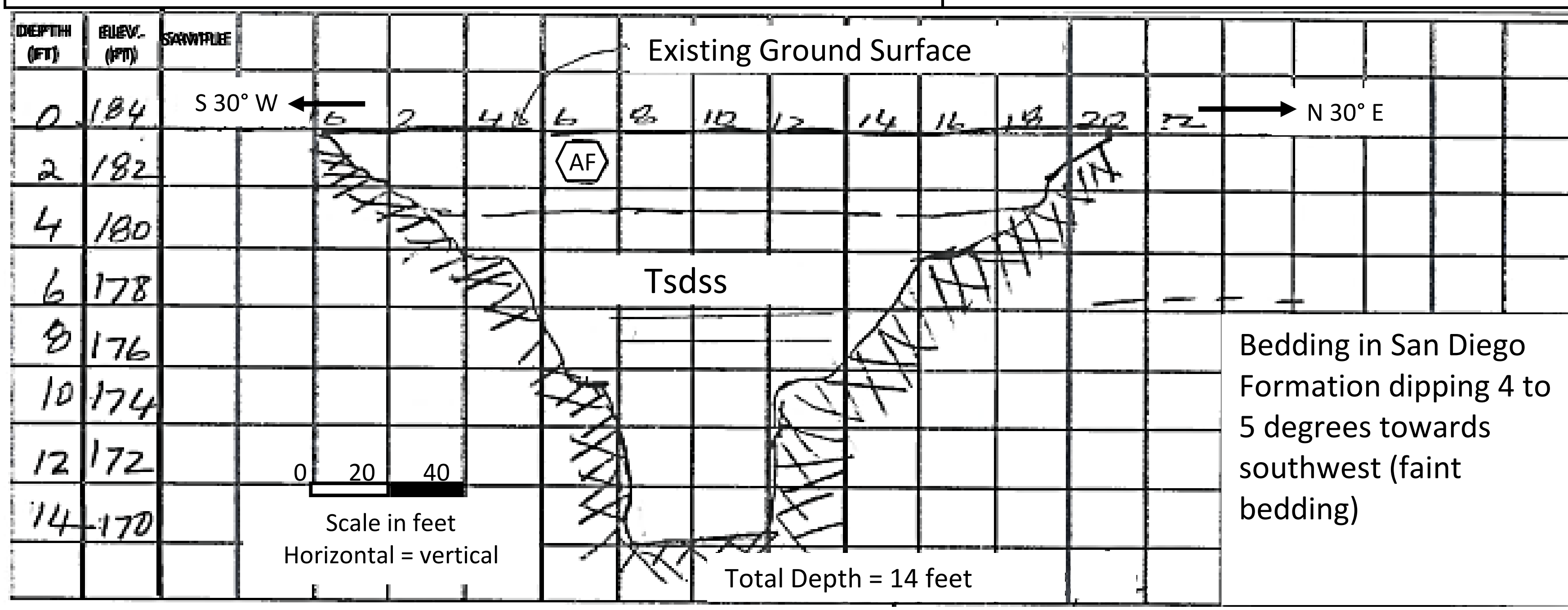
: Approx. limits of excavation

\- - - - - - ?--?--?--? : Lithologic contact queried where line uncertain

PLAN



Not to scale



UNIFIED SOIL CLASSIFICATION SYSTEM

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

COARSE-GRAINED SOILS (more than 50% of material is larger than No. 200 sieve size.)		
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	Clean Gravels (Less than 5% fines)	
	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
	GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
	Gravels with fines (More than 12% fines)	
	GM	Silty gravels, gravel-sand-silt mixtures
	GC	Clayey gravels, gravel-sand-clay mixtures
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	Clean Sands (Less than 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 (More than 12% fines)	
	SM	Silty sands, sand-silt mixtures
	SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (50% or more of material is smaller than No. 200 sieve size.)		
SILTS AND CLAYS Liquid limit less than 50%	ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts with slight plasticity
	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
	OL	Organic silts and organic silty clays of low plasticity
SILTS AND CLAYS Liquid limit 50% or greater	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 silts
HIGHLY ORGANIC SOILS	PT	Peat and other highly organic soils

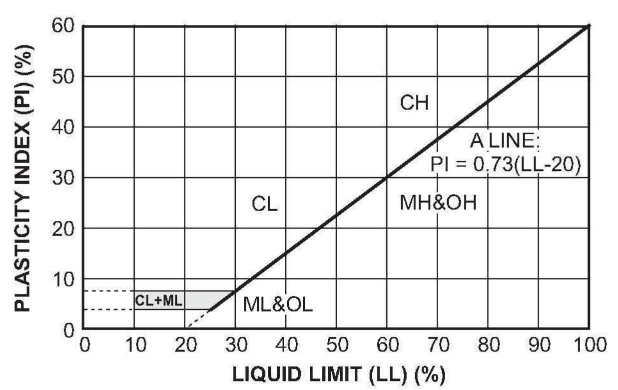
LABORATORY CLASSIFICATION CRITERIA

GW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
GP	Not meeting all gradation requirements for GW	
GM	Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
GC	Atterberg limits above "A" line with P.I. greater than 7	
SW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
SP	Not meeting all gradation requirements for GW	
SM	Atterberg limits below "A" line or P.I. less than 4	Limits plotting in shaded zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols.
SC	Atterberg limits above "A" line with P.I. greater than 7	

Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:

Less than 5 percent .... GW, GP, SW, SP
More than 12 percent .... GM, GC, SM, SC
5 to 12 percent .... Borderline cases requiring dual symbols

PLASTICITY CHART



BORING LOG KEY - EXPLANATION OF TERMS

SURFACE COVER: General discription with thickness to the inch, ex. Topsoil, Concrete, Asphalt, etc,

FILL: General description with thickness to the 0.5 feet. Ex. Roots, Debris, Processed Materials (Pea Gravel, etc.)

NATIVE GEOLOGIC MATERIAL: Deposit type, 1.Color, 2.moisture, 3.density, 4.SOIL TYPE, other notes - Thickness to 0.5 feet

1. Color - Generalized

Light Brown (usually indicates dry soil, rock, caliche)
Brown (usually indicates moist soil)
Dark Brown (moist to wet soil, organics, clays)
Reddish (or other bright colors) Brown (moist, indicates some soil development/or residual soil)
Greyish Brown (Marine, sub groundwater - not the same as light brown above)
Mottled (brown and gray, indicates groundwater fluctuations)

2. Moisture

dry - only use for wind-blown silts in the desert
damp - soil with little moisture content
moist - near optimum, has some cohesion and stickyness
wet - beyond the plastic limit for clayey soils, and feels wet to the touch for non clays
saturated - Soil below the groundwater table, sampler is wet on outside

3. Density (based on blow counts or hand evaluation)

SPT	Ring	Granular	Cohesive		
0-5	0-7	very loose	very soft	Unsuitable	Thumb penetrates through
5-10	7-14	loose	soft	<1,500psf	Thumb penetrates part way
10-20	14-28	medium dense	firm	<3,000psf	Thumb dents only
20-75	28-100	dense	stiff	>3,000psf	Thumbnail dents
75+	100+	very dense	hard	Hard Dig	Thumbnail does not dent

4. Classification

Determine percent Gravel (bigger than 3/8")
Determine percent fines (silt and clay feel soft, with no grit)
Determine percent sand (between silt and clay, feels gritty)
Determine if clayey (make soil moist, if it easily roll into a snake it is clayey)

Sands and gravels (more gravel starts with G, more sand starts with S)

GP	SP	Mostly sand and gravel, with less than 5 % fines	sandy GRAVEL	SAND
GP-GM	SP-SM	Mostly sand and gravel 7-12% fines, non-clayey	sandy GRAVEL with silt	SAND with Silt
GP-GC	SP-SC	Mostly sand and gravel 7-12% fines, clayey	sandy GRAVEL with clay	SAND with clay
GC	SC	Mostly sand and gravel >12% fines clayey	clayey GRAVEL	clayey SAND
GM	SM	Mostly sand and gravel >12% fines non-clayey	silty GRAVEL	silty SAND

Cohesive Soil (generaly forms long chunks (more than 2 inches) in sampler

ML	Soft, non clayey	SILT with sand
MH	Very rare, holds a lot of water, and is pliable with very low strength	high plasticity SILT
CL	If sandy can be hard when dry, will be stiff/plastic when wet	CLAY with sand/silt
CH	Hard and resiliant when dry, very strong/sticky when wet (may have sand in it)	FAT CLAY

H = Liquid limit over 50%, L - LL under 50%
C = Clay
M = Silt

Samplers

S = Standard split spoon (SPT)
R = Modified ring
Bulk = Excavation spoils
ST = Shelby tube
C = Rock core

Boring Number:	B1		Page 1 of 1
Location:	See Figure	Date Started:	1/25/2018
Site Address:	517 Shinohara Lane Chula Vista, CA 91911	Date Completed:	1/25/2018
		Depth to Groundwater:	N/A
Project Number:	17-199602.3	Field Technician:	JM
Drill Rig Type:	LAR DUAL RIG 75	**Partner Engineering and Science**	
Sampling Equipment:	SPT	**2154 Torrance Blvd, Suite 201**	
Borehole Diameter:	8"	**Torrance, CA 90501**	

Depth	Sample	N-Value	USCS	Description
0				**SURFACE COVER**: Grass/Dirt
1				**FILL**: Brown, moist, loose, fine to medium-grained sand, silty SAND
2				
3				
4				
5	S	18	SM	**SAN DIEGO FORMATION:** gray, moist, medium dense, fine to medium-grained, silty SAND
6				
7				
8				
9				
10	S	29		Dense
11				
12				
13				
14				
15	S	27		
16				Boring Terminated at 16.5 feet
17				Backfilled with spoils upon completion
18				Groundwater not encountered
19				
20				
21				
22				
23				
24				
25				
26				
27				
28				
29				
30				

Boring Number:	B2		Page 1 of 1
Location:	See Figure	Date Started:	1/25/2018
Site Address:	517 Shinohara Lane Chula Vista, CA 91911	Date Completed:	1/25/2018
		Depth to Groundwater:	N/A
Project Number:	17-199602.3	Field Technician:	J.M.
Drill Rig Type:	LAR DUAL RIG 75	**Partner Engineering and Science**	
Sampling Equipmer	SPT	**2154 Torrance Blvd, Suite 201**	
Borehole Diameter	8"	**Torrance, CA 90501**	

Depth	Sample	N-Value	USCS	Description
0				**SURFACE COVER**: Grass/Dirt
1				Topsoil mixed wth fill
2				
3				
4				
5		10	SM	**FILL**: Brown, moist, loose, fine to medium-grained sand, silty SAND
6				
7				
8				
9				
10		32	SM	**SAN DIEGO FORMATION:** Yellowish-brown, moist, dense, fine to medium-grained, silty SAND
11				
12				
13				
14				
15		16		Gray, medium dense, fine-grained, silty SAND
16				Boring Terminated at 16.5 feet
17				Backfilled with spoils upon completion
18				Groundwater not encountered
19				
20				
21				
22				
23				
24				
25				
26				
27				
28				
29				
30				

Boring Number:	B3		Page 1 of 1
Location:	See Figure	Date Started:	1/25/2018
Site Address:	517 Shinohara Lane Chula Vista, CA 91911	Date Completed:	1/25/2018
		Depth to Groundwater:	N/A
Project Number:	17-199602.3	Field Technician:	J.M.
Drill Rig Type:	LAR DUAL RIG 75	**Partner Engineering and Science** **2154 Torrance Blvd, Suite 201** **Torrance, CA 90501**	
Sampling Equipment	SPT		
Borehole Diameter:	8"		

Depth	Sample	N-Value	USCS	Description
0				**SURFACE COVER**: Grass/Dirt
1				
2				
3				
4				
5	■	11	SM	**FILL**:Brown, moist, loose, fine to medium-grained, silty SAND
6				**SAN DIEGO FORMATION:** Yellowish-brown, moist, dense, fine to medium-grained, silty SAND
7				
8				
9				
10	■	44		dense
11				
12				encountered harder drilling around 11-13'
13				
14				
15	■	19		medium dense
16				
17				
18				Boring Terminated at 16.5 feet
19				Backfilled with spoils upon completion
20				Groundwater not encountered
21				
22				
23				
24				
25				
26				
27				
28				
29				
30				

Boring Number:	B4	Page _1_ of _1_	
Location:	Near center of property	Date Started:	1/25/2018
Site Address:	517 Shinohara Lane Chula Vista, CA 91911	Date Completed:	1/25/2018
		Depth to Groundwater:	N/A
Project Number:	17-199602.3	Field Technician:	J.M.
Drill Rig Type:	LAR DUAL RIG 75	**Partner Engineering and Science**	
Sampling Equipment:	SPT	**2154 Torrance Blvd, Suite 201**	
Borehole Diameter:	8"	**Torrance, CA 90501**	

Depth	Sample	N-Value	USCS	Description
0				**SURFACE COVER**: Grass/Dirt
1				
2				
3				
4				
5		37	SM	**FILL**: Brown (reddish), moist, dense, fine to medium-grained,
6				silty SAND with little clay (Moisture Content: 7%; NP)
7				
8				
9				
10		16	SM	**SAN DIEGO FORMATION**: Brown, moist, medium dense, fine to medium-grained, silty SAND
11				
12				
13				
14				layer of gravel and silt
15		47		dense
16				
17				Boring terminated at 16.5 feet
18				Backfilled with spoils upon completion
19				Groundwater not encountered
20				
21				
22				
23				
24				
25				
26				
27				
28				
29				
30				

Boring Number:	B5	Page 1 of 1	
Location:	See Figure	Date Started:	1/25/2018
Site Address:	517 Shinohara Lane Chula Vista, CA 91911	Date Completed:	1/25/2018
		Depth to Groundwater:	N/A
Project Number:	17-199602.3	Field Technician:	J.M.
Drill Rig Type:	LAR DUAL RIG 75	**Partner Engineering and Science**	
Sampling Equipment:	SPT	**2154 Torrance Blvd, Suite 201**	
Borehole Diameter:	8"	**Torrance, CA 90501**	

Depth	Sample	N-Value	USCS	Description
0				
1				
2				
3				
4	■	50/4"	SM	**FILL**: Brown, moist, dense, fine to medium-grained, silty SAND,
5				some gravel and some clay
6				
7				
8				Boring terminated at 6 feet
9				Backfilled with spoils upon completion
10				Groundwater not encountered
11				
12				
13				
14				
15				
16				
17				
18				
19				
20				
21				
22				
23				
24				
25				
26				
27				
28				
29				
30				

