PETRA GEOSCIENCES INC.

SOLID AS A ROCK

2ND REVISION TO UPDATED GEOTECHNICAL EVALUATION
PROPOSED SUMMIT AVENUE DUPLEXES
10939 SUMMIT AVENUE, APN 378-190-01
SANTEE, SAN DIEGO COUNTY, CALIFORNIA

WARMINGTON RESIDENTIAL

December 17, 2024
J.N. 23-248

ENGINEERS + GEOLOGISTS + ENVIRONMENTAL SCIENTISTS

December 17, 2024
J.N. 23-248
Revision 2

WARMINGTON RESIDENTIAL
10125 Channel Road
Lakeside, California 92040

Attention: Mr. Matthew Esquivel

Subject: 2ND Revision to Updated Geotechnical Evaluation, Proposed Summit Avenue Duplexes, 10939 Summit Avenue, APN 378-190-01, Santee, San Diego County, California

Dear Mr. Esquivel:

Petra Geosciences, Inc. (Petra) is submitting herewith our second revision to the updated design-level geotechnical evaluation report for the three-story duplexes project located at 10939 Summit Avenue (APN 378-190-01) in the city of Santee, San Diego County, California. This work was performed in general accordance with the scope of work outlined in our Proposal No. 23-248P dated June 26, 2023. This report presents the results of our current field explorations, the requirements of the 2022 California Building Code (CBC) and our engineering judgment, opinions, conclusions, and recommendations pertaining to geotechnical design aspects for the proposed multi-family residential development. This revision provided our stability analyses for the proposed retaining walls within the easterly and southeasterly areas of the site.

It has been a pleasure to be of service to you on this project. Should you have questions regarding the contents of this report or should you require additional information, please contact this office.

Respectfully submitted,

PETRA GEOSCIENCES, INC.

Jim Larwood, CEG
Principal Geologist

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Senior Principal Engineer

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2ND REVISION TO UPDATED GEOTECHNICAL EVALUATION
PROPOSED SUMMIT AVENUE DUPLEXES
10939 SUMMIT AVENUE, APN 378-190-01
SANTEE, SAN DIEGO COUNTY, CALIFORNIA

INTRODUCTION

Petra Geosciences, Inc. (Petra) is presenting herein the results of our second revision to the updated design-level geotechnical evaluation of the subject 4.9±-acre property. This 2nd revision update report was prepared in response to revisions to the site plan and development concept as well as comments and questions by the City of Santee in the project 3rd review received by Petra on November 22, 2024. Our evaluation included a review of regional geological maps published by the California Geological Survey (CGS) and other sources that encompass the site, including review of limited online imagery (Google Earth Imagery, 1994-2023) of the project site.

PURPOSE AND SCOPE OF SERVICES

The purposes of this 2nd revision to the updated geotechnical evaluation were to present information on the subsurface geologic and soil conditions within the project area, evaluate the field and laboratory data, and provide conclusions and recommendations for design and construction of the proposed buildings and other site improvements as influenced by the subsurface conditions.

The scope of our evaluation consisted of the following:

- Reconnaissance of the site to evaluate existing conditions, mark-out borings for DigAlert notification, and contact DigAlert.
- Review of available published and unpublished data and maps concerning geologic and soil conditions within and adjacent to the site, which could have an impact on the proposed improvements.
- Drilling of a total of six exploratory borings up to 19.5 feet below ground surface (bgs), utilizing a truck-mounted hollow-stem auger drill rig with one boring (B-6) being excavated and drilled by a mini-excavator equipped with a 6-inch diameter flight auger, to evaluate the stratigraphy of the subsurface soils and collect representative undisturbed and bulk samples for laboratory testing.
- Two of the borings were drilled near the lower elevations at the site in the western area. No grading plans or preliminary water quality basin locations were made available to Petra at the time of one of the borings (Boring B-2) for testing. The boring was used to conduct a preliminary percolation test. One additional percolation boring (Boring B-6) was installed based on a request from the City of Santee in their 2nd review sheet. Based on the conceptual density study prepared by KTGY, 2024, the tests are located in the area of the proposed stormwater basin.
- Log and visually classify soil materials encountered in the borings in accordance with the Unified Soil Classification System.
- Conduct laboratory testing of representative samples (bulk and undisturbed) obtained from the exploratory borings to determine their engineering properties.

- Perform engineering and geologic analysis of the data with respect to the proposed improvements including slope stability analysis for design cut slopes and retaining walls.

- Preparation of this report, including pertinent figures and appendices, presenting the results of our evaluation and recommendations for the proposed improvements in general conformance with the requirements of the 2022 California Building Code (CBC), as well as in accordance with applicable local jurisdictional requirements.

SITE LOCATION

The rectangular-shaped subject site consists of 4.9± acres of land located east of Summit Avenue and north of Noble Way in the city of Santee. A site location map is included as Figure 1.

SITE DESCRIPTION

The subject rectangular-shaped property is comprised of level land. The site slopes gently to the west with existing elevations on the order of approximately 526± feet above mean sea level (msl) along the southwest portion of the site, to 597± feet above msl along the northeast portion of the site.

The subject property is currently the site of an occupied single-family residence with detached garages and shade structures. Grasses and weeds cover most of the subject site. Few trees are located near and around the residence. Vehicular access to the subject property is via dirt and partially paved driveways from Summit Avenue to the subject site. Multiple vehicles are located in proximity to the residences and garages. The subject site is secured by short fencing along the perimeter. An unlined shallow drainage channel is located in the northwest corner of the site. Woodglen Vista Creek is located approximately 500 feet to the west. Existing detached homes are present to the south with a vinyl fence separating the site.

Overhead communication and electrical lines are located along the west property boundary and extend overhead into the properties. The resident noted that the house is serviced by septic system leach field. The presumed septic system is most likely located on the southern side of the existing home. Information provided by the occupant indicated a water well services the property and is located along the north central area of the property. The occupant thought the well may be approximately 200 feet in depth. Depth to groundwater was not known.

Based on aerial photographic information obtained on Google Earth (1994 - 2024), the subject site appears mostly unchanged from current conditions, except variations in seasonal vegetation and vegetation clearing and dirt roads. An aerial photograph of the subject property from December 2022 is provided below in Figure A.

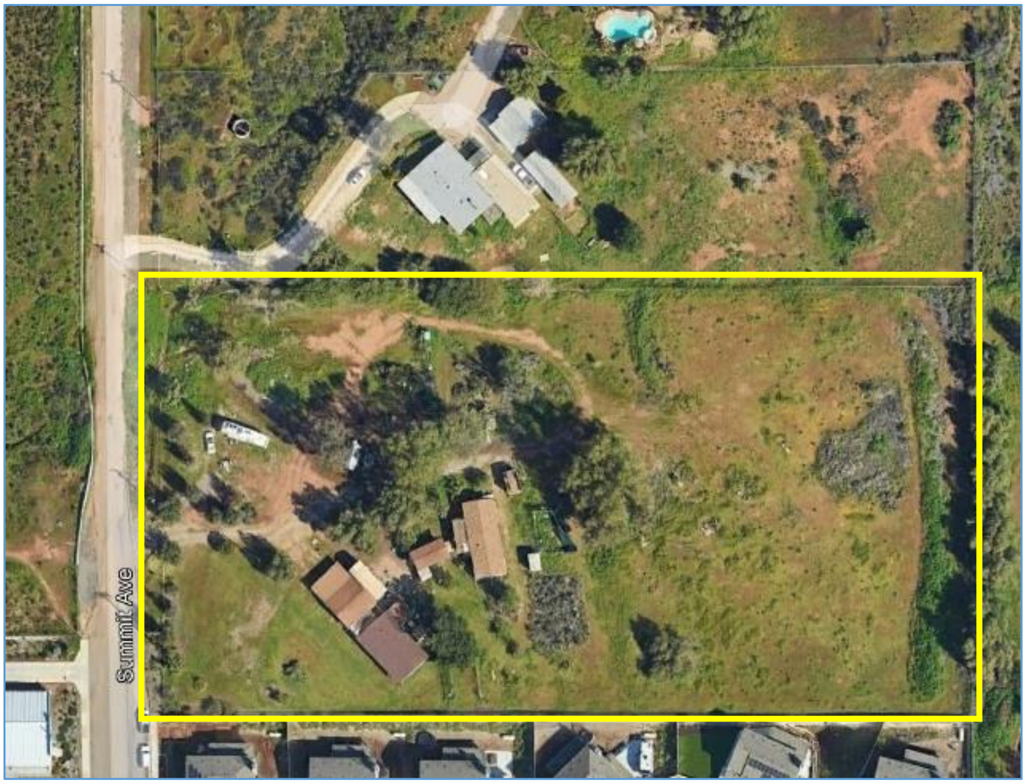

Figure A – Aerial photograph of the subject property (Google Earth, February 2022)

PROPOSED DEVELOPMENT

Based on a review of the referenced grading plan by Rick Engineering Company, received December 13, 2024, we understand that the proposed development will consist of 21 three-story, multi-unit duplex-style condominium buildings totaling 42 units. The complex will be accessed by a proposed site entry off Summit Avenue on the west. A straight interior drive connects the proposed duplexes with alleys. A 70-foot-wide corridor within the northern boundary is reserved for the future Magnolia Avenue and is not a part of this study. A 50-foot-wide fire setback adjacent to the Summit Avenue right-of-way is also designated along the western boundary.

It is expected that the buildings will be of typical wood-frame construction supported on conventional slab-on-ground foundations. Appurtenant structures will likely include paved drive isles and parking stalls, trash enclosures, masonry block screen walls, retaining walls, a tot lot, landscaped areas, and above- and below-ground utilities. Perimeter cut slopes up to 20± feet in height at 2:1 h:v are proposed along the eastern, southern, and northern boundaries. Fill slopes less than approximately 5 feet in height at 2:1 h:v are proposed along the northern, western, and southern boundaries. Retaining walls are generally proposed up to 7 feet along the southern boundary. A proposed retaining wall along the eastern and southeastern corner boundary is shown as 13 feet to 19 feet in height. An existing offsite residential home is located approximately 23 feet horizontally from the top of the proposed wall. It should be noted, however, that final grading plans were not available at the time of the study and the ultimate volume of cut and fill required throughout the project is not available.

Literature Review

Petra was not provided geotechnical reports for review pertaining to the subject property by the client. Petra researched and reviewed available published and unpublished geologic data pertaining to regional geology, groundwater, faulting, and geologic hazards that may affect the site. The results of this review are discussed under the Findings and Conclusions sections presented in this report.

Subsurface Exploration

A subsurface exploration program was performed by a geologist from Petra on July 20, 2023 (Petra, 2023a) and April 25, 2024. The exploration involved the drilling of six exploratory borings (B-1 through B-6) to a maximum depth of approximately 19.5 feet below existing grade (bgs). Earth materials encountered within the six exploratory borings were classified and logged by a geologist, under the supervision a professional geologist, in accordance with the visual-manual procedures of the Unified Soil Classification System. The approximate locations of the exploratory borings are shown on the Boring Location Map, Figure 2. The boring logs are presented in Appendix A.

Disturbed bulk samples and relatively undisturbed ring samples of soil materials were collected for classification, laboratory testing and engineering analyses. Undisturbed samples were obtained using a 3-inch outside diameter modified California split-spoon soil sampler lined with brass rings. The soil sampler was driven with successive 30-inch drops of a free-fall, 140-pound automatic trip hammer. The central portions of the driven-core samples were placed in sealed containers and transported to our laboratory for testing. The number of blows required to drive the split-spoon sampler 18 inches into the soil were recorded for each 6-inch driving increment; however, the number of blows required to drive the sampler for the final 12 inches was noted in the boring logs as *Blows per Foot*.

Standard Penetration Tests (SPT) were also performed at selected depth intervals in accordance with ASTM D 1586. This method consists of mechanically driving an unlined, 2.0-inch outside diameter (OD) standard penetrometer sampler 18 inches into the soil with successive 30-inch drops of the 140-pound automatic trip hammer. Blow counts for each 6-inch driving increment were recorded on the exploration logs. The number of blows required to drive the sampler for the last 12 of the 18 inches was identified as the uncorrected standard penetration resistance (N). Disturbed soil samples from the unlined sampler were placed in sealed plastic bags and transported to our laboratory for testing.

Boring B-6 was drilled with a mini-excavator equipped with a 6-inch diameter flight auger. Disturbed samples were obtained used to describe the soil and bedrock conditions. Samples were discarded upon completion of the boring.

Two borings (B-2 and B-6) were drilled near the lower elevations at the site in the western area in the proposed basin site. The borings were used to conduct percolation tests. Perforated pipe and gravel were placed in the boring, followed by pre-soaking the borings with water. Following presoak, a falling-head percolation test was conducted. Upon completion of logging and/or testing, all boreholes were subsequently backfilled with borehole cuttings.

Laboratory Testing

In-situ dry density and moisture content, maximum dry density and optimum moisture content, expansion index, direct shear, and corrosivity (sulfate and chloride content, pH, and resistivity), for selected samples of onsite soils materials was conducted. A description of laboratory test methods and summaries of the laboratory test data are presented in Appendix B. The in-site dry density and moisture content results are presented on the boring logs (Appendix A).

FINDINGS

Regional Geologic Setting

The site lies within the northern portion of the Peninsular Ranges Geomorphic Province (CGS, 2002). The Peninsular Range Province extends from the tip of Baja California north to the Transverse Ranges Geomorphic Province and is characterized by northwest trending mountain ranges separated by subparallel fault zones. The San Bernardino Mountains, located on the north side of the valley, provides the boundary between the Peninsula Range Province and the Transverse Ranges Province. In general, the province is underlain primarily of plutonic rock of the Southern California Batholith. These rocks formed from the cooling of molten magma deep within the earth's crust. Intense heat associated with the plutonic magma metamorphosed the ancient sedimentary rocks into which the plutons intruded. The Peninsular Range Geomorphic Province is generally characterized by alluviated basins and elevated erosion surfaces.

Most of the subject site is mapped on regional geologic maps as being underlain by late-Pleistocene Alluvial Deposits (Tan. 2002a and 2002b). These soils are described as moderately consolidated, poorly sorted flood plain deposits consisting of gravelly sandy silt and clay. The eastern portion of the subject site is mapped as Cretaceous-age Granodiorite described as tonalite and monzogranite which is medium to coarse grained. A portion of the geologic map is provided below in Figure B.

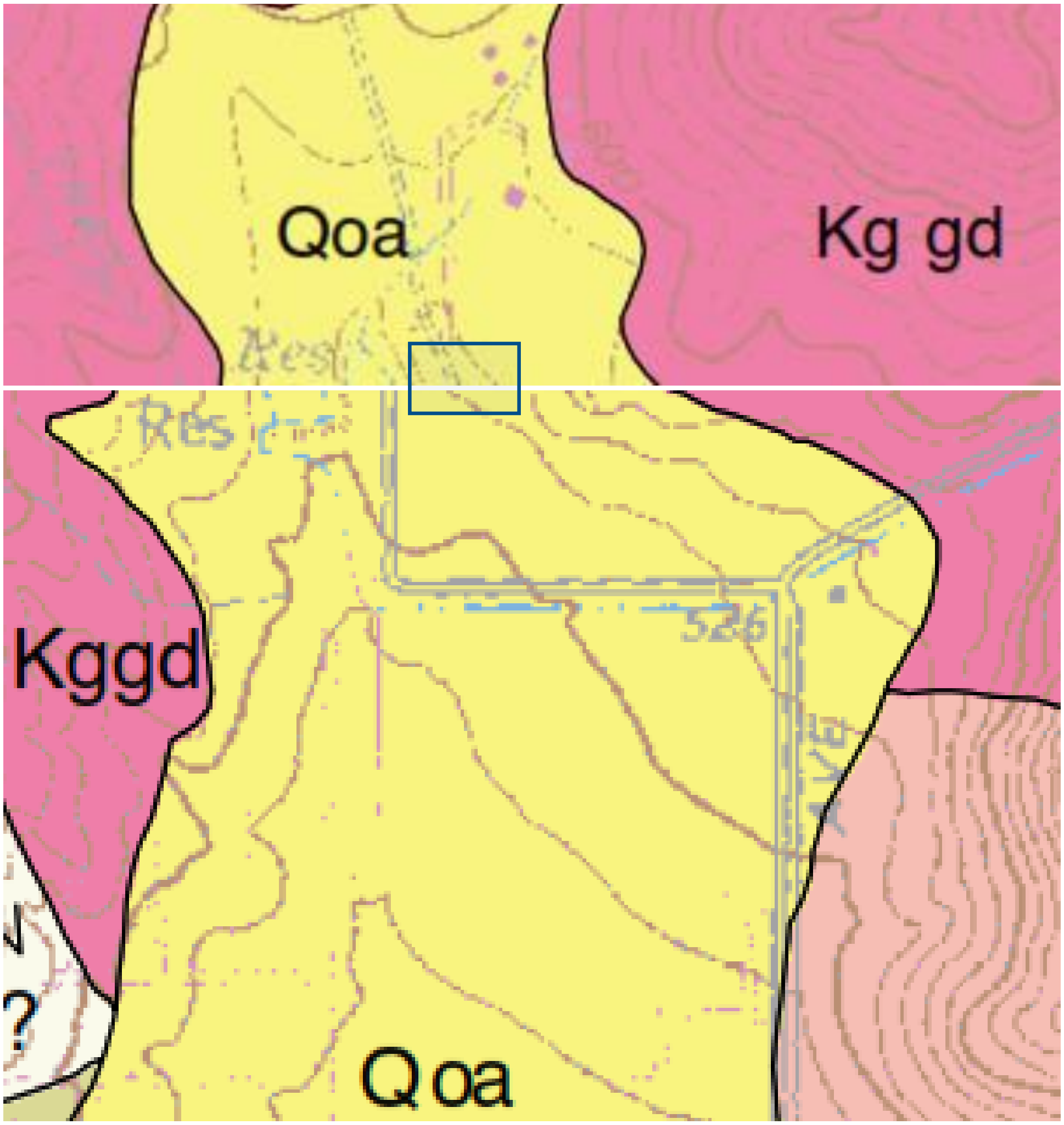

Figure B – Geologic Maps (Tan, 2002a and 2002b)

Qls	Landslide deposits (Holocene to Pleistocene). Landslide slump and rock fall deposits. On map, the deposit is depicted by landslide arrows (see "MAP SYMBOLS"). Queried where questionable.
Qoa	Late Pleistocene alluvial deposits; moderately consolidated, poorly-sorted flood plain deposits consisting of gravelly, sandy silt and clay.
Kgt	Tonalite (Cretaceous); includes some granodiorite and quartz diorite; medium-grained; generally dark colored and severely weathered.
Kggd	Granodiorite (Cretaceous); includes some tonalite and monzogranite; medium-to coarse-grained.

The site does not lie within an Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007; CGS, 2022) or a landslide hazard zone (City of Santee General Plan, Safety Element, 2020).

Local Geology and Subsurface Soil Conditions

Earth units encountered onsite consisted of minor amounts of artificial fill underlain by older alluvial flood plain deposits to a depth of approximately 3.5 to 8 feet below the ground surface. Granitic bedrock was found underlying the alluvial fan deposits.

Older alluvial deposits were observed to consist predominately of dry to moist, loose to dense, silty fine- to coarse-grain sand and clayey sand. Generally, the upper one to five feet of soil encountered within the subject property were loose. Logs of exploratory borings are presented in Appendix A and boring locations are presented on the Boring Location Map (Figure 2).

Groundwater

The site is located within the San Diego River Valley Groundwater Sub-Basin (9-015) (California Department of Water Resources, 2023a). Based California Department of Water Resources (DWR) Water Data Library interactive Station Map, no municipal wells are mapped on or in proximity to the subject property (California Department of Water Resources, 2023b). However, information provided by the occupant of 10939 Summit Avenue indicated a water well services the property and is located along the north central area of the property. The occupant thought the well may be approximately 200 feet in depth. Depth to groundwater was not known. Groundwater was not encountered in the borings to an explored depth of 19.5 feet bgs. Regional groundwater is not anticipated to affect the proposed development.

Surface Water

No surface water was observed onsite during our recent field exploration (Petra, 2023a). Based on Flood Insurance Map (FIRM), the site is located within an area of minimal flood zone hazard (FEMA, 2023). A portion of the Flood Insurance Rate Map (FIRM) for the subject property and vicinity is provided below in Figure C.

Figure C – FIRM Map, Map Number 06073C1400G (May 16, 2012)

Petra reviewed the California Division of Dam Safety (CDODS) Dam Breach Inundation Map Web Publisher (CDODS, 2023). There is one dam, San Vicente Dam, identified by CDODS near the site. However, the San Vicente Dam is not up gradient from the subject property. The site is not in a downstream breach hazard.

Faulting

Based on our review of the referenced geologic maps and literature, no active faults are known to project through the property. Furthermore, the site does not lie within the boundaries of an "Earthquake Fault Zone" as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act.

The California Geological Survey's (CGS) Special Publication 42 (revised 2018) defines a Holocene-active fault that has had displacement within the Holocene epoch or last 11,700 years. A pre-Holocene fault is defined as a fault that does not display evidence of movement within the last 11,700 years, but has moved within the Quaternary period, the last 2.6 million years. Pre-Holocene faults are not placed within Alquist-Priolo Earthquake Fault Zones, but are considered when placing such critical structures as dams and nuclear power plants, etc.

By definition, age-undetermined faults are "where the recency of fault movement has not been determined. Faults can be 'age-undetermined' if the fault in question has simply not been studied in order to determine its recency of movement. Faults can also be age-undetermined due to limitations in the ability to constrain the timing of the recency of faulting. Examples of such faults are instances where datable materials are not present in the geologic record, or where evidence of recency of movement does not exist due to stripping (either by natural or anthropogenic processes) of Holocene-age deposits. Within the framework of the A-P Act, age-undetermined faults within regulatory Earthquake Fault Zones are considered Holocene-active until proved otherwise (CGS, 2018). Age-undetermined faults are located in the western portion of the subject property, where surficial soils have been disturbed by previous agricultural activities.

The main objective of the AP Act is to prevent the construction of dwellings on top of active faults that could displace the ground surface resulting in loss of life and property.

However, it should be noted that according to the USGS Unified Hazard Tool website and/or 2023 CGS California Earthquake Hazard Zone Application (EQZapp), the Rose Canyon Fault zone, located approximately 23.3 kilometers (14.5± miles) southwest of the site, would probably generate the most severe site ground motions and, therefore, is the majority contributor to the deterministic minimum component of the ground motion models. This fault is reported to be capable of generating a magnitude 6.96 event.

Seismic Design Parameters

Earthquake loads on earthen structures and buildings are a function of ground acceleration which may be determined from the site-specific ground motion analysis. Alternatively, a design response spectrum can be developed for certain sites based on the code guidelines. To provide the design team with the parameters necessary to construct the design acceleration response spectrum for this project, we used two computer applications. Specifically, the first computer application, which was jointly developed by the Structural Engineering Association of California (SEAOC) and California's Office of Statewide Health Planning and Development (OSHPD), the SEA/OSHPD Seismic Design Maps Tool website, https://seismicmaps.org, is used to calculate the ground motion parameters. The second computer application, the United Stated Geological Survey (USGS) Unified Hazard Tool website, https://earthquake.usgs.gov/hazards/interactive/, is used to estimate the earthquake magnitude and the distance to surface projection of the fault.

To run the above computer applications, site latitude and longitude, seismic risk category and knowledge of site class are required. The site class definition depends on the direct measurement and the ASCE 7-16 recommended procedure for calculating average small-strain shear wave velocity, V_{s30}, within the upper 30 meters (approximately 100 feet) of site soils.

A seismic risk category of II was assigned to the proposed building(s) in accordance with 2022 CBC, Table 1604.5. Based on our engineering geology judgement, the bedrock at the site appears to exhibit the characteristics of a Site Class B, i.e., competent rock with moderate fracturing and weathering; however, no direct, small-strain shear wave measurement of shear wave velocity was performed. Therefore, an average shear wave velocity in the range of 2,500 to 5,000 feet per second for the upper 100 feet was considered for the site based on engineering judgment and geophysical experience. As such, in accordance with ASCE 7-16, Table 20.3-1, Site Class B (B - Estimated as per SEA/OSHPD software) has been assigned to the subject site.

The following table, Table 1, provides parameters required to construct the design acceleration response spectrum based on the 2022 CBC guidelines. Please note that for Site Class B - Estimated, Site Coefficients, F_a, F_v, and F_{PGA} should be taken as unity (1.0), as reflected in Table 1. A printout of the computer output is attached in Appendix C.

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TABLE 1

Seismic Design Parameters

Ground Motion Parameters	Specific Reference	Parameter Value	Unit
Site Latitude (North)	-	32.8760	°
Site Longitude (West)	-	-116.9747	°
Site Class Definition	Section 1613.2.2 (1), Chapter 20 (2)	B-est (4)	-
Assumed Seismic Risk Category	Table 1604.5 (1)	II	-
M_w - Earthquake Magnitude	USGS Unified Hazard Tool (3)	6.96 (3)	-
R – Distance to Surface Projection of Fault	USGS Unified Hazard Tool (3)	23.3 (3)	km
S_s - Mapped Spectral Response Acceleration Short Period (0.2 second)	Figure 1613.2.1(1) (1)	0.776 (4)	g
S_1 - Mapped Spectral Response Acceleration Long Period (1.0 second)	Figure 1613.2.1(3) (1)	0.286 (4)	g
F_a – Short Period (0.2 second) Site Coefficient	Table 1613.2.3(1) (1)	1 (4)	-
F_v – Long Period (1.0 second) Site Coefficient	Table 1613.2.3(2) (1)	1 (4)	-
S_{MS} – MCE_R Spectral Response Acceleration Parameter Adjusted for Site Class Effect (0.2 second)	Equation 16-20 (1)	0.776 (4)	g
S_{M1} - MCE_R Spectral Response Acceleration Parameter Adjusted for Site Class Effect (1.0 second)	Equation 16-21 (1)	0.286 (4)	g
S_{DS} - Design Spectral Response Acceleration at 0.2-s	Equation 16-22 (1)	0.517 (4)	g
S_{D1} - Design Spectral Response Acceleration at 1-s	Equation 16-23 (1)	0.19 (4)	g
$T_o = 0.2\ S_{D1}/\ S_{DS}$	Section 11.4.6 (2)	0.073	s
$T_s = S_{D1}/\ S_{DS}$	Section 11.4.6 (2)	0.367	s
T_L - Long Period Transition Period	Figure 22-14 (2)	8 (4)	s
PGA - Peak Ground Acceleration Maximum Considered Earthquake Geometric Mean, MCE_G (*)	Figure 22-9 (2)	0.332	g
F_{PGA} - Site Coefficient Adjusted for Site Class Effect (2)	Table 11.8-1 (2)	1 (4)	-
PGA_M –Peak Ground Acceleration (2) Adjusted for Site Class Effect	Equation 11.8-1 (2)	0.332 (4)	g
Design PGA ≈ (⅔ PGA_M) - Slope Stability (†)	Similar to Eqs. 16-22 & 16-23 (2)	0.221	g
Design PGA ≈ (0.4 S_{DS}) – Short Retaining Walls (‡)	Equation 11.4-5 (2)	0.206	g
C_{RS} - Short Period Risk Coefficient	Figure 22-18A (2)	0.927 (4)	-
C_{R1} - Long Period Risk Coefficient	Figure 22-19A (2)	0.928 (4)	-
SDC - Seismic Design Category (§)	Section 1613.2.5 (1)	D (4)	-

References:
(1) California Building Code (CBC), 2022, California Code of Regulations, Title 24, Part 2, Volume I and II.
(2) American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI), 2016, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, Standards 7-16.
(3) USGS Unified Hazard Tool - https://earthquake.usgs.gov/hazards/interactive/ [Dynamic: Conterminous U.S. 2014 (update) (v4.2.0)]
(4) SEI/OSHPD Seismic Design Map Application – https://seismicmaps.org [Reference: ASCE 7-16]
Related References:
Federal Emergency Management Agency (FEMA), 2015, NEHRP (National Earthquake Hazards Reduction Program) Recommended Seismic Provision for New Building and Other Structures (FEMA P-1050).

Notes:
* PGA Calculated at the MCE return period of 2475 years (2 percent chance of exceedance in 50 years).
† PGA Calculated at the Design Level of ⅔ of MCE; approximately equivalent to a return period of 475 years (10 percent chance of exceedance in 50 years).
‡ PGA Calculated for short, stubby retaining walls with an infinitesimal (zero) fundamental period.
§ The designation provided herein may be superseded by the structural engineer in accordance with Section 1613.2.5.1, if applicable.

CONCLUSIONS

Site Suitability

From a geotechnical engineering and engineering geologic point of view, the subject property is considered suitable for the proposed development provided the following conclusions and recommendations are incorporated into the design criteria and project specifications.

Primary Geologic/Geotechnical Considerations

Groundwater

No groundwater or perched groundwater was encountered in the six exploratory borings at a depth of 19.5 feet bgs. Regional groundwater is not anticipated to affect the subject development. The onsite groundwater well should be abandoned appropriately per regulatory agency requirements.

Fault Rupture

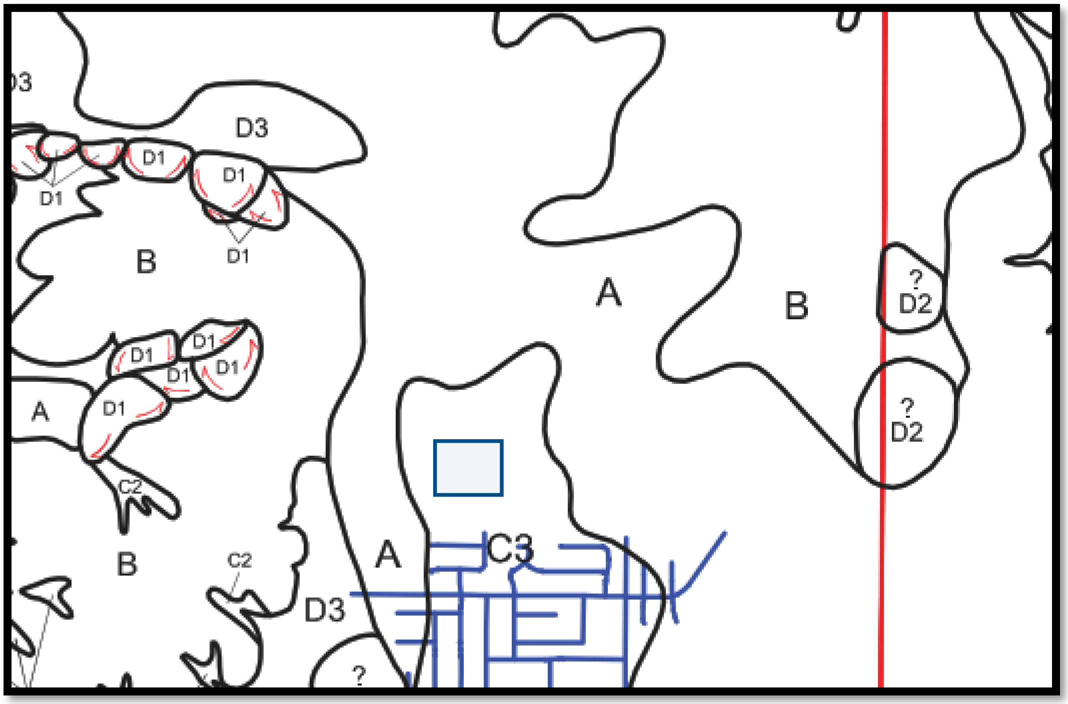
The site is not located within a currently designated State of California Alquist-Priolo Earthquake Fault Zone (CGS, 2023). In addition, no known active faults have been identified on the site. While fault rupture would most likely occur along previously established fault traces, fault rupture could occur at other locations. However, the potential for active fault rupture at the site is considered to be very low.

Strong Ground Motions

The site is located in a seismically active area of southern California and will likely be subjected to very strong seismically related ground shaking during the anticipated life span of the project. Structures within the site should therefore be designed and constructed to resist the effects of strong ground motion in accordance with the 2022 CBC and the seismic parameters included in Table 1, above.

Liquefaction, Landslides and Secondary Seismic Effects

The proposed residential development is mapped mostly within zones of “nominal” liquefaction potential, Zone A (Figure D) based on the property being underlain by granitic bedrock. The western area of the site is shown as being in “low to moderate” liquefaction potential, Zone C3 (Figure D) due to the presence of older alluvium. However, the site should be considered Nominal Liquefaction potential due to lack of groundwater and granitic bedrock within 5 to 8 feet of the ground surface based on boring logs (Appendix A).



Legend	Soil Type	Location	Relative Landslide Susceptibiliy	Liquefaction Hazard	Expansion Condition
A	Granitic Rock	Hard Rock Outcrops and Decomposed Granitics, Northern Slopes (Fanita Ranch), Central Area (Ramsgate Way), Southwestern Area (Rancho Fanita Drive, Cowles Mountain)	Least Susceptible	Nominal	Very Low
B	Stadium Conglomerate	Northwestern and Northern Slopes (Fanita Ranch), Southern Undeveloped Area	Marginally Susceptible (Generally Susceptible Debris To Flow)	Nominal	Low
C1	Alluvium	Main Drainage Channels, Possible Shallow Groundwater, San Diego River	Marginally Susceptible	Moderate to High	Variable
C2	Alluvium/Debris Flow	Secondary Drainage and Tributary Channels, Fluctuating Groundwater	Variable	Nominal to Low	Moderate
C3	Terrace Deposits/ Older Alluvium	Gentle Slopes Western Area, Flanks of the San Diego River Valley (Carlton Oaks Drive), Central Area (Woodpark Drive)	Generally To Marginally susceptible (Where Underlain by Friars Formation)	Low to Moderate	Variable

Figure D – City of Santee Geotechnical/Seismic Hazard Map, excerpt from Figure 8-3 of Santee General Plan, 2020

The site and immediate area exhibit level topography with no existing slopes within or immediately adjacent to the subject property. The closest slope is situated approximately 200 feet to the northeast which is a natural slope. Elevation data from Google Earth suggests the natural slope, ascending from the eastern property boundary is about 200 feet in height over about 800 linear feet. No landslides or rockfalls are mapped within or in proximity to the subject site. A landslide is mapped about 2,500 feet southwest of the site on the east-facing slopes of the foothills. The location is shown on the above Figure B.

Secondary effects of seismic activity normally considered as possible hazards to a site include several types of ground failure. Such ground failures, which might occur as a consequence of severe ground shaking at the site, include ground subsidence, ground lurching and lateral spreading. The probability of occurrence of each type of ground failure depends on the severity of the earthquake, distance from faults, topography, subsoils, and groundwater conditions, in addition to other factors. Based on site conditions underlying the subject site, proposed grading, the lack of shallow groundwater, and gentle topography across most of the site, landsliding, liquefaction, ground subsidence, ground lurching and lateral spreading are considered unlikely at the site (Petra, 2023b). The potential for seismic flooding due to a tsunami or seiche is considered negligible.

Compressible Soils

One of the most significant geotechnical factors affecting the project site is the presence of near-surface compressible soil materials within the subject property consisting of undocumented fill and the upper portion of weathered older alluvium. Such materials consist of loose undocumented fill, which is not considered suitable for support of fill or structural loads in its present condition. Based on our subsurface assessment and laboratory test results, remedial removal depths of existing fill soils and highly weathered alluvium underlying the proposed structures expected to be on the order of 2 to 5 feet below existing grades. Accordingly, these materials will require removal to competent existing fill or alluvial soils and replacement with properly compacted fill.

Flooding

No surface water was observed onsite during our recent field exploration. Based on Flood Insurance Map (FIRM), the site is located within an area of minimal flood zone hazard (FEMA, 2023). A portion of the Flood Insurance Rate Map (FIRM) for the subject property and vicinity is provided in Figure C.

Stability of Temporary Excavations and Backcuts

Due to restrictions imposed by the adjacent property, the temporary excavations for the proposed retaining wall along the eastern and southeastern boundary will likely be a near-vertical cut of up to approximately 20 feet in height, or at gentler gradient in excess of 20 feet. This cut is primarily expected to expose moderately hard to hard granitic bedrock with the uppermost portion of the excavation consisting of about 2 to 4 feet of topsoil and older alluvium. This condition is depicted on cross-sections A-A’, B-B’, and C-C’, and is incorporated onto Figure 3.

Temporary unsupported sidewalls constructed at the recommended maximum slope ratio are expected to remain stable during the remedial grading, however, all temporary slopes should be observed by a representative of the project geotechnical consultant for any evidence of potential instability. Depending on the results of these observations, revised temporary slope configurations may be necessary. Recommendations for remedial grading are provided in the “Earthwork” section of this report.

SLOPE STABILITY OF PERMANENT CUTS

As part of grading for the proposed development, Petra understands that cut sections in excess of 20 feet are proposed within the eastern half of the site. The cut sections generally include a vertical cut supported by retaining wall up to 13 feet high below a 1.5H:1V (horizontal to vertical) slope which is up to 7 feet high. In addition, the proposed cut sections also include a 14-foot-high slope inclined at 1.65H:1V with no retaining wall. Per the City of Santee, Code of Ordinances, Title 11, Chapter 11.40, Article 3 – Design Standards (accessed online at https://ecode360.com/43753753#43753775), requests for approval of cut slopes steeper than 2H:1V must be accompanied by a geotechnical report that establishes such slopes will be stable.

To evaluate the stability of the proposed cut sections, slope stability analyses were performed along three cross-sections, A-A' through C-C'. The locations of the cross-sections are shown in Figure 2. The cross-sections are shown on Figure 3. This section discusses the details of the stability analyses including the subsurface model, analysis methods, stability criteria, results of the analyses, and a discussion of the results.

Analysis Methods

Slope stability was evaluated by performing two-dimensional limit equilibrium analyses and calculating a factor of safety (FS) against sliding for both static and seismic (pseudo-static) conditions. For the seismic condition, a horizontal seismic coefficient, K_h equal to ⅔ of peak ground acceleration adjusted for site class effect, 0.221g, (obtained from Table 1 – Seismic Design Parameters, of this report) was conservatively applied as an additional driving force. The computer program Slide2 (Rocscience) was used to perform the Morgenstern and Price limit equilibrium analysis method (Morgenstern and Price, 1965) and the Spencer Method of Slices to calculate the FS which is defined as the ratio of resisting forces to driving forces.

Two sets of analyses were performed for sections A-A' and B-B' – one considering failure above the retaining wall that includes the proposed 1.5H:1V cut slope and a second analysis considering the global stability of the whole section where the failure surface exits at the base of the retaining wall. For section C-C' (no retaining wall), only one analysis was performed.

Stability Criteria

The minimum stability factors of safety required under static and pseudo-static loading conditions are 1.5 and 1.1, respectively, following accepted geotechnical practices and agency guidelines.

Subsurface Model

The proposed cut sections will encompass the older fan deposits (Qof) as well as granitic bedrock (Kgr). Based on the boring logs, the upper 4 to 8 feet consists of Qof underlain by weathered Kgr that varies in thickness from 2 to 5 feet underlain by relatively unweathered bedrock to the termination depth of the borings. These geologic units were modeled using Mohr-Coulomb strength criteria. The strength parameters for Qof were obtained from a direct shear test performed on a Qof sample obtained from boring B-5. The strength parameters for the bedrock unit were based on engineering judgment and experience. Soil strength parameters used in the analysis are summarized in the Table 2, below.

TABLE 2

Idealized Soil Parameters

Soil Type	Unit Weight (pcf)	Strength Type	Strength Parameters			
			Static		Seismic	
			Φ (degree)	Cohesion (psf)	Φ (degree)	Cohesion (psf)
Qof	125	Mohr-Coulomb	28.9	270	26.2	522
Weathered Kgr	135	Mohr-Coulomb	34	500	34	500
Kgr	135	Mohr-Coulomb	38	500	38	500

Analysis Results

The calculated static and pseudo-static slope stability factors of safety for the cross sections analyzed are presented in Table 3, below. The outputs from Slide2 are presented in Appendix D.

TABLE 3

Slope Stability Analysis Results

Cross Section	Factor of Safety	
	Static	Seismic (K_h = 0.221)
A-A' : Failure above Wall	4.04	3.61
A-A' : Global Failure	1.59	1.38
B-B' : Failure above Wall	3.49	2.89
B-B': Global Failure	1.72	1.40
C-C'	2.73	1.94

Discussion

Based on the results of slope stability evaluation, all three cross-sections meet or exceed the minimum factor of safety requirements established in the Stability Criteria section. Therefore, the proposed cut section which includes 1.5H:1V slopes is anticipated to be stable from geotechnical standpoint.

EARTHWORK RECOMMENDATIONS

Earthwork Criteria

Earthwork should be performed in accordance with the Grading Code of the City of Santee and/or County of San Diego, in addition to the applicable provisions of the 2022 CBC. Grading should also be performed in accordance with the following site-specific recommendations prepared by Petra based on the proposed construction.

Geotechnical Observations and Testing

Prior to the start of earthwork, a meeting should be held at the site with the owner, contractor, and geotechnical consultant to discuss the work schedule and geotechnical aspects of the grading. Earthwork, which in this instance will generally entail removal and re-compaction of the near surface soils, should be accomplished under full-time observation and testing of the geotechnical consultant. A representative of the project geotechnical consultant should be present onsite during all earthwork operations to document proper placement and compaction of fills, as well as to document compliance with the other recommendations presented herein.

Clearing and Grubbing

All existing weeds, grasses, brush, shrubs, trees/tree stumps, root balls, and similar vegetation existing within areas to be graded should be stripped and removed from the site. Clearing operations should also include the demolition and removal of existing improvements such as septic systems and any remaining trash, debris, vegetation, and similar deleterious materials. The existing water well should be abandoned under regulatory permit by a C-57 licensed water well drilling contractor. Any cavities or excavations created upon removal of buried structures or root balls or any unknown subsurface structures should be cleared of loose soil, shaped to provide access for backfilling and compaction equipment and then backfilled with properly compacted (engineered) fill. Note that deleterious materials may be encountered within the site and may need to be removed by hand, i.e., root pickers, during the grading operations.

The project geotechnical consultant should provide periodic observation and testing services during clearing and grubbing operations to document compliance with the above recommendations. In addition, should unusual or adverse soil conditions or buried structures be encountered during grading that are not described herein, these conditions should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations.

Existing Septic Systems

Based on our investigation, it is likely that there are septic tanks and leach fields in the back yards of the existing residences. Removal of the entire septic system is required and should consist of complete removal of the leach lines, plumbing and septic tank. The excavated areas should be backfilled with compacted fill, placed under full-time geotechnical observation and testing. It is recommended that the septic removal and disposal be conducted in accordance with current local, state, and federal disposal regulations.

Excavation Characteristics

The existing site soils are expected to be readily excavated with conventional earthmoving equipment. If oversize rocks (i.e., 12-inches in one dimension or greater) are encountered, they should either be disposed of either offsite or properly buried within the planned deeper fills, if available, in an approved engineered fashion, a minimum of 10 feet below finish pad grade(s) and 15 feet from slope faces.

Based on the results of our exploratory borings, surficial native soil deposits, including topsoil, old alluvial fan deposits and highly weathered bedrock, are expected to be readily excavatable with conventional heavy earthmoving equipment. Our evaluation of bedrock rippability is discussed below.

- The property is underlain by soft to very hard, granitic bedrock which locally forms outcrops. Exploratory borings were drilled with a hollow-stem auger truck-mounted drill rig to 19.5 feet bgs within areas of the site. Drilling progress was difficult but steady.

- Based on the data obtained in the areas where the borings were drilled, the granitic rock encountered is anticipated to be rippable utilizing a D-9 dozer or equivalent, with a single shank in the center slot.

Remedial Grading - General

To create a uniform compacted fill mat below the proposed buildings and reduce the potential for distress due to excessive differential settlement, it is recommended that all near surface low-density native materials and/or insufficiently compacted existing fill soils be removed to underlying competent existing fill or alluvial materials and replaced as properly compacted fill materials. It must be noted that the depths of remedial grading provided herein are estimates only and are based on conditions observed at the boring locations. Subsurface conditions can and usually do vary between points of exploration. For this reason, the actual removal depths will have to be determined on the basis of in-grading observations and testing performed by a representative of the project geotechnical consultant. The Client, civil engineer, and project grading contractor should allow contingencies for additional earthwork quantities should adverse conditions and deeper removals be required.

Ground Preparation – Building Pads

Based on our subsurface exploration and laboratory test results, remedial removal depths on the order of 2 to 5 feet below existing grades are expected in the proposed building areas. The horizontal limits of removal and re-compaction should extend to a minimum distance of 5 feet beyond the proposed building footprints. Unsuitable soil removals may also need to be locally deeper, depending on the exposed conditions encountered during grading. The minimum depth of compacted (engineered) fill within the finished building pad should be 4 feet, The actual depths and horizontal limits of removals and over-excavations should be evaluated during grading on the basis of observations and testing performed by the project geotechnical consultant.

It should be noted that the northwestern most proposed building pad, in the vicinity of Boring B-1, is located in close proximity to the existing offsite drainage swale. Remedial excavation for the building pad should extend into competent material at a 1:1 projection downward from the proposed building edge and not encroach into the swale influence zone. Deepened footings in this area may be necessary or other stabilization methods.

The suitability of the existing fill should be evaluated during the course of remedial removals by testing the fill for satisfactory in-situ compaction, as well as verification by observation of the consistency of the fill. Deeper removal depths may be warranted should the condition of the existing fill prove to be unsatisfactory.

Prior to placing engineered fill, <u>all</u> exposed bottom surfaces in the removal areas should be approved by a representative of the project geotechnical consultant and then scarified to a minimum depth of 12 inches, moisture-conditioned to attain approximately 2 percent above optimum moisture, and compacted in-place to no less than 90 percent relative compaction with reference to per ASTM D 1557. All fills should be placed in 6- to 8-inch-thick maximum lifts, watered or air dried as necessary to achieve slightly above-optimum moisture conditions, and then compacted to a minimum relative compaction of 90 percent.

Ground Preparation – Drive Isles and Parking Areas

For proposed drive isles and parking areas, the existing ground surfaces should be over-excavated to a minimum depth of 2 feet below the existing ground surface or 2 feet below the proposed subgrade elevations or the deepest proposed utility if in granitic bedrock, whichever is deeper. After completion of over-excavation, the areas should be scarified to a minimum depth of 6 inches, moisture-conditioned, and re-compacted to a minimum 90 percent relative compaction during rough grading activities. The excavated materials may be replaced as properly compacted fill. The horizontal limits of over-excavation should extend to a minimum horizontal distance of 12 inches beyond the perimeter of the proposed improvements. All fills should be placed in 6- to 8-inch-thick maximum lifts, watered or air dried as necessary to achieve slightly above-optimum moisture conditions, and then compacted to a minimum relative compaction of 90 percent with reference to per ASTM D 1557. Prior to paving, the pavement subgrade soils will require rework to a depth of 12 inches to achieve no less than 95 percent relative compaction prior to placement of aggregate base.

Ground Preparation – Retaining Wall Foundation Zone

Cantilever retaining walls must yield outward slightly in order to develop ‘active’ soil pressures. This is accomplished with a relatively small settlement of the wall footing. Where the footing is founded in hard rock, such yielding will not occur, resulting in a higher lateral earth pressure, defined as the ‘at-rest’ pressure. The at-rest pressure is typically utilized for design where retaining walls are restrained from yielding, such as basement walls.

In order to develop active soil pressure, the retaining wall footing subgrade should consist of compacted fill, rather than cut hard bedrock. Accordingly, retaining wall footings should be underlain by no less than 2 feet of compacted fill. This applies to wall footings that would otherwise be founded directly on hard bedrock. The need for such over-excavation should be evaluated in the field during the course of grading by an engineering geologist, as weathered bedrock may not warrant over-excavation. It should be noted that such remedial recommendations conditions do not, necessarily, apply to other retaining wall systems, such as gravity walls, segmental walls and tie-back walls.

Suitability of Site Soils as Fill

Site soils are suitable for use in engineered fills provided they are clean from organics and/or debris. Wet older alluvial soils may also be encountered during site grading (depending upon the time of year grading occurs) and may require drying back before being reused as fill.

Fill Placement

Fill materials should be placed in approximately 6- to 8-inch-thick loose lifts, watered or air-dried as necessary to achieve a moisture content approximately 2 percent above optimum moisture condition, and then compacted in-place to no less than 90 percent relative compaction. The laboratory maximum dry density and optimum moisture content for each major soil type should be determined in accordance with ASTM D 1557.

Import Soils for Grading

If import soils are needed to achieve final design grades, import soils should be free of deleterious materials, oversize rock, and any hazardous materials. The soils should also be non-expansive and essentially non-corrosive and approved by the project geotechnical consultant *prior* to being brought onsite. The geotechnical consultant should inspect the potential borrow site and conduct testing of the soil at least three days before the commencement of import operations.

Shrinkage and Subsidence

Volumetric changes in earth quantities will occur when excavated onsite soils are replaced as properly compacted fill. Following is an estimate of shrinkage factors for the alluvial soil present onsite. These estimates are based on in-place densities of the various materials and on the estimated average degree of relative compaction achieved during grading.

- Disturbed Surface Soils (0 - 8± feet).…………………………………Shrinkage of 5 to 10%±
- Older Alluvium (below 2 - 5± feet)……………………………………. Shrinkage of 0 to 5%±
- Weathered Granitic Bedrock and Granitic Bedrock (below 5± feet) ……..... Bulking 0 to 5%±

Subsidence from scarification and re-compaction of exposed bottom surfaces in removal areas to receive fill is expected to vary from negligible to approximately 0.1 foot. The above estimates of shrinkage, bulking and subsidence are intended as an aid for project engineers in determining earthwork quantities. However, these estimates should not be considered as absolute values and should be used with some caution. Contingencies should be made for balancing earthwork quantities based on actual shrinkage, bulking and subsidence that occurs during the grading operations.

Temporary Excavations

Temporary excavations to a depth possibly as much as 20± feet below existing grades may be required to accommodate the recommended over-excavation of unsuitable materials or to construct retaining walls and subsurface storm water disposal structures. Based on the physical properties of the onsite soils, temporary excavations which are constructed exceeding 4 feet in height should be cut back to a ratio of 0.5:1 (h:v) or flatter for the duration of the over-excavation of unsuitable soil material and replacement as compacted fill, as well as placement of underground utilities. However, the temporary excavations should be observed by a representative of the project geotechnical consultant for evidence of potential instability. Depending on the results of these observations, revised slope configurations may be necessary or appropriate. Other factors which should be considered with respect to the stability of the temporary slopes include construction traffic and/or storage of materials on or near the tops of the slopes, construction scheduling, presence of nearby walls or structures on adjacent properties and weather conditions at the time of construction. Applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health act of 1970 and the Construction Safety Act should also be followed.

No temporary excavations along the property lines should be left open without proper protections to reduce safety hazards. The grading contractor is solely responsible for ensuring the safety of construction personnel and the general public, and for appointing a designated "Competent Person" to observe and classify temporary excavation sidewalls pursuant to 29 CFR Part 1926 (OSHA Safety and Health Regulations for Construction).

Cut Slope Construction and Protection

Observations during grading of individual cut slopes by the project engineering geologist to document favorable geologic conditions of the exposed slopes is recommended. The finish surface of cut slope faces should be scaled of any loose rocks and embedded rock fragments prone to raveling. If significant fractured or loose rock is exposed following grading, additional recommendations may be warranted.

Monitoring of Adjacent Properties

Existing adjoining residential structures in the immediate vicinity of temporary excavations may have pre-existing damage, which go unnoticed (hairline cracks, etc.) until some construction activity draws attention to such conditions. Then, it becomes difficult to identify whether damage was pre-existing or has been caused by the construction. To help reduce the risk of such conflicts, it is advisable, though not required, to perform a pre-construction condition survey of existing structures, especially those located directly along the property lines. This would involve visual inspection and photo and video documentation.

The proposed construction is likely to create vibrations in the vicinity of adjoining structures, due to activities such as excavations into hard or dense earth materials. At your discretion, vibrations be monitored on or near existing buildings and structures in order to reduce the risk of damage to existing buildings and defend against potential future claims.

FOUNDATION DESIGN RECOMMENDATIONS

Allowable Soil Bearing Capacities

Pad Footings

An allowable soil bearing capacity of 1,500 pounds per square foot may be utilized for design of isolated 24-inch-square footings founded at a minimum depth of 12 inches below the lowest adjacent final grade for pad footings that are not a part of the slab system and are used for support of such features as roof overhang, second-story decks, patio covers, etc. This value may be increased by 20 percent for each additional foot of depth and by 10 percent for each additional foot of width, to a maximum value of 2,500 pounds per square foot. The recommended allowable bearing value includes both dead and live loads and may be increased by one-third for short duration wind and seismic forces.

Continuous Footings

An allowable soil bearing capacity of 1,500 pounds per square foot may be utilized for design of continuous footings founded at a minimum depth of 12 inches below the lowest adjacent final grade. This value may be increased by 20 percent for each additional foot of depth and by 10 percent for each additional foot of width, to a maximum value of 2,500 pounds per square foot. The recommended allowable bearing value includes both dead and live loads and may be increased by one-third for short duration wind and seismic forces. Increased bearing capacity for large continuous footings, such as retaining walls, may be appropriate, depending on the as-graded soil/bedrock conditions.

Estimated Footing Settlement

Based on the allowable bearing values provided above, total static settlement of the footings under the anticipated loads is expected to be on the order of 3/4 inch. Differential settlement is expected to be less than 1/2 inch over a horizontal span of 30 feet. The majority of settlements are likely to take place as footing loads are applied or shortly thereafter.

Lateral Resistance

A passive earth pressure of 300 pounds per square foot per foot of depth, to a maximum value of 2,500 pounds per square foot, may be used to determine lateral bearing resistance for footings. The above value may be increased by one-third when designing for transient wind or seismic forces. In addition, a coefficient of friction of 0.35 times the dead load forces may be used between concrete and the supporting soils to determine lateral sliding resistance. It should be noted that the above values are based on the condition where footings are cast in direct contact with compacted fill or competent native soils. In cases where the footing sides are formed, all backfill placed against the footings upon removal of forms should be compacted to at least 90 percent of the applicable maximum dry density.

Guidelines for Footings and Slab-on-Ground Design and Construction

The results of our laboratory tests performed on representative samples of near-surface soils within the site during our investigation indicate that these materials predominantly exhibit expansion indices that range from 21 to 50. As such, the site soils are classified as "expansive" as defined in Section 1803.5.3 of the 2022 California Building Code (2022, CBC). The design of foundations and slabs on-ground should therefore be performed in accordance with the procedures outlined in Sections 1808.6.1 and 1808.6.2 of the 2022 CBC.

General

Briefly, Section 1808.6.1 of the 2022 CBC requires that foundations placed on or within the active zone of expansive soils shall be designed to resist differential volume changes and to prevent structural damage to the supported structure. Section 1808.6.2 of the 2022 CBC requires that non-prestressed slabs on-grade or mat foundations constructed on expansive soils be designed in accordance with the latest Code-adopted edition of *WRI/CRSI Design of Slab-on-Ground Foundations.* The 2022 CBC also requires that post-tensioned slabs on-grade or mat foundations placed on expansive soils be designed in accordance with the latest Code-adopted edition of *PTI DC 10.5,* with the provision that the analyses used to determination of moments, shears and deflections are performed accordingly. It should be noted that, under certain conditions, the 2022 CBC allows for alternative, rational methods of analysis and design of such slabs provided that these methods account for soil-structure interaction, the deformed shape of the soil support, plate, or stiffened plate action of the slab, as well as both center lift and edge lift conditions.

The design and construction guidelines that follow are based on the above soil conditions and may be considered for reducing the effects of variability in fabric, composition and, therefore, the detrimental behavior of the site soils such as excessive short- and long-term total and differential heave and settlement. These guidelines have been developed on the basis of the previous experience of this firm on projects with similar soil conditions. Although construction performed in accordance with these guidelines has been found to reduce post-construction movement and/or distress, they generally do not positively eliminate all potential effects of variability in soils characteristics and future settlement.

It should also be noted that the suggestions for dimension and reinforcement provided herein are performance-based and intended only as preliminary guidelines to achieve adequate performance under the anticipated soil conditions. However, they should not be construed as replacement for structural engineering analyses, experience, and judgment. The project structural engineer, architect and/or civil engineer should make appropriate adjustments to slab and footing dimensions, and reinforcement type, size and spacing to account for internal concrete forces (e.g., *thermal, shrinkage and expansion), as well as external forces (e.g., applied loads) as deemed necessary. Consideration should also be given to minimum design criteria as dictated by local building code requirements.*

Post-Tensioned Slab-on-Gound System

As stated above, onsite soils should be considered to be expansive per Section 1803.5.3 of the 2022 CBC. Section 1808.6.2 of the 2022 CBC specifies that post-tensioned slab-on-ground foundations (floor slabs) resting on expansive materials should be designed in accordance with the latest Code-adopted edition of the Post-Tensioning Institute publication, PTI DC 10.5.