Boring Number:	B6	Page 1 of 1	
Location:	See Figure	Date Started:	1/25/2018
Site Address:	517 Shinohara Lane Chula Vista, CA 91911	Date Completed:	1/25/2018
		Depth to Groundwater:	N/A
Project Number:	17-199602.3	Field Technician:	J.M.
Drill Rig Type:	LAR DUAL RIG 75	**Partner Engineering and Science** **2154 Torrance Blvd, Suite 201** **Torrance, CA 90501**	
Sampling Equipment:	SPT		
Borehole Diameter:	8"		

Depth	Sample	N-Value	USCS	Description
0				**SURFACE COVER**: Grass/Dirt
1				
2				
3			3'	layers of gravel
4				
5	■	45	SM	**FILL**: Brown mottled with white, dense, silty SAND, little gravel
6				
7				
8				
9				
10	■	39	SC	**SAN DIEGO FORMATION**: Brown, moist, fine-grained, clayey SAND
11				(Moisture Content: 10%, PI=15)
12				hard layer of gravel at 11'
13				
14				
15	■	25	SM	Brownish yellow, moist, fine-grained, silty SAND
16				
17				Boring Terminated at 16.5 feet
18				Backfilled with spoils upon completion
19				Groundwater not encountered
20				
21				
22				
23				
24				
25				
26				
27				
28				
29				
30				

Boring Number:	B-7	Boring Log Page 1 of 1	
Location:	Slope Center (SE proposed building	Date Started:	2/12/2019
Site Address:	517 Shinohara Lane	Date Completed:	2/12/2019
	Chula Vista, California	Depth to Groundwater:	N/A
Project Number:	17-199602.7	Field Technician:	J. Eudell
Drill Rig Type:	FRASTE	**Partner Engineering and Science**	
Sampling Equipment:	6" H.S.A, SPT & Ring sampler	**2154 Torrance Boulevard, Suite 200**	
Borehole Diameter:	6"	**Torrance, California 90501**	

Depth	Sample	N-Value	USCS	Description
0				**SURFACE COVER**: Grass covered topsoil
1				
2				
3				
4				
5	S	62	SM	**SAN DIEGO FORMATION**: Brown, moist, dense, silty SAND with clay and few rocks
6				
7				
8				
9				
10	S	38		
11				
12				
13				
14				
15	S	32		
16				Boring terminated at 16.5' bgs
17				Boring backfilled with spoils upon completion
18				Groundwater not encountered
19				
20				
21				
22				
23				
24				
25				
26				
27				
28				
29				

Boring Number:	B-10	Boring Log Page 1 of 1	
Location:	20' NE of stake	Date Started:	2/12/2019
Site Address:	517 Shinohara Lane	Date Completed:	2/12/2019
	Chula Vista, California	Depth to Groundwater:	N/A
Project Number:	17-199602.7	Field Technician:	J. Eudell
Drill Rig Type:	FRASTE	**Partner Engineering and Science**	
Sampling Equipment:	6" H.S.A, SPT & Ring sampler	**2154 Torrance Boulevard, Suite 200**	
Borehole Diameter:	6"	**Torrance, California 90501**	

Depth	Sample	N-Value	USCS	Description
0				**SURFACE COVER**: Grass covered topsoil
1				
2				
3				
4				
5	S	37	SM	**SAN DIEGO FORMATION**: Brown, damp, dense, silty SAND
6				
7				
8				
9				
10	S	23		
11				
12				
13				
14				
15	S	20		
16				Boring terminated at 16.5' bgs
17				Boring backfilled with spoils upon completion
18				Groundwater not encountered
19				
20				
21				
22				
23				
24				
25				
26				
27				
28				
29				

Boring Number:	B-12	Boring Log Page 1 of 1		
Location:	Middle of cliff, north side	Date Started:	2/12/2019	
Site Address:	517 Shinohara Lane	Date Completed:	2/12/2019	
	Chula Vista, California	Depth to Groundwater:	N/A	
Project Number:	17-199602.7	Field Technician:	J. Eudell	
Drill Rig Type:	FRASTE	**Partner Engineering and Science**		
Sampling Equipment:	6" H.S.A, SPT & Ring sampler	**2154 Torrance Boulevard, Suite 200**		
Borehole Diameter:	6"	**Torrance, California 90501**		

Depth	Sample	N-Value	USCS	Description
0				**SURFACE COVER**: Grass/Dirt
1				
2	S	24	SP	**FILL**: Brown to gray, damp, dense, poorly graded SAND
3				
4				
5	R	52	SM	**SAN DIEGO FORMATION**: Reddish brown, damp, dense, silty SAND
6				
7	S	23	ML	Yellowish gray, damp, stiff, SILT
8				
9				
10	R	32		
11				
12				
13				
14				
15	S	24		
16				Boring terminated at 16.5' bgs
17				Boring backfilled with spoils upon completion
18				Groundwater not encountered
19				
20				
21				
22				
23				
24				
25				
26				
27				
28				
29				

Boring Number:	B-13	Boring Log Page 1 of 1	
Location:	Bottom of cliff, north end	Date Started:	2/12/2019
Site Address:	517 Shinohara Lane	Date Completed:	2/12/2019
	Chula Vista, California	Depth to Groundwater:	N/A
Project Number:	17-199602.7	Field Technician:	J. Eudell
Drill Rig Type:	FRASTE	**Partner Engineering and Science**	
Sampling Equipment:	6" H.S.A, SPT & Ring sampler	**2154 Torrance Boulevard, Suite 200**	
Borehole Diameter:	6"	**Torrance, California 90501**	

Depth	Sample	N-Value	USCS	Description
0				**SURFACE COVER**: Grass/Dirt
1				
2	S	12	SP	**FILL**: Gray to brown, damp, medium dense, poorly graded SAND
3				
4				
5	S	29	SM	**SAN DIEGO FORMATION**: Yellowish gray to brown, damp, dense, silty SAND
6				
7				
8				
9				
10	S	27		
11				
12				
13				
14				
15	S	32	SP	Brown, damp, dense, poorly graded SAND
16				Boring terminated at 16.5' bgs
17				Boring backfilled with spoils upon completion
18				Groundwater not encountered
19				
20				
21				
22				
23				
24				
25				
26				
27				
28				
29				

Boring Number:	B-14	Boring Log Page 1 of 1	
Location:	New Southern fence	Date Started:	2/12/2019
Site Address:	517 Shinohara Lane	Date Completed:	2/12/2019
	Chula Vista, California	Depth to Groundwater:	N/A
Project Number:	17-199602.7	Field Technician:	J. Eudell
Drill Rig Type:	FRASTE	**Partner Engineering and Science**	
Sampling Equipment:	6" H.S.A, SPT & Ring sampler	**2154 Torrance Boulevard, Suite 200**	
Borehole Diameter:	6"	**Torrance, California 90501**	

Depth	Sample	N-Value	USCS	Description
0				**SURFACE COVER**: Grass/Dirt
1				
2	S	42	ML	**SAN DIEGO FORMATION:** Dark brown, moist, stiff, sandy SILT
3				
4				
5	S	33		
6				
7				
8				
9				
10	S	27		
11				
12				
13				
14				
15	S	32		
16				Boring terminated at 16.5' bgs
17				Boring backfilled with spoils upon completion
18				Groundwater not encountered
19				
20				
21				
22				
23				
24				
25				
26				
27				
28				
29				

Boring Number:	B-15		Boring Log Page 1 of 1
Location:	Southern fence	Date Started:	2/12/2019
Site Address:	517 Shinohara Lane	Date Completed:	2/12/2019
	Chula Vista, California	Depth to Groundwater:	N/A
Project Number:	17-199602.7	Field Technician:	J. Eudell
Drill Rig Type:	FRASTE	**Partner Engineering and Science**	
Sampling Equipment:	6" H.S.A, SPT & Ring sampler	**2154 Torrance Boulevard, Suite 200**	
Borehole Diameter:	6"	**Torrance, California 90501**	

Depth	Sample	N-Value	USCS	Description
0				**SURFACE COVER**: Grass/dirt
1				
2	R	55	ML	**SAN DIEGO FORMATION:** Dark brown, moist, stiff, sandy SILT
3				
4				
5	S	40		Some clay present
6				
7	R	40	CL	Brown, damp, dense, silty CLAY with rocks
8				
9				
10	S	24		
11				
12				
13				
14				
15	S	24	SM	Brown, damp, dense, silty SAND
16				Boring terminated at 16.5' bgs
17				Boring backfilled with spoils upon completion
18				Groundwater not encountered
19				
20				
21				
22				
23				
24				
25				
26				
27				
28				
29				

Legend to Engineering Geologic Mapping, Rock Boring and Trenches

Igneous and Metamorphic Rock Grain Size Descriptors

Descriptors	Average Crystal Diameter
Very coarse-grained or pegmatic	> 10mm (> 3/8 in)
Coarse-grained	5-10 mm (3/16 – 3/8 in)
Medium-grained	1-5 mm (1/32 – 3/16 in)
Fine-grained	0.1-1 mm (0.04 – 1/32 in)
Aphanitic (cannot be seen with the unaided eye)	< 0.1 mm (<0.04 in)

Bedding, Foliation or Flow Texture Descriptors

Descriptor	Thickness / Spacing
Massive	> 10 ft (> 3 m)
Very thickly (bedded, foliated or banded)	3 – 10 ft (1 – 3 m)
Thickly	1 – 3 ft (300 mm – 1 m)
Moderately	0.3 – 1 ft (100 – 300 mm)
Thinly	0.1 – 0.3 ft (30 – 100 mm)
Very Thinly	0.03 {3/8 in} – 0.1 ft (10 – 30 mm)
Laminated (intensely foliated or banded)	< 0.03 ft {3/8/ in} (< 10 mm)

Weathering Descriptors of Rocks

Descriptor	Symbol	Diagnostic Features
Fresh	FR	No Discoloration, not oxidized
Slightly Weathered	SW	Discoloration or oxidation is limited to surface or, or short distance from, fractures: some feldspar crystals are dull
Moderately Weathered	MW	Discoloration or oxidation extends from fractures, usually throughout: Fe-Mg minerals are “rusty,” feldspar crystals are “cloudy.”
Highly Weathered	HW	Discoloration or oxidation throughout: all feldspars and Fe-Mg minerals are altered to clay to some extend; or chemical alteration produces in-situ disaggregation. See grain boundary conditions
Completely Weathered	XW	Discoloration or oxidation throughout; but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay

RQD: Rock Quality Designation Criteria

RQD%	Velocity Index	Rock Mass Quality
90 – 100	0.8 – 1.0	Excellent
75 - 90	0.6 – 0.8	Good
50 - 75	0.4 – 0.6	Fair
25 - 50	0.2 – 0.4	Poor
0 - 25	0 – 0.2	Very Poor

$$\% \text{ Recovery} = \frac{\text{Total Length of Core Recovered}}{\text{Length of Coring Interval}} \times 100$$

$$\text{R.Q.D.*} = \frac{\text{Sum of Length of all core pieces equal to or greater than 4 inches}}{\text{Length of coring intercal}}$$

* No mechanically induced core breaks are included in the computation of the RQD

* R.Q.D. Rock Quality Designation

Rock Hardness / Strength Descriptors

Descriptor	Symbol	Diagnostic Features
Very Hard	VH	Cannot be scratched with knife or sharp pick.
Hard	H	Can be scratched with knife or sharp pick with difficulty (heavy pressure). Heavy hammer blow required to break specimen.
Moderately Hard	MH	Can be scratched with knife or sharp pick with light or moderate pressure. Core or fragment breaks with moderate hammer blow.
Soft	S	Can be grooved or gouged easily by knife or sharp pick with light pressure, can be scratched with fingernail. Breaks with light to moderate manual pressure.
Very Soft	VS	Can be readily indented, grooved or gouged with fingernail. Breaks with light manual pressure.

Weathering Conditions of Fracture Surfaces

Symbol	Diagnostic Features
H	High weathering effects on fracture surface
S	Slight weathering effects on fracture surface
M	Moderate weathering effects on fracture surface

Fracture Conditions

Symbol	Diagnostic Features
G	Good fit between the fracture sides
P	Poor fit between the facture sides

References: 1) Engineering Geology Field Manual, Second Edition, Volume 1, U.S Department of the Interior., Bureau of Reclamation, 1998

2) Naval Facilities Engineering Command, NAVFAC DM-7.1, 7.2, 7.3, Design Manual. Soil Mechanics and Foundations. May 1982, April 1983

Note: At this project location, RQD criteria pertains to very soft to moderately hard rock

Boring Number:	B8-A		
Project:	Encompass Health Hospitals	Date Started:	3/15/2019
Site Address:	517 Shinohara Lane Chula Vista, California	Date Completed:	3/15/2019
		Depth to Groundwater:	Not encountered
Project Number:	17-199602.4	Eng/Geo:	R. Quraishi
Drill Rig Type:	CME-95	Total Depth: 50'	**Partner Engineering and Science** **2154 Torrance Blvd, Suite 201** **Torrance, CA 90501**
Sampling Equipment	Continuous Coring Sampler	Surface Elevation:	
Borehole Diameter:	8" Auger Hole	204'	

Depth	Sample	N-Value	Recovery (ft)	RQD (%)	Fracture SK C	Weathering	Strength	Formation	Description
1									From 0 to 0.5': Top Soil / root fragments
2									
3	R-1	NA	4.75	NA				Artifical Fill	Reddish brown, silty
4									fine to medium SAND, moist
5									(SM*)
6	SPT**	11, 14, 13	1.3	NA					Yellowish brown to gray,
7									sandy SILT, moist
8	R-2	NA	4.2	NA					(SM/ML)*
9									
10									
11									Yellowish brown to
12									reddish brown
13	R-3	NA	4.0	80					silty SAND, slightly micaceous,
14									moist (SM*)
15								San Diego Formation: Early Pleistocene/ Late Pliocene	
16									Gray, silty SAND / sandy SILT
17					No Natural Fractures Visible				slightly micaceous
18	R-4	NA	3.0	60		MW to HW	MH to S		moist (SM, SM/ML)*
19									(Mostly fine grained silty sandstone
20									moderately to highly weathered)
21									Same as above
22									
23	R-5	NA	4.8	96					
24	SPT**	8, 9, 12, 14	0	NA					
25									

Boring Number:	B8-A Contintued		Page 2 of 2
Project:	Encompass Health Hospitals	Date Started:	3/15/2019
Site Address:	517 Shinohara Lane Chula Vista, California	Date Completed:	3/15/2019
		Depth to Groundwater:	Not encountered
Project Number:	17-199602.4	Eng/Geo:	R. Quraishi
Drill Rig Type:	CME-95	Total Depth: 50'	**Partner Engineering and Science**
Sampling Equipment	Continuous Coring Sampler	Surface Elevation:	**2154 Torrance Blvd, Suite 201**
Borehole Diameter:	8" Auger Hole	204'	**Torrance, CA 90501**

Depth	Sample	N-Value	Recovery (ft)	RQD (%)	Fracture SK C	Weathering	Strength	Formation	Description
26–30	R-6	NA	2.0	40	No Natural Fractures Visible	MW to HW	MH to S	San Diego Formation: Early Pleistocene/ Late Pliocene	Alternate seams of yellowish brown to gray, fine, sandy SILT (SM/ML)* micaceous, moist (Mostly fine grained silty sandstone, moderately to highly weathered, moderately hard to soft)
31–35	R-7	NA	4.5	90					Same as above
36–40	R-8	NA	4.5	90					Same as above
41–45	R-9	NA	3.6	72					Yellowish brown to gray fine sandy SILT (SM/ML)* Slightly micaceous, moist (Mostly fine grained silty sandstone, moderately to highly weathered, moderately hard to soft)
46–50	R-10	NA	4.8	96					Yellowish brown to gray fine to medium SAND, little silt, slightly micaceous (SP-SM)* (Mostly fine grained silty sandstone, moderately weathered, soft to moderately hard)
									END OF BORING AT 50 FT

Notes:

1: Borings backfilled with drill cuttings/soil spoils upon completion
2: NA / Not Applicable
3: For Boring location see Figure 2
4: For RQD, weathering and strength criteria see Figure 15
5: * Unified soil classification system
6: R-1 Core Run Designtation
7: N-Value: actual blow counts per 6 inches
8: SPT** blow counts pertains to boring location B-8 approximately 10 feet Southeast of Boring 8A

Boring Number:	B11-A		
Project:	Encompass Health Hospitals	Date Started:	3/15/2019
Site Address:	517 Shinohara Lane Chula Vista, California	Date Completed:	3/15/2019
		Depth to Groundwater:	Not encountered
Project Number:	17-199602.4	Eng/Geo:	R. Quraishi
Drill Rig Type:	CME-95	Total Depth: 44.25'	**Partner Engineering and Science**
Sampling Equipment	Continuous Coring Sampler	Surface Elevation:	**2154 Torrance Blvd, Suite 201**
Borehole Diameter:	8" Auger Hole	210'	**Torrance, CA 90501**

Depth	Sample	N-Value	Recovery (ft)	RQD (%)	Fracture SK C	Weathering	Strength	Formation	Description
1–5	R-1	NA	4.2	84	No Natural Fractures Visible			Artificial Fill	From 0 to 0.5': Top Soil / root fragments Yellowish brown to gray fine sandy SILT (SM/ML)* damp
6	*SPT	5, 8, 14	1.2	NA				San Diego Formation: Early Pleistocene/ Late Pliocene	
7–10	R-2	NA	4.2	84					Yellowish brown to gray fine sandy SILT (SM/ML)*
11–15	R-3	NA	5.0	100					Yellowish brown to gray fine sandy SILT (SM/ML)* (Hard drilling)
16–20	R-4	NA	2.0	40		MW to HW	MH to S		Grayish white fine sand, some silt (SM)*, damp (Mostly fine grained silty sandstone, moderately to highly weathered, moderately hard to soft)
21–25	R-5	NA	5.0	100					Yellow to gray fine sandy SILT (SM/ML)*, moist (Mostly fine grained silty sandstone, moderately weathered to highly weathered, moderately hard to soft, moist)

Boring Number:	B11-A Contintued		
Project:	Encompass Health Hospitals	Date Started:	3/15/2019
Site Address:	517 Shinohara Lane Chula Vista, California	Date Completed:	3/15/2019
		Depth to Groundwater:	Not encountered
Project Number:	17-199602.4	Eng/Geo:	R. Quraishi
Drill Rig Type:	CME-95	Total Depth: 44.25'	**Partner Engineering and Science** **2154 Torrance Blvd, Suite 201** **Torrance, CA 90501**
Sampling Equipment	Continuous Coring Sampler	Surface Elevation:	
Borehole Diameter:	8" Auger Hole	212'	

Depth	Sample	N-Value	Recovery (ft)	RQD (%)	Fracture SK C	Weathering	Strength	Formation	Description
26					No Natural Fractures Visible	MW to HW	MH to S	San Diego Formation: Early Pleistocene/ Late Pliocene	Yellowish gray, micaceous
27									fine SAND, little silt
28	R-6	NA	5.0	100					(SP-SM), damp
29									Mostly fine grained, micaceous sandstone, moderately to
30									Highly weathered, moderately hard to soft
31									
32									Same as above
33	R-7	NA	5.0	100					
34									
35									
36									Same as above
37									(hard drilling)
38	R-8	NA	5.0	100					
39									
40									
41									Same as above
42	R-9	NA	3.0	60					Fine grained micaceous sandstone
43									Mostly moderately weathered to highly weathered
44	SPT**	23, 40, 50/3"	1.25	NA					Very hard drilling (SP-SM)*
45									
46									END OF BORING AT 44.25 FT
47									
48									
49									
50									

Notes:
1: Borings backfilled with drill cuttings/soil spoils upon completion
2: NA / Not Applicable
3: For Boring location see Figure 2
4: For RQD, weathering and strength criteria see Ffigure 15
5: * Unified soil classification system
6: R-1 Core Run Designtation
7: N-Value: actual blow counts per 6 inches
8: SPT** blow counts pertains to boring location B-11, located approximately 10 feet Northwest of boring B11-A

Boring Number:	B16-A		
Project:	Encompass Health Hospitals	Date Started:	3/15/2019
Site Address:	517 Shinohara Lane Chula Vista, California	Date Completed:	3/15/2019
		Depth to Groundwater:	Not encountered
Project Number:	17-199602.4	Eng/Geo:	R. Quraishi
Drill Rig Type:	CME-95	Total Depth: 40'	**Partner Engineering and Science** **2154 Torrance Blvd, Suite 201** **Torrance, CA 90501**
Sampling Equipment	Continuous Coring Sampler	Surface Elevation:	
Borehole Diameter:	8" Auger Hole	206'	

Depth	Sample	N-Value	Recovery (ft)	RQD (%)	Fracture SK C	Weathering	Strength	Formation	Description
1–5	R-1	NA	5.0	NA	No Natural Fractures Visible	MW	MH to S	Artificial Fill	From 0 to 0.5': Top Soil / root fragments Gray to brown, fine sandy SILT (SM/ML)* moist
6–10	R-2	NA	4.6	NA				San Diego Formation: Early Pleistocene/ Late Pliocene	Gray to brown, fine sandy SILT (SM/ML)* moist
11–15	Drill Cuttings	NA	Drill Cuttings	NA					Gray to brown, fine sandy SILT (SM/ML)* moist (Observations from drill cuttings)
16–20		NA		NA					Observation form drill cuttings yellow brown to gray, fine SAND, little silt (SP-SM)*
21–25	R-3	NA	2.6	52					Yellowish brown to gray fine SAND, little silt (SP-SM)* slightly micaceous Alternate gray to brown bandings (Mostly fine grained silty sandstone, moderately weathered, moderately hard to soft)

Boring Number:	B16-A Continued		Page 2 of 2
Project:	Encompass Health Hospitals	Date Started:	3/15/2019
Site Address:	517 Shinohara Lane Chula Vista, California	Date Completed:	3/15/2019
		Depth to Groundwater:	Not encountered
Project Number:	17-199602.4	Eng/Geo:	R. Quraishi
Drill Rig Type:	CME-95	Total Depth: 40'	**Partner Engineering and Science**
Sampling Equipment	Continuous Coring Sampler	Surface Elevation:	**2154 Torrance Blvd, Suite 201**
Borehole Diameter:	8" Auger Hole	206'	**Torrance, CA 90501**

Depth	Sample	N-Value	Recovery (ft)	RQD (%)	Fracture SK C	Weathering	Strength	Formation	Description
26									Reddish brown to yellow
27									brown to gray
28	R-4	NA	4.8	96					fine SAND, little silt, slightly micaceous (SP-SM)*
29									(Mostly fine grained sandstone,
30									moderately weathered, moderately hard)
31									Mostly fine grained, gray, fine silty sandstone
32									slightly micaceous
33	R-5	NA	5.0	100	No Natural Fractures Visible	MW	MH	San Diego Formation: Early Pleistocene/ Late Pliocene	moderately weathered
34									moderately hard
35									Alternate Iron oxide buildup
36									(moderate to hard drilling)
37									
38	R-6	NA	5.0	100					
39									
40									
41									END OF BORING AT 40 FT
42									
43									
44									
45									
46									
47									
48									
49									
50									

Notes:
1: Borings backfilled with drill cuttings/soil spoils upon completion
2: NA / Not Applicable
3: For Boring location see figures
4: For RQD, weathering and strength criteria see figures
5: * Unified soil classification system
6: R-1 Core Run Designtation

APPENDIX C

Laboratory Test Results

HAMILTON
& Associates

1641 Border Avenue • Torrance, CA 90501 T 310.618.2190 888.618.2190 F 310.618.2191 W hamilton-associates.net

August 2, 2018
H&A Project No. 18-2487
Partner Project No. 17-199602.4

Partner Engineering and Science, Inc.
4518 N.12 Street Suite 201
Phoenix, AZ 85016

Attention: Mr. Matthew Marcus, Technical Director- Geotechnical Engineering

Subject: Laboratory Testing of Soil Samples: Partner (Chula Vista)
517 Shinohara Lane, Chula Vista, CA 91911

Dear Mr. Marcus:

We have completed the laboratory tests on the samples provided for the subject project. Enclosed is a summary of laboratory test results.

We thank you for the opportunity to provide laboratory testing services. If there are any questions, please do not hesitate to contact the undersigned.

Respectfully submitted,
HAMILTON & ASSOCIATES, INC.

Rosa E. Murrieta
Laboratory Supervisor | Staff Geologist

David T. Hamilton, PE, GE
President

Distribution: (1) Matthew Marcus, mmarcus@partneresi.com
(2) Brett Bova, bbova@partneresi.com

MOISTURE CONTENT AND DENSITY TESTS

Relatively undisturbed soil retained within the rings of the Modified California barrel sampler were tested in the laboratory to determine in-place dry density and moisture content. Test results are presented in Table 1.

DIRECT SHEAR TESTS

Direct shear (ASTM D3080) tests were performed on selected relatively undisturbed samples to determine the shear strength parameters of various soil samples, respectively. The results of these tests are shown graphically on the appended “D” Plates.

ATTERBERG LIMITS

Atterberg Limits (ASTM D 4318) tests were performed on selected samples to determine the liquid limit, plastic limit, and the plasticity index of soils. Test Pit 1 at 5.5 and 11 feet has mostly sand with some fines, therefore non-plastic limits and Atterberg limits cannot be determined.

PARTICLE SIZE ANALYSIS WITHOUT HYDROMETER

Grain size analyses were performed on selected samples to determine soil characteristics in accordance with ASTM D422. The results of this test are shown graphically on the appended ‘G’ Plates.

HAMILTON & Associates

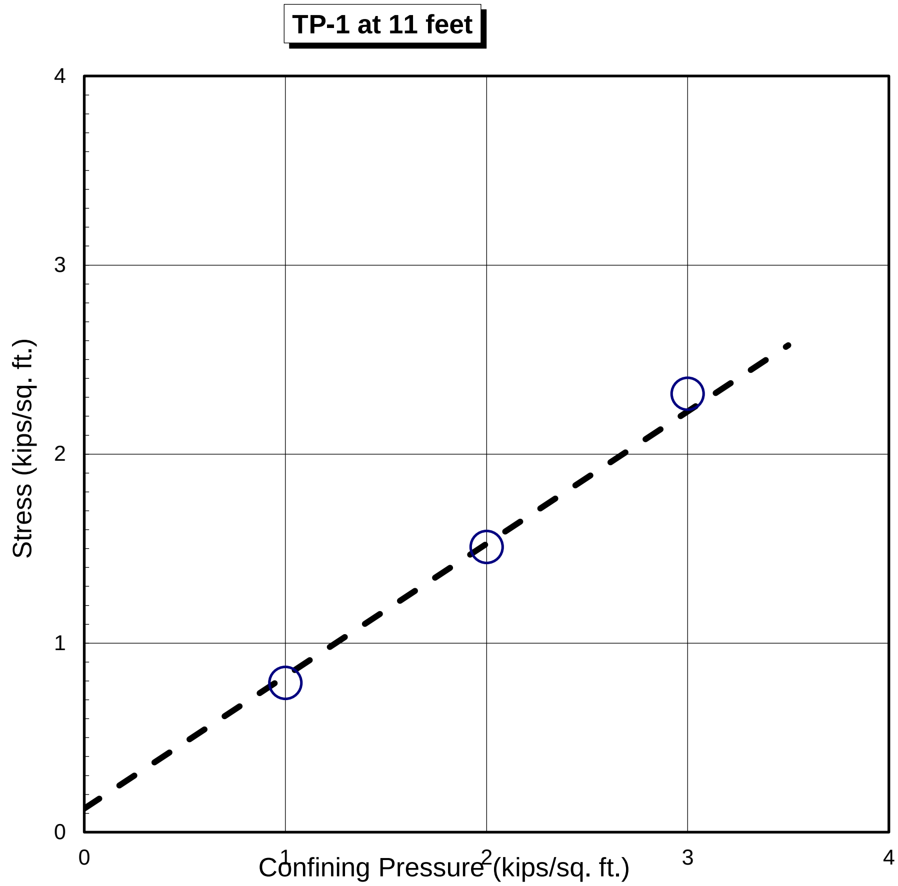
TABLE 1: LABORATORY RESULTS

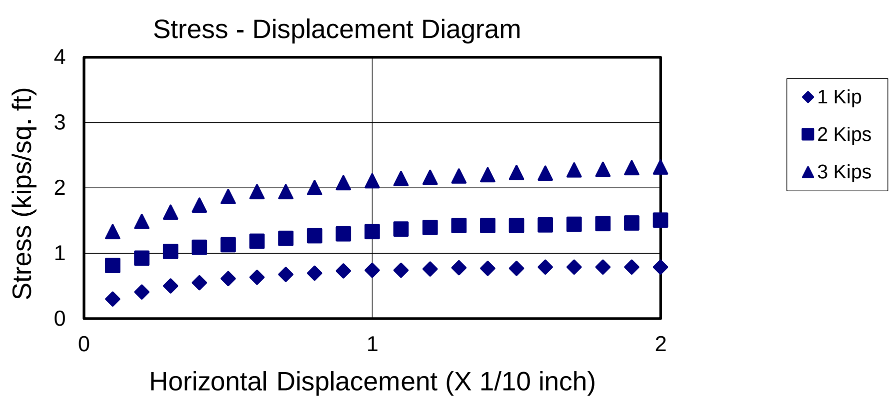
JOB TITLE: Partner (Chula Vista)
H&A PROJECT NO. 18-2487
SCHEDULED BY: FC
DATE SAMPLES DROPPED OFF: 7/31/2018
DATE ASSIGNED: **7/31/2018**
SHEET: 1 of 1