To comply with Section 1808.6.2 of the 2022 CBC and the PTI publication, in addition to performing appropriate tests on representative samples of site soils, certain assumptions regarding the site environmental (climatic) condition and the composition of the subsurface soils were made. The following table, Table 4, presents our recommendations for soil and climatic parameters for design of post-tensioned slabs on-grade based on our laboratory testing, engineering analysis, as well as our engineering judgment and experience on similar sites.

TABLE 4

Presumptive Post-Tensioned Slab on-Grade Design Parameters for PTI Procedure

Tentative Design Parameters	
Approximate Depth of Constant Suction, feet	9
Approximate Soil Suction, pF	3.9
Inferred Thornthwaite Index:	-20
Average Edge Moisture Variation Distance, e_m in feet: Center Lift Edge Lift	 9.0 4.7
Anticipated Swell, y_m in inches: Center Lift Edge Lift	 0.35 0.65

It should be noted that some of the non-climatic site parameters, which may impact slabs on-grade performance, are not known at this time, as it is the case for many projects at the design stage. Some of these site parameters include unsaturated soils diffusion conditions pre- and post-construction (e.g., casting the slabs at the end of long, dry, or wet periods, maintenance during long, dry and wet periods, etc.), landscaping, alterations in site surface gradient, irrigation, trees, etc. While the effects of any or a combination of these parameters on slab performance cannot be accurately predicted, maintaining *moisture content equilibrium within the soils mass and planting trees at a distance greater than half of their mature height away from the edge of foundation may reduce the potential for the adverse impact of these site parameters on slabs on-grade performance.*

<u>Modulus of Subgrade Reaction</u>

The modulus of subgrade reaction for design of load bearing elements depends on the size of the element and soil-structure interaction. However, as a first level of approximation, this value may be assumed to be 125 pounds per cubic inch.

<u>Minimum Design Recommendations</u>

The soil values provided above may be utilized by the project structural engineer to design post-tensioned slabs on-ground in accordance with Section 1808.6.2 of the 2022 CBC and the PTI publication. Thicker floor slabs and larger footing sizes may be required for structural reasons and should govern the design if more restrictive than the minimum recommendations provided below:

<u>Footings</u>

1. Exterior continuous footings for three- and four-story structures should be founded at a minimum depth of 18 inches below the lowest adjacent finished ground surface. Interior footings may be founded at a minimum depth of 12 inches below the tops of the adjacent finish floor slabs. Interior continuous footings width and spacing should be designed by the project structural engineer.

2. In accordance with Table 1809.7 of 2022 CBC for light-frame construction, all continuous footings should have minimum widths of 15 and 18 inches for three- and four-story construction. We recommend all continuous footings should be reinforced with a minimum of two No. 4 bars, one top and one bottom. Alternatively, post-tensioned tendons may be utilized in the perimeter continuous footings in lieu of the reinforcement bars.

3. A minimum 12-inch-wide grade beam founded at the same depth as adjacent footings should be provided across the garage entrances or similar openings (such as large doors or bay windows). The grade beam should be reinforced in a similar manner as provided above.

4. Interior isolated pad footings, if required, should be a minimum of 24 inches square and founded at a minimum depth of 15 inches below the bottoms of the adjacent floor slabs. Pad footings should be reinforced with No. 4 bars spaced a maximum of 18 inches on centers, both ways, placed near the bottoms of the footings.

5. Exterior isolated pad footings intended for support of colonnades, roof overhangs, upper-story decks, patio covers, and similar construction should be a minimum of 24 inches square, and founded at a minimum depth of 18 inches below the lowest adjacent final grade. The pad footings should be reinforced with No. 4 bars spaced a maximum of 18 inches on centers, both ways, placed near the bottoms of the footings. Exterior isolated pad footings may need to be connected to adjacent pad and/or continuous footings via tie beams at the discretion of the project structural engineer.

6. The thickness of the floor slabs should be determined by the project structural engineer with consideration given to the expansion potential of the on-site soils; however; we recommend that a minimum slab thickness of 4 inches be considered.

7. As an alternative to designing 4-inch-thick post-tensioned slabs with perimeter footings as described in Items 1 and 2 above, the structural engineer may design the foundation system using a thickened slab design. The minimum thickness of this uniformly thick slab should be 8 inches. The engineer in charge of post-tensioned slab design may also opt to use any combination of slab thickness and footing embedment depth as deemed appropriate based on their engineering experience and judgment.

8. Living area concrete floor slabs and areas to receive moisture sensitive floor covering should be underlain with a moisture vapor retarder consisting of a minimum 10-mil-thick polyethylene or polyolefin membrane that meets the minimum requirements of ASTM E96 and ASTM E1745 for vapor retarders (such as Husky Yellow Guard®, Stego® Wrap, or equivalent). All laps within the membrane should be sealed, and at least 2 inches of clean sand should be placed over the membrane to promote uniform curing of the concrete.

   In general, to reduce the potential for punctures, the membrane should be placed on a pad surface that has been graded smooth without any sharp protrusions. If a smooth surface cannot be achieved by grading, consideration should be given to lowering the pad finished grade an additional inch and then placing a 1-inch-thick leveling course of sand across the pad surface prior to the placement of the membrane. Foot traffic on the membrane should be reduced to a minimum. Additional steps would also need to be taken to prevent puncturing of the vapor retarder during concrete placement.

   At the present time, some slab designers, geotechnical professionals, and concrete experts view the sand layer below the slab (blotting sand) as a place for entrapment of excess moisture that could adversely impact moisture-sensitive floor coverings. As a preventive measure, the potential for moisture intrusion into the concrete slab could be reduced if the concrete is placed directly on the vapor retarder. However, if this sand layer is omitted, appropriate curing methods must be implemented to ensure that the concrete slab cures uniformly. A qualified contractor with experience in slab construction and curing should provide recommendations for alternative methods of curing and supervise the construction process to ensure uniform slab curing.

9. Garage floor slabs should be a minimum 4 inches thick and reinforced in a similar manner as living area floor slabs. Consideration should be given to placement of a moisture vapor retarder below the garage slab, similar to that provided in Item 6 above, should the garage slab be overlain with moisture sensitive floor covering.

10. Presaturation of the subgrade below floor slabs will not be required; however, prior to placing concrete, the subgrade below all dwelling and garage floor slab areas should be thoroughly moistened to achieve a moisture content that is at least equal to or slightly greater than optimum moisture content to a minimum depth of 12 inches below the bottoms of the slabs.

11. The minimum footing dimensions and foundation design parameters recommended herein are based on our experience, judgement and professional interpretation of the prevailing site soils' characteristics and the inferred site environmental/climatic conditions. At this time, we do not have information regarding potential improvements to be located and/or absent within the zone of influence of the foundation system that could adversely impact the foundation's performance. Such improvements may include, but are not limited to, adjacent lawn/planter areas, irrigation regime, trees located within a horizontal distance of less than half of their mature height from the foundation, and vertical and/or

horizontal moisture barriers. A knowledge of these features may allow the designers to perform a refined analysis. However, in the absence of such process, the minimum dimensions provided herein may be modified (increased or decreased subject to the constraints of Chapter 18 of the 2022 CBC and PTI DC 10.5) by the structural engineer responsible for foundation design based on his/her calculations, engineering experience, and judgment.

Footing Observations

Foundation footing trenches should be observed by the project geotechnical consultant to document into competent bearing-soils. The foundation excavations should be observed prior to the placement of forms, reinforcement, or concrete. The excavations should be trimmed neat, level, and square. Prior to placing concrete, all loose, sloughed, or softened soils and/or construction debris should be removed. Excavated soils derived from footing and utility trench excavations should not be placed in slab-on-grade areas unless the soils are compacted to a relative compaction of 90 percent or more.

Preliminary Infiltration Rate

The field falling-head percolation test data was utilized in determining the test infiltration rate, I_t, expressed in units of inches/hour, utilizing the Porchet Method (SARWQCB, 2013). Field testing was conducted in two perforated-cased boreholes (with pea gravel surrounding the pipe) at both 30-minute intervals for a period of approximately 7 hours. Test data are attached in Appendix E. The infiltration rate, I_t, was calculated by determining the volumetric water flow through the wetted borehole surface area, expressed in terms of inches per hour. Un-factored test results are summarized in the following Table 5.

TABLE 5

Summary of Percolation Test Results*

Test Location	Total Depth (ft.)	Percolation Rate (gal/day/ft^2)	Un-Factored Infiltration Test Rate, I_t (in/hr)
B-2	10.5	3.2	0.45
B-6	10.0	3.5	0.49

*** Note – Percolation test was performed in approximately the lower 4± feet of the test boreholes.**

In view of the test data, the shallow subsurface granitic bedrock exhibits low permeability. It should be noted that these results are un-factored. Although the tests did not indicate localized impermeability, variability can be possible due to changes in both material density and gradation. The boring logs indicate that shallow bedrock exists across the site and, therefore, infiltration rates could be consistently very low.

However, no infiltration due to the infiltration testing results, soil characteristics, and shallow bedrock. Form I-8 is included in Appendix D.

Preliminary Pavement Design Recommendations

Based upon our experience in Santee, an R-value of 30 was estimated for the subject site. A traffic index (TI) of 5 was assumed for parking lot drive aisles and parking stalls. A TI of 7 was assumed for the collector streets Summit Avenue and Princess Joann. The traffic index and the estimated design R-value were utilized for preliminary pavement section design. The pavement section has been computed in accordance with Caltrans design procedures and presented in Table 6.

TABLE 6

Preliminary Pavement Design

Location	Design R-value	Traffic Index	Pavement Section
Building Access Aisles and Parking Stalls	30	5.0	3 in. AC / 6 in. AB*
Project Entry and Loop Road	30	6.0	4 in. AC / 6 in. AB
Summit Avenue and Princess Joann	30	7.5	5 in. AC / 9 in. AB

Notes:
AC = Asphalt Concrete
AB = Aggregate Base
*Minimum 6” aggregate base section per City of Santee

Subgrade soils immediately below the base should be compacted to 95 percent or more relative compaction based on ASTM D 1557 to a depth of 12 inches or more. Final subgrade compaction should be performed prior to placing base or asphalt-concrete and after utility-trench backfills have been compacted and tested. Subgrade should be firm and unyielding, as exhibited by proof-rolling, prior to placement of aggregate base.

Base materials should consist of Caltrans Class 2 aggregate base. Base materials should be compacted to 95 percent or more relative compaction based on ASTM D 1557. The base materials should be near optimum-moisture content when compacted. Asphalt concrete materials should conform to Section 203-6 of the most recent Standard Specifications for Public Works Construction (Greenbook) or as required by the City of Santee Public Works Department - Standard Specifications and Detail Drawings.

General Corrosivity Screening

As a screening level study, limited chemical and electrical tests were performed on samples considered representative of the onsite soils to identify potential corrosive characteristics of these soils. The common indicators that are generally associated with soil corrosivity, among other indicators, include water-soluble sulfate (a measure of soil corrosivity on concrete), water-soluble chloride (a measure of soil corrosivity on metals embedded in concrete), pH (a measure of soil acidity), and minimum electrical resistivity (a measure of corrosivity on metals embedded in soils). Test methodology and results are presented in Appendix B.

> *It should be noted that Petra does not practice corrosion engineering; therefore, the test results, opinion and engineering judgment provided herein should be considered as general guidelines only. Additional analyses, and/or determination of other indicators, would be warranted, especially, for cases where buried metallic building materials (such as copper and cast or ductile iron pipes) in contact with site soils are planned for the project. In many cases, the project geotechnical engineer may not be informed of these choices. Therefore, for conditions where such elements are considered, we recommend that other, relevant project design professionals (e.g., the architect, landscape architect, civil and/or structural engineer, etc.) to be involved. We also recommend considering a qualified corrosion engineer to conduct additional sampling and testing of near-surface soils during the final stages of site grading to provide a complete assessment of soil corrosivity. Recommendations to mitigate the detrimental effects of corrosive soils on buried metallic and other building materials that may be exposed to corrosive soils should be provided by the corrosion engineer as deemed appropriate.*

In general, a soil's water-soluble sulfate levels and pH relate to the potential for concrete degradation; water-soluble chlorides in soils impact ferrous metals embedded or encased in concrete, e.g., reinforcing steel; and electrical resistivity is a measure of a soil's corrosion potential to a variety of buried metals used in the building industry, such as copper tubing and cast or ductile iron pipes. Table 7, below, presents test results with an interpretation of current code approach and guidelines that are commonly used in building construction industry. The table includes the code-related classifications of the soils as they relate to the various tests, as well as a general recommendation for possible mitigation measures in view of the potential adverse impact of corrosive soils on various components of the proposed structures in direct contact with site soils. The guidelines provided herein should be evaluated and confirmed, or modified, in their entirety by the project structural engineer, corrosion engineer and/or the contractor responsible for concrete placement for structural concrete used in exterior and interior footings, interior slabs on-ground, garage slabs, wall foundations and concrete exposed to weather such as driveways, patios, porches, walkways, ramps, steps, curbs, etc.

TABLE 7

Soil Corrosivity Screening Results

Test (Test Method Designation)	Test Results	Classification	General Recommendations
Soluble Sulfate (Cal 417)	SO_4^{2-} < 0.10 % by weight	**S0(1) - Not Applicable**	Type II cement; minimum f_c' = 2,500 psi; no water/cement ratio restrictions.
pH (Cal 643)	7.8 – 8.2	**Moderately Alkaline**	No special recommendations
Soluble Chloride (Cal 422)	Cl^{1-} < 500 ppm	**C1(2) - Moderate**	No special recommendations; f_c' should not be less than 2,500 psi.
Resistivity (Cal 643)	2,345 ohm-cm	**Highly Corrosive(5)**	**Consult a Corrosion Engineer**

Notes:
1. ACI 318-14, Section 19.3
2. ACI 318-14, Section 19.3
3. Pierre R. Roberge, "Handbook of Corrosion Engineering"
4. Exposure classification C2 applies specifically to swimming pools and appurtenant concrete elements
5. f_c': 28-day unconfined compressive strength of concrete

POST-GRADING RECOMMENDATIONS

Site Drainage

Surface drainage systems consisting of sloping concrete flatwork, graded earth swales and/or an underground area drain system are anticipated to be constructed to collect and direct all surface waters to the adjacent streets and storm drain facilities. In addition, the ground surface around the proposed buildings should be sloped at a positive gradient away from the structures. The purpose of the precise grading is to prevent ponding of surface water within the level areas of the site and against building foundations and associated site improvements. The drainage systems should be properly maintained throughout the life of the proposed development.

It should be emphasized that the slopes away from the structures area drain inlets and storm drain structures to be properly maintained, not to be obstructed, and that future improvements not to alter established gradients unless replaced with suitable alternative drainage systems.

Slope Landscaping and Maintenance

Adequate slope and pad drainage facilities are essential in the design of grading for the subject site. An anticipated rainfall equivalency on the order of 60 to 100 inches per year at the site can result due to irrigation. The overall stability of the graded slopes should not be adversely affected provided drainage provisions are properly constructed and maintained thereafter and provided engineered slopes are landscaped immediately following grading with a deep-rooted, drought-tolerant, and maintenance-free plant species, as recommended by the project landscape architect. Additional comments and recommendations are presented below with respect to slope drainage, landscaping, and irrigation.

A common type of slope failure in hillside areas is the surficial instability and usually involves the upper 1 to 6 feet. For a given gradient, these surficial slope failures are generally caused by a wide variety of conditions, such as overwatering, cyclic changes in moisture content and density of slope soils from both seasonal and irrigation-induced wetting and drying, soil expansiveness, time lapse between slope construction and slope planting, type and spacing of plant materials used for slope protection, rainfall intensity and/or lack of a proper maintenance program. Based on this discussion, the following recommendations are presented to mitigate potential surficial slope failures.

- Proper drainage provisions for engineered slopes should consist of concrete terrace drains, downdrains and energy dissipaters (where required) constructed in accordance with the Grading Code of the City of Santee. Provisions should also be made for construction of compacted-earth berms along the tops of engineered slopes.

- Permanent engineered slopes should be landscaped as soon as practical at the completion of grading. As noted, the landscaping should consist of a deep-rooted, drought-tolerant, and maintenance-free plant species. If landscaping cannot be provided within a reasonable period of time, jute matting (or equivalent) or a spray-on product designed to seal slope surfaces should be considered as a temporary measure to inhibit surface erosion until such time permanent landscape plants have become well-established.

- Irrigation systems should be installed on the engineered slopes and a watering program then implemented which maintains a uniform, near-optimum moisture condition in the soils. Overwatering and subsequent saturation of the slope soils should be avoided. On the other hand, allowing the soils to dry-out is also detrimental to slope performance.

- Irrigation systems should be constructed at the surface only. Construction of sprinkler lines in trenches should not be allowed without prior approval from the geotechnical engineer and engineering geologist.

- A permanent slope-maintenance program should be initiated for major slopes not maintained by individual homeowners. Proper slope maintenance should include the care of drainage- and erosion-control provisions, rodent control, and repair of leaking or damaged irrigation systems.

- Homeowners should be advised of the potential problems that can develop when drainage on the pads and slopes is altered. Drainage can be altered due to the placement of fill and construction of garden walls, retaining walls, walkways, patios, swimming pools, spas, and planters.

Utility Trenches

Utility-trench backfill within street rights-of-way, utility easements, under sidewalks, driveways and building-floor slabs should be compacted to a relative compaction of 90 percent or more. Where onsite soils are utilized as backfill, mechanical compaction should be used. Density testing, along with probing, should be performed by the project geotechnical consultant or his representative to document adequate compaction. Utility-trench sidewalls deeper than about 4 feet should be laid back at a ratio of 0.5:1 (h:v) or flatter or shored. A trench box may be used in lieu of shoring. If shoring is anticipated, the project geotechnical consultant should be contacted to provide design parameters.

For trenches with vertical walls, backfill should be placed in approximately 1- to 2-foot-thick loose lifts and then mechanically compacted with a hydra-hammer, pneumatic tampers, or similar compaction equipment. For deep trenches with sloped walls, backfill materials should be placed in approximately 8- to 12-inch-thick loose lifts and then compacted by rolling with a sheepsfoot tamper or similar equipment.

Where utility trenches are proposed in a direction that parallels any building footing (interior and/or exterior trenches), the bottom of the trench should not be located within a 1:1 (h:v) plane projected downward from the outside bottom edge of the adjacent footing.

EXTERIOR CONCRETE FLATWORK

General

Near-surface compacted fill soils within the site are variable in fines content and expansion behavior with an expectation for the majority of these soils to exhibit an Expansion Index in the range of low expansion potential. For this reason, we recommend that additional testing of subgrade soils be performed at the completion of precise grading in order to provide specific recommendations for all exterior concrete flatwork. However, owing to typical project scheduling constraints, it may not be feasible to collect additional samples of subgrade soils for testing to verify their expansion characteristics in a timely manner; i.e., immediately prior to pouring concrete. As such, we recommend that all exterior concrete flatwork such as sidewalks, patio slabs, large decorative slabs, concrete subslabs that will be covered with decorative pavers, private and/or public vehicular parking, driveways and/or access roads within and adjacent to the site be designed by the project architect, civil and/or structural engineer with consideration given to mitigating the potential cracking, curling, uplift, etc. that can develop in soils exhibiting expansion index values that fall in the upper ranges of the values provided above.

The guidelines that follow should be considered as minimums and are subject to review and revision by the project architect, civil engineer, structural engineer and/or landscape consultant as deemed appropriate. If sufficient time will be allowed in the project schedule for verification sampling and testing prior to the concrete pour, the test results may dictate that a somewhat less conservative design could be used.

Subgrade Preparation

Compaction

To reduce the potential for distress to concrete flatwork, the subgrade soils below concrete flatwork areas to a minimum depth of 12 inches (or deeper, as either prescribed elsewhere in this report or determined in the field) should be moisture conditioned to at least equal to, or slightly greater than, the optimum moisture content and then compacted to a minimum relative compaction of 90 percent. Where concrete public roads, concrete segments of roads and/or concrete access driveways and heavy recreational vehicles parking are proposed, the upper 6 inches of subgrade soil should be compacted to a minimum 95 percent relative compaction.

Pre-Moistening

As a further measure to reduce the potential for concrete flatwork distress, subgrade soils should be thoroughly moistened prior to placing concrete. The moisture content of the soils should be at least 1.2 times the optimum moisture content and penetrate to a minimum depth of 12 inches into the subgrade. Flooding or ponding of the subgrade is not considered feasible to achieve the above moisture conditions since this method would likely require construction of numerous earth berms to contain the water. Therefore, moisture conditioning may be achieved with sprinklers or a light spray applied to the subgrade over a period of few to several days just prior to pouring concrete. Pre-watering of the soils is intended to promote uniform curing of the concrete, reduce the development of shrinkage cracks and reduce the potential for differential expansion pressure on freshly poured flatwork. A representative of the project geotechnical consultant should observe and verify the density and moisture content of the soils, and the depth of moisture penetration prior to pouring concrete.

Thickness and Joint Spacing

To reduce the potential of unsightly cracking, concrete walkways, patio-type slabs, large decorative slabs and concrete subslabs to be covered with decorative pavers should be at least 4 inches thick and provided with construction joints or expansion joints every 6 feet or less. Private driveways that will be designed for the use of passenger cars for access to private garages should also be at least 4 inches thick and provided with construction joints or expansion joints every 10 feet or less. Concrete pavement that will be designed based on an unlimited number of applications of an 18-kip single-axle load in public access areas, segments of road that will be paved with concrete (such as bus stops and cross-walks) or access roads and driveways, which serve multiple residential units or garages, that will be subject to heavy truck loadings and recreational vehicles parking should have a minimum thickness of 5 inches and be provided with control joints spaced at maximum 10-foot intervals. A modulus of subgrade reaction of 125 pounds per cubic foot may be used for design of the public and access roads.

Reinforcement

All concrete flatwork having their largest plan-view panel dimensions exceeding 10 feet should be reinforced with a minimum of No. 3 bars spaced 18 inches for 4-inch-thick slabs and No. 4 bars spaced 24 inches for 5-inch-thick slabs on centers, both ways. Alternatively, the slab reinforcement may consist of welded wire mesh of the sheet type (not rolled) with 6x6/W1.4xW1.4 designations for 4-inch-thick slabs and 6x6/W2.9xW2.9 designations for 5-inch-thick slabs in accordance with the Wire Reinforcement Institute (WRI). The reinforcement should be properly positioned near the middle of the slabs. All foot and equipment traffic on the reinforcement should be avoided or reduced to a minimum.

> *The reinforcement recommendations provided herein are intended as a guideline to achieve adequate performance for anticipated soil conditions. As such, this guideline may not satisfy certain acceptable approaches, e.g. the area of reinforcement to be equal to or greater that 0.2 percent of the area of concrete. The project architect, civil and/or structural engineer should make appropriate adjustments in reinforcement type, size and spacing to account for concrete internal (e.g., shrinkage and thermal) and external (e.g., applied loads) forces as deemed necessary.*

Edge Beams (Optional)

Where the outer edges of concrete flatwork are to be bordered by landscaping, it is recommended that consideration be given to the use of edge beams (thickened edges) to prevent excessive infiltration and accumulation of water under the slabs. Edge beams, if used, should be 6 to 8 inches wide, extend 8 inches below the tops of the finish slab surfaces. Edge beams are not mandatory; however, their inclusion in flatwork construction adjacent to landscaped areas is intended to reduce the potential for vertical and horizontal movement and subsequent cracking of the flatwork related to uplift forces that can develop in expansive soils.

Drainage

Drainage from patios and other flatwork areas should be directed to local area drains and/or graded earth swales designed to carry runoff water to the adjacent streets or other approved drainage structures. The concrete flatwork should be sloped at a minimum gradient of one percent, or as prescribed by project civil engineer or local codes, away from building foundations, retaining walls, masonry garden walls and slope areas.

Tree Wells

Tree wells are not recommended in concrete flatwork areas because they typically introduce excessive water into the subgrade soils and allow root invasion, both of which can cause heaving and cracking of the flatwork.

Retaining Walls

As stated earlier, retaining walls are generally proposed up to 7 feet along the southern boundary. A proposed retaining wall along the eastern and southeastern corner boundary is shown as 13 feet to 19 feet in height. Additional details were not available at the time of preparation of this report. As such, the following recommendations should be considered tentative. Upon preparation of detailed retaining wall design, they should be provided to us to perform retaining wall stability analysis and verify the recommendations provided herein.

Footing Embedment

The base of retaining-wall footings constructed on level ground may be founded at a depth of 12 inches or more below the lowest adjacent final grade for low height walls. Where retaining walls are proposed on or within 15 feet from the top of adjacent descending fill slope, the footings should be deepened such that a horizontal clearance of 7 feet or more is maintained between the outside bottom edges of the footings and the face of the slope. The above-recommended footing setback is preliminary and may be revised based on site-specific soil conditions. Footing trenches should be observed by the project geotechnical representative to document that the footing trenches have been excavated into competent bearing soils and to the embedment recommended above. These observations should be performed prior to placing forms or reinforcing steel.

Allowable Bearing Values and Lateral Resistance

Retaining wall footings that are embedded in compacted fill, competent native soils, and weathered granitic bedrock may be designed using the allowable bearing values and lateral resistance values provided previously for building foundations; however, when calculating passive resistance, the resistance of the upper 6 inches of the soil cover in front of the wall should be ignored in areas where the front of the wall will not be covered with concrete flatwork. It should be noted that the above values are based on the condition where footings are cast in direct contact with compacted fill or competent native soils/weathered bedrock. In cases where the footing sides are formed, all backfill placed against the footings upon removal of forms should be compacted to at least 90 percent of the applicable maximum dry density.

Retaining wall footings that are embedded in unweathered granitic bedrock may be designed for an allowable soil bearing capacity of 4,500 pounds per square foot may be utilized for design of footings founded at a minimum depth of 24 inches below the lowest adjacent final grade The recommended allowable bearing value includes both dead and live loads and may be increased by one-third for short duration wind and seismic forces. Further, a passive earth pressure of 1,000 pounds per square foot may be used to determine lateral bearing resistance for footings at a minimum embedment depth of 24 inches. In addition, a coefficient of friction of 0.35 times the dead load forces may be used between concrete and the supporting soils to determine lateral sliding resistance.

Active Earth Pressures

As of the date of this report, it is uncertain whether the proposed retaining walls will be backfilled with on-site soils or imported granular materials. For this reason, active and at-rest earth pressures are provided below for both conditions. However, considering that the onsite earth materials have an expansion index corresponding to both Very Low and Low expansion potentials, the use of imported granular materials for backfilling behind the retaining walls, as described in the following sections, is optional.

1. <u>Onsite Soils Used for Backfill</u>

   Assuming onsite soils are Low in expansion potential, active earth pressures equivalent to fluids having a density of 40 psf/ft and 61 psf/ft should be used for design of cantilevered walls retaining a level backfill and ascending 2:1 backfill, respectively. For walls that are restrained at the top, at-rest earth pressures of 60 pounds per cubic foot (equivalent fluid pressures) should be used. The above values are for retaining walls that have been supplied with a proper subdrain system (see Figure RW-1). All walls should be designed to support any adjacent structural surcharge loads imposed by other nearby walls or footings in addition to the above recommended active and at-rest earth pressures.

2. Imported Sand, Pea Gravel or Rock Used for Wall Backfill

   Imported clean sand exhibiting a sand equivalent value (SE) of 30 or greater, pea gravel or crushed rock may be used for wall backfill to reduce the lateral earth pressures provided these granular backfill materials extend behind the walls to a minimum horizontal distance equal to one-half the wall height. In addition, the sand, pea gravel or rock backfill materials should extend behind the walls to a minimum horizontal distance of 2 feet at the base of the wall or to a horizontal distance equal to the heel width of the footing, whichever is greater (see Figures RW-2 and RW-3). For the above conditions, cantilevered walls retaining a level backfill and ascending 2:1 backfill may be designed to resist active earth pressures equivalent to fluids having densities of 30 and 41 pounds per cubic foot, respectively. For walls that are restrained at the top, at-rest earth pressures equivalent to fluids having densities of 45 and 62 pounds per cubic foot are recommended for design of restrained walls supporting a level backfill and ascending 2:1 backfill, respectively. These values are also for retaining walls supplied with a proper subdrain system. Furthermore, as with native soil backfill, the walls should be designed to support any adjacent structural surcharge loads imposed by other nearby walls or footings in addition to the recommended active and at-rest earth pressures.

All structural calculations and details should be provided to the project geotechnical consultant for verification purposes prior to grading and construction phases.

Earthquake Loads on Retaining Walls

Section 1803.5.12 of the 2022 CBC requires the determination of lateral loads from earthquake forces on retaining walls supporting more than 6 feet of backfill height for structures in Seismic Design Categories D and E.