Test Pit/ Boring	Depth (ft)	Sampler /No. Rings	Field Dry Desnity (pcf)	Field Moisture (%)	Atterberg	Consolidation	Corrosivity Suite	Direct Shear	Expansion Index	Fines Fraction (Minus No. 200)	Hydraulic Conductivity	Maximum Density/ Optimum Moisture	No. 200 Wash / Grain Size Analysis (Particle Size)	Particle Size w/ Hydrometer	R-Value	Reshear (4/7 passes)	Specific Gravity	Sulfate	Triaxial (UU)	Triaxial (CU)	Unconfined Compression	Other	Other	Remarks
TP-1	5.5	Bulk																						
	11	Bulk																						
TP-1	5.5	R	79.7	6.3	X								X											
	11	R	77.3	7.0	X			X					X											

SPECIAL INSTRUCTIONS: 1 WEEK RUSH - Complete by Tu. 8/7

SHEAR TEST RESULTS





Tan sitly sand samples were submerged for at least 24 hours.
The samples had a density of 77.3 lbs./cu.ft. and a moisture content of 7 %
Cohesion = 125 psf
Friction Angle = 35 degrees
Based on Ultimate Strength

Geotechnical Engineering Investigation 517 Shinohara Lane Chula Vista, CA	Project No.	18-2487
	Plate	D-1

HAMILTON & ASSOCIATES

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No. 200 Wash and Grain Analysis
ASTM D 1140

Project Name:	**Partner (Chula Vista)**	**Tested By:**	**RM**
Project No.:	**18-2487**	**Checked By:**	
Boring No.:	**TP-1**	**Depth (ft.):**	**5.5**
Sample No.:	**N/A**	**Date:**	**8/1/2018**

Soil Description: **Tan silty sand**

Moisture Determination

Tare No.	L-34
Tare Weight (g)	3.6
Wet Weight of Soil plus Tare (g)	99.3
Oven Dried Weight of Soil plus Tare (g)	93.6
Moisture Content (%)	6.3

Grain Analysis

Post #200 Wash Mass of Oven Dried Soil for Grain Analysis plus Tare (g)	62.8	
Mass of Soil Retained on Seive (g)	3"	0.0
	1 1/2"	0.0
	1"	0.0
	3/4"	0.0
	3/8"	0.0
	#4	0.0
	#10	0.0
	#20	0.9
	#40	0.8
	#60	0.8
	#100	1.6
	#140	12.4
	#200	42.9
	Pass #200	4.2

0.0	% Gravel
66.0	% Sand
38.9	% Fines

Plate G-1

HAMILTON & Associates

No. 200 Wash and Grain Analysis
ASTM D 1140

Project Name:	**Partner (Chula Vista)**	**Tested By:**	**RM**
Project No.:	**18-2487**	**Checked By:**	
Boring No.:	**TP-1**	**Depth (ft.):**	**11**
Sample No.:	**N/A**	**Date:**	**8/1/2018**

Soil Description: **Tan silty sand**

Moisture Determination

Tare No.	91.0
Tare Weight (g)	3.7
Wet Weight of Soil plus Tare (g)	91.9
Oven Dried Weight of Soil plus Tare (g)	86.1
Moisture Content (%)	7.0

Grain Analysis

Post #200 Wash Mass of Oven Dried Soil for Grain Analysis plus Tare (g)	62.5	
Mass of Soil Retained on Seive (g)	3"	0.0
	1 1/2"	0.0
	1"	0.0
	3/4"	0.0
	3/8"	0.0
	#4	0.0
	#10	0.1
	#20	0.8
	#40	0.9
	#60	0.9
	#100	2.7
	#140	14.7
	#200	33.1
	Pass #200	5.3

0.1	% Gravel
64.3	% Sand
35.1	% Fines

Plate G-2

HAMILTON
& Associates

1641 Border Avenue • Torrance, CA 90501 T 310.618.2190 888.618.2190 F 310.618.2191 W hamilton-associates.net

March 11, 2019
H&A Project No. 18-2487
Partner Project No. 17-199602.7

Partner Engineering and Science, Inc.
4518 N.12 Street Suite 201
Phoenix AZ, 85016

Attention: Mr. Matthew Marcus, Technical Director- Geotechnical Engineering

Subject: Laboratory Testing of Soil Samples, Partner (Chula Vista)
517 Shinohara Lane, Chula Vista, California 91911

Dear Mr. Marcus:

We have completed the laboratory tests on the samples provided for the subject project. Enclosed is a summary of laboratory test results.

We thank you for the opportunity to provide laboratory testing services. If there are any questions, please do not hesitate to contact the undersigned.

Respectfully submitted,
HAMILTON & ASSOCIATES, INC.

Rosa E. Murrieta
Laboratory Supervisor | Staff Geologist

David T. Hamilton, PE, GE
President

Distribution: (1) Matthew Marcus
mmarcus@partneresi.com
(2) Brett Bova
bbova@partneresi.com

MOISTURE CONTENT AND DENSITY TESTS

Relatively undisturbed soil retained within the rings of the Modified California barrel sampler was tested in the laboratory to determine in-place dry density and moisture content. Test results are presented in Table 1.

DIRECT SHEAR TESTS

Direct shear (ASTM D3080) tests were performed on selected relatively undisturbed samples to determine the shear strength parameters of various soil samples, respectively. The results of these tests are shown graphically on the appended “D” Plates.

HAMILTON
& Associates

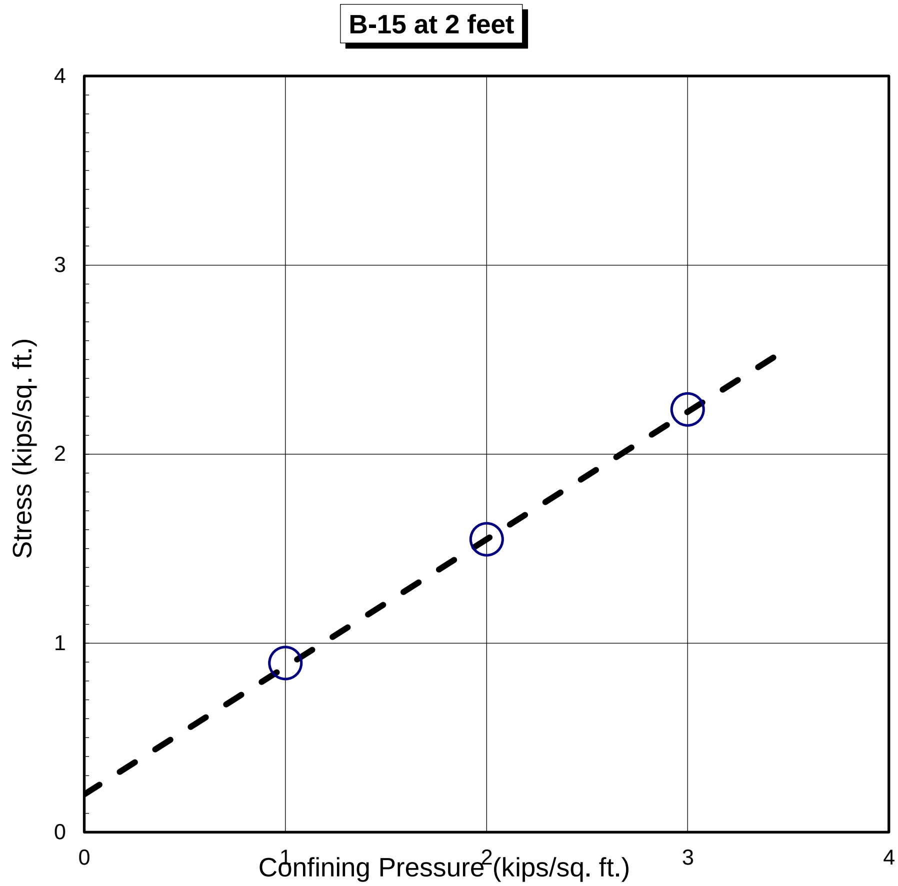
LABORATORY SCHEDULE SHEET

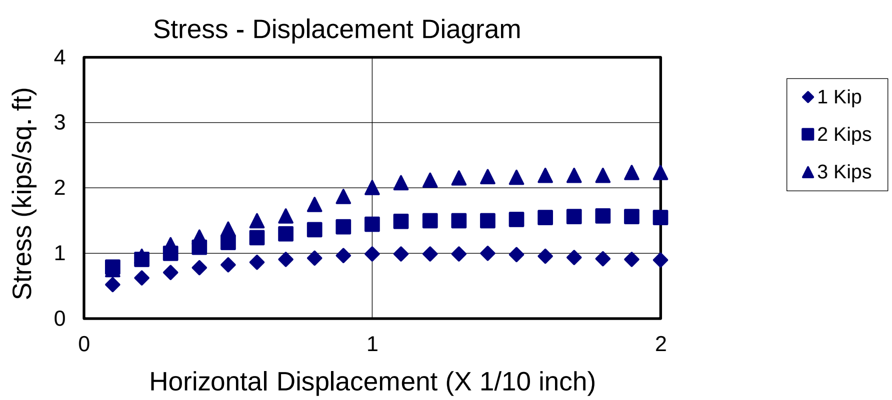
JOB TITLE: Partner (Chula Vista)
H&A PROJECT NO. 18-2487
SCHEDULED BY: FC/YK
DATE RECEIVED: **3.5.19**
DATE ASSIGNED: **3.4.19**
SHEET: 1 of 1

Test Pit/ Boring No.	Depth (ft)	Sampler/No. Rings	Field Dry Desnity (pcf)	Field Moisture (%)	Atterberg	Consolidation	Corrosivity Suite	Direct Shear	Expansion Index	Fines Fraction (Minus No. 200)	Hydraulic Conductivity	Maximum Density/ Optimum Moisture	No. 200 Wash / Grain Size Analysis (Particle Size)	Particle Size w/ Hydrometer	R-Value	Reshear (4/7 passes)	Specific Gravity	Sulfate	Triaxial (UU)	Triaxial (CU)	Unconfined Compression	Other	Other	Remarks
B-12	5	R	103.5	5.3				X																
	10	R	90.2	8.9				X																
B-15	2	R	103.7	7.7				X																

SPECIAL INSTRUCTIONS: 1 WEEK RUSH

SHEAR TEST RESULTS





Clayey silt samples were submerged for at least 24 hours.

The samples had a density of 103.7 lbs./cu.ft. and a moisture content of 7.7 %

Cohesion = 200 psf

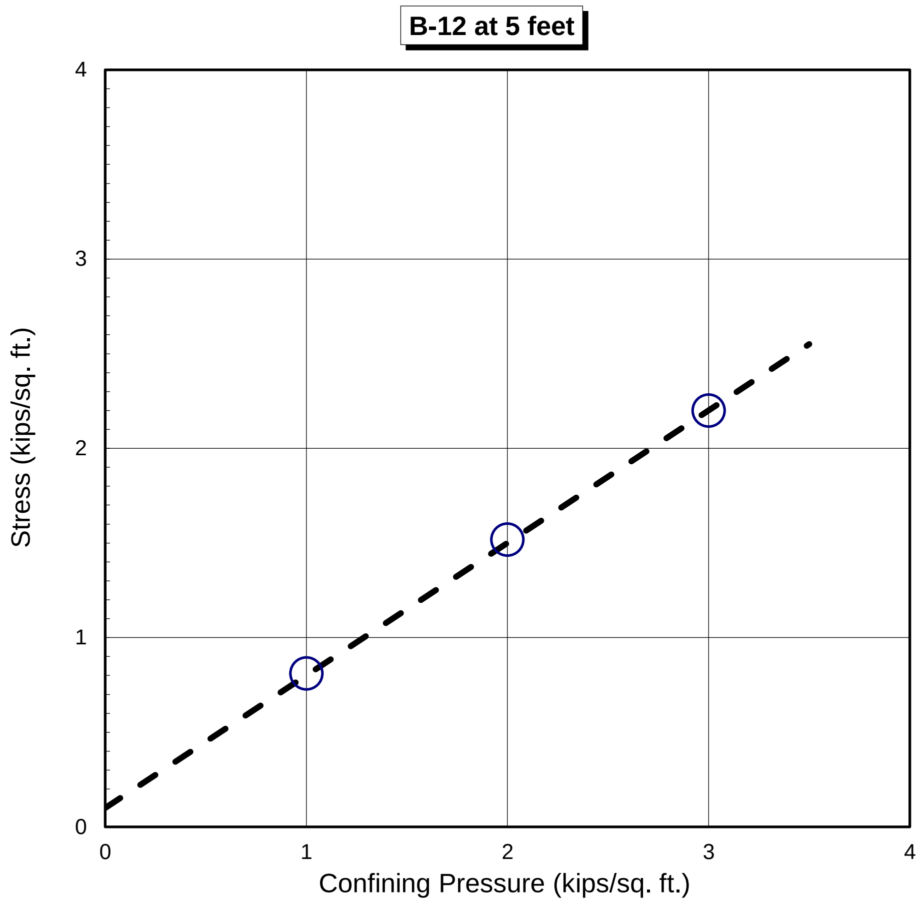
Friction Angle = 34 degrees

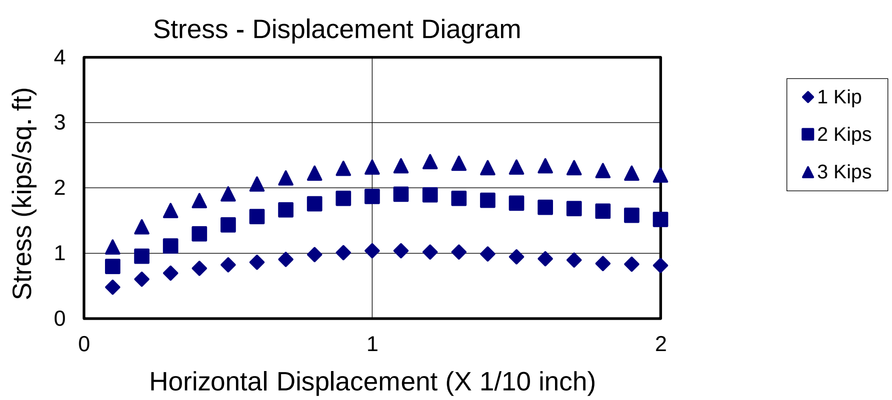
Based on Ultimate Strength

Geotechnical Engineering Investigation 517 Shinohara Lane Chula Vista, California	Project No.	18-2487
	Plate	D-1

HAMILTON & ASSOCIATES

SHEAR TEST RESULTS





Silty sand samples were submerged for at least 24 hours.