The 2022 CBC allows that the peak ground acceleration (PGA) may be assumed equal to S_{Ds} / 2.5. This procedure gives a PGA value of 0.206g for this site. This PGA value can be considered a free field acceleration.

According to the research of Mikola and Sitar (2013), the simplified Seed and Whitman calculation is appropriate for use for both cantilever retaining walls and restrained walls.

Cantilever Walls

For cantilever walls, Rankine's active coefficient (ΔK_{ae}) used to determine the additional seismic loading on a retaining wall may be assumed to equal the 40% of the peak free field ground acceleration (Mikola and Sitar, 2013). Thus, ΔK_{ae} = (0.4* 0.206) = 0.08g

From Seed and Whitman (1970), the additional seismic lateral load increment (ΔP_{ae}) on a retaining structure can be determined by the following equation:

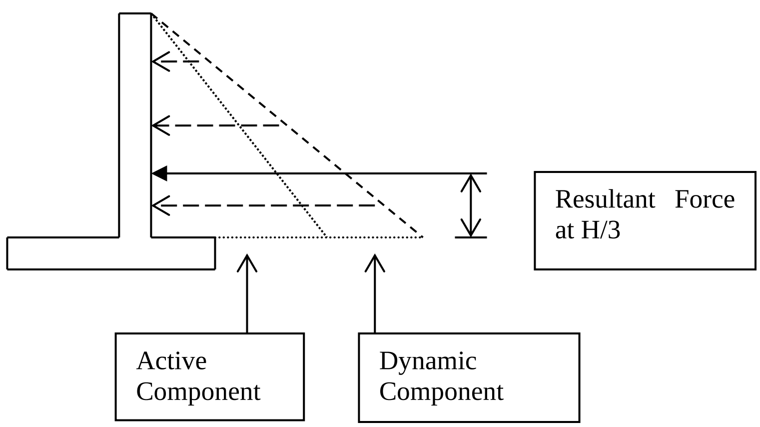
$\Delta P_{ae} = (½)\ \Upsilon\ H^2\ \Delta K_{ae}$

where ΔP_{ae} = additional seismic load increment,
Υ = weight of soil = 125 pcf,
ΔK_{ae} = additional seismic load increment for Rankine's active earth pressure coefficient,
H = retained height of wall in ft.

thus, $\Delta P_{ae} = (½)\ (125\ \text{pcf})\ (H^2)\ (0.08) = 5.0\ H^2$; use 5 H^2 lbs. (per unit foot width).

For cantilever retaining walls, Mikola and Sitar (2013), indicate that the seismic earth pressures have a triangular distribution with the largest load occurring at the bottom of the wall.

The distribution of the seismic lateral load for both types of walls is as follows:



Geotechnical Observation and Testing

All grading associated with retaining wall construction, including backcut excavations, observation of the footing trenches, installation of the subdrainage systems, and placement of backfill should be provided by a representative of the project geotechnical consultant.

Backdrains

To reduce the likelihood of the entrapment of water in the backfill soils, weepholes or open vertical masonry joints may be considered for retaining walls not exceeding a height of 3 feet. Weepholes, if used, should be 3-inches minimum diameter and provided at intervals of 6 feet or less along the wall. Open vertical masonry joints, if used, should be provided at 32-inch intervals. A continuous gravel fill, 3 inches by 12 inches, should be placed behind the weepholes or open masonry joints. The gravel should be wrapped in filter fabric to prevent infiltration of fines and subsequent clogging of the gravel. Filter fabric may consist of Mirafi 140N or equivalent.

A perforated pipe-and-gravel backdrain should be constructed behind retaining walls exceeding a height of 3 feet (see Figure RW-1). Perforated pipe should consist of 4-inch-minimum diameter PVC Schedule 40, or ABS SDR-35, with the perforations laid down. The pipe should be encased in a 1-foot-wide column of ¾-inch to 1½-inch open-graded gravel. If on-site soils are used as backfill, the open-graded gravel should extend above the wall footings to a minimum height equal to one-third the wall height or to a minimum height of 1.5 feet above the footing, whichever is greater. The open-graded gravel should be completely wrapped in filter fabric consisting of Mirafi 140N or equivalent. Solid outlet pipes should be connected to the subdrains and then routed to a suitable area for discharge of accumulated water.

Waterproofing

The backfilled sides of retaining walls should be coated with an approved waterproofing compound or covered with a similar material to inhibit migration of moisture through the walls.

Temporary Excavations

Temporary slopes may be cut at a gradient no steeper than 1:1 (h:v). However, the project geotechnical engineer should observe temporary slopes for evidence of potential instability. Depending on the results of these observations, flatter slopes may be necessary. The potential effects of various parameters such as weather, heavy equipment travel, storage near the tops of the temporary excavations and construction scheduling should also be considered in the stability of temporary slopes.

Wall Backfill

Recommended active and at-rest earth pressures for design of retaining walls are based on the physical and mechanical properties of the onsite soil materials. The backfill behind the proposed retaining walls, they should be placed in approximately 6- to 8-inch-thick maximum lifts, watered as necessary to achieve near optimum moisture conditions, and then mechanically compacted in place to a minimum relative compaction of 90 percent. Flooding or jetting of the backfill materials should be avoided. A representative of the project geotechnical consultant should observe the backfill procedures and test the wall backfill to verify adequate compaction.

Masonry Block Screen Walls

Construction On or Near the Tops of Descending Slopes

Continuous footings for masonry walls proposed on or within 5 feet from the top of a descending cut or fill slope should be deepened such that a horizontal clearance of 5 feet is maintained between the outside bottom edge of the footing and the slope face. The footings should be reinforced with two No. 4 bars, one top and one bottom. Plans for top-of-slope masonry walls proposing pier and grade beam footings should be reviewed by the project geotechnical consultant prior to construction.

Construction on Level Ground

Where masonry walls are proposed on level ground and 5 feet or more from the tops of descending slopes, the footings for these walls may be founded 18 inches or more below the lowest adjacent final grade. These footings should also be reinforced with two No. 4 bars, one top and one bottom.

Construction Joints

In order to reduce the potential for unsightly cracking related to the effects of differential settlement, positive separations (construction joints) should be provided in the walls at horizontal intervals of approximately 20 to 25 feet and at each corner. The separations should be provided in the blocks only and not extend through the footings. The footings should be placed monolithically with continuous rebars to serve as effective "grade beams" along the full lengths of the walls.

CONSTRUCTION SERVICES

This report has been prepared for the exclusive use of Warmington Residential to assist the project engineers and architect in the design of the proposed development. It is recommended that Petra be engaged to review the final-design drawings and specifications prior to construction. This is to document that the recommendations contained in this report have been properly interpreted and are incorporated into the project specifications. If Petra is not accorded the opportunity to review these documents, we can take no responsibility for misinterpretation of our recommendations.

We recommend that Petra be retained to provide soil-engineering services during construction of the excavation and foundation phases of the work. This is to observe compliance with the design, specifications, or recommendations and to allow design changes if subsurface conditions differ from those anticipated prior to start of construction.

If the project plans change significantly (e.g., building loads or type of structures), we should be retained to review our original design recommendations and their applicability to the revised construction. If conditions are encountered during construction that appear to be different than those indicated in this report, this office should be notified immediately. Design and construction revisions may be required.

LIMITATIONS

This report is based on the project as described and the geotechnical data obtained from the field tests performed and our laboratory test data. The materials encountered on the project site and utilized in our laboratory evaluation are believed representative of the total area. However, soil materials can vary in characteristics between excavations, both laterally and vertically.

The conclusions and opinions contained in this report are based on the results of the described geotechnical evaluations and represent our professional judgment. The findings, conclusions and opinions contained in this report are to be considered tentative only and subject to confirmation by the undersigned during the construction process. Without this confirmation, this report is to be considered incomplete and Petra or the undersigned professionals assume no responsibility for its use. In addition, this report should be reviewed and updated after a period of one year or if the site ownership or project concept changes from that described herein.

The professional opinions contained herein have been derived in accordance with current standards of practice and no warranty is expressed or implied.

Respectfully submitted,

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STATE OF CALIFORNIA

JL/SJ/lv

W:\2020-2025\2023\200\23-248\Reports\23-248 110 2nd Revised Updated Geotechnical Evaluation.docx

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FIGURES

Reference: Bing Maps, 2023

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COSTA MESA TEMECULA VALENCIA PALM DESERT CORONA SAN DIEGO

Site Location Map

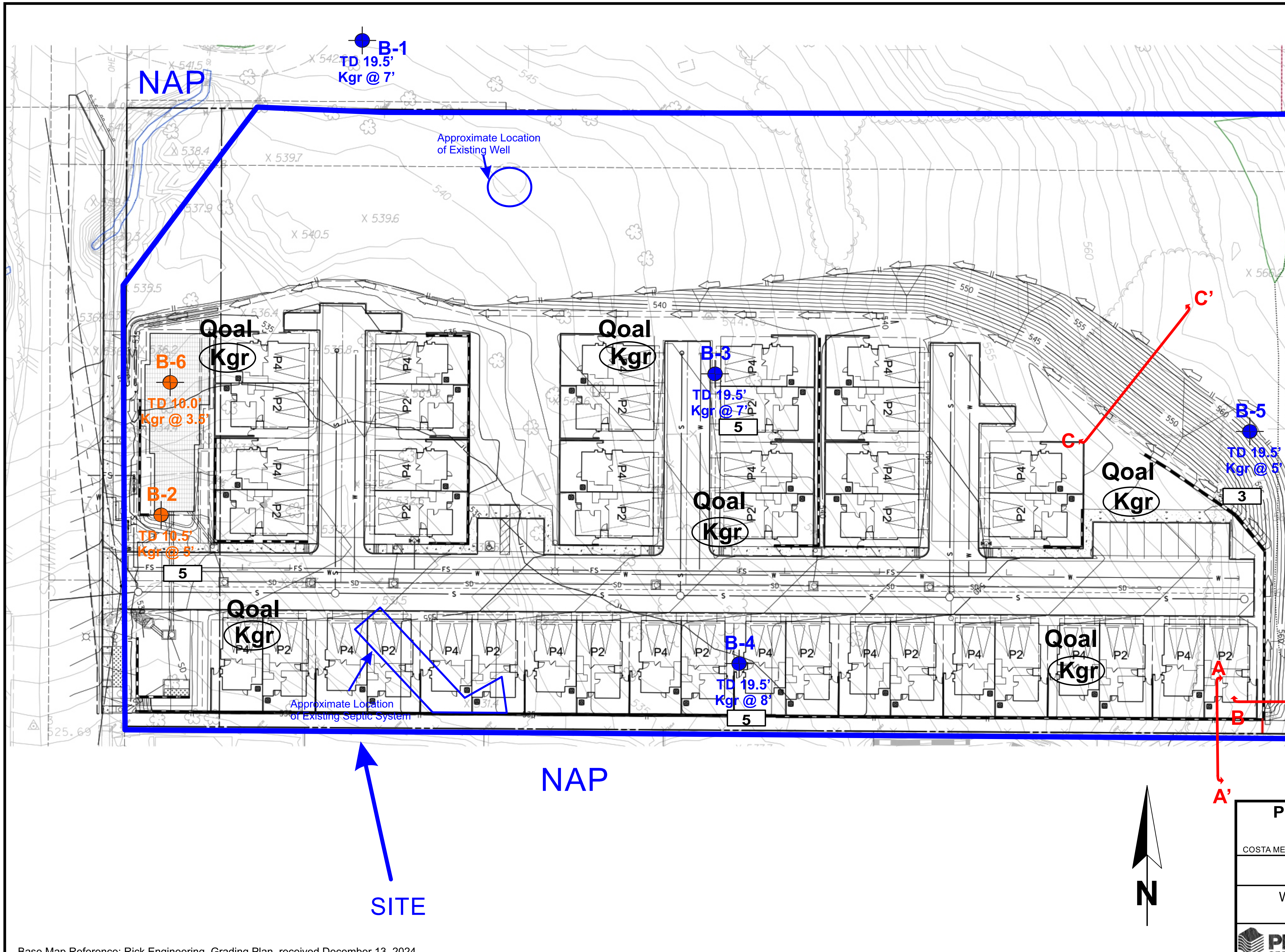
Warmington Homes Summit Avenue Project
City of Santee, California

PETRA GEOSCIENCES INC.

DATE: D□□ember 2024

J.N.: 23-248

Figure 1



EXPLANATION

Qoal	Quaternary Older Alluvium, Circled Where Buried
Kgr	Granitic Bedrock, Circled Where Buried
B-1 TD 19.5' Kgr @ 5'	Approximate Location of Geotechnical Boring, Showing Total Depth and Depth to Kgr
B-6 TD 10.5' Kgr @ 5'	Approximate Location of Percolation Test Boring Showing Total Depth and Depth to Kgr
5	Estimated Depth of Remedial Removals
C — C'	Cross-Section Locations Modified from Rick Engineering Received November 25, 2024

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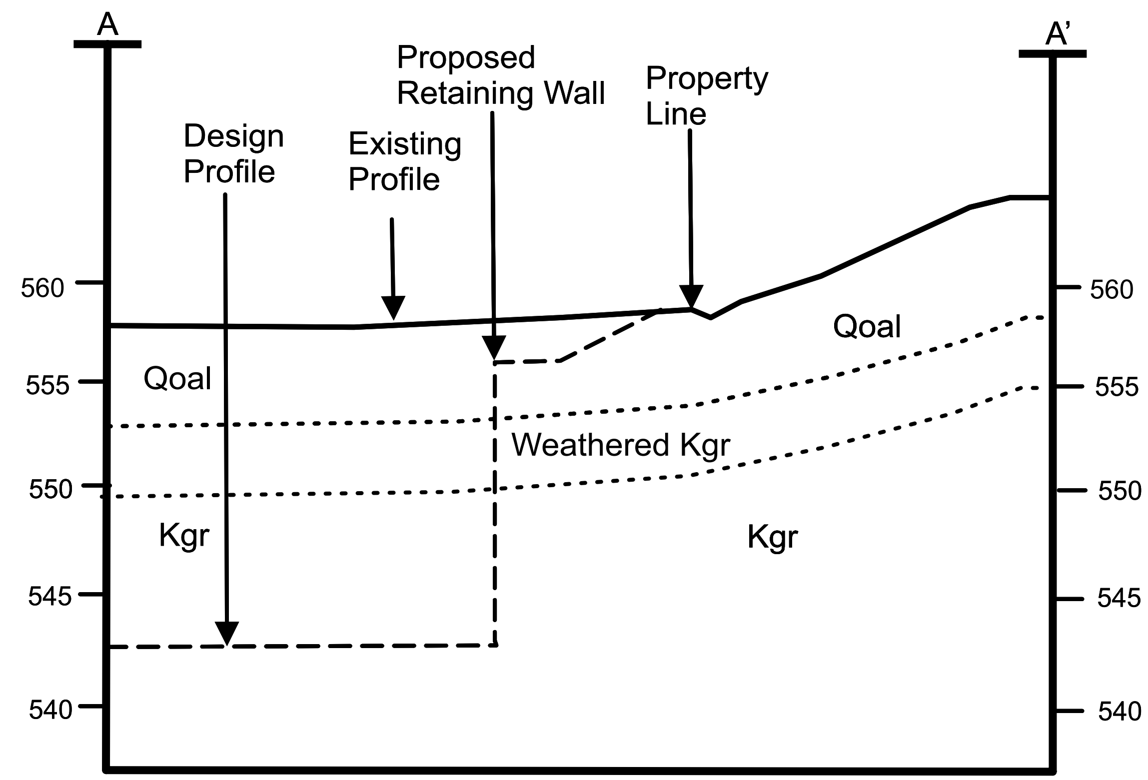
BORING LOCATION MAP

Warmington - Summit Avenue Project
Santee, California

PETRA GEOSCIENCES INC.

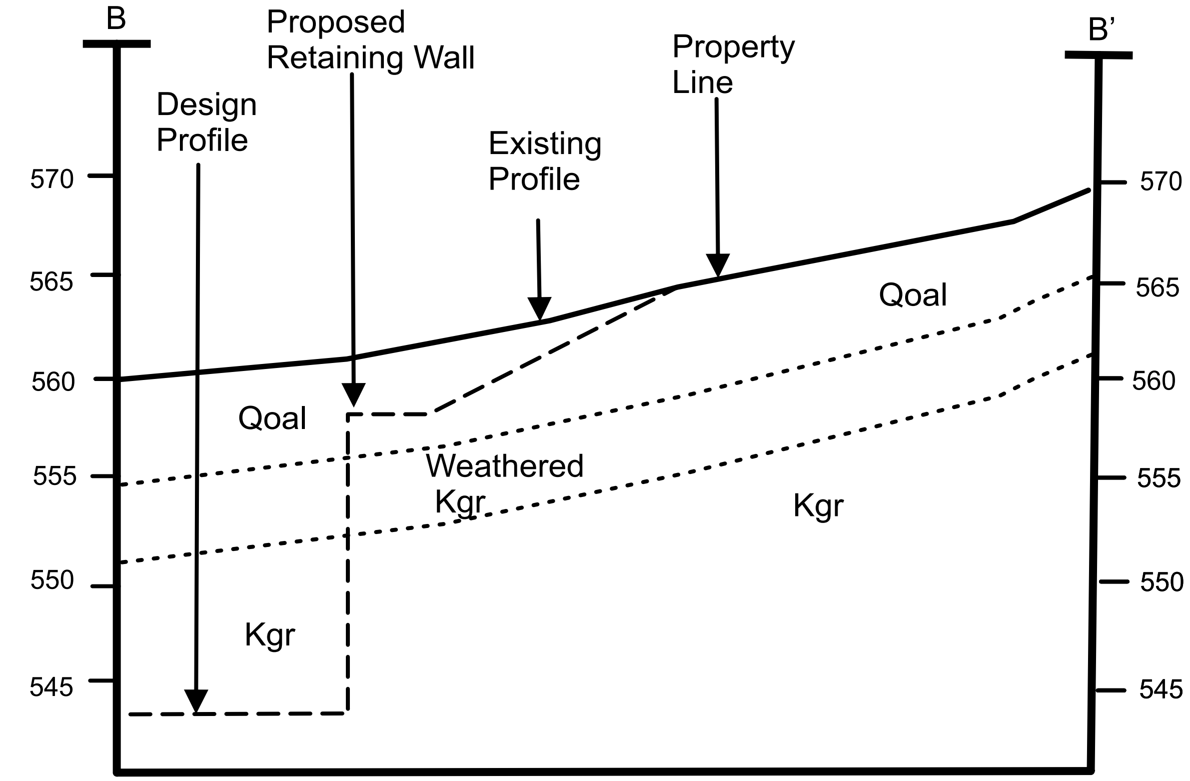
DATE: December 2024
J.N.: 23-248

Figure 2



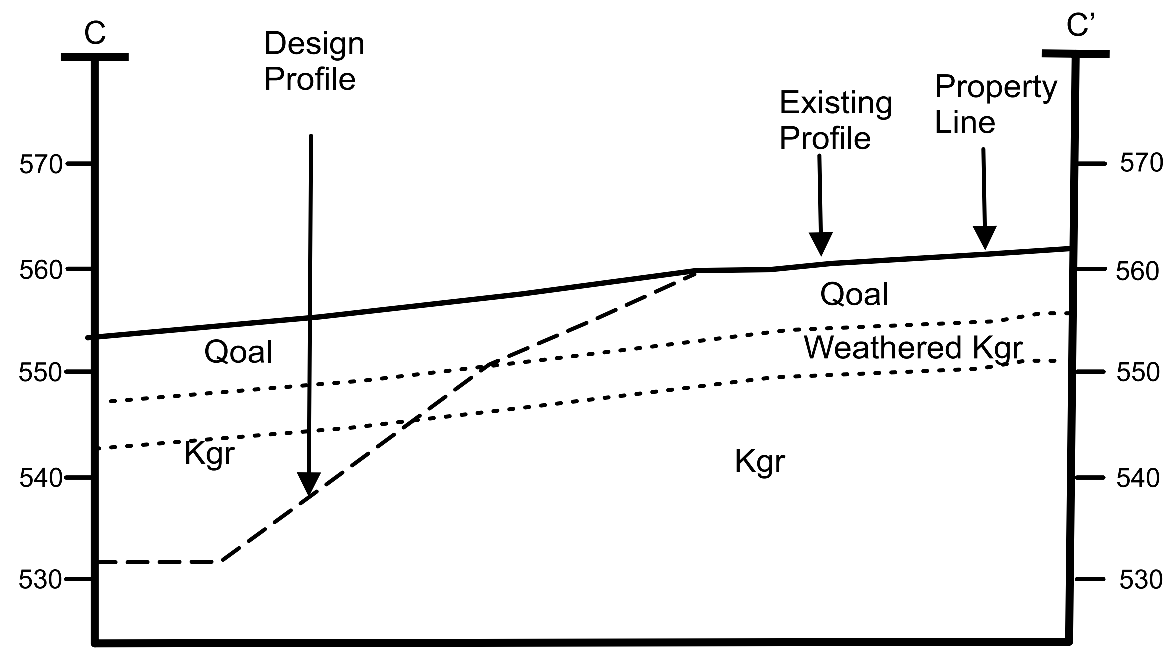
CROSS-SECTION A-A'

N - S



CROSS-SECTION B-B'

W - E



CROSS-SECTION C-C'

S30W - N30W

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GEOTECHNICAL PLANS	Summit Avenue Santee, California	PETRA GEOSCIENCES INC.	DATE: December 2024 J.N.: 23-248	**Figure 3**

APPENDIX A

EXPLORATION LOGS

Key to Soil and Bedrock Symbols and Terms

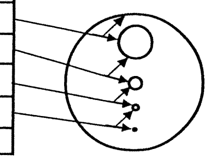
PETRA

Unified Soil Classification System

Coarse-grained Soils > 1/2 of materials is larger than #200 sieve	The No. 200 U.S. Standard Sieve is about the smallest particle visible to the naked eye	GRAVELS more than half of coarse fraction is larger than #4 sieve	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	GM	Silty Gravels, poorly-graded gravel-sand-silt mixtures
				GC	Clayey Gravels, poorly-graded gravel-sand-clay mixtures
		SANDS more than half of coarse fraction is smaller than #4 sieve	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	SM	Silty Sands, poorly-graded sand-gravel-silt mixtures
				SC	Clayey Sands, poorly-graded sand-gravel-clay mixtures
Fine-grained Soils > 1/2 of materials is smaller than #200 sieve		SILTS & CLAYS Liquid Limit Less Than 50		ML	Inorganic silts & very fine sands, silty or clayey fine sands, clayey silts with slight plasticity
				CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
				OL	Organic silts & clays of low plasticity
		SILTS & CLAYS Liquid Limit Greater Than 50		MH	Inorganic silts, micaceous or diatomaceous fine sand or silt
				CH	Inorganic clays of high plasticity, fat clays
				OH	Organic silts and clays of medium-to-high plasticity
Highly Organic Soils				PT	Peat, humus swamp soils with high organic content

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		>12"	>12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19"	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock salt-sized
	fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized to
Fines		Passing #200	<0.0029"	Flour-sized and smaller



Laboratory Test Abbreviations

MAX	Maximum Dry Density	MA	Mechanical (Particle Size) Analysis
EXP	Expansion Potential	AT	Atterberg Limits
SO4	Soluble Sulfate Content	#200	#200 Screen Wash
RES	Resistivity	DSU	Direct Shear (Undisturbed Sample)
pH	Acidity	DSR	Direct Shear (Remolded Sample)
CON	Consolidation	HYD	Hydrometer Analysis
SW	Swell	SE	Sand Equivalent
CL	Chloride Content	OC	Organic Content
RV	R-Value	COMP	Mortar Cylinder Compression

Modifiers

Trace	< 1 %
Few	1 - 5%
Some	5 - 12 %
Numerous	12 - 20 %

Sampler and Symbol Descriptions

- Approximate Depth of Seepage
- Approximate Depth of Standing Groundwater
- Modified California Split Spoon Sample
- Standard Penetration Test
- Bulk Sample
- Shelby Tube
- No Recovery in Sampler

Bedrock Hardness

Soft	Can be crushed and granulated by hand; "soil like" and structureless
Moderately Hard	Can be grooved with fingernails; gouged easily with butter knife; crumbles under light hammer blows
Hard	Cannot break by hand; can be grooved with a sharp knife; breaks with a moderate hammer blow
Very Hard	Sharp knife leaves scratch; chips with repeated hammer blows

Notes:
Blows Per Foot: Number of blows required to advance sampler 1 foot (unless a lesser distance is specified). Samplers in general were driven into the soil or bedrock at the bottom of the hole with a standard (140 lb.) hammer dropping a standard 30 inches unless noted otherwise in Log Notes. Drive samples collected in bucket auger borings may be obtained by dropping non-standard weight from variable heights. When a SPT sampler is used the blow count conforms to ASTM D-1586

EXPLORATION LOG

Project: **Summitt Ave.**		Boring No.:	**B-1**
Location: **Santee**		Elevation:	**546±**
Job No.: **23-248**	Client: **Warmington Residential**	Date:	**7/20/23**
Drill Method: **8" Hollow Stem Auger**	Driving Weight: **140 lbs / 30 "**	Logged By:	**SS**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**OLDER FAN DEPOSITS (Qof)** Silty SAND (SM): Light brown, dry, loose, fine- to coarse-grained.							
		Clayey SAND (SC): Brown to red, dry, dense, fine- to coarse-grained.		17 17 25			9.3	119.0	MAX. EXP, CORR
		medium dense.		16 16 18			9.6	113.8	
5									
		BEDROCK - Granitics (Kgr) Weathered Granite BEDROCK: Light gray, dry, hard.		50/5"			3.2		
10		Granite BEDROCK: Brown to pale red, dry to slightly moist, hard, excavates as fine- to coarse-grained sand, some clay, orange iron staining.		19 50/3"					
15				50/3"					
				50/1"					
20		Total Depth = 19.5' No groundwater Boring backfilled with cuttings.							
25									
30									

PLATE A-1

Petra Geosciences, Inc.

EXPLORATION LOG

Project:	**Summitt Ave.**	Boring No.:	**B-2**
Location:	**Santee**	Elevation:	**531±**
Job No.: **23-248**	Client: **Warmington Residential**	Date:	**7/20/23**
Drill Method: **8 " Hollow Stem Auger**	Driving Weight: **140 lbs / 30 "**	Logged By:	**SS**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**OLDER FAN DEPOSITS (Qof)** Silty SAND (SM): Light brown, dry, loose, fine- to coarse-grained.							
		Clayey SAND (SC): Red to brown, dry, dense, fine- to coarse-grained, trace gravel.		16 23 25			7.6	119.6	
				23 50/6"			13.2	112.0	
5		**BEDROCK - Granitics (Kgr)** Weathered Granite BEDROCK: Light gray, hard, hard, excavates as fine- to coarse-grained sand.							
		Granite BEDROCK: Brown to red, dry to slightly moist, hard, excavates as fine- to coarse-grained sand, trace clay, orange iron staining.		28 48 38					SIEVE
				50/6"					
10		Total Depth = 10.5' No groundwater Percolation test hole Backfilled with cuttings.							
15									
20									
25									
30									

PLATE A-2

Petra Geosciences, Inc.

E X P L O R A T I O N   L O G

Project: **Summitt Ave.**		Boring No.:	**B-3**
Location: **Santee**		Elevation:	**543±**
Job No.: **23-248**	Client: **Warmington Residential**	Date:	**7/20/23**
Drill Method: **8 " Hollow Stem Auger**	Driving Weight: **140 lbs / 30 "**	Logged By:	**SS**

Depth (Feet)	Material Description	WATER	Blows per 6 in.	Core	Bulk	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
0	**OLDER FAN DEPOSITS (Qof)** Silty SAND (SM): Light brown, dry, loose, fine- to coarse-grained.							
	Brown to red, dry, medium dense, fine- to coarse-grained.		6 8 5	■		3.8	108.3	
	dry to slightly moist, loose.		6 6 6	■		5.0	110.4	
5								
	BEDROCK - Granitics (Kgr) Weathered Granite BEDROCK: Brown to red, dry, hard, orange iron staining.		50/5"	■		3.2		
10								
	Granite BEDROCK: Light gray, dry, hard.		25 50/1"	◩				
15								
			50/1"	▬				
20	Total Depth = 19.5' No groundwater Boring backfilled with cuttings.							
25								
30								

PLATE A-3

Petra Geosciences, Inc.