The samples had a density of 103.5 lbs./cu.ft. and a moisture content of 5.3 %

Cohesion = 100 psf

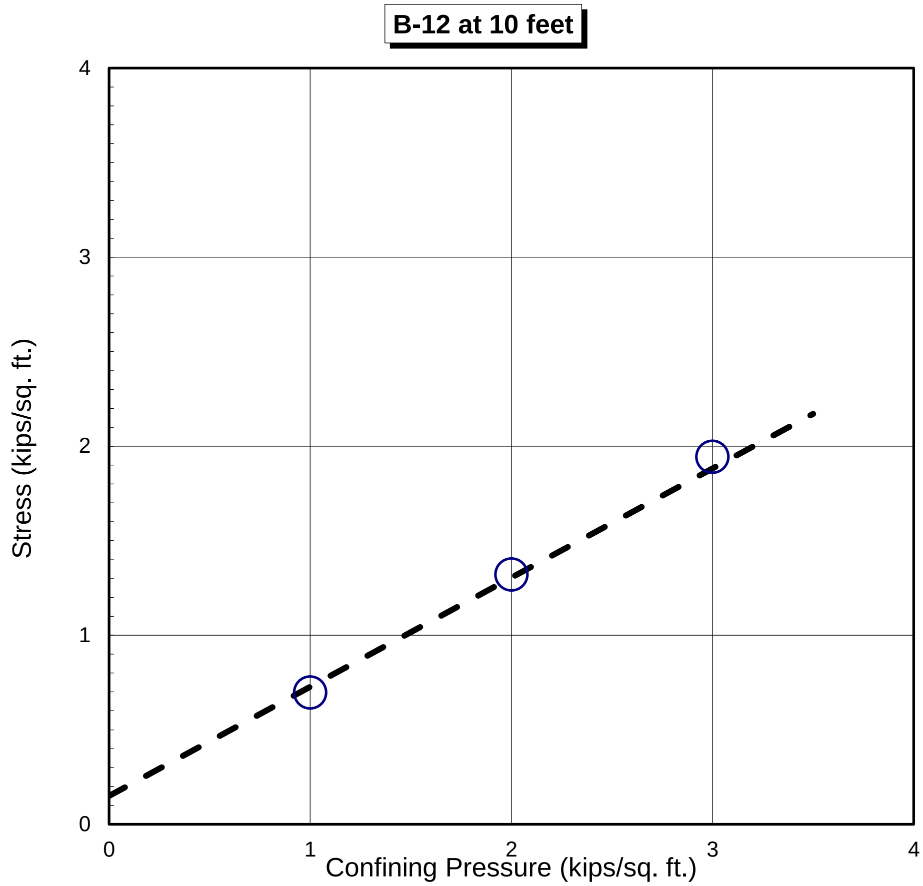
Friction Angle = 35 degrees

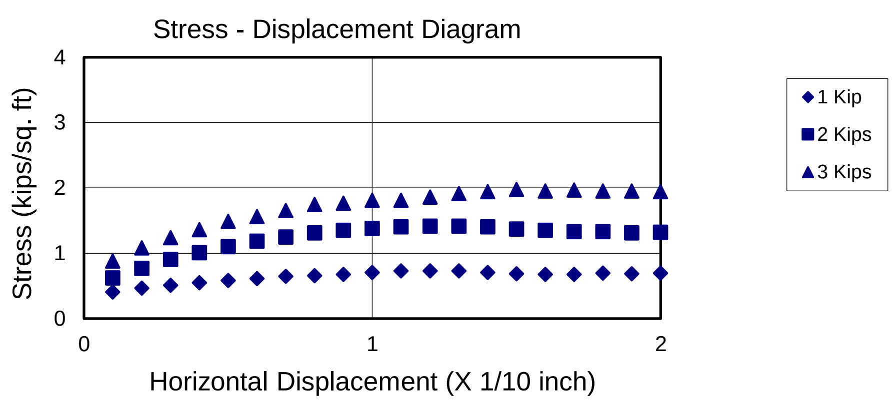
Based on Ultimate Strength

Geotechnical Engineering Investigation 517 Shinohara Lane Chula Vista, California	Project No.	18-2487
	Plate	D-2

HAMILTON & ASSOCIATES

SHEAR TEST RESULTS





Silty/clayey fine sands samples were submerged for at least 24 hours.

The samples had a density of 90.2 lbs./cu.ft. and a moisture content of 8.9 %

Cohesion = 150 psf

Friction Angle = 30 degrees

Based on Ultimate Strength

Geotechnical Engineering Investigation 517 Shinohara Lane Chula Vista, California	Project No.	18-2487
	Plate	D-3

HAMILTON & ASSOCIATES

April 10, 2019
H&A Project No. 18-2487
Partner Project No. 17-199602.4

Partner Engineering and Science, Inc.
4518 N.12 Street Suite 201
Phoenix AZ, 85016

Attention: Mr. Matthew Marcus, Technical Director- Geotechnical Engineering

Subject: Laboratory Testing of Soil Samples, Partner (Chula Vista)
517 Shinohara Lane, Chula Vista, California 91911

Dear Mr. Marcus:

We have completed the laboratory tests on the samples provided for the subject project. Enclosed is a summary of laboratory test results.

We thank you for the opportunity to provide laboratory testing services. If there are any questions, please do not hesitate to contact the undersigned.

Respectfully submitted,
HAMILTON & ASSOCIATES, INC.

Rosa E. Murrieta
Laboratory Supervisor | Staff Geologist

David T. Hamilton, PE, GE
President

Distribution: (1) Matthew Marcus
mmarcus@partneresi.com
(2) Brett Bova
bbova@partneresi.com

NO. 200 SIEVE (WASH)
No. 200 Sieves (Wash) were performed on selected samples to determine the fines content. The results of these tests are shown on Table 1.

ATTERBERG LIMITS
Atterberg Limits (ASTM D-4318) tests were performed on selected samples to determine the liquid limit, plastic limit, and the plasticity index of soils. The samples from Boring 12 at 10 feet is granular sand, therefore non-plastic limits and Atterberg limits cannot be determined. The results of these tests are shown on the appended “E” Plates.

HAMILTON & Associates

TABLE 1: LABORATORY RESULTS

JOB TITLE: Partner (Chula Vista)
H&A PROJECT NO. 18-2487
SCHEDULED BY: FC
DATE RECEIVED: **4.5.19**
DATE ASSIGNED: **4.5.19**
SHEET: 1 of 1

Test Pit/ Boring No.	Depth (ft)	Sampler/No. Rings	Field Dry Density (pcf)	Field Moisture (%)	Atterberg	Consolidation	Corrosivity Suite	Direct Shear	Expansion Index	Fines Fraction (Minus No. 200)	Hydraulic Conductivity	Maximum Density/ Optimum Moisture	No. 200 Wash / Grain Size Analysis (Particle Size)	Particle Size w/ Hydrometer	R-Value	Reshear (4/7 passes)	Specific Gravity	Sulfate	Triaxial (UU)	Triaxial (CU)	Unconfined Compression	Other	Other	Remarks
B-12	10				X					18.1														
B-15	2				X					40.4														

SPECIAL INSTRUCTIONS:

ATTERBERG LIMITS
ASTM D4318

HAMILTON & Associates

Project Name: Partner (Chula Vista)
Project No. : 18-2487
Boring No. : B-15
Sample No. : N/A

Tested By: RM
Checked By: RM
Depth (ft.): 2
Date: 4/9/2019

Visual Sample Description: Brown silty, clayey sand

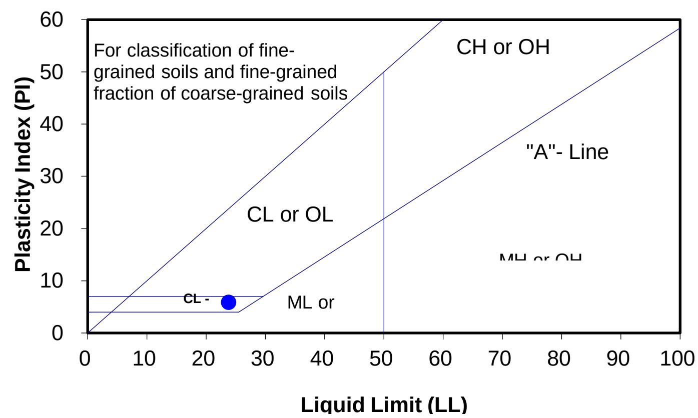
	PLASTIC LIMIT		LIQUID LIMIT			
	1	2	1	2	3	4
Number of Blows [N]:			32	28	23	
Tare No.:	P-8	J-1	J-2	J-3	P-1	
Wt. of Tare (gm):	15.79	15.59	15.56	16.03	14.98	
Wet Wt. of Soil + Tare (gm):	20.90	21.00	46.33	46.25	45.45	
Dry Wt. of Soil + Tare (gm):	20.10	20.20	40.60	40.60	39.40	
Moisture Content (%) [Wn]:	18.56	17.35	22.88	23.00	24.77	

Liquid Limit	24
Plastic Limit	18
Plasticity Index	6
USCS Classification	CL-ML

PI at "A" - Line = 0.73(LL-20) = 2.777973

One - Point Liquid Limit Calculation

LL =Wn(N/25) $^{0.121}$



PROCEDURES USED

- [ ] Wet Preparation Multipoint - Wet
- [X] Dry Preparation Multipoint - Dry
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test

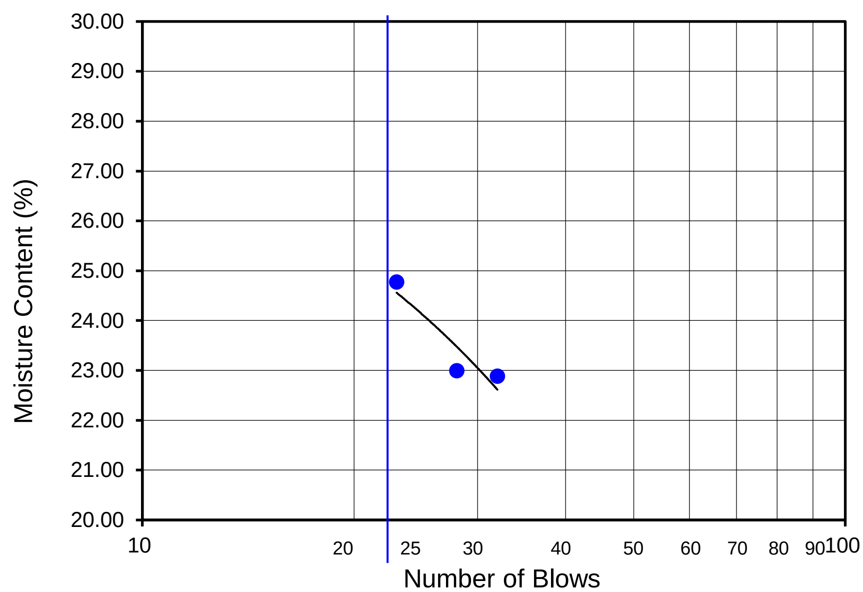


Plate E-1

APPENDIX D

General Geotechnical Design and Construction Considerations

Subgrade Preparation

Earthwork – Structural Fill/Excavations

Underground Pipeline Installation – Structural Backfill

Cast-in-Place Concrete

Foundations

Laterally Loaded Structures

Excavations and Dewatering

Waterproofing and Drainage

Chemical Treatment of Soils

Paving

Site Grading and Drainage

PARTNER

SUBGRADE PREPARATION

1. In general, construction should proceed per the project specifications and contract documents, as well as governing jurisdictional guidelines for the project site, including but not limited to the applicable State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Subgrade preparation in this section is considered to apply to the initial modifications to existing site conditions to prepare for new planned construction.
3. Prior to the start of subgrade preparation, a detailed conflict study including as-builts, utility locating, and potholing should be conducted. Existing features that are to be demolished should also be identified and the geotechnical study should be referenced to determine the need for subgrade preparation, such as over-excavation, scarification and compaction, moisture conditioning, and/or other activities below planned new structural fills, slabs on grade, pavements, foundations, and other structures.
4. The site conflicts, planned demolitions, and subgrade preparation requirements should be discussed in a pre-construction meeting with the pertinent parties, including the geotechnical engineer, inspector, contractors, testing laboratory, surveyor, and others.
5. In the event of preparations that will require work near to existing structures to remain in-place, protection of the existing structures should be considered. This also includes a geotechnical review of excavations near to existing structures and utilities and other concerns discussed in General Geotechnical Design and Construction Considerations, EARTHWORK and UNDERGROUND PIPELINE INSTALLATION.
6. Features to be demolished should be completely removed and disposed of per jurisdictional requirements and/or other conditions set forth as a part of the project. Resulting excavations or voids should be backfilled per the recommendations in the General Geotechnical Design and Construction Considerations, EARTHWORK section.
7. Vegetation, roots, soils containing organic materials, debris and/or other deleterious materials on the site should be removed from structural areas and should be disposed of as above. Replacement of such materials should be in accordance with the recommendations in the General Geotechnical Design and Construction Considerations, EARTHWORK section
8. Subgrade preparation required by the geotechnical report may also call for as over-excavation, scarification and compaction, moisture conditioning, and/or other activities below planned structural fills, slabs on grade, pavements, foundations, and other structures. These requirements should be provided within the geotechnical report. The execution of this work should be observed by the geotechnical engineering representative or inspector for the site. Testing of the subgrade preparation should be performed per the recommendations in the General Geotechnical Design and Construction Considerations, EARTHWORK section.

9. Subgrade Preparation cannot be completed on frozen ground or on ground that is not at a proper moisture condition. Wet subgrades may be dried under favorable weather if they are disked and/or actively worked during hot, dry, weather, when exposed to wind and sunlight. Frozen ground or wet material can be removed and replaced with suitable material. Dry material can be pre-soaked, or can have water added and worked in with appropriate equipment. The soil conditions should be monitored by the geotechnical engineer prior to compaction. Following this type of work, approved subgrades should be protected by direction of surface water, covering, or other methods, otherwise, re-work may be needed.