EXPLORATION LOG

Project:	**Summitt Ave.**	Boring No.:	**B-4**
Location:	**Santee**	Elevation:	**537±**
Job No.: **23-248**	Client: **Warmington Residential**	Date:	**7/20/23**
Drill Method: **8 " Hollow Stem Auger**	Driving Weight: **140 lbs / 30 "**	Logged By:	**SS**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**OLDER FAN DEPOSITS (Qof)** Silty SAND (SM): Light brown, dry, loose, fine- to coarse-grained.							
		Light brown to red, dry, medium dense, fine- to coarse-grained, trace clay.		7 7 13	■		3.4	103.0	
5				9 9 12	■		3.6	114.1	
				17 30 50/3"	■				
		BEDROCK - Granitics (Kgr) Weathered Granite BEDROCK: Brown to red, dry, hard, excavates as fine- to coarse-grained sand.							
10									
		Granite BEDROCK: Light gray, dry, hard, excavates as fine- to coarse-grained sand.		50/2"	■				
15									
				50/4"	■				
20		Total Depth = 19.5' No groundwater Boring backfilled with cuttings.							
25									
30									

PLATE A-4

Petra Geosciences, Inc.

E X P L O R A T I O N   L O G

Project: **Summitt Ave.**		Boring No.: **B-5**	
Location: **Santee**		Elevation: **562±**	
Job No.: **23-248**	Client: **Warmington Residential**	Date: **7/20/23**	
Drill Method: **8 " Hollow Stem Auger**	Driving Weight: **140 lbs / 30 "**	Logged By: **SS**	

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**OLDER FAN DEPOSITS (Qof)** Silty SAND (SM): Light brown, dry, loose, fine- to coarse-grained.							
		Brown to red, dry, medium dense, fine- to coarse-grained.		8 9 12			3.7	114.7	MAX, EXP, CORR, DSR
5				8 12 12			6.8	115.4	
		BEDROCK - Granitics (Kgr) Weathered Granite BEDROCK: Brown to red, dry, dense, excavates as fine- to coarse-grained sand.							
		Granite BEDROCK: Brown to red, dry, dense, excavates as fine- to coarse-grained sand.		19 22 27			7.0	112.7	
10									
		Light gray, hard.		50/4"					CORR
15									
				50/4"					
20		Total Depth = 19.5' No groundwater Boring backfilled with cuttings.							
25									
30									

PLATE A-5

Petra Geosciences, Inc.

EXPLORATION LOG

Project:	10939 **Summit Ave.**	Boring No.:	**B-6**
Location:	**Santee**	Elevation:	**531±**
Job No.:	**23-248** Client: **Warmington Residential**	Date:	4/25/24
Drill Method:	JD MX3 M**ini-Ex w 6" auger** Driving Weight: **NA**	Logged By:	JL

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**OLDER FAN DEPOSITS (Qof)** Silty SAND (SM): R□ddi□h □ro□n, dry, loose, fine- to coarse-grained.							
		Clayey SAND (SC): Red to brown, dry, dense, fine- to coarse-grained, trace gravel□f□□ □raniti□ □la□t□.							
5		**BEDROCK - Granitics (Kgr)** Weathered Granite BEDROCK: Light gray, hard, hard, excavates as fine- to coarse-grained sand.							
		Granite BEDROCK: Brown to red, dry to slightly moist, hard, excavates as fine- to coarse-grained sand, trace clay, orange iron staining.							
10		Total Depth = □0' No groundwater Percolation test hole □□t Backfilled with cuttings.							
15									
20									
25									
30									

PLATE A-2

Petra Geosciences, Inc.

APPENDIX B

LABORATORY TEST PROCEDURES

LABORATORY DATA SUMMARY

LABORATORY TEST PROCEDURES

Soil Classification

Soils encountered within the exploratory borings were initially classified in the field in general accordance with the visual-manual procedures of the Unified Soil Classification System (ASTM D 2488). The samples were re-examined in the laboratory and the classifications reviewed and then revised where appropriate. The assigned group symbols are presented in the Boring Logs (Appendix A).

In-Situ Moisture and Density

Moisture content and unit dry density of in-place soils were determined in representative strata. Test data are summarized in the Boring Logs (Appendix A).

Maximum Dry Density and Optimum Moisture Content

The maximum dry density and optimum moisture content of the on-site soils were determined for selected bulk samples in accordance with current version of ASTM D 1557. The results of these tests are presented on Plate B-1.

Expansion Index

The expansion index of onsite soils was determined per ASTM D 4829. The expansion index and expansion potential are presented in Plate B-1.

Corrosivity Tests

Chemical analyses were performed on a selected sample to determine concentrations of soluble sulfate and chloride, as well as pH and resistivity. These tests were performed in accordance with California Test Method Nos. 417 (sulfate), 422 (chloride) and 643 (pH and resistivity). Test results are presented in Plate B-1.

Direct Shear

The Coulomb shear strength parameters, i.e., angle of internal friction and cohesion, were determined for a selected, reconstituted-bulk sample of onsite soil. The test was performed in general accordance with the current version of Test Method ASTM D 3080. Three specimens were prepared for each test. The test specimens were inundated and then sheared under various normal loads at a constant strain rate of 0.005 inch per minute. The results of the direct shear test are graphically presented on Plate B-2.

LABORATORY DATA SUMMARY

Laboratory Maximum Dry Density

Sample Location	Soil Type	Optimum Moisture (%)	Maximum Dry Density (pcf)
B-1 @ 0 – 5’	Red Brown, clayey fine- to coarse-grained SAND	9.0	131.5
B-5 @ 0 – 5’	Dark Brown, Silty fine to coarse grained SAND	8.0	135.5

PER ASTM D 1557 & ASTM D 4718-15

Corrosivity

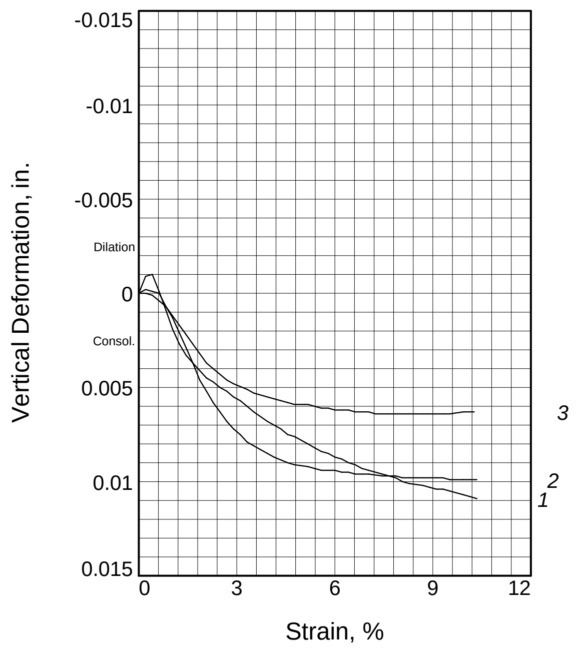
Sample Location	Sulfate[1] (%)	Chloride[2] (ppm)	pH[3]	Resistivity[3] (ohm-cm)
B-1 @ 0 – 5’ (Older Alluvium)	0.0110	62.5	7.8	2,345
B-5 @ 0 – 5’ (Older Alluvium)	0.0112	69.3	8.2	4,422
B-5 @ 12’ (Granitic Bedrock)	0.0063	45.1	8.0	11,250

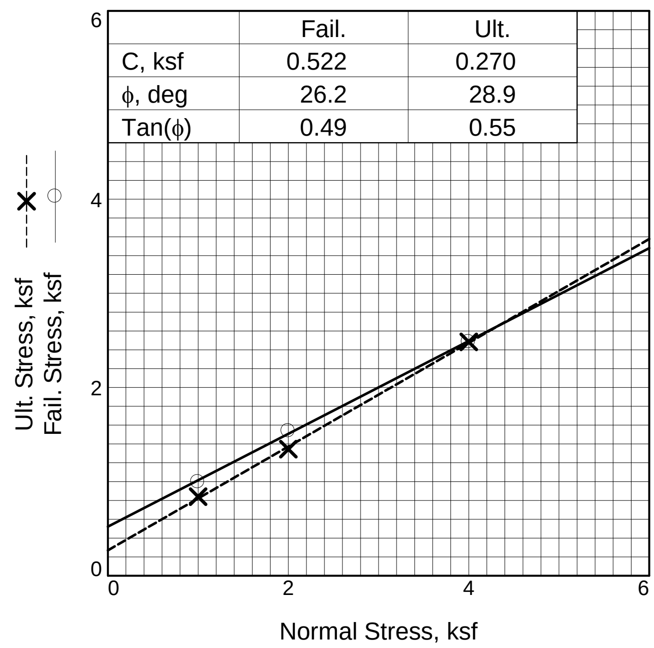
(1) PER CALIFORNIA TEST METHOD NO. 417
(2) PER CALIFORNIA TEST METHOD NO. 422
(3) PER CALIFORNIA TEST METHOD NO. 643

Expansion Index

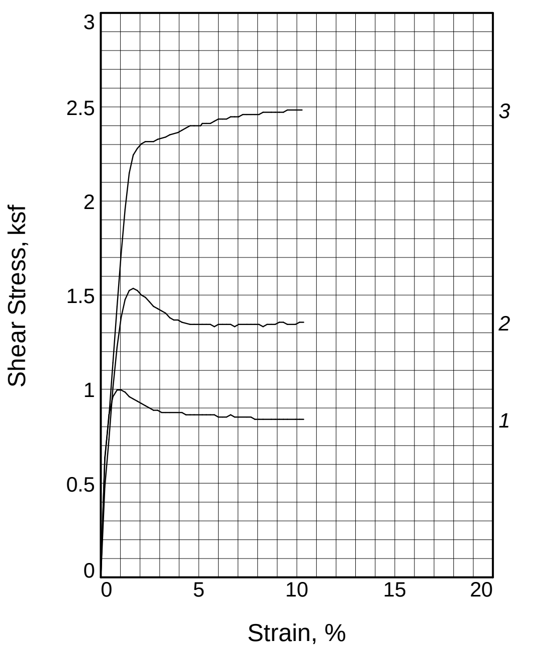
Sample Location Depth (feet)	Soil Type	Expansion[1] Index	Expansion Potential
B-1 @ 0 – 5’	Red brown, Clayey fine- to coarse-grained SAND	27	Low
B-5 @ 0 – 5’	Dark brown, Silty fine- to coarse-grained SAND	0	Very Low

(1) PER ASTM D 4829





	Fail.	Ult.
C, ksf	0.522	0.270
ϕ, deg	26.2	28.9
Tan(ϕ)	0.49	0.55



	Sample No.	1	2	3
Initial	Water Content, %	5.0	5.0	5.0
	Dry Density, pcf	125.0	125.3	124.6
	Saturation, %	40.8	41.2	40.3
	Void Ratio	0.3239	0.3205	0.3279
	Diameter, in.	2.416	2.416	2.416
	Height, in.	1.004	1.002	1.007
At Test	Water Content, %	11.9	10.9	11.3
	Dry Density, pcf	125.3	126.4	126.1
	Saturation, %	98.7	93.7	95.8
	Void Ratio	0.3199	0.3089	0.3118
	Diameter, in.	2.416	2.416	2.416
	Height, in.	1.001	0.993	0.995
	Normal Stress, ksf	1.000	2.000	4.000
	Fail. Stress, ksf	0.996	1.536	2.484
	Strain, %	1.0	1.7	10.3
	Ult. Stress, ksf	0.840	1.344	2.484
	Strain, %	10.3	9.9	10.3
	Strain rate, in./min.	0.040	0.040	0.040

Sample Type: Remolded
Description: Dark Brown Silty Fine to Coarse Sand

Specific Gravity= 2.65
Remarks:

Figure ________

Client: Warmington Residential

Project: Santee

Source of Sample: 23L187 **Depth:** 0-5
Sample Number: B-5
Proj. No.: 23-248 **Date Sampled:**

PETRA GEOSCIENCES INC.

Tested By: DI

APPENDIX C

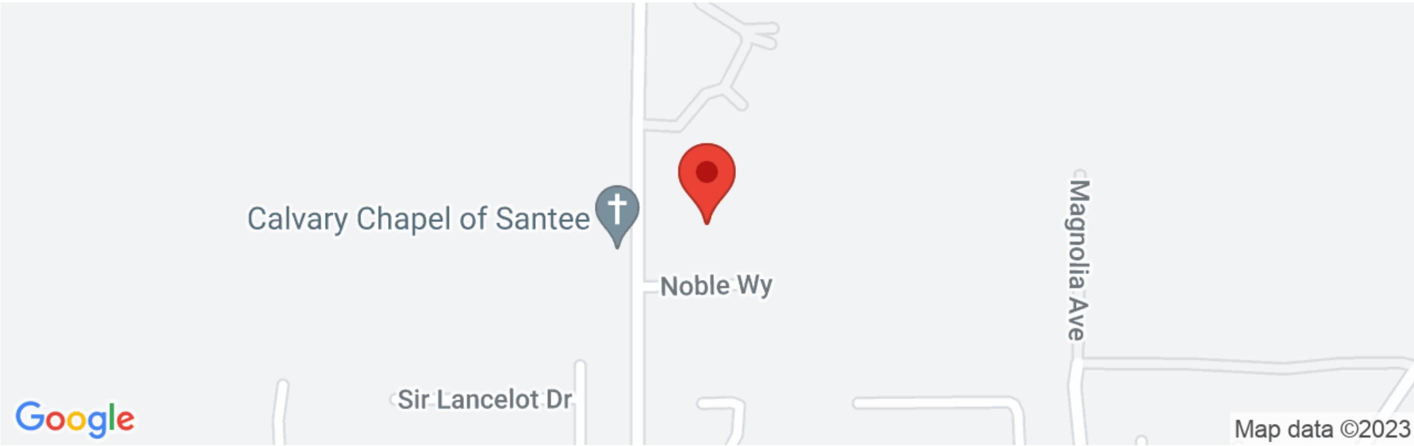
SEISMIC DESIGN PARAMETERS

USGS web services were down for some period of time and as a result this tool wasn't operational, resulting in *timeout* error.
USGS web services are now operational so this tool should work as expected.

23-248

10939 Summit Ave, Santee, CA 92071, USA

Latitude, Longitude: 32.8760469, -116.9747269



Date	8/16/2023, 10:09:25 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	B - Estimated (see Section 11.4.3)

Type	Value	Description
S_S	0.776	MCE_R ground motion. (for 0.2 second period)
S_1	0.286	MCE_R ground motion. (for 1.0s period)
S_{MS}	0.776	Site-modified spectral acceleration value
S_{M1}	0.286	Site-modified spectral acceleration value
S_{DS}	0.517	Numeric seismic design value at 0.2 second SA
S_{D1}	0.19	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	1	Site amplification factor at 1.0 second
PGA	0.332	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.332	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	0.776	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	0.837	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.286	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.308	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.332	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.927	Mapped value of the risk coefficient at short periods
C_{R1}	0.928	Mapped value of the risk coefficient at a period of 1 s
C_V	0.9	Vertical coefficient

DISCLAIMER

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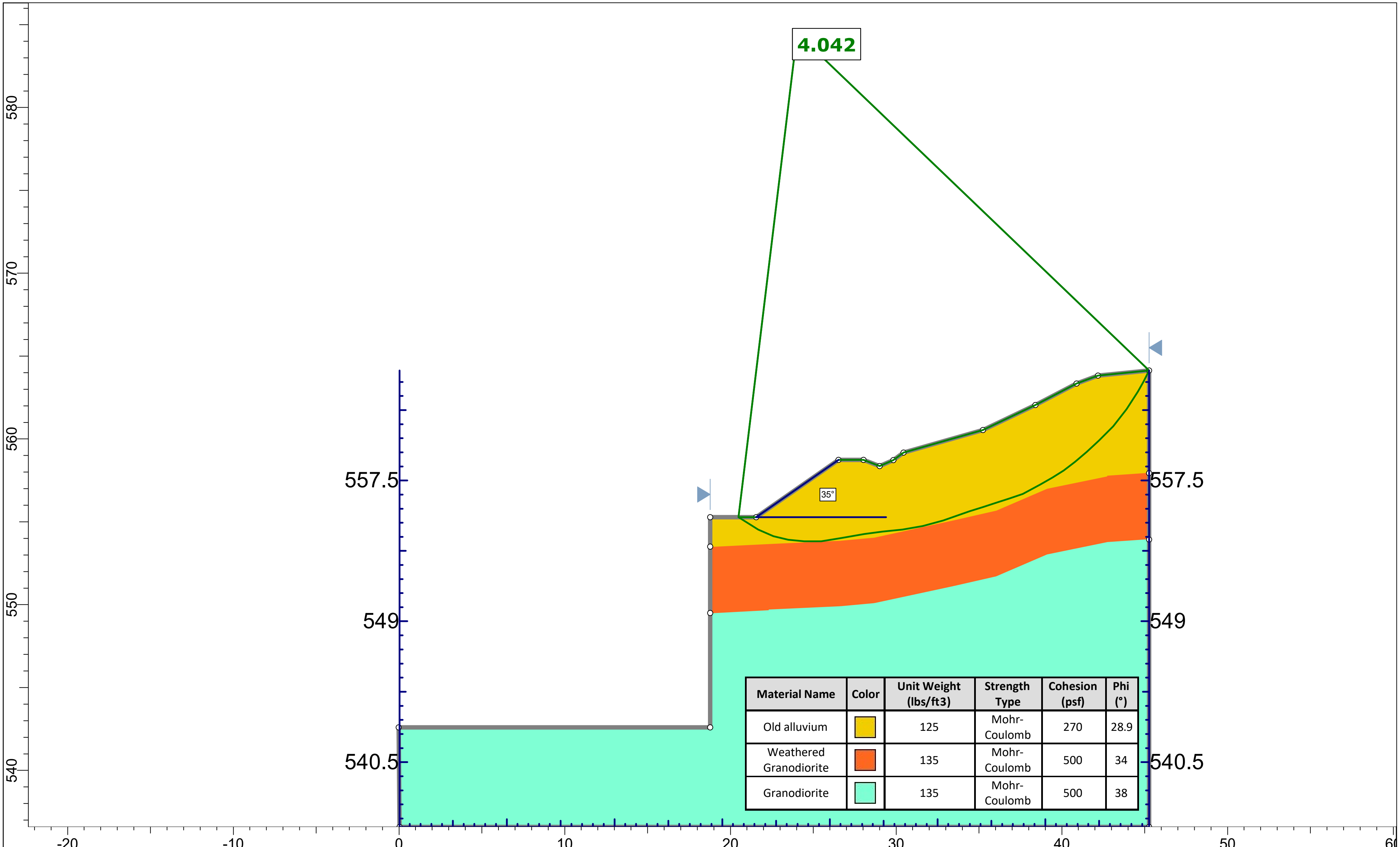
APPENDIX D

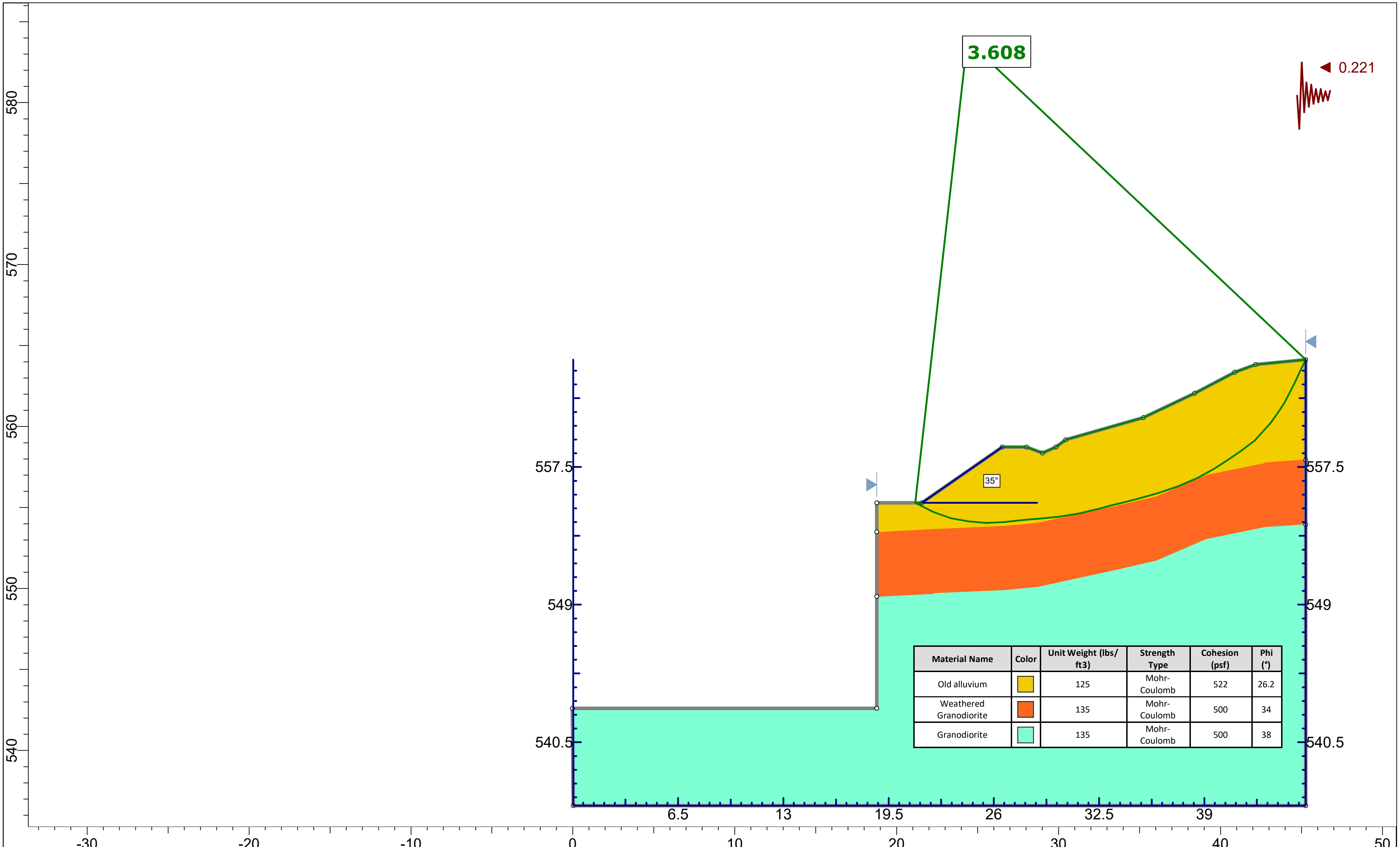
SLOPE STABILITY ANALYSIS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Old alluvium		125	Mohr-Coulomb	270	28.9
Weathered Granodiorite		135	Mohr-Coulomb	500	34
Granodiorite		135	Mohr-Coulomb	500	38

Project	Slide2 - An Interactive Slope Stability Program		
Group	Group 1	*Scenario*	Failure above wall
Drawn By		*Company*	
Date	10/10/2024, 10:37:03 AM	*File Name*	A-A' static.slmd

SLIDEINTERPRET 9.037

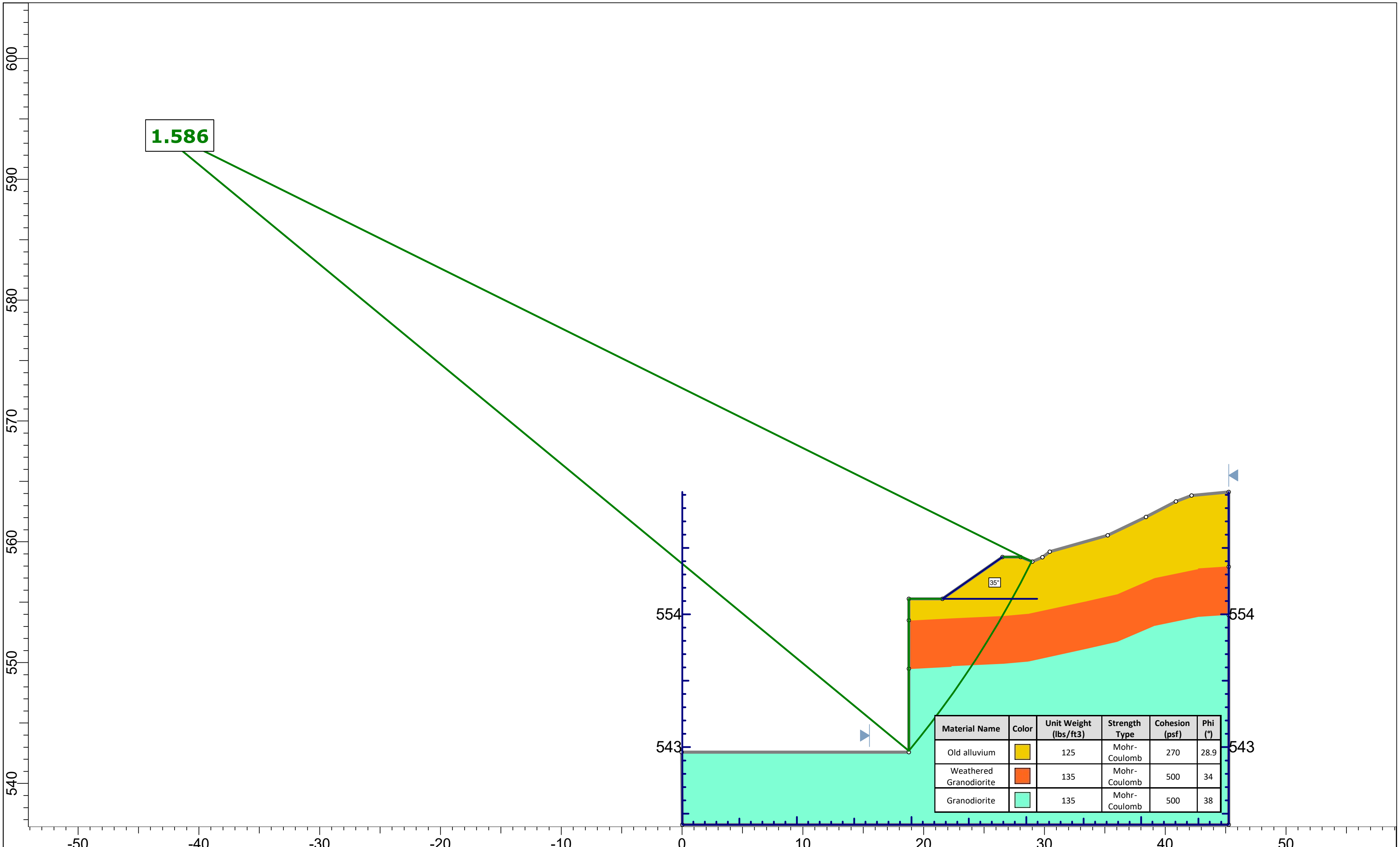


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Old alluvium		125	Mohr-Coulomb	522	26.2
Weathered Granodiorite		135	Mohr-Coulomb	500	34
Granodiorite		135	Mohr-Coulomb	500	38

rocscience

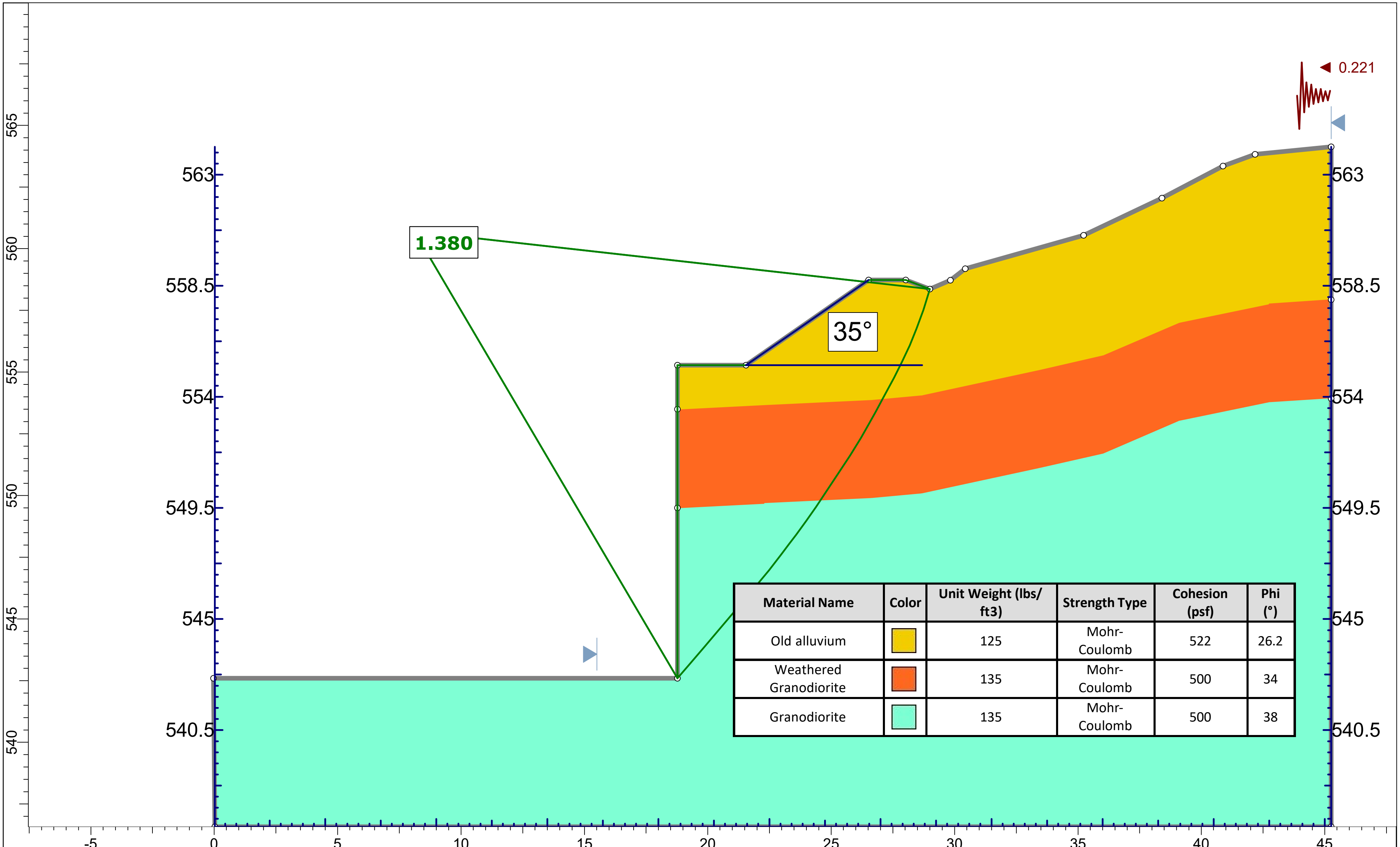
SLIDEINTERPRET 9.037

Project	Slide2 - An Interactive Slope Stability Program		
Group	Group 1	*Scenario*	Failure above wall
Drawn By		*Company*	
Date	10/10/2024, 10:37:03 AM	*File Name*	A-A' seismic.slmd



Project	Slide2 - An Interactive Slope Stability Program	
Group Group 1	*Scenario* Global Failure	
Drawn By	*Company*	
Date 10/10/2024, 10:37:03 AM	*File Name* A-A' static.slmd	

SLIDEINTERPRET 9.037

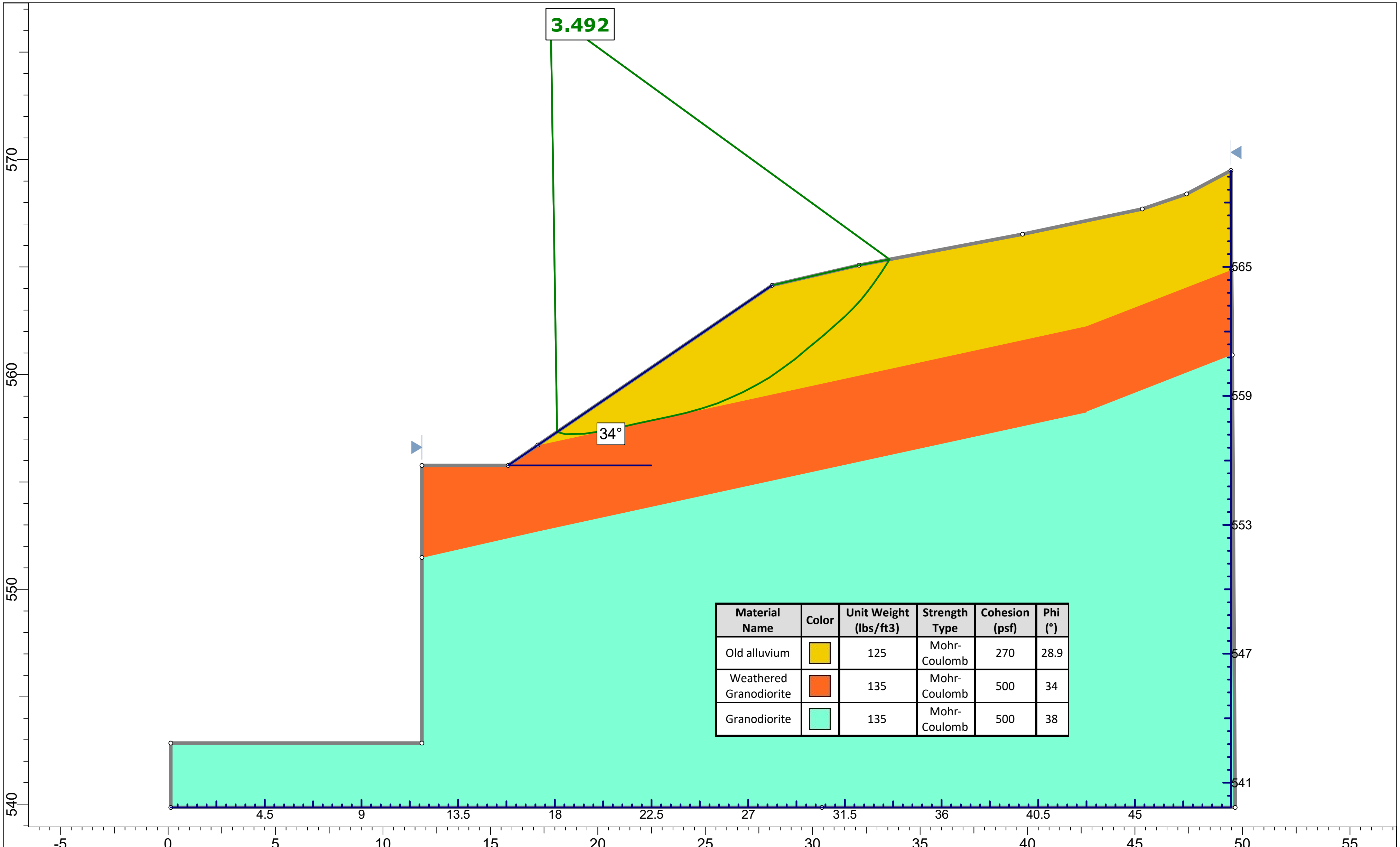


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Old alluvium		125	Mohr-Coulomb	522	26.2
Weathered Granodiorite		135	Mohr-Coulomb	500	34
Granodiorite		135	Mohr-Coulomb	500	38

rocscience

SLIDEINTERPRET 9.037

Project	Slide2 - An Interactive Slope Stability Program		
Group	Group 1	*Scenario*	Global Failure
Drawn By		*Company*	
Date	10/10/2024, 10:37:03 AM	*File Name*	A-A' seismic.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Old alluvium		125	Mohr-Coulomb	270	28.9
Weathered Granodiorite		135	Mohr-Coulomb	500	34
Granodiorite		135	Mohr-Coulomb	500	38

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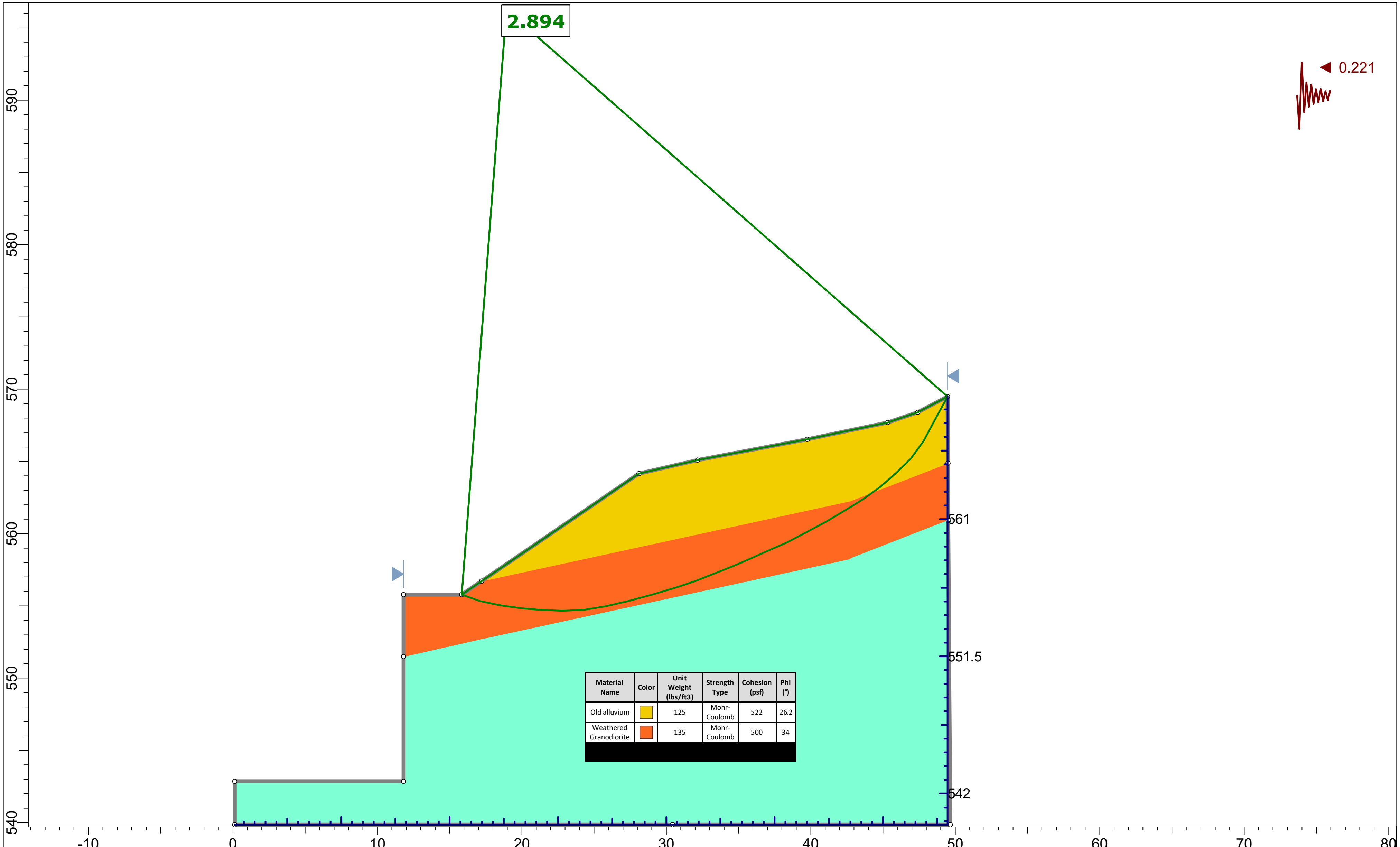
SLIDEINTERPRET 9.037

Project	Slide2 - An Interactive Slope Stability Program		
Group	Group 1	*Scenario*	Failure above wall
Drawn By		*Company*	
Date	10/10/2024, 10:37:03 AM	*File Name*	B-B' static.slmd



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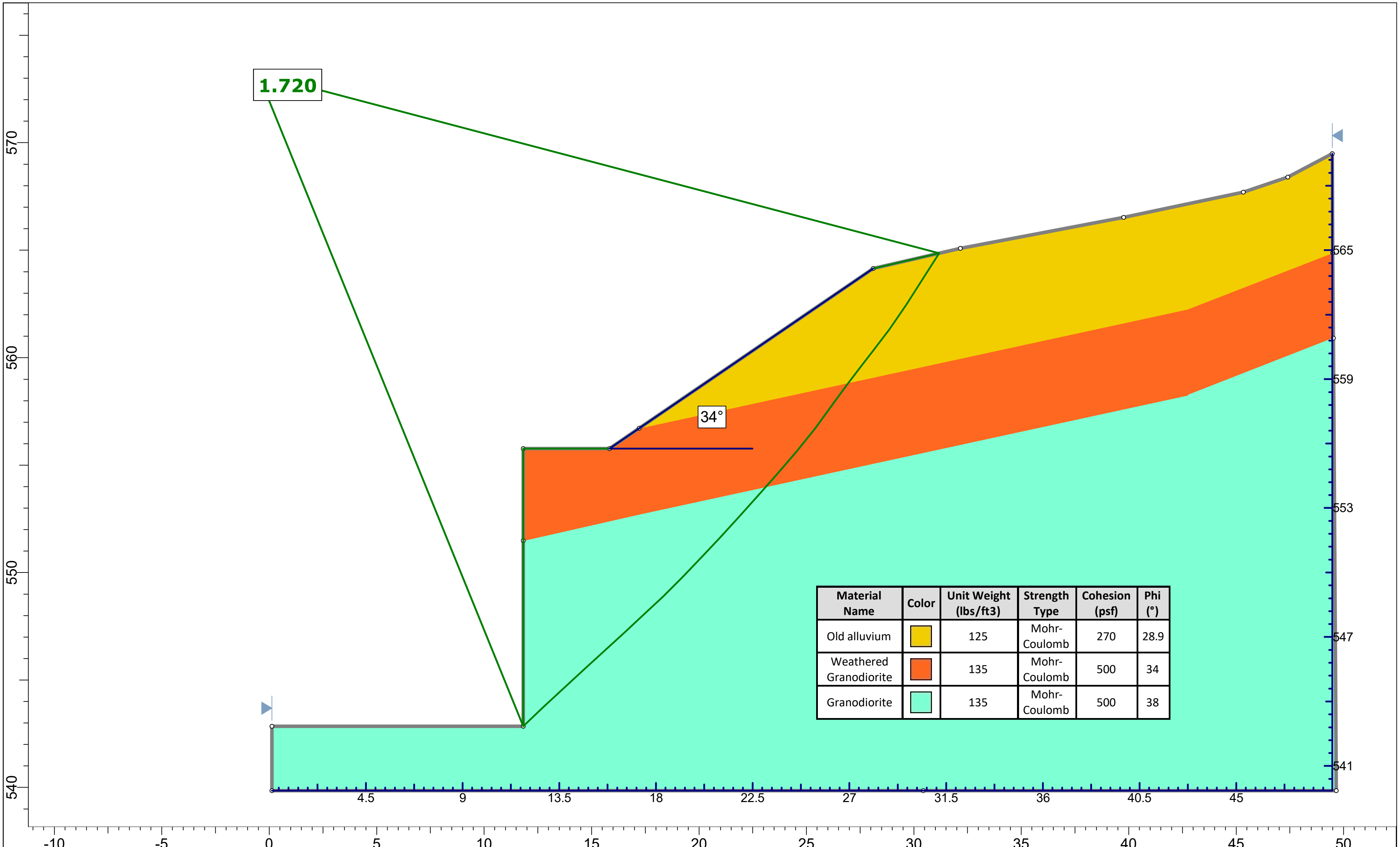
SLIDEINTERPRET 9.037

Project Slide2 - An Interactive Slope Stability Program	
Group Group 1	*Scenario* Seismic
Drawn By	*Company*
Date 10/10/2024, 10:37:03 AM	*File Name* B-B' seismic.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Old alluvium		125	Mohr-Coulomb	270	28.9
Weathered Granodiorite		135	Mohr-Coulomb	500	34
Granodiorite		135	Mohr-Coulomb	500	38

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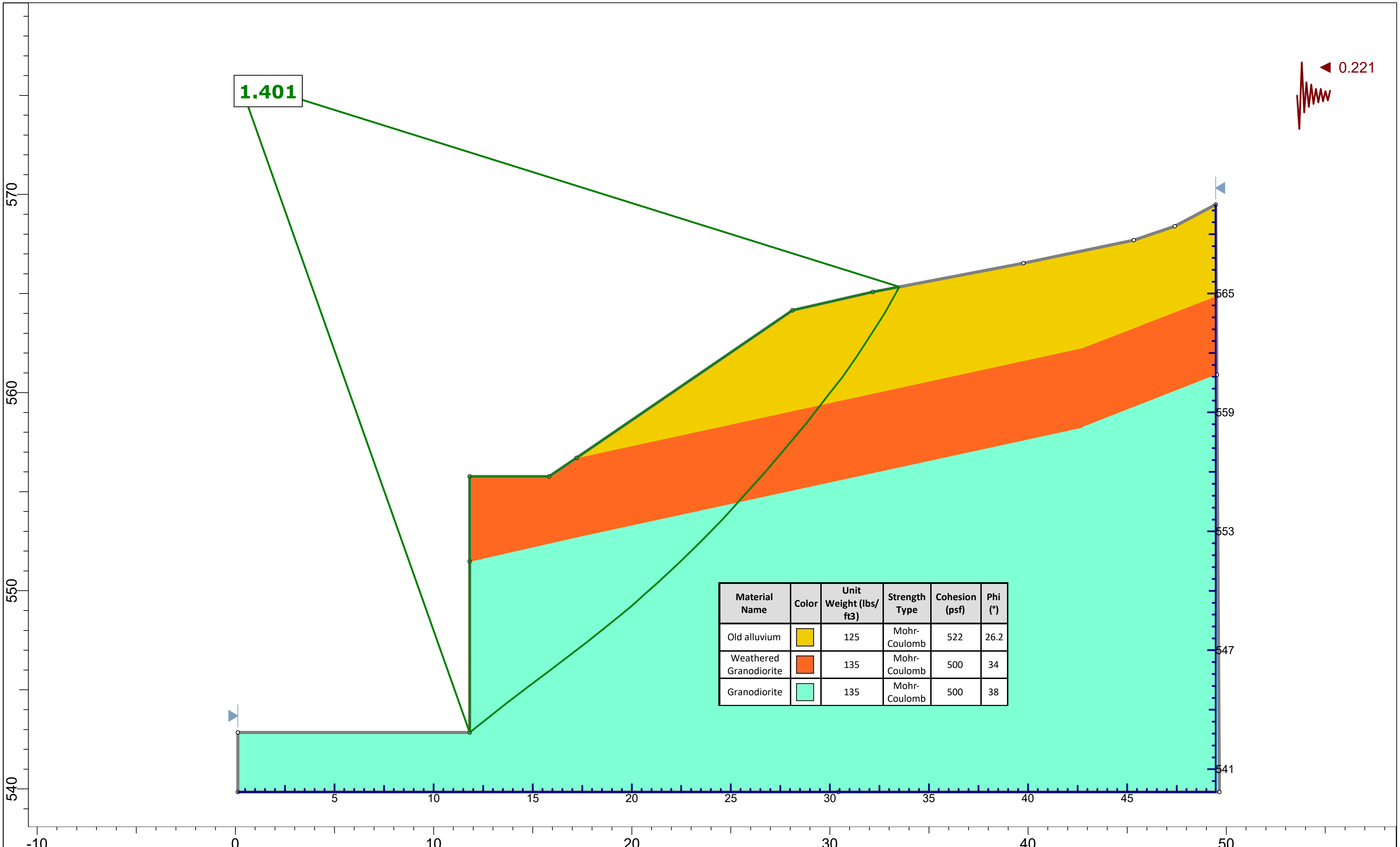
SLIDEINTERPRET 9.037

Project	Slide2 - An Interactive Slope Stability Program
Group Group 1	*Scenario* Global Failure
Drawn By	*Company*
Date 10/10/2024, 10:37:03 AM	*File Name* B-B' static.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Old alluvium		125	Mohr-Coulomb	522	26.2
Weathered Granodiorite		135	Mohr-Coulomb	500	34
Granodiorite		135	Mohr-Coulomb	500	38

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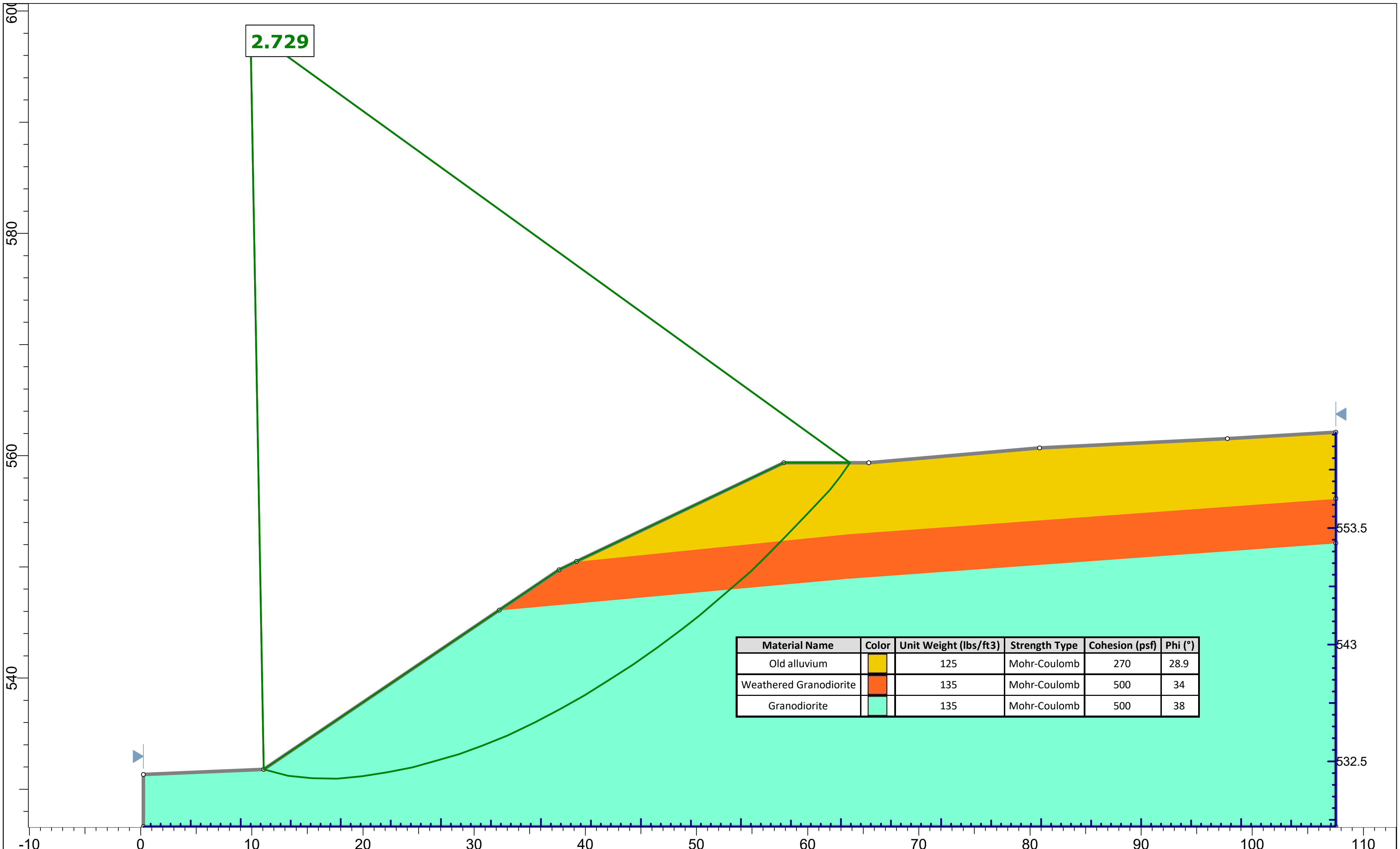
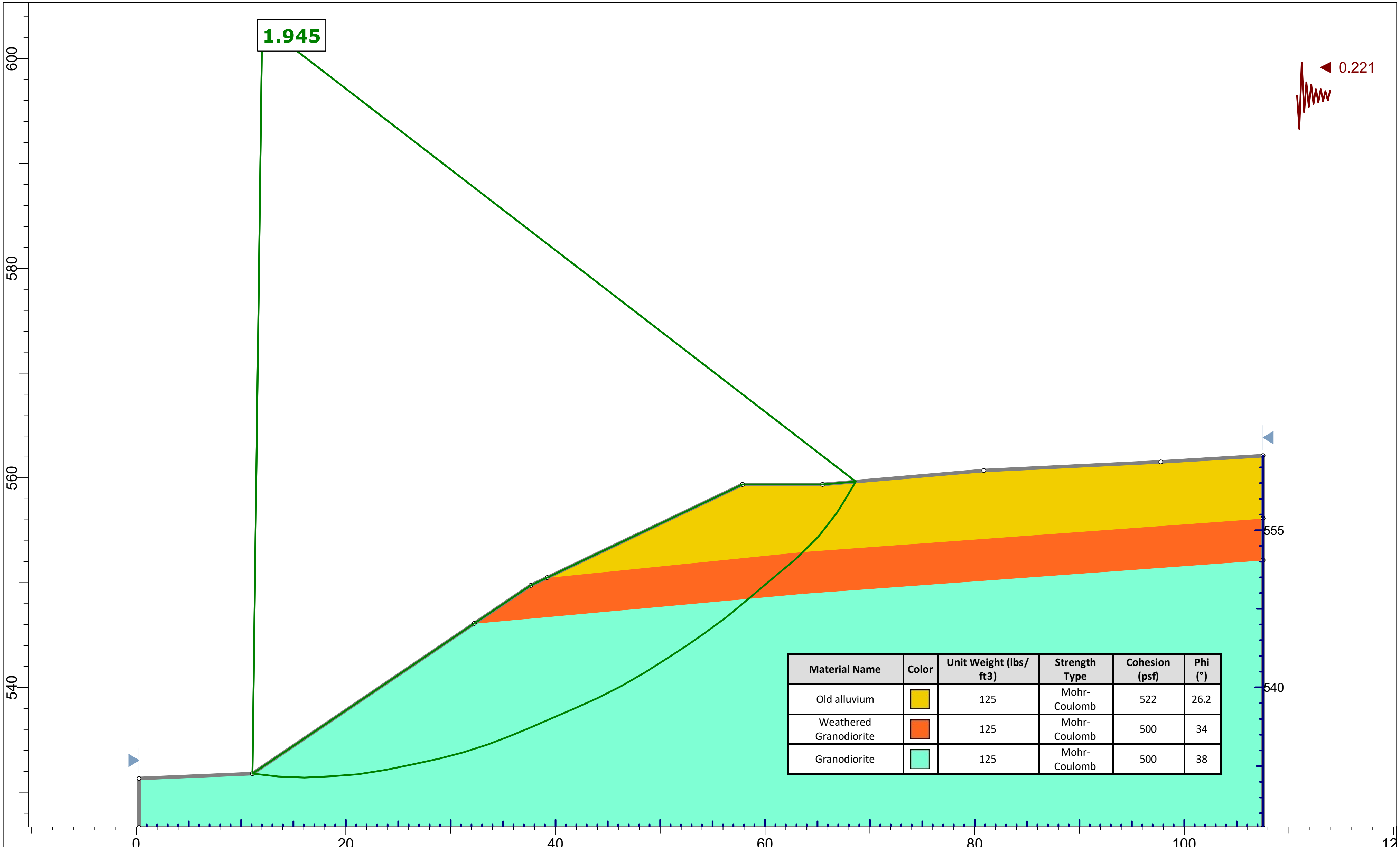
SLIDEINTERPRET 9.037

Project	Slide2 - An Interactive Slope Stability Program		
Group	Group 1	*Scenario*	Global Failure
Drawn By		*Company*	
Date	10/10/2024, 10:37:03 AM	*File Name*	B-B' seismic.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Old alluvium		125	Mohr-Coulomb	270	28.9
Weathered Granodiorite		135	Mohr-Coulomb	500	34
Granodiorite		135	Mohr-Coulomb	500	38

rocscience	*Project* Slide2 - An Interactive Slope Stability Program		
	Group Group 1	*Scenario* sTATIC	
	Drawn By	*Company*	
SLIDEINTERPRET 9.037	*Date* 10/10/2024, 10:37:03 AM	*File Name* C-C' static.slmd	



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Old alluvium		125	Mohr-Coulomb	522	26.2
Weathered Granodiorite		125	Mohr-Coulomb	500	34
Granodiorite		125	Mohr-Coulomb	500	38

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SLIDEINTERPRET 9.037

Project	Slide2 - An Interactive Slope Stability Program		
Group	Group 1	*Scenario*	Seismic
Drawn By		*Company*	
Date	10/10/2024, 10:37:03 AM	*File Name*	C-C' seismic.slmd

APPENDIX E

PERCOLATION TEST DATA

Test Number: B-2

Deep Percolation Test Method

Total Depth of Boring, D_t (ft): 10.5
Diameter of Hole, D (in): 8
Diameter of Pipe, d (in): 3
Agg. Correction (% Voids): 40
Pre-soak depth (ft): 5-9

Time Interval (min)	Depth to Water Surface D_w (ft)		Change in Head (in)	Perc. Rate (min/in)	Perc. Rate (gal/day/ft^2)
	1st Reading	2nd Reading			
25	5.00	5.62	7.44	3.36	5.59
25	5.00	5.45	5.40	4.63	3.97
30	5.00	5.52	6.24	4.81	3.86
30	5.00	5.48	5.76	5.21	3.54
30	5.00	5.46	5.52	5.43	3.39
30	5.00	5.44	5.28	5.68	3.23
30	5.00	5.44	5.28	5.68	3.23
30	5.00	5.44	5.28	5.68	3.23
30	5.00	5.44	5.28	5.68	3.23
30	5.00	5.45	5.40	5.56	3.31
30	5.00	5.44	5.28	5.68	3.23
30	5.00	5.44	5.28	5.68	3.23
30	5.00	5.44	5.28	5.68	3.23
30	5.00	5.44	5.28	5.68	3.23