EARTHWORK – STRUCTURAL FILL

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Earthwork in this section is considered to apply to the re-shaping and grading of soil, rock, and aggregate materials for the purpose of supporting man-made structures. Where earthwork is needed to raise the elevation of the site for the purpose of supporting structures or forming slopes, this is referred to as the placement of structural fill. Where lowering of site elevations is needed prior to the installation of new structures, this is referred to as earthwork excavations.
3. Prior to the start of earthwork operations, the geotechnical study should be referenced to determine the need for subgrade preparation, such as over-excavation or scarification and compaction of unsuitable soils below planned structural fills, slabs on grade, pavements, foundations, and other structures. These required preparations should be discussed in a pre-construction meeting with the pertinent parties, including the geotechnical engineer, inspector, contractors, testing laboratory, surveyor, and others. The preparations should be observed by the inspector or geotechnical engineer representative, and following such subgrade preparation, the geotechnical engineer should observe the prepared subgrade to approve it for the placement of earthwork fills or new structures.
4. Structural fill materials should be relatively free of organic materials, man-made debris, environmentally hazardous materials, and brittle, non-durable aggregate, frozen soil, soil clods or rocks and/or any other materials that can break down and degrade over time.
5. In deeper structural fill zones, expansive soils (greater than 1.5 percent swell at 100 pounds per square foot surcharge) and rock fills (fills containing particles larger than 4 inches and/or containing more than 35 percent gravel larger than ¾-inch diameter or more than 50 percent gravel) may be used with the approval and guidance of the geotechnical report or geotechnical engineer. This may require the placement of geotextiles or other added costs and/or conditions. These conditions may also apply to corrosive soils (less than 2,000 ohm-cm resistivity, more than 50 ppm chloride content, more than 0.1 percent sulfates)
6. For structural fill zones that are closer in depth below planed structures, low expansive materials, and materials with smaller particle size are generally recommended, as directed by the geotechnical report (see criteria above in 5). This may also apply to corrosive soils.
7. For structural fill materials, in general the compaction equipment should be appropriate for the thickness of the loose lift being placed, and the thickness of the loose lift being placed should be at least two times the maximum particle size incorporated in the fill.
8. Fill lift thickness (including bedding) should generally be proportioned to achieve 95 percent or more of a standard proctor (ASTM D689) maximum dry density (MDD) or 90 percent or more of a modified proctor (ASTM D1557) MDD, depending on the state practices. For subgrades below

roadways, the general requirement for soil compaction is usually increased to 100 percent or more of the standard proctor MDD and 95 percent or more of the modified proctor MDD.

9. Soil compaction should be performed at a moisture content generally near optimum moisture content determined by either standard or modified proctor, and ideally within 3 percent below to 1 percent over the optimum for a standard proctor, and from 2 percent below to 2 percent above optimum for a modified proctor.
10. In some instances fill areas are difficult to access. In such cases a low-strength soil-cement slurry can be used in the place of compacted fill soil. In general such fills should be rated to have a 28-day strength of 75 to 125 psi, which in some areas is referred to as a "1-sack" slurry. It should be noted that these materials are wet during placement, and require a period of 2 days (24 hours) to cure before additional fill can be placed above them. Testing of this material can be done using concrete cylinder compression strength testing equipment, but care is needed in removing the test specimens from the molds. Field testing using the ball method, and spread or flow testing is also acceptable.
11. For fills to be placed on slopes, benching of fill lifts is recommended, which may require cutting into existing slopes to create a bench perpendicular to the slope where soil can be placed in a relatively horizontal orientation. For the construction of slopes, the slopes should be over-built and cut back to grade, as the material in the outer portion of the slope may not be well compacted.
12. For subgrade below roadways, runways, railways or other areas to receive dynamic loading, a proofroll of the finished, compacted subgrade should be performed by the geotechnical engineer or inspector prior to the placement of structural aggregate, asphalt or concrete. Proofrolling consists of observing the performance of the subgrade under heavy-loaded equipment, such as full, 4,000 Gallon water truck, loaded tandem-axel dump truck or similar. Areas that exhibit instability during proofroll should be marked for additional work prior to approval of the subgrade for the next stage of construction.
13. Quality control testing should be provided on earthwork. Proctor testing should be performed on each soil type, and one-point field proctors should be used to verify the soil types during compaction testing. If compaction testing is performed with a nuclear density gauge, it should be periodically correlated with a sand cone test for each soil type. Density testing should be performed per project specifications and or jurisdictional requirements, but not less than once per 12 inches elevation of any fill area, with additional tests per 12-inch fill area for each additional 7,500 square-foot section or portion thereof.
14. For earthwork excavations, OSHA guidelines should be referenced for sloping and shoring. Excavations over a depth of 20 feet require a shoring design. In the event excavations are planned near to existing structures, the geotechnical engineer should be consulted to evaluate whether such excavation will call for shoring or underpinning the adjacent structure. Pre-construction and post-construction condition surveys and vibration monitoring might also be helpful to evaluate any potential damage to surrounding structures.
15. Excavations into rock, partially weathered rock, cemented soils, boulders and cobbles, and other hard soil or "hard-pan" materials, may result in slower excavation rates, larger equipment with

specialized digging tools, and even blasting. It is also not unusual in these situations for screening and or crushing of rock to be called for. Blasting, hard excavating, and material processing equipment have special safety concerns and are more costly than the use of soil excavation equipment. Additionally, this type of excavation, especially blasting, is known to cause vibrations that should be monitored at nearby structures. As above, a pre-blast and post-blast conditions assessment might also be warranted.

UNDERGROUND PIPELINE – STRUCTURAL BACKFILL

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable State Department of Transportation, the State Department of Environmental Quality, the US Environmental Protection Agency, City and/or County Public Works, Occupational Safety and Health Administration (OSHA), Private Utility Companies, and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered, and in some cases work may take place to multiple different standards. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Underground pipeline in this section is considered to apply to the installation of underground conduits for water, storm water, irrigation water, sewage, electricity, telecommunications, gas, etc. Structural backfill refers to the activity of restoring the grade or establishing a new grade in the area where excavations were needed for the underground pipeline installation.
3. Prior to the start of underground pipeline installation, a detailed conflict study including as-builts, utility locating, and potholing should be conducted. The geotechnical study should be referenced to determine subsurface conditions such as caving soils, unsuitable soils, shallow groundwater, shallow rock and others. In addition, the utility company responsible for the line also will have requirements for pipe bedding and support as well as other special requirements. Also, if the underground pipeline traverses other properties, rights-of-way, and/or easements etc. (for roads, waterways, dams, railways, other utility corridors, etc.) those owners may have additional requirements for construction.
4. The required preparations above should be discussed in a pre-construction meeting with the pertinent parties, including the geotechnical engineer, inspector, contractors, testing laboratory, surveyor, and other stake holders.
5. For pipeline excavations, OSHA guidelines should be referenced for sloping and shoring. Excavations over a depth of 20 feet require a shoring design. In the event excavations are planned near to existing structures or pipelines, the geotechnical engineer should be consulted to evaluate whether such excavation will call for shoring or supporting the adjacent structure or pipeline. A pre-construction and post-construction condition survey and vibration monitoring might also be helpful to evaluate any potential damage to surrounding structures.
6. Excavations into rock, partially weathered rock, cemented soils, boulders and cobbles, and other hard soil or "hard-pan" materials, may result in slower excavation rates, larger equipment with specialized digging tools, and even blasting. It is also not unusual in these situations for screening and or crushing of rock to be called for. Blasting, hard excavating and material processing equipment have special safety concerns and are more costly than the use soil excavation equipment. Additionally, this type of excavation, especially blasting, is known to cause vibrations that should be monitored at nearby structures. As above, a pre-blast and post-blast conditions assessment might also be warranted.
7. Bedding material requirements vary between utility companies and might depend of the type of pipe material and availability of different types of aggregates in different locations. In

general, bedding refers to the material that supports the bottom of the pipe, and extends to 1 foot above the top of the pipe. In general the use of aggregate base for larger diameter pipes (6-inch diameter or more) is recommended lacking a jurisdictionally specified bedding material. Gas lines and smaller diameter lines are often backfilled with fine aggregate meeting the ASTM requirements for concrete sand. In all cases bedding with less than 2,000 ohm-cm resistivity, more than 50 ppm chloride content or more than 0.1 percent sulfates should not be used.

8. Structural backfill materials above the bedding should be relatively free of organic materials, man-made debris, environmentally hazardous materials, frozen material, and brittle, non-durable aggregate, soil clods or rocks and/or any other materials that can break down and degrade over time.
9. In general the backfill soil requirements will depend on the future use of the land above the buried line, but in most cases, excessive settlement of the pipe trench is not considered advisable or acceptable. As such, the structural backfill compaction equipment should be appropriate for the thickness of the loose lift being placed. The thickness of the loose lift being placed should be at least two times the maximum particle size incorporated in the fill. Care should be taken not to damage the pipe during compaction or compaction testing.
10. Fill lift thickness (including bedding) should generally be proportioned to achieve 95 percent or more of a standard proctor (ASTM D689) maximum dry density (MDD) or 90 percent or more of a modified proctor (ASTM D1557) MDD, depending on the state practices (in general the modified proctor is required in California and for projects in the jurisdiction of the Army Corps of Engineers). For backfills within the upper poritons of roadway subgrades, the general requirement for soil compaction is usually increased to 100 percent or more of the standard proctor MDD and 95 percent or more of the modified proctor MDD.
11. Soil compaction should be performed at a moisture content generally near optimum moisture content determined by either standard or modified proctor, and ideally within 3 percent below to 1 percent over the optimum for a standard proctor, and from 2 percent below to 2 percent above optimum for a modified proctor.
12. In some instances fill areas are difficult to access. In such cases a low-strength soil-cement slurry can be used in the place of compacted fill soil. In general such fills should be rated to have a 28-day strength of 75 to 125 psi, which in some areas is referred to as a "1-sack" slurry. It should be noted that these materials are wet, and require a period of 2 days (24 hours) to cure before additional fill can be placed above it. Testing of this material can be done using concrete cylinder compression strength testing equipment, but care is needed in removing the test specimens from the molds. Field testing using the ball method, and spread or flow testing is also acceptable.
13. Quality control testing should be provided on structural backfill to assist the contractor in meeting project specifications. Proctor testing should be performed on each soil type, and one-point field proctors should be used to verify the soil types during compaction testing. If compaction testing is performed with a nuclear density gauge, it should be periodically correlated with a sand cone test for each soil type.

14. Density testing should be performed on structural backfill per project specifications and or jurisdictional requirements, but not less than once per 12 inches elevation in each area, and additional tests for each additional 500 linear-foot section or portion thereof.

CAST-IN-PLACE CONCRETE
SLABS-ON-GRADE/STRUCTURES/PAVEMENTS

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Cast-in-place concrete (concrete) in this section is considered to apply to the installation of cast-in-place concrete slabs on grade, including reinforced and non-reinforced slabs, structures, and pavements.
3. In areas where concrete is bearing on prepared subgrade or structural fill soils, testing and approval of this work should be completed prior to the beginning of concrete construction.
4. In locations where a concrete is approved to bear on in-place (native) soil or in locations where approved documented fills have been exposed to weather conditions after approval, a concrete subgrade evaluation should be performed prior to the placement of reinforcing steel and or concrete. This can consist of probing with a "t"-handled rod, borings, penetrometer testing, dynamic cone penetration testing and/or other methods requested by the geotechnical engineer and/or inspector. Where unsuitable, wet, or frozen bearing material is encountered, the geotechnical engineer should be consulted for additional recommendations.
5. Slabs on grade should be placed on a 4-inch thick or more capillary barrier consisting of non-corrosive (more than 2,000 ohm-cm resistivity, less than 50 ppm chloride content and less than 0.1 percent sulfates) aggregate base or open-graded aggregate material. This material should be compacted or consolidated per the recommendations of the structural engineer or otherwise would be covered by the General Considerations for EARTHWORK.
6. Depending on the site conditions and climate, vapor barriers may be required below in-door grade-slabs to receive flooring. This reduces the opportunity for moisture vapor to accumulate in the slab, which could degrade flooring adhesive and result in mold or other problems. Vapor barriers should be specified by the structural engineer and/or architect. The installation of the barrier should be inspected to evaluate the correct product and thickness is used, and that it has not been damaged or degraded.
7. At times when rainfall is predicted during construction, a mud-mat or a thin concrete layer can be placed on prepared and approved subgrades prior to the placement of reinforcing steel or tendons. This serves the purpose of protecting the subgrades from damage once the reinforcement placement has begun.
8. Prior to the placement of concrete, exposed subgrade or base material and forms should be wetted, and form release compounds should be applied. Reinforcement support stands or ties should be

checked. Concrete bases or subgrades should not be so wet that they are softened or have standing water.

9. For a cast-in-place concrete, the form dimensions, reinforcement placement and cover, concrete mix design, and other code requirements should be carefully checked by an inspector before and during placement. The reinforcement should be specified by the structural engineering drawings and calculations.
10. For post-tension concrete, an additional check of the tendons is needed, and a tensioning inspection form should be prepared prior to placement of concrete.
11. For Portland cement pavements, forms an additional check of reinforcing dowels should performed per the design drawings.
12. During placement, concrete should be tested, and should meet the ACI and jurisdictional requirements and mix design targets for slump, air entrainment, unit weight, compressive strength, flexural strength (pavements), and any other specified properties. In general concrete should be placed within 90 minutes of batching at a temperature of less than 90 degrees Fahrenheit. Adding of water to the truck on the jobsite is generally not encouraged.
13. Concrete mix designs should be created by the accredited and jurisdictionally approved supplier to meet the requirements of the structural engineer. In general a water/cement ratio of 0.45 or less is advisable, and aggregates, cement, flyash, and other constituents should be tested to meet ASTM C-33 standards, including Alkali Silica Reaction (ASR). To further mitigate the possibility of concrete degradation from corrosion and ASR, Type II or V Portland Cement should be used, and fly ash replacement of 25 percent is also recommended. Air entrained concrete should be used in areas where concrete will be exposed to frozen ground or ambient temperatures below freezing.
14. Control joints are recommended to improve the aesthetics of the finished concrete by allowing for cracking within partially cut or grooved joints. The control joints are generally made to depths of about 1/4 of the slab thickness and are generally completed within the first day of construction. The spacing should be laid out by the structural engineer, and is often in a square pattern. Joint spacing is generally 5 to 15 feet on-center but this can vary and should be decided by the structural engineer. For pavements, construction joints are generally considered to function as control joints. Post-tensioned slabs generally do not have control joints.
15. Some slabs are expected to meet flatness and levelness requirements. In those cases, testing for flatness and levelness should be completed as soon as possible, usually the same day as concrete placement, and before cutting of control joints if possible. Roadway smoothness can also be measured, and is usually specified by the jurisdictional owner if is required.
16. Prior to tensioning of post-tension structures, placement of soil backfills or continuation of building on newly-placed concrete, a strength requirement is generally required, which should be specified by the structural engineer. The strength progress can be evaluated by the use of concrete compressive strength cylinders or maturity monitoring in some jurisdictions. Advancing with backfill, additional concrete work or post-tensioning without reaching strength benchmarks could result in damage and failure of the concrete, which could result in danger and harm to nearby people and property.

17. In general, concrete should not be exposed to freezing temperatures in the first 7 days after placement, which may require insulation or heating. Additionally, in hot or dry, windy weather, misting, covering with wet burlap or the use of curing compounds may be called for to reduce shrinkage cracking and curling during the first 7 days.

FOUNDATIONS

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Foundations in this section are considered to apply to the construction of structural supports which directly transfer loads from man-made structures into the earth. In general, these include shallow foundations and deep foundations. Shallow foundations are generally constructed for the purpose of distributing the structural loads horizontally over a larger area of earth. Some types of shallow foundations (or footings) are spread footings, continuous footings, mat foundations, and reinforced slabs-on-grade. Deep foundations are generally designed for the purpose of distributing the structural loads vertically deeper into the soil by the use of end bearing and side friction. Some types of deep foundations are driven piles, auger-cast piles, drilled shafts, caissons, helical piers, and micro-piles.
3. For shallow foundations, the minimum bearing depth considered should be greater than the maximum design frost depth for the location of construction. This can be found on frost depth maps (ICC), but the standard of practice in the city and/or county should also be consulted. In general the bearing depth should never be less than 18 inches below planned finished grades.
4. Shallow continuous foundations should be sized with a minimum width of 18 inches and isolated spread footings should be a minimum of 24 inches in each direction. Foundation sizing, spacing, and reinforcing steel design should be performed by a qualified structural engineer.
5. The geotechnical engineer will provide an estimated bearing capacity and settlement values for the project based on soil conditions and estimated loads provided by the structural engineer. It is assumed that appropriate safety factors will be applied by the structural engineer.
6. In areas where shallow foundations are bearing on prepared subgrade or structural fill soils, testing and approval of this work should be completed prior to the beginning of foundation construction.
7. In locations where the shallow foundations are approved to bear on in-place (native) soil or in locations where approved documented fills have been exposed to weather conditions after approval, a foundation subgrade evaluation should be performed prior to the placement of reinforcing steel. This can consist of probing with a "t"-handled rod, borings, penetrometer testing, dynamic cone penetration testing and/or other methods requested by the geotechnical engineer and/or inspector. Where unsuitable foundation bearing material is encountered, the geotechnical engineer should be consulted for additional recommendations.
8. For shallow foundations to bear on rock, partially weathered rock, hard cemented soils, and/or boulders, the entire foundation system should bear directly on such material. In this case, the rock surface should be prepared so that it is clean, competent, and formed into a roughly horizontal, stepped base. If that is not possible, then the entire structure should be underlain by a zone of

structural fill. This may require the over-excavation in areas of rock removal and/or hard dig. In general this zone can vary in thickness but it should be a minimum of 1 foot thick. The geotechnical engineer should be consulted in this instance.

9. At times when rainfall is predicted during construction, a mud-mat or a thin concrete layer can be placed on prepared and approved subgrades prior to the placement of reinforcing steel. This serves the purpose of protecting the subgrades from damage once the reinforcing steel placement has begun.
10. For cast-in-place concrete foundations, the excavations dimensions, reinforcing steel placement and cover, structural fill compaction, concrete mix design, and other code requirements should be carefully checked by an inspector before and during placement.

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11. For deep foundations, the geotechnical engineer will generally provide design charts that provide foundations axial capacity and uplift resistance at various depths given certain-sized foundations. These charts may be based on blow count data from drilling and or laboratory testing. In general safety factors are included in these design charts by the geotechnical engineer.
12. In addition, the geotechnical engineer may provide other soil parameters for use in the lateral resistance analysis. These parameters are usually raw data, and safety factors should be provided by the shaft designer. Sometimes, direct shear and or tri-axial testing is performed for this analysis.
13. In general the spacing of deep foundations is expected to be 6 shaft diameters or more. If that spacing is reduced, a group reduction factor should be applied by the structural engineer to the foundation capacities per FHWA guidelines. The spacing should not be less than 2.5 shaft diameters.
14. For deep foundations, a representative of the geotechnical engineer should be on-site to observe the excavations (if any) to evaluate that the soil conditions are consistent with the findings of the geotechnical report. Soil/rock stratigraphy will vary at times, and this may result in a change in the planned construction. This may require the use of fall protection equipment to perform observations close to an open excavation.
15. For driven foundations, a representative of the geotechnical engineer should be on-site to observe the driving process and to evaluate that the resistance of driving is consistent with the design assumptions. Soil/rock stratigraphy will vary at times and may this may result in a change in the planned construction.
16. For deep foundations, the size, depth, and ground conditions should be verified during construction by the geotechnical engineer and/or inspector responsible. Open excavations should be clean, with any areas of caving and groundwater seepage noted. In areas below the groundwater table, or areas where slurry is used to keep the trench open, non-destructive testing techniques should be used as outlined below.
17. Steel members including structural steel piles, reinforcing steel, bolts, threaded steel rods, etc. should be evaluated for design and code compliance prior to pick-up and placement in the foundation. This includes verification of size, weight, layout, cleanliness, lap-splices, etc. In addition, if non-destructive testing such as crosshole sonic logging or gamma-gamma logging is required, access tubes should be attached to the steel reinforcement prior to placement, and should be

relatively straight, capped at the bottom, and generally kept in-round. These tubes must be filled with water prior to the placement of concrete.

18. In cases where steel welding is required, this should be observed by a certified welding inspector.
19. In many cases, a crane will be used to lower steel members into the deep foundations. Crane picks should be carefully planned, including the ground conditions at placement of outriggers, wind conditions, and other factors. These are not generally provided in the geotechnical report, but can usually be provided upon request.
20. Cast-in-place concrete, grout or other cementations materials should be pumped or distributed to the bottom of the excavation using a tremmie pipe or hollow stem auger pipe. Depending on the construction type, different mix slumps will be used. This should be carefully checked in the field during placement, and consolidation of the material should be considered. Use of a vibrator may be called for.
21. For work in a wet excavation (slurry), the concrete placed at the bottom of the excavation will displace the slurry as it comes up. The upper layer of concrete that has interacted with the slurry should be removed and not be a part of the final product.
22. Bolts or other connections to be set in the top after the placement is complete should be done immediately after final concrete placement, and prior to the on-set of curing.
23. For shafts requiring crosshole sonic logging or gamma-gamma testing, this should be performed within the first week after placement, but not before a 2 day curing period. The testing company and equipment manufacturer should provide more details on the requirements of the testing.
24. Load testing of deep foundations is recommended, and it is often a project requirement. In some cases, if test piles are constructed and tested, it can result in a significant reduction of the amount of needed foundations. The load testing frame and equipment should be sized appropriately for the test to be performed, and should be observed by the geotechnical engineer or inspector as it is performed. The results are provided to the structural engineer for approval.

LATERALLY LOADED STRUCTURES - RETAINING WALLS/SLOPES/DEEP FOUNDATIONS/MISCELLANEOUS

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Laterally loaded structures for this section are generally meant to describe structures that are subjected to loading roughly horizontal to the ground surface. Such structures include retaining walls, slopes, deep foundations, tall buildings, box culverts, and other buried or partially buried structures.
3. The recommendations put forth in General Geotechnical Design and Construction Considerations for FOUNDATIONS, CAST-IN-PLACE CONCRETE, EARTHWORK, and SUBGRADE PREPARATION should be reviewed, as they are not all repeated in this section, but many of them will apply to the work. Those recommendations are incorporated by reference herein.
4. Laterally loaded structures are generally affected by overburden pressure, water pressure, surcharges, and other static loads, as well as traffic, seismic, wind, and other dynamic loads. The structural engineer must account for these loads. In addition, eccentric loading of the foundation should be evaluated and accounted for by the structural engineer. The structural engineer is also responsible for applying the appropriate factors of safety to the raw data provided by the geotechnical engineer.
5. The geotechnical report should provide data regarding soil lateral earth pressures, seismic design parameters, and groundwater levels. In the report the pressures are usually reported as raw data in the form of equivalent fluid pressures for three cases. 1. Static is for soil pressure against a structure that is fixed at top and bottom, like a basement wall or box culvert. 2. Active is for soil pressure against a wall that is free to move at the top, like a retaining wall. 3. Passive is for soil that is resisting the movement of the structure, usually at the toe of the wall where the foundation and embedded section are located. The structural engineer is responsible for deciding on safety factors for design parameters and groundwater elevations based on the raw data in the geotechnical report.
6. Generally speaking, direct shear or tri-axial shear testing should be performed for this evaluation in cases of soil slopes or unrestrained soil retaining walls over 6 feet in height or in lower walls in some cases based on the engineer's judgment. For deep foundations and completely buried structures, this testing will be required per the discretion of the structural engineer.
7. For non-confined retaining walls (walls that are not attached at the top) and slopes, a geotechnical engineer should perform overall stability analysis for sliding, overturning, and global stability. For walls that are structurally restrained at the top, the geotechnical engineer does not generally perform this analysis. Internal wall stability should be designed by the structural engineer.

8. Cut slopes into rock should be evaluated by an engineering geologist, and rock coring to identify the orientation of fracture plans, faults, bedding planes, and other features should be performed. An analysis of this data will be provided by the engineering geologist to identify modes of failure including sliding, wedge, and overturning, and to provide design and construction recommendations.
9. For laterally loaded deep foundations that support towers, bridges or other structures with high lateral loads, geotechnical reports generally provide parameters for design analysis which is performed by the structural engineer. The structural engineer is responsible for applying appropriate safety factors to the raw data from the geotechnical engineer.
10. Construction recommendations for deep foundations can be found in the General Geotechnical Design and Construction Considerations-FOUNDATIONS section.
11. Construction of retaining walls often requires temporary slope excavations and shoring, including soil nails, soldier piles and lagging or laid-back slopes. This should be done per OSHA requirements and may require specialty design and contracting.
12. In general, surface water should not be directed over a slope or retaining wall, but should be captured in a drainage feature trending parallel to the slope, with an erosion protected outlet to the base of the wall or slope.
13. Waterproofing for retaining walls is generally required on the backfilled side, and they should be backfilled with an 18-inch zone of open graded aggregate wrapped in filter fabric or a synthetic draining product, which outlets to weep holes or a drain at the base of the wall. The purpose of this zone, which is immediately behind the wall is to relieve water pressures from building behind the wall.
14. Backfill compaction around retaining walls and slopes requires special care. Lighter equipment should be considered, and consideration to curing of cementitious materials used during construction will be called for. Additionally, if mechanically stabilized earth walls are being constructed, or if tie-backs are being utilized, additional care will be necessary to avoid damaging or displacing the materials. Use of heavy or large equipment, and/or beginning of backfill prior to concrete strength verification can create dangers to construction and human safety. Please refer to the General Geotechnical Design and Construction Considerations-CAST-IN-PLACE CONCRETE section. These concerns will also apply to the curing of cell grouting within reinforced masonry walls.
15. Usually safety features such as handrails are designed to be installed at the top of retaining walls and slopes. Prior to their installation, workers in those areas will need to be equipped with appropriate fall protection equipment.

EXCAVATION AND DEWATERING

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Excavation and Dewatering for this section are generally meant to describe structures that are intended to create stable, excavations for the construction of infrastructure near to existing development and below the groundwater table.
3. The recommendations put forth in General Geotechnical Design and Construction Considerations for LATERALLY LOADED STRUCTURES, FOUNDATIONS, CAST-IN-PLACE CONCRETE, EARTHWORK, and SUBGRADE PREPARATION should be reviewed, as they are not all repeated in this section, but many of them will apply to the work. Those recommendations are incorporated by reference herein.
4. The site excavations will generally be affected by overburden pressure, water pressure, surcharges, and other static loads, as well as traffic, seismic, wind, and other dynamic loads. The structural engineer must account for these loads as described in Section 5.2 of this report. In addition, eccentric loading of the foundation should be evaluated and accounted for by the structural engineer. The structural engineer is also responsible for applying the appropriate factors of safety to the raw data provided by the geotechnical engineer.
5. The geotechnical report should provide data regarding soil lateral earth pressures, seismic design parameters, and groundwater levels. In the report the pressures are usually reported as raw data in the form of equivalent fluid pressures for three cases. 1. Static is for soil pressure against a structure that is fixed at top and bottom, like a basement wall or box culvert. 2. Active is for soil pressure against a wall that is free to move at the top, like a retaining wall. 3. Passive is for soil that is resisting the movement of the structure, usually at the toe of the wall where the foundation and embedded section are located. The structural engineer is responsible for deciding on safety factors for design parameters and groundwater elevations based on the raw data in the geotechnical report.
6. The parameters provided above are based on laboratory testing and engineering judgement. Since numerous soil layers with different properties will be encountered in a large excavation, assumptions and judgement are used to generate the equivalent fluid pressures to be used in design. Factors of safety are not included in those numbers and should be evaluated prior to design.
7. Groundwater, if encountered will dramatically change the stability of the excavation. In addition, pumping of groundwater from the bottom of the excavation can be difficult and costly, and it can result in potential damage to nearby structures if groundwater drawdown occurs. As such, we recommend that groundwater monitoring be performed across the site during design and prior to construction to assist in the excavation design and planning.
8. Groundwater pumping tests should be performed if groundwater pumping will be needed during construction. The pumping tests can be used to estimate drawdown at nearby properties, and also

will be needed to determine the hydraulic conductivity of the soil for the design of the dewatering system.

9. For excavation stabilization in granular and dense soil, the use of soldier piles and lagging is recommended. The soldier pile spacing and size should be determined by the structural engineer based on the lateral loads provided in the report. In general, the spacing should be more than two pile diameters, and less than 8 feet. Soldier piles should be advanced 5 feet or more below the base of the excavation. Passive pressures from Section 5.2 can be used in the design of soldier piles for the portions of the piles below the excavation.
10. If the piles are drilled, they should be grouted in-place. If below the groundwater table, the grouting should be accomplished by tremmie pipe, and the concrete should be a mix intended for placement below the groundwater table. For work in a wet excavation, the concrete placed at the bottom of the excavation will displace the water as it comes up. The upper layer of concrete that has interacted with the water should be removed and not be a part of the final product. Lagging should be specially designed timber or other lagging. The temporary excavation will need to account for seepage pressures at the toe of the wall as well as hydrostatic forces behind the wall.
11. Depending on the loading, tie back anchors and/or soil nails may be needed. These should be installed beyond the failure envelope of the wall. This would be a plane that is rotated upward 55 degrees from horizontal. The strength of the anchors behind this plane should be considered, and bond strength inside the plane should be ignored. If friction anchors are used, they should extend 10 feet or more beyond the failure envelope. Evaluation of the anchor length and encroachment onto other properties, and possible conflicts with underground utilities should be carefully considered. Anchors are typically installed 25 to 40 degrees below horizontal. The capacity of the anchors should be checked on 10% of locations by loading to 200% of the design strength. All should be loaded to 120% of design strength, and should be locked off at 80%
12. The shoring and tie backs should be designed to allow less than ½ inch of deflection at the top of the excavation wall, where the wall is within an imaginary 1:1 line extending downward from the base of surrounding structures. This can be expanded to 1 inch of deflection if there is no nearby structure inside that plane. An analysis of nearby structures to locate their depth and horizontal position should be conducted prior to shored excavation design.
13. Assuming that the excavations will encroach below the groundwater table, allowances for drainage behind and through the lagging should be made. The drainage can be accomplished by using an open-graded gravel material that is wrapped in geotextile fabric. The lagging should allow for the collected water to pass through the wall at select locations into drainage trenches below the excavation base. These trenches should be considered as sump areas where groundwater can be pumped out of the excavation.
14. The pumped groundwater needs to be handled properly per jurisdictional guidelines.
15. In general, surface water should not be directed over a slope or retaining wall, but should be captured in a drainage feature trending parallel to the slope, with an erosion protected outlet to the base of the wall or slope.