Percolation Rate: 5.68 Minutes/Inch

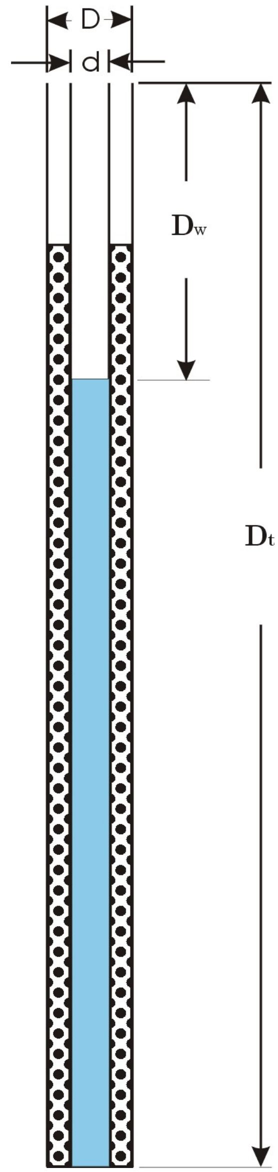
3.23 gal/day/ft^2

Infiltration Rate: 0.45 Inches/Hour*
(Porchet Method)

where Infiltration Rate, $I_t = \Delta H\ (60r) / \Delta t\ (r + 2H_{avg})$

$r = D / 2$
$H_o = D_t - D_o$
$H_f = D_t - D_f$
$\Delta H = \Delta D = H_o - H_f$
$H_{avg} = (H_o + H_f) / 2$

*Raw Number, Does Not Include a Factor of Safety



Reference: San Diego County BMP Design Manual, Appendix D, effective January 1, 2019

PETRA GEOSCIENCES, INC.
40880 County Center Drive, Suite M
Temecula, CA 92591
PHONE: (951) 600-9271
COSTA MESA TEMECULA VALENCIA PALM DESERT CORONA SAN DIEGO

PERCOLATION TEST SUMMARY

Warmington Residential
Santee, California

DATE: Aug, 2023
J.N.: 23-248

Figure 1

Test Number: B-6

Deep Percolation Test Method

Total Depth of Boring, D_t (ft):	10
Diameter of Hole, D (in):	8
Diameter of Pipe, d (in):	3
Agg. Correction (% Voids):	40
Pre-soak depth (ft):	5-10

Time Interval (min)	Depth to Water Surface D_w (ft)		Change in Head (in)	Perc. Rate (min/in)	Perc. Rate (gal/day/ft^2)
	1st Reading	2nd Reading			
25	6.04	6.62	6.96	3.59	5.26
25	6.05	6.52	5.64	4.43	4.21
30	6.08	6.65	6.84	4.39	4.35
30	6.05	6.62	6.84	4.39	4.31
30	6.10	6.58	5.76	5.21	3.64
30	6.00	6.51	6.12	4.90	3.78
30	6.06	6.53	5.64	5.32	3.52
30	6.05	6.52	5.64	5.32	3.51
30	6.00	6.48	5.76	5.21	3.54
30	6.02	6.49	5.64	5.32	3.48
30	6.10	6.59	5.88	5.10	3.72
30	6.05	6.51	5.52	5.43	3.43
30	6.00	6.48	5.76	5.21	3.54
30	6.00	6.48	5.76	5.21	3.54

Percolation Rate: 5.21 Minutes/Inch

3.54 gal/day/ft^2

Infiltration Rate: 0.49 Inches/Hour*
(Porchet Method)

where Infiltration Rate, $I_t = \Delta H\,(60r) / \Delta t\,(r + 2H_{avg})$

$r = D / 2$

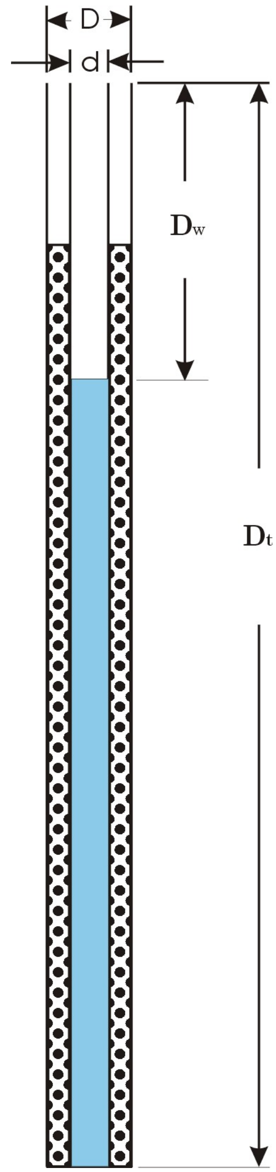
$H_o = D_t - D_o$

$H_f = D_t - D_f$

$\Delta H = \Delta D = H_o - H_f$

$H_{avg} = (H_o + H_f) / 2$

*Raw Number, Does Not Include a Factor of Safety



Reference: San Diego County BMP Design Manual, Appendix D, effective January 1, 2019

PETRA GEOSCIENCES, INC.
40880 County Center Drive, Suite M
Temecula, CA 92591
PHONE: (951) 600-9271
COSTA MESA TEMECULA VALENCIA PALM DESERT CORONA SAN DIEGO

PERCOLATION TEST SUMMARY

Warmington Residential
Santee, California

DATE: April, 2024
J.N.: 23-248

Figure 2

Categorization of Infiltration Feasibility Condition			Form I-8
Part 1 - Full Infiltration Feasibility Screening Criteria **Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?**			
Criteria	Screening Question	**Yes**	**No**
1	**Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		No
Provide basis: The site will be graded such that compacted fill will cover the majority of the area and underlain by relatively impermeable older alluvium granitic bedrock. The compacted fill and granitic bedrock possess little void space and therefore the infiltration rate is considered to be less than 0.5 inches per hour. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
2	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		No
Provide basis: The site will be graded such that compacted fill will cover the majority of the area and underlain by relatively impermeable older alluvium granitic bedrock. The compacted fill and granitic bedrock possess little void space. Areas at the site could not be designed for infiltration without increasing risk of geotechnical hazards. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			

Form I-8 Page 2 of 4			
Criteria	Screening Question	**Yes**	**No**
3	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	Yes	

Provide basis:

Based on Appendix C.3 screening, the site is not known to be in an area of groundwater contamination and/or soil pollution. Groundwater is greater than 10 feet below existing ground surface.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.

4	**Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.		No

Provide basis:

Infiltration should not be allowed due to presence of nearby Woodglen Vista Creek.
.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.

Part 1 Result *	If all answers to rows 1 - 4 are "**Yes**" a full infiltration design is potentially feasible. The feasibility screening category is **Full Infiltration** If any answer from row 1-4 is "**No**", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design. Proceed to Part 2	No

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

Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria

Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
5	**Do soil and geologic conditions allow for infiltration in any appreciable rate or volume?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		No

Provide basis:

The site will be graded such that compacted fill will cover the majority of the area as well as be underlain by relatively impermeable older alluvium and granitic bedrock. The compacted fill, older alluvium, and granitic bedrock possess litlle void space. Areas at the site could be designed for infiltration rates less than 0.5 inches per hour.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

Criteria	Screening Question	Yes	No
6	**Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level**? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		No

Provide basis:

The site will be graded such that compacted fill will cover the majority of the area and underlain by relatively impermeable older alluvium and granitic bedrock. The compacted fill and granitic bedrock possess little void space. Areas at the site should not be designed for infiltration without increasing due to risk of geotechnical hazards.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

Form I-8 Page 4 of 4			
Criteria	Screening Question	**Yes**	**No**
7	**Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.		No
Provide basis: Based on Appendix C.3 screening, the site is not known to be in an area of groundwater contamination and/or soil pollution. Groundwater is greater than 10 feet below existing ground surface. However, infiltration may initiate perched groundwater creating a shallow water table and/or associated storm water pollutants. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.			
8	**Can infiltration be allowed without violating downstream water rights**? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.		No
Provide basis: Based on Appendix C.3 screening, the site is not known to be in an area which would impact downstrean water rights. However, nearby Woodglen Vista Creek may be impacted. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.			
Part 2 Result*	If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is **Partial Infiltration.** If any answer from row 5-8 is no, then infiltration of any volume is considered to be **infeasible** within the drainage area. The feasibility screening category is **No Infiltration.**		No Infiltration

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

APPENDIX F

STANDARD GRADING SPECIFICATIONS

STANDARD GRADING SPECIFICATIONS

These specifications present the usual and minimum requirements for projects on which Petra Geosciences, Inc. (Petra) is the geotechnical consultant. No deviation from these specifications will be allowed, except where specifically superseded in the preliminary geology and soils report, or in other written communication signed by the Soils Engineer and Engineering Geologist of record (Geotechnical Consultant).

I. GENERAL

A. The Geotechnical Consultant is the Owner's or Builder's representative on the project. For the purpose of these specifications, participation by the Geotechnical Consultant includes that observation performed by any person or persons employed by, and responsible to, the licensed Soils Engineer and Engineering Geologist signing the soils report.

B. The contractor should prepare and submit to the Owner and Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" and the estimated quantities of daily earthwork to be performed prior to the commencement of grading. This work plan should be reviewed by the Geotechnical Consultant to schedule personnel to perform the appropriate level of observation, mapping, and compaction testing as necessary.

C. All clearing, site preparation, or earthwork performed on the project shall be conducted by the Contractor in accordance with the recommendations presented in the geotechnical report and under the observation of the Geotechnical Consultant.

D. It is the Contractor's responsibility to prepare the ground surface to receive the fills to the satisfaction of the Geotechnical Consultant and to place, spread, mix, water, and compact the fill in accordance with the specifications of the Geotechnical Consultant. The Contractor shall also remove all material considered unsatisfactory by the Geotechnical Consultant.

E. It is the Contractor's responsibility to have suitable and sufficient compaction equipment on the job site to handle the amount of fill being placed. If necessary, excavation equipment will be shut down to permit completion of compaction to project specifications. Sufficient watering apparatus will also be provided by the Contractor, with due consideration for the fill material, rate of placement, and time of year.

F. After completion of grading a report will be submitted by the Geotechnical Consultant.

II. SITE PREPARATION

A. Clearing and Grubbing

1. All vegetation such as trees, brush, grass, roots, and deleterious material shall be disposed of offsite. This removal shall be concluded prior to placing fill.

2. Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipe lines, etc., are to be removed or treated in a manner prescribed by the Geotechnical Consultant.

III. FILL AREA PREPARATION

A. Remedial Removals/Overexcavations

1. Remedial removals, as well as overexcavation for remedial purposes, shall be evaluated by the Geotechnical Consultant. Remedial removal depths presented in the geotechnical report and shown on the geotechnical plans are estimates only. The actual extent of removal should be determined by the Geotechnical Consultant based on the conditions exposed during grading. All soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as determined by the Geotechnical Consultant.

2. Soil, alluvium, or bedrock materials determined by the Soils Engineer as being unsuitable for placement in compacted fills shall be removed from the site. Any material incorporated as a part of a compacted fill must be approved by the Geotechnical Consultant.

3. Should potentially hazardous materials be encountered, the Contractor should stop work in the affected area. An environmental consultant specializing in hazardous materials should be notified immediately for evaluation and handling of these materials prior to continuing work in the affected area.

B. Evaluation/Acceptance of Fill Areas

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

C. Processing

After the ground surface to receive fill has been declared satisfactory for support of fill by the Geotechnical Consultant, it shall be scarified to a minimum depth of 6 inches and until the ground surface is uniform and free from ruts, hollows, hummocks, or other uneven features which may prevent uniform compaction.

The scarified ground surface shall then be brought to optimum moisture, mixed as required, and compacted to a minimum relative compaction of 90 percent.

D. Subdrains

Subdrainage devices shall be constructed in compliance with the ordinances of the controlling governmental agency, and/or with the recommendations of the Geotechnical Consultant. (Typical Canyon Subdrain details are given on Plate SG-1).

E. Cut/Fill & Deep Fill/Shallow Fill Transitions

In order to provide uniform bearing conditions in cut/fill and deep fill/shallow fill transition lots, the cut and shallow fill portions of the lot should be overexcavated to the depths and the horizontal limits discussed in the approved geotechnical report and replaced with compacted fill. (Typical details are given on Plate SG-7.)

IV. COMPACTED FILL MATERIAL

A. General

Materials excavated on the property may be utilized in the fill, provided each material has been determined to be suitable by the Geotechnical Consultant. Material to be used for fill shall be essentially free of organic material and other deleterious substances. Roots, tree branches, and other matter missed during clearing shall be removed from the fill as recommended by the Geotechnical Consultant. Material that is spongy, subject to decay, or otherwise considered unsuitable shall not be used in the compacted fill.

Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

B. Oversize Materials

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 12 inches in diameter, shall be taken offsite or placed in accordance with the recommendations of the Geotechnical Consultant in areas designated as suitable for rock disposal (Typical details for Rock Disposal are given on Plate SG-4).

Rock fragments less than 12 inches in diameter may be utilized in the fill provided, they are not nested or placed in concentrated pockets; they are surrounded by compacted fine grained soil material and the distribution of rocks is approved by the Geotechnical Consultant.

C. Laboratory Testing

Representative samples of materials to be utilized as compacted fill shall be analyzed by the laboratory of the Geotechnical Consultant to determine their physical properties. If any material other than that previously tested is encountered during grading, the appropriate analysis of this material shall be conducted by the Geotechnical Consultant as soon as possible.

D. Import

If importing of fill material is required for grading, proposed import material should meet the requirements of the previous section. The import source shall be given to the Geotechnical Consultant at least 2 working days prior to importing so that appropriate tests can be performed and its suitability determined.

V. FILL PLACEMENT AND COMPACTION

A. Fill Layers

Material used in the compacting process shall be evenly spread, watered, processed, and compacted in thin lifts not to exceed 6 inches in thickness to obtain a uniformly dense layer. The fill shall be placed and compacted on a horizontal plane, unless otherwise approved by the Geotechnical Consultant.

B. Moisture Conditioning

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly above optimum moisture content.

C. Compaction

Each layer shall be compacted to 90 percent of the maximum density in compliance with the testing method specified by the controlling governmental agency. (In general, ASTM D 1557-02, will be used.)

If compaction to a lesser percentage is authorized by the controlling governmental agency because of a specific land use or expansive soils condition, the area to received fill compacted to less than 90 percent shall either be delineated on the grading plan or appropriate reference made to the area in the soils report.

D. Failing Areas

If the moisture content or relative density varies from that required by the Geotechnical Consultant, the Contractor shall rework the fill until it is approved by the Geotechnical Consultant.

E. Benching

All fills shall be keyed and benched through all topsoil, colluvium, alluvium or creep material, into sound bedrock or firm material where the slope receiving fill exceeds a ratio of 5 horizontal to 1 vertical, in accordance with the recommendations of the Geotechnical Consultant.

VI. SLOPES

A. Fill Slopes

The contractor will be required to obtain a minimum relative compaction of 90 percent out to the finish slope face of fill slopes, buttresses, and stabilization fills. This may be achieved by either overbuilding the slope and cutting back to the compacted core, or by direct compaction of the slope face with suitable equipment, or by any other procedure that produces the required compaction.

B. Side Hill Fills

The key for side hill fills shall be a minimum of 15 feet within bedrock or firm materials, unless otherwise specified in the soils report. (See detail on Plate SG-5.)

C. Fill-Over-Cut Slopes

Fill-over-cut slopes shall be properly keyed through topsoil, colluvium or creep material into rock or firm materials, and the transition shall be stripped of all soils prior to placing fill. (see detail on Plate SG-6).

D. Landscaping

All fill slopes should be planted or protected from erosion by other methods specified in the soils report.

E. Cut Slopes

1. The Geotechnical Consultant should observe all cut slopes at vertical intervals not exceeding 10 feet.

2. If any conditions not anticipated in the preliminary report such as perched water, seepage, lenticular or confined strata of a potentially adverse nature, unfavorably inclined bedding, joints or fault planes are encountered during grading, these conditions shall be evaluated by the Geotechnical Consultant, and recommendations shall be made to treat these problems (Typical details for stabilization of a portion of a cut slope are given in Plates SG-2 and SG-3.).

3. Cut slopes that face in the same direction as the prevailing drainage shall be protected from slope wash by a non-erodible interceptor swale placed at the top of the slope.

4. Unless otherwise specified in the soils and geological report, no cut slopes shall be excavated higher or steeper than that allowed by the ordinances of controlling governmental agencies.

5. Drainage terraces shall be constructed in compliance with the ordinances of controlling governmental agencies, or with the recommendations of the Geotechnical Consultant.

VII. GRADING OBSERVATION

A. General

All cleanouts, processed ground to receive fill, key excavations, subdrains, and rock disposals must be observed and approved by the Geotechnical Consultant prior to placing any fill. It shall be the Contractor's responsibility to notify the Geotechnical Consultant when such areas are ready.

B. Compaction Testing

Observation of the fill placement shall be provided by the Geotechnical Consultant during the progress of grading. Location and frequency of tests shall be at the Consultants discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations may be selected to verify adequacy of compaction levels in areas that are judged to be susceptible to inadequate compaction.

C. Frequency of Compaction Testing

In general, density tests should be made at intervals not exceeding 2 feet of fill height or every 1000 cubic yards of fill placed. This criteria will vary depending on soil conditions and the size of the job. In any event, an adequate number of field density tests shall be made to verify that the required compaction is being achieved.

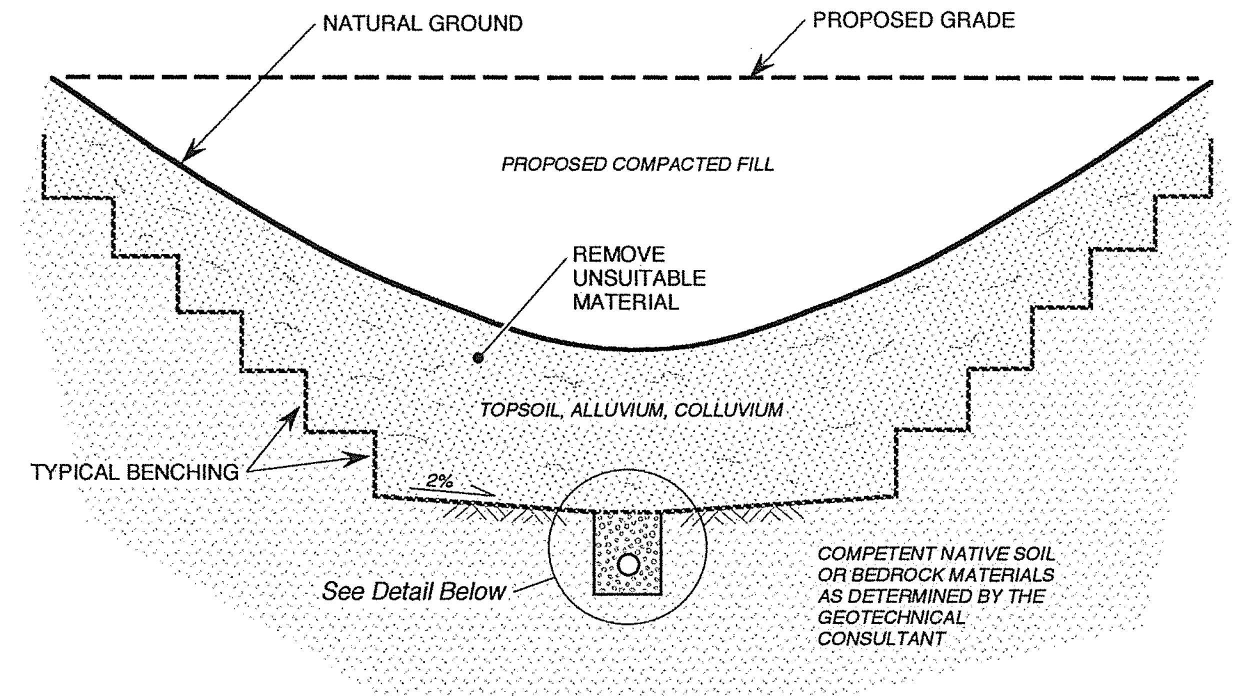
VIII. CONSTRUCTION CONSIDERATIONS

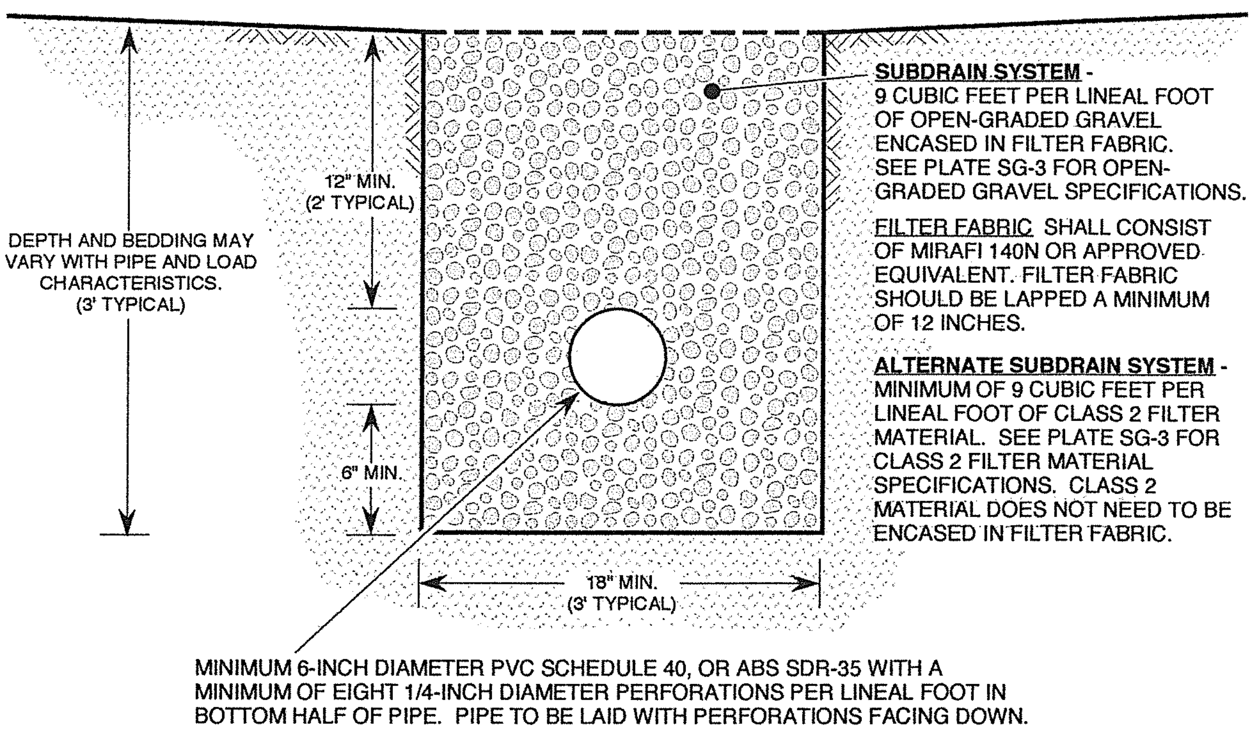
A. Erosion control measures, when necessary, shall be provided by the Contractor during grading and prior to the completion and construction of permanent drainage controls.

B. Upon completion of grading and termination of observations by the Geotechnical Consultant, no further filling or excavating, including that necessary for footings, foundations, large tree wells, retaining walls, or other features shall be performed without the approval of the Geotechnical Consultant.

C. Care shall be taken by the Contractor during final grading to preserve any berms, drainage terraces, interceptor swales, or other devices of permanent nature on or adjacent to the property.

S:\!BOILERS-WORK\REPORT INSERTS\STANDARD GRADING SPECS

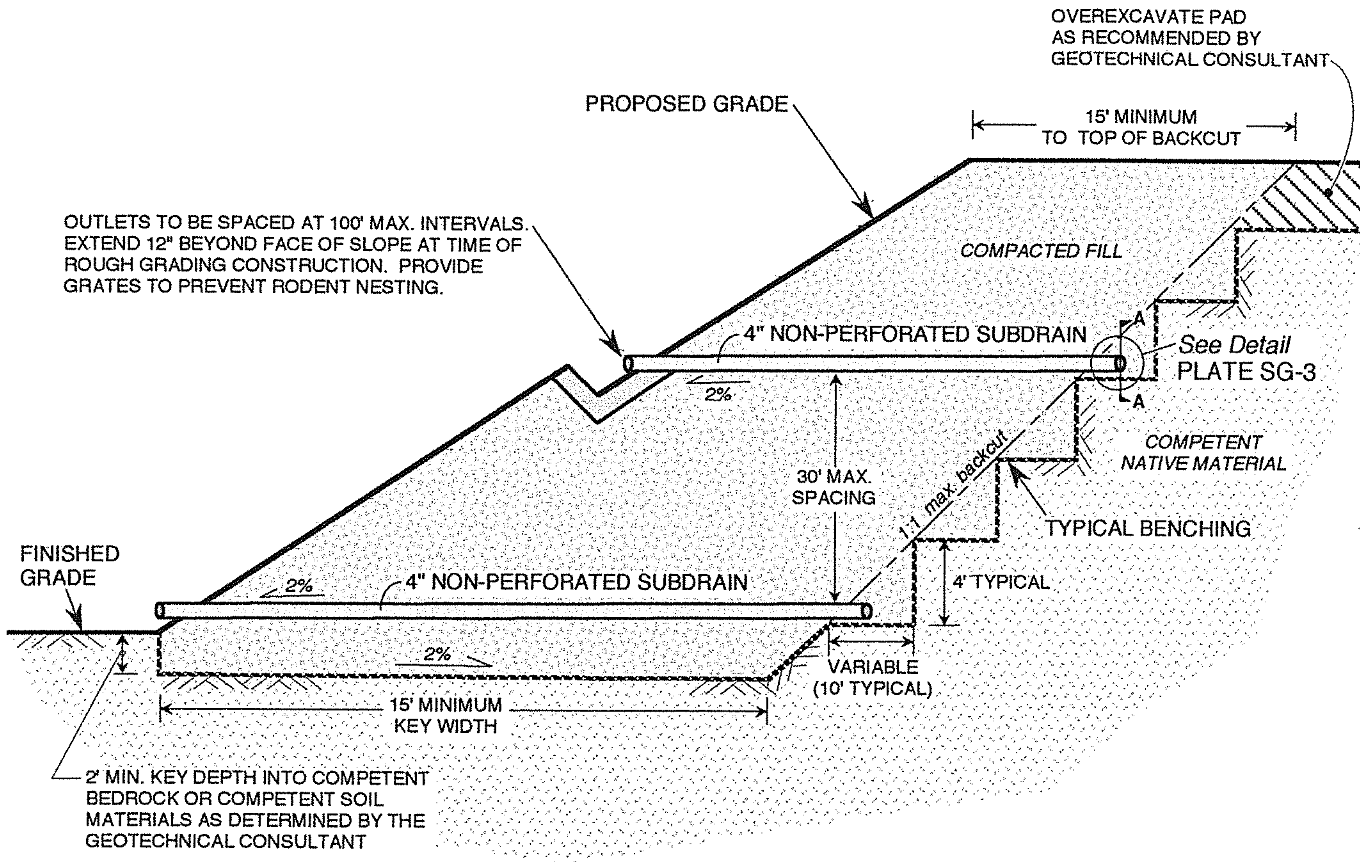




NOTES:

1. FOR CONTINUOUS RUNS IN EXCESS OF 500 FEET USE 8-INCH DIAMETER PIPE.
2. FINAL 20 FEET OF PIPE AT OUTLET SHALL BE NON-PERFORATED AND BACKFILLED WITH FINE-GRAINED MATERIAL.

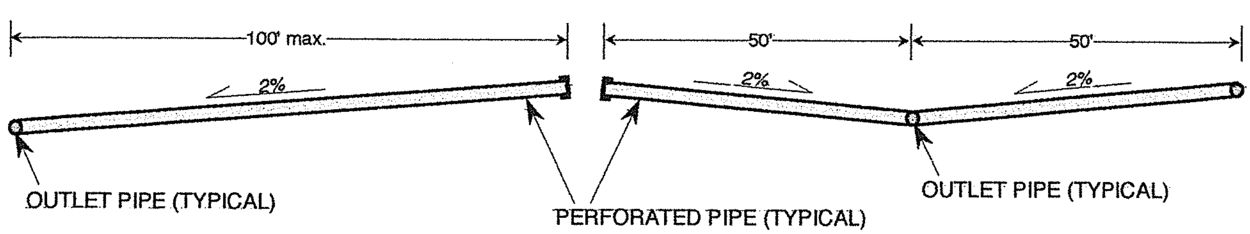
PETRA	**CANYON SUBDRAIN DETAIL**	**PLATE SG-1**



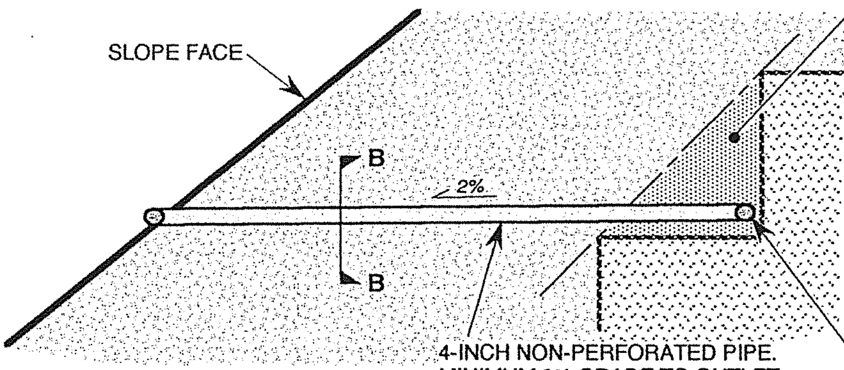
NOTES:

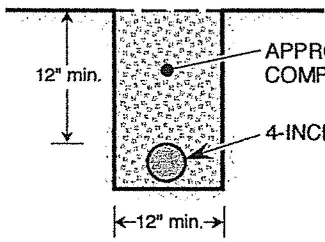
1. 30' MAXIMUM VERTICAL SPACING BETWEEN SUBDRAIN SYSTEMS.
2. 100' MAXIMUM HORIZONTAL DISTANCE BETWEEN NON-PERFORATED OUTLET PIPES. *(See Below)*
3. MINIMUM GRADIENT OF 2% FOR ALL PERFORATED AND NON-PERFORATED PIPE.

SECTION A-A (PERFORATED PIPE PROFILE)



PETRA	**BUTTRESS OR STABILIZATION FILL DETAIL**	**PLATE SG-2**





SECTION B-B (OUTLET PIPE)

PIPE SPECIFICATIONS:

1. 4-INCH MINIMUM DIAMETER, PVC SCHEDULE 40 OR ABS SDR-35.
2. FOR PERFORATED PIPE, MINIMUM 8 PERFORATIONS PER FOOT ON BOTTOM HALF OF PIPE.

FILTER MATERIAL/FABRIC SPECIFICATIONS:

OPEN-GRADED GRAVEL ENCASED IN FILTER FABRIC.
(MIRAFI 140N OR EQUIVALENT)

OPEN-GRADED GRAVEL

SIEVE SIZE	PERCENT PASSING
1 1/2-INCH	88 - 100
1-INCH	5 - 40
3/4-INCH	0 - 17
3/8-INCH	0 - 7
No. 200	0 - 3

ALTERNATE:

CLASS 2 PERMEABLE FILTER MATERIAL PER CALTRANS STANDARD SPECIFICATION 68-1.025.

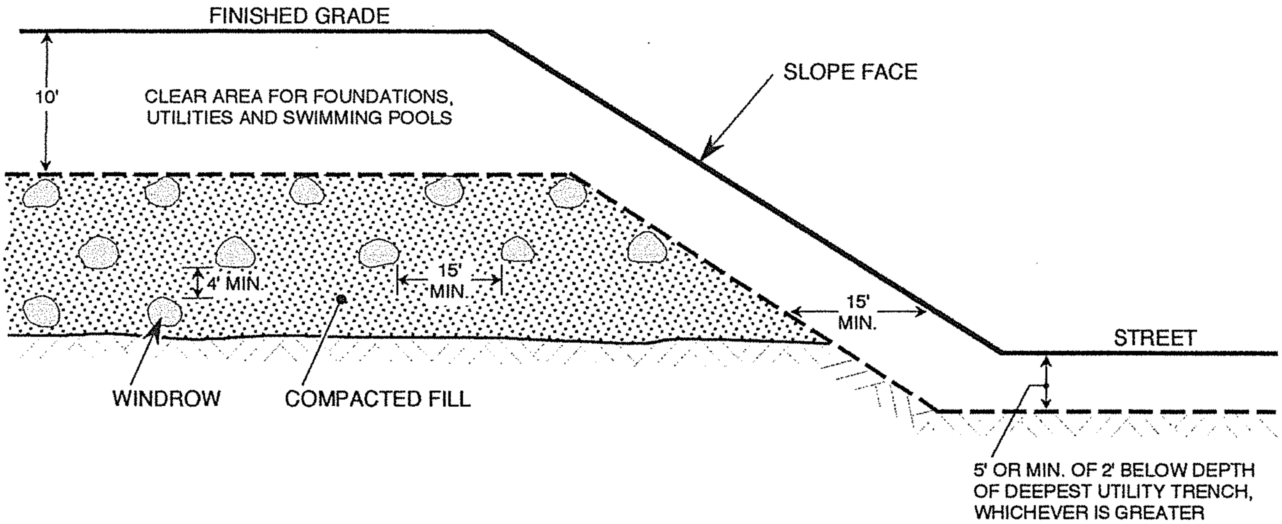
CLASS 2 FILTER MATERIAL

SIEVE SIZE	PERCENT PASSING
1-INCH	100
3/4-INCH	90 - 100
3/8-INCH	40 - 100
No. 4	25 - 40
No. 8	18 - 33
No. -30	5 - 15
No. -50	0 - 7
No. 200	0 - 3

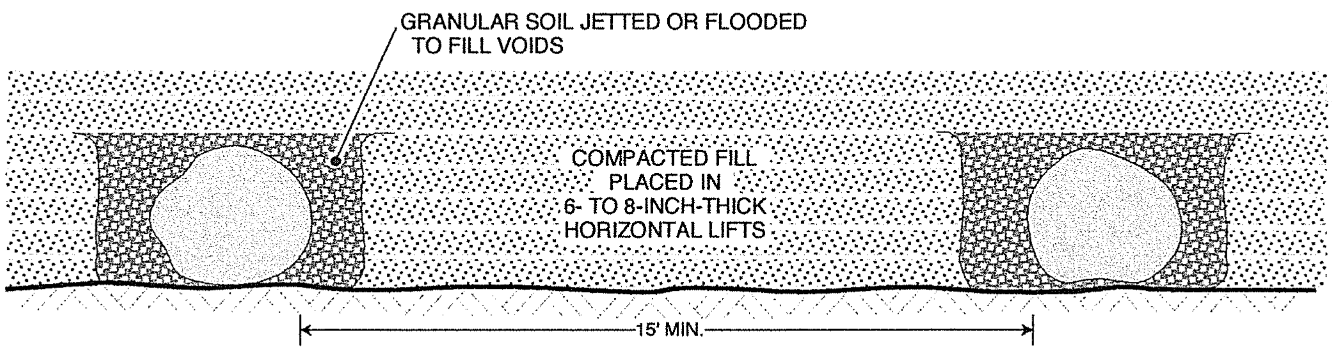
PETRA

BUTTRESS OR STABILIZATION FILL SUBDRAIN

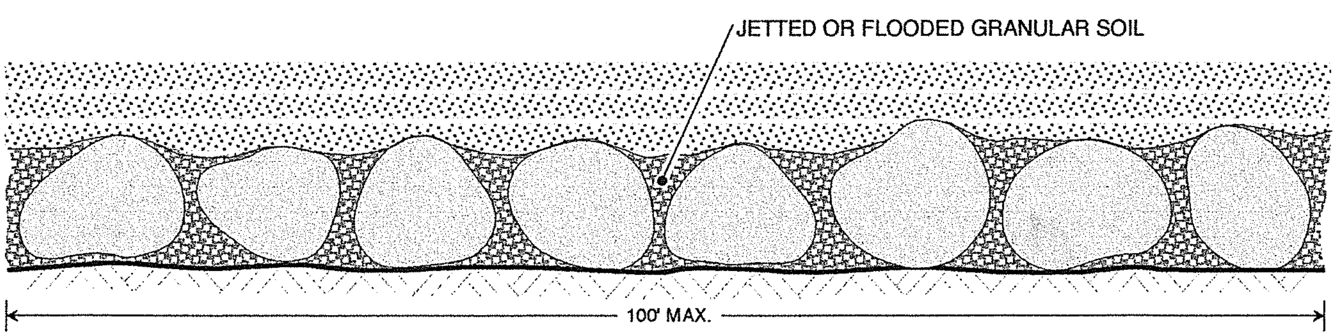
PLATE SG-3



TYPICAL WINDROW DETAIL (END VIEW)

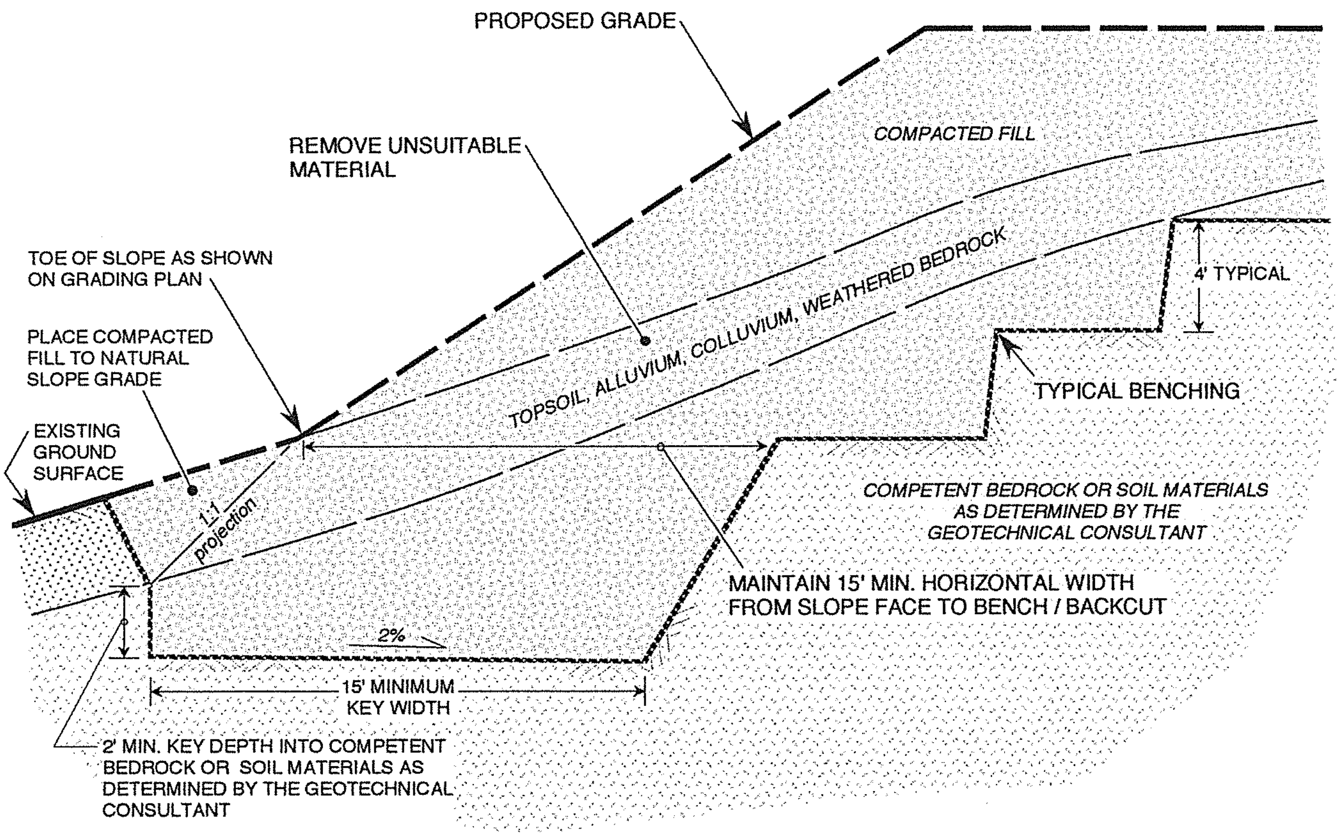


TYPICAL WINDROW DETAIL (PROFILE VIEW)



NOTE: OVERSIZE ROCK IS DEFINED AS CLASTS HAVING A MAXIMUM DIMENSION OF 12" OR LARGER

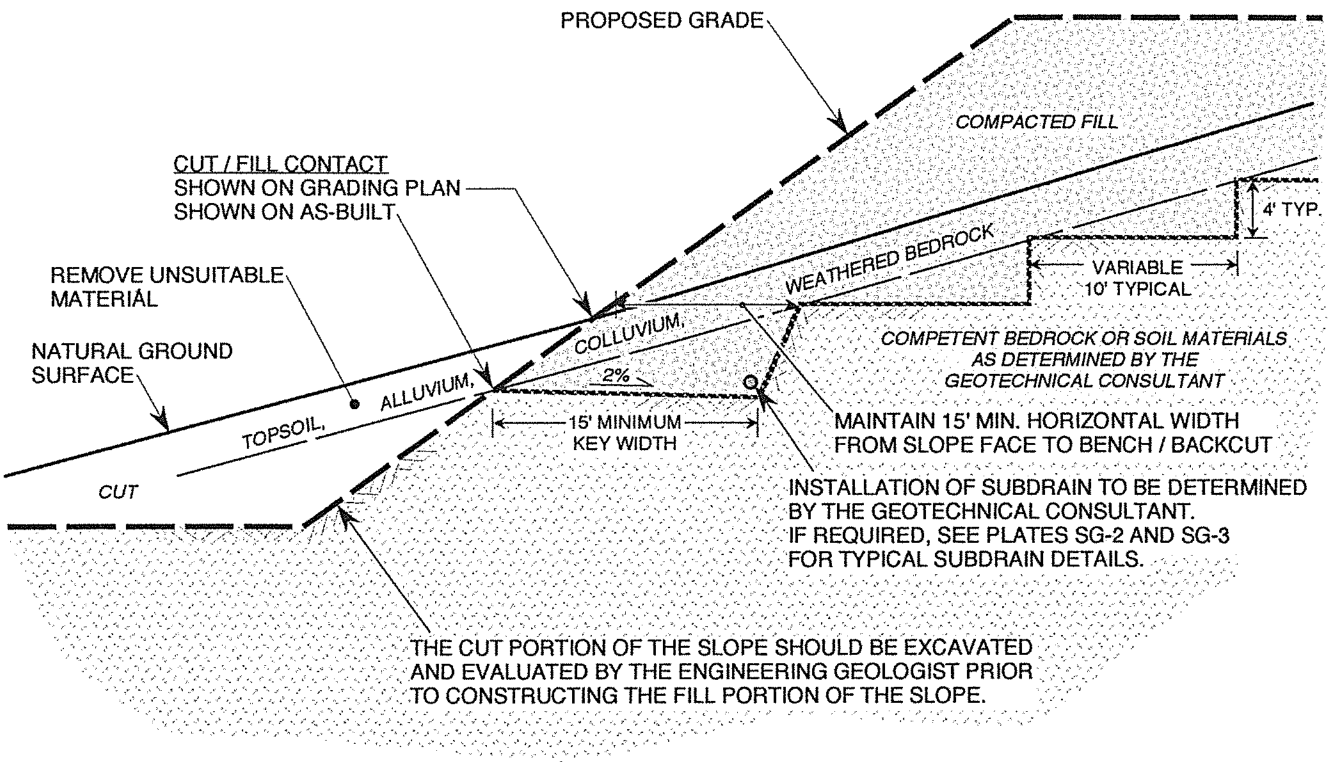
PETRA	TYPICAL ROCK DISPOSAL DETAIL	PLATE SG-4



NOTES:

1. WHERE NATURAL SLOPE GRADIENT IS 5:1 OR LESS, BENCHING IS NOT NECESSARY; HOWEVER, FILL IS NOT TO BE PLACED ON COMPRESSIBLE OR UNSUITABLE MATERIAL.
2. SOILS ENGINEER TO DETERMINE IF SUBDRAIN IS REQUIRED.

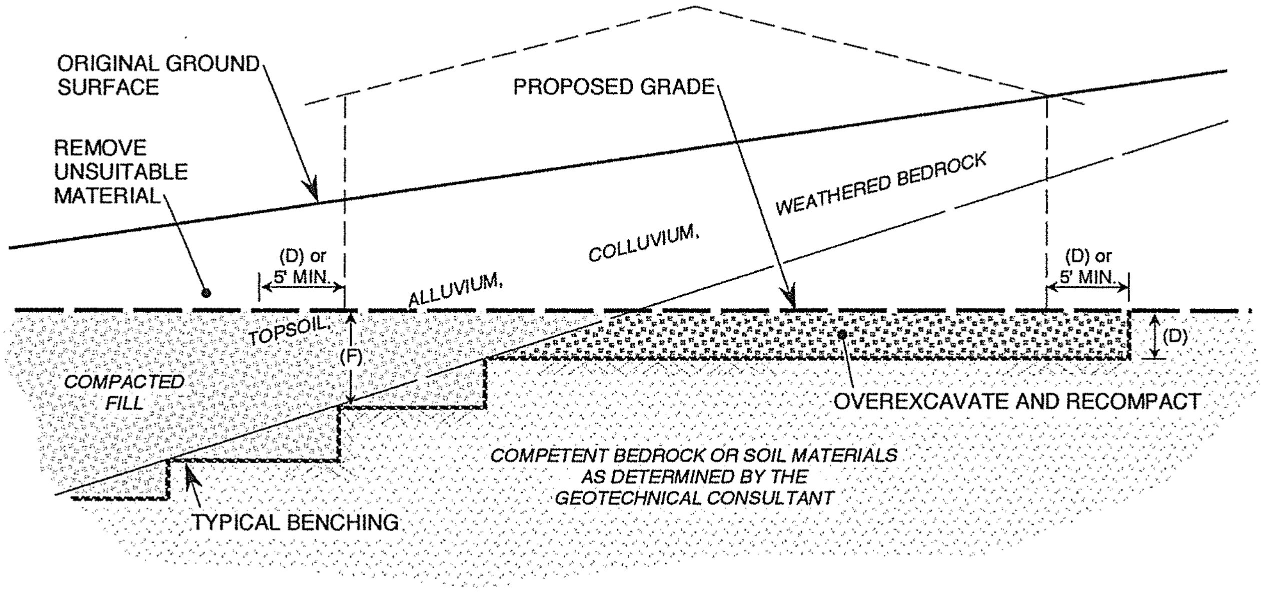
PETRA	FILL SLOPE ABOVE NATURAL SLOPE	PLATE SG-5



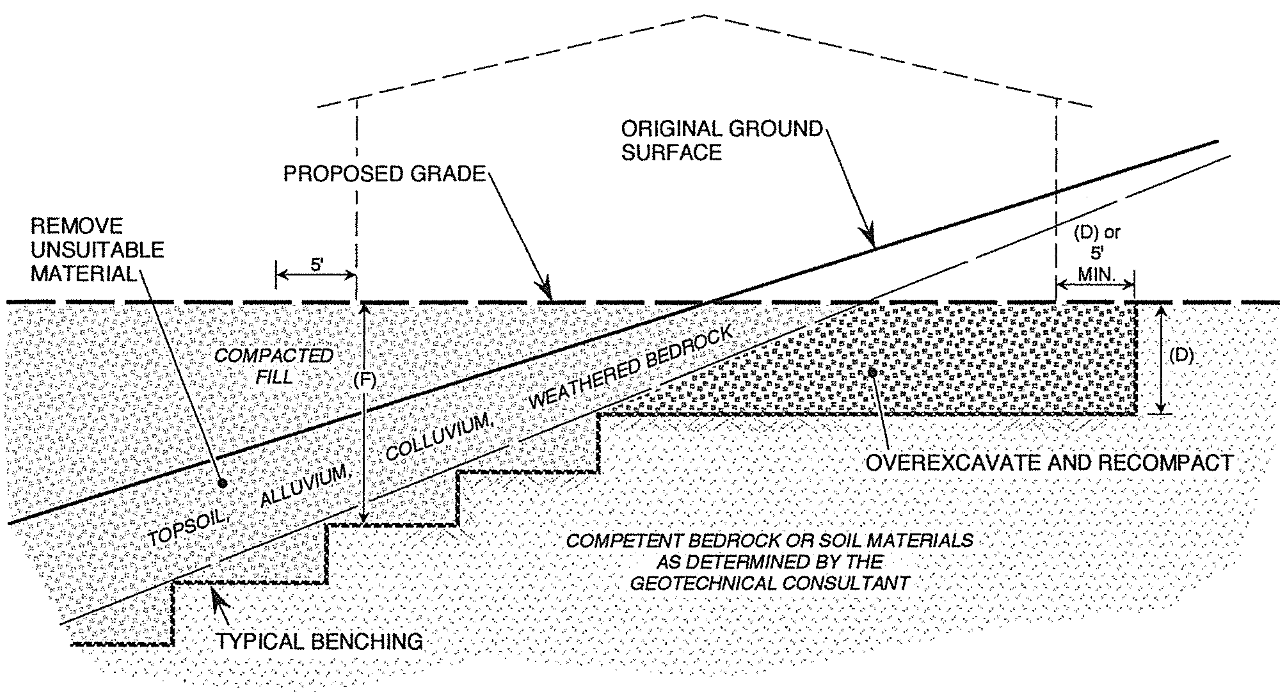
PETRA	FILL SLOPE ABOVE CUT SLOPE	PLATE SG-6

CUT LOT

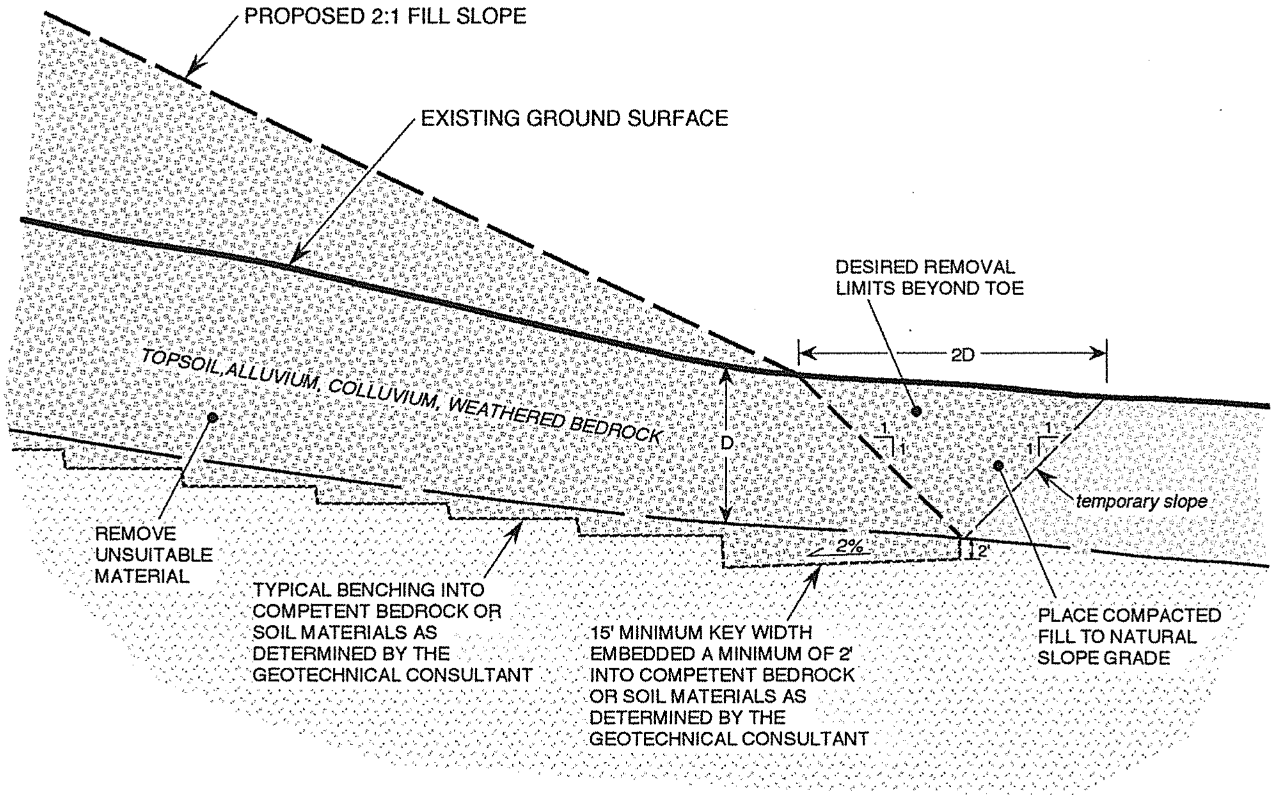
UNSUITABLE MATERIAL EXPOSED IN PORTION OF CUT PAD



CUT-FILL TRANSITION LOT

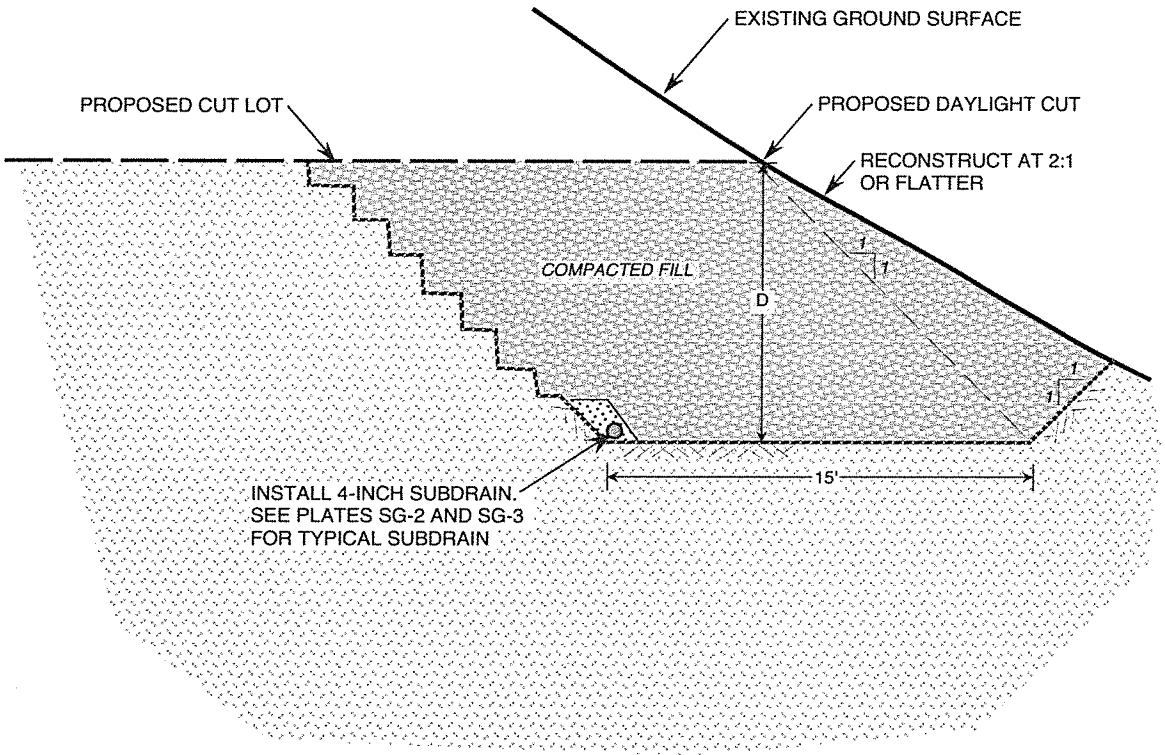


MAXIMUM FILL THICKNESS (F)	DEPTH OF OVEREXCAVATION (D)
FOOTING DEPTH TO 3 FEET	EQUAL DEPTH
3 TO 6 FEET	3 FEET
GREATER THAN 6 FEET	1/2 THE THICKNESS OF DEEPEST FILL PLACED WITHIN THE "FILL" PORTION (F) TO 15 FEET MAXIMUM



D = RECOMMENDED DEPTH OF REMOVAL
PER GEOTECHNICAL REPORT

PETRA	TYPICAL REMOVALS BEYOND TOE OF PROPOSED FILL SLOPE	PLATE SG-8



NOTE:

1. "D" SHALL BE 10 FEET MINIMUM OR AS DETERMINED BY SOILS ENGINEER.

PETRA	SHEAR KEY ON DAYLIGHT CUT LOTS	PLATE SG-9