16. Safety features such as handrails or barriers are to be designed to be installed at the top of retaining walls and slopes. Prior to their installation, workers in those areas will need to be equipped with appropriate fall protection equipment.

Waterproofing and Back Drainage

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Waterproofing and Back drainage structures for this section are generally meant to describe permanent subgrade structures that are planned to be below the historic high groundwater elevation of 20 feet below existing grades.
3. The recommendations put forth in General Geotechnical Design and Construction Considerations for FOUNDATIONS, CAST-IN-PLACE CONCRETE, EARTHWORK, and SUBGRADE PREPARATION should be reviewed, as they are not all repeated in this section, but many of them will apply to the work. Those recommendations are incorporated by reference herein.
4. In general, surface water should not be directed over a slope or retaining wall, but should be captured in a drainage feature trending parallel to the slope, with an erosion protected outlet to the base of the wall or slope.
5. Waterproofing for retaining walls is generally required on the backfilled side, and they should be backfilled with an 18-inch zone of open graded aggregate wrapped in filter fabric or a synthetic draining product, which outlets to weep holes or a drain at the base of the wall. The purpose of this zone, which is immediately behind the wall is to relieve water pressures from building behind the wall.
6. For the basement walls on this site, sump pumps will be needed to reduce the build-up of water in the basement. The design should be for a historic high groundwater level of 20 feet bgs. The pumping system should be designed to keep the slab and walls relatively dry so that mold, efflorescence, and other detrimental effects to the concrete structure will not result.
7. Backfill compaction around retaining walls and slopes requires special care. Lighter equipment should be considered, and consideration to curing of cementitious materials used during construction will be called for. Additionally, if mechanically stabilized earth walls are being constructed, or if tie-backs are being utilized, additional care will be necessary to avoid damaging or displacing the materials. Use of heavy or large equipment, and/or beginning of backfill prior to concrete strength verification can create dangers to construction and human safety. Please refer to the General Geotechnical Design and Construction Considerations-CAST-IN-PLACE CONCRETE section. These concerns will also apply to the curing of cell grouting within reinforced masonry walls.

CHEMICAL TREATMENT OF SOIL

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, State Department of Environmental Quality, the US Environmental Protection Agency, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Chemical treatment of soil for this section is generally meant to describe the process of improving soil properties for a specific purpose, using cement or chemical lime.
3. A mix design should be performed by the geotechnical engineer to help it meet the specific strength, plasticity index, durability, and/or other desired properties. The mix design should be performed using the proposed chemical lime or cement proposed for use by the contractor, along with samples of the site soil that are taken from the material to be used in the process.
4. For the mix design the geotechnical engineer should perform proctor testing to determine optimum moisture content of the soil, and then mix samples of the soil at 3 percent above optimum moisture content with varying concentrations of lime or cement. The samples will be prepared and cured per ASTM standards, and then after 7-days for curing, they will be tested for compression strength. Durability testing goes on for 28 days.
5. Following this testing, the geotechnical engineer will provide a recommended mix ratio of cement or chemical lime in the geotechnical report for use by the contractor. The geotechnical engineer will generally specify a design ratio of 2 percent more than the minimum to account for some error during construction.
6. Prior to treatment, the in-place soil moisture should be measured so that the correct amount of water can be used during construction. Work should not be performed on frozen ground.
7. During construction, special considerations for construction of treated soils should be followed. The application process should be conducted to prevent the loss of the treatment material to wind which might transport the materials off site, and workers should be provided with personal protective equipment for dust generated in the process.
8. The treatment should be applied evenly over the surface, and this can be monitored by use of a pan placed on the subgrade. This can also be tested by preparing test specimens from the in-place mixture for laboratory testing.
9. Often, after or during the chemical application, additional water may be needed to activate the chemical reaction. In general, it should be maintained at about 3 percent or more above optimum moisture. Following this, mixing of the applied material is generally performed using specialized equipment.
10. The total amount of chemical provided can be verified by collecting batch tickets from the delivery trucks, and the depth of the treatment can be verified by digging of test pits, and the use of reagents that react with lime and or cement.

11. For the use of lime treatment, compaction should be performed after a specified amount of time has passed following mixing and re-grading. For concrete, compaction should be performed immediately after mixing and re-grading. In both cases, some swelling of the surface should be expected. Final grading should be performed the following day of the initial work for lime treatment, and within 2 to 4 hours for soil cement.
12. Quality control testing of compacted treated subgrades should be performed per the recommendations of the geotechnical report, and generally in accordance with General Geotechnical Design and Construction Considerations - EARTHWORK

PAVING

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Paving for this section is generally meant to describe the placement of surface treatments on travel-ways to be used by rubber-tired vehicles, such as roadways, runways, parking lots, etc.
3. The geotechnical engineer is generally responsible for providing structural analysis to recommend the thickness of pavement sections, which can include asphalt, concrete pavements, aggregate base, cement or lime treated aggregate base, and cement or lime treated subgrades.
4. The civil engineer is generally responsible for determining which surface finishes and mixes are appropriate, and often the owner, general contractor and/or other party will decide on lift thickness, the use of tack coats and surface treatments, etc.
5. The geotechnical engineer will generally be provided with the planned traffic loading, as well as reliability, design life, and serviceability factors by the jurisdiction, traffic engineer, designer, and/or owner. The geotechnical study will provide data regarding soil resiliency and strength. A pavement modeling software is generally used to perform the analysis for design, however, jurisdictional minimum sections also must be considered, as well as construction considerations and other factors.
6. The geotechnical report report will generally provide pavement section thicknesses if requested.
7. For construction of overlays, where new pavement is being placed on old pavement, an evaluation of the existing pavement is needed, which should include coring the pavement, evaluation of the overall condition and thickness of the pavement, and evaluation of the pavement base and subgrade materials.
8. In general, the existing pavement is milled and treated with a tack coat prior to the placement of new pavement for the purpose of creating a stronger bond between the old and new material. This is also a way of removing aged asphalt and helping to maintain finished grades closer to existing conditions grading and drainage considerations.
9. If milling is performed, a minimum of 2 inches of existing asphalt should be left in-place to reduce the likelihood of equipment breaking through the asphalt layer and destroying its integrity. After milling and before the placement of tack coat, the surface should be evaluated for cracking or degradation. Cracked or degraded asphalt should be removed, spanned with geosynthetic reinforcement, or be otherwise repaired per the direction of the civil and or geotechnical engineer prior to continuing construction. Proofrolling may be requested.
10. For pavements to be placed on subgrade or base materials, the subgrade and base materials should be prepared per the General Geotechnical Design and Construction Considerations – EARTHWORK section.

11. Following the proofrolling as described in the General Geotechnical Design and Construction Considerations – EARTHWORK section, the application of subgrade treatment, base material, and paving materials can proceed per the recommendations in the geotechnical report and/or project plans. The placement of pavement materials or structural fills cannot take place on frozen ground.
12. The placement of aggregate base material should conform to the jurisdictional guidelines. In general, the materials should be provided by an accredited supplier, and the material should meet the standards of ASTM C-33. Material that has been stockpiled and exposed to weather including wind and rain should be retested for compliance since fines could be lost. Frozen material cannot be used.
13. The placement of asphalt material should conform to the jurisdictional guidelines. In general, the materials should be provided by an accredited supplier, and the material should meet the standards of ASTM C-33. The material can be placed in a screed by end-dumping, or it can be placed directly on the paving surface. The temperature of the mix at placement should generally be on the order of 300 degrees Fahrenheit at time of placement and screeding.
14. Compaction of the screeded asphalt should begin as soon as practical after placement, and initial rolling should be performed before the asphalt has cooled significantly. Compaction equipment should have vibratory capabilities, and should be of appropriate size and weight given the thickness of the lift being placed and the sloping of the ground surface.
15. In cold and/or windy weather, the cooling of the screeded asphalt is a quality issue, so preparations should be made to perform screeding immediately after placement, and compaction immediately after screeding.
16. Quality control testing of the asphalt should be performed during placement to verify compaction and mix design properties are being met and that delivery temperatures are correct. Results of testing data from asphalt laboratory testing should be provided within 24 hours of the paving.

SITE GRADING AND DRAINAGE

1. In general, construction should proceed per the governing jurisdictional guidelines for the project site, including but not limited to the applicable American Concrete Institute (ACI), International Code Council (ICC), State Department of Transportation, State Department of Environmental Quality, the US Environmental Protection Agency, City and/or County, Army Corps of Engineers, Federal Aviation, Occupational Safety and Health Administration (OSHA), and any other governing standard details and specifications. In areas where multiple standards are applicable the more stringent should be considered. Work should be performed by qualified, licensed contractors with experience in the specific type of work in the area of the site.
2. Site grading and drainage for this section is generally meant to describe the effect of new construction on surface hydrology, which impacts the flow of rainfall or other water running across, onto or off-of, a newly constructed or modified development.
3. This section does not apply to the construction of site grading and drainage features. Recommendations for the construction of such features are covered in General Geotechnical Design and Construction Considerations for Earthwork – Structural Fills section and Underground Pipeline Installation – Backfill section.
4. In general, surface water flows should be directed towards storm drains, natural channels, retention or detention basins, swales, and/or other features specifically designed to capture, store, and or transmit them to specific off-site outfalls.
5. The surface water flow design is generally performed by a site civil engineer, and it can be impacted by hydrology, roof lines, and other site structures that do not allow for water to infiltrate into the soil, and that modify the topography of the site.
6. Soil permeability, density, and strength properties are relevant to the design of storm drain systems, including dry wells, retention basins, swales, and others. These properties are usually only provided in a geotechnical report if specifically requested, and recommendations will be provided in the geotechnical report in those cases.
7. Structures or site features that are not a part of the surface water drainage system should not be exposed to surface water flows, standing water or water infiltration. In general, roof drains and scuppers, exterior slabs, pavements, landscaping, etc. should be constructed to drain water away from structures and foundations. The purpose of this is to reduce the opportunity for water damage, erosion, and/or altering of structural soil properties by wetting. In general, a 5 percent or more slope away from foundations, structural fills, slopes, structures, etc. should be maintained.
8. Special considerations should be used for slopes and retaining walls, as described in the General Geotechnical Design and Construction Considerations - LATERALLY LOADED STRUCTURES section.
9. Additionally, landscaping features including irrigation emitters and plants that require large amounts of water should not be placed near to new structures, as they have the potential to alter soil moisture states. Changing of the moisture state of soil that provides structural support can lead to damage to the supported structures.

APPENDIX E

Percolation Test

Pecolation Test Data Sheet

Project:	EHS Chula Vista
Project No.:	17-199602.7
Date:	3/14/2019
Test Hole:	P1
Tested by:	MM
Depth of Hole, ft, D:	3.25
Boring Radius, in:	6
UCSD:	SP

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

Pre-Soak Procedure (See notes)						Calculations	
Reading #	Start Time	Stop Time	Δ t Time Interval	Do Initial Depth to Water Level	Df Final Depth to Water Level	Δ D Change in Water Level	Greater than 6"
	hr:mm	hr:mm	min	in	in	in	(y/n)
1	10:30	11:00	30	12	19	7.0	
2	11:10	11:40	30	19	28	9.0	

IN RIVERSIDE, 2Y=SAND: 10 min intervals for 1 hour. **IF NOT SAND:** 12 intervals at 30 min each, refilling each time

IN SAN DIEGO, Presoak for at least 2 hours if sandy soils. Rates of fall are measured for six hours, refilling each half hour (or 10 minutes for sand). Tests are generally repeated until consistent results are obtained.

Raw Data						Calculations		
Reading #	Start Time	Stop Time	Δ t Time Interval (10 or 30)	Do Initial Depth to Water Level	Df Final Depth to Water Level	Δ D Change in Water Level	Percolation Rate	Corrected Infiltration Rate
	hr:mm	hr:mm	min	inches (0.25" precision)			min/ in	in/hr
1	13:40	14:00	20	4.5	5.0	0.5	40.0	0.12
2	14:00	14:20	20	5.0	5.5	0.5	40.0	0.12
3	14:20	14:30	20	5.5	5.8	0.3	80.0	**0.06**
4								
5								
6								
7								
8								
9								
10								
11								
12								

Sources:

Appendix D, Approved Infiltration Rate Assessment Methods for Selection of Storm Water BMPs (San Diego)

Appendix A, Infiltration Testing (Riverside County)

Appendix D, Infiltration Rate Protocol, 2011 (Orange County)

Pecolation Test Data Sheet	
Project:	EHS Chula Vista
Project No.:	17-199602.7
Date:	3/14/2019
Test Hole:	P2
Tested by:	MM
Depth of Hole, ft, D:	3
Boring Radius, in:	6
UCSD:	SP

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

Pre-Soak Procedure (See notes)						Calculations	
Reading #	Start Time	Stop Time	Δ t Time Interval	Do Initial Depth to Water Level	Df Final Depth to Water Level	Δ D Change in Water Level	Greater than 6"
	hr:mm	hr:mm	min	in	in	in	(y/n)
1	10:40	11:10	30	12	24	12.0	
2	11:10	11:40	30	24	36	12.0	

IN RIVERSIDE, 2Y=SAND: 10 min intervals for 1 hour. **IF NOT SAND:** 12 intervals at 30 min each, refilling each time

IN SAN DIEGO, Presoak for at least 2 hours if sandy soils. Rates of fall are measured for six hours, refilling each half hour (or 10 minutes for sand). Tests are generally repeated until consistent results are obtained.

Raw Data						Calculations		
Reading #	Start Time	Stop Time	Δ t Time Interval (10 or 30)	Do Initial Depth to Water Level	Df Final Depth to Water Level	Δ D Change in Water Level	Percolation Rate	Corrected Infiltration Rate
	hr:mm	hr:mm	min	inches (0.25" precision)			min/ in	in/hr
1	13:40	14:00	20	0.0	5.3	5.3	3.8	1.30
2	14:00	14:20	20	5.3	8.0	2.8	7.3	0.76
3	14:20	14:30	10	0.0	2.3	2.3	4.4	1.07
4	14:13	14:23	20	2.3	5.0	2.8	7.3	0.70
5	14:23	14:33	10	5.0	6.3	1.3	8.0	**0.67**
6								
7								
8								
9								
10								
11								
12								

Sources:

Appendix D, Approved Infiltration Rate Assessment Methods for Selection of Storm Water BMPs (San Diego)

Appendix A, Infiltration Testing (Riverside County)

Appendix D, Infiltration Rate Protocol, 2011 (Orange County)