GEOTECHNICAL INVESTIGATION

OTAY RANCH VILLAGE 7
NEIGHBORHOOD R-8
CHULA VISTA, CALIFORNIA

PREPARED FOR

BALDWIN & SONS

NOVEMBER 18, 2021
REVISED MAY 9, 2024
PROJECT NO. 06862-52-67

GEOCON
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GEOTECHNICAL
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MATERIALS

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Project No. 06862-52-67
November 18, 2021
Revised May 9, 2024

Baldwin & Sons, Inc.
20 Corporate Plaza
Newport Beach, California 92660

Attention: Ms. Maria Miller

Subject: GEOTECHNICAL INVESTIGATION
OTAY RANCH VILLAGE 7
NEIGHBORHOOD R-8
CHULA VISTA, CALIFORNIA

Dear Ms. Miller:

In accordance with your request and authorization of our Proposal No. LG-19506 dated December 27, 2019, we herein submit the results of our geotechnical investigation for the subject project. We performed our investigation to evaluate the underlying soil and geologic conditions and potential geologic hazards, and to assist in the design of the proposed development and associated improvements. We have revised this report to address the 3rd party comments prepared by Michael Baker International.

The accompanying report presents the results of our study and conclusions and recommendations pertaining to geotechnical aspects of the proposed project. The site is suitable for the proposed development and improvements provided the recommendations of this report are incorporated into the design and construction of the planned project.

Should you have questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

GEOCON INCORPORATED

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for a new residential development at Otay Ranch Village 7, Neighborhood R-8 in the City of Chula Vista, California as shown on the Vicinity Map.

Vicinity Map

The purpose of the geotechnical investigation is to evaluate the surface and subsurface soil conditions and general site geology, and to identify geotechnical constraints that may affect development of the property including faulting, liquefaction and seismic shaking based on the 2022 CBC seismic design criteria. In addition, we provided recommendations for remedial grading, shallow foundations, concrete slab-on-grade, concrete flatwork, pavement and retaining walls.

We reviewed the following plans and reports in preparation of this report:

1. *Final Report of Testing and Observation Services Performed During Site Grading, Otay Ranch Village 7, Neighborhood R-2, Chula Vista, California*, prepared by Geocon Incorporated, dated April 12, 2006 (Project No. 06862-52-03A).

2. *Geotechnical Investigation, Otay Ranch Village 7, R-2 and Village 4 Community Park, Chula Vista, California*, prepared by Geocon Incorporated, dated May 5, 2004 (Project No. 06862-52-03).

3. *Conceptual Site Plan, Otay Ranch Village 7 – Neighborhood R-8, City of Chula Vista, California*, prepared by ARKArchitects, Inc., dated September 22, 2023.(Project No. 2022-114)

The scope of this investigation included reviewing readily available published and unpublished geologic literature (see List of References), performing engineering analyses and preparing this report. We also advanced 3 exploratory borings to a maximum depth of about 60 feet, sampled soil and performed laboratory testing. Appendix A presents the exploratory boring logs and details of the field investigation. The details of the laboratory tests and a summary of the test results are shown in Appendix B and on the boring logs in Appendix A. The slope stability analyses are shown in Appendix C and the Recommended Grading Specifications are shown in Appendix D.

2. SITE AND PROJECT DESCRIPTION

The property is an irregularly shaped parcel location south of the Otay Ranch Village 7 development and a storm water drainage channel, east of La Media Road, west of a communications tower (Vortac Site) and north of Santa Luna Street. The site slopes from an elevation of about 540 feet Mean Sea Level (MSL) at the eastern limits to 400 feet MSL at the northwest limits. The area was previously graded during the development of Otay Ranch Village 7 and the installation of La Media Road. The Existing Site Plan shows the current site conditions.

Existing Site Plan

We understand the planned development will consist of a multi-family residential development with accommodating utilities, landscaping, storm water management devices, driveways and surface parking areas. A recreation area will be installed on the southern portion of the site near the neighborhood access point. Cut and fill slopes are planned on the east and north/west, respectively, with heights of up to about

70 feet. The Proposed Site Plan shows the preliminary layout of the planned buildings and improvements.

Proposed Site Plan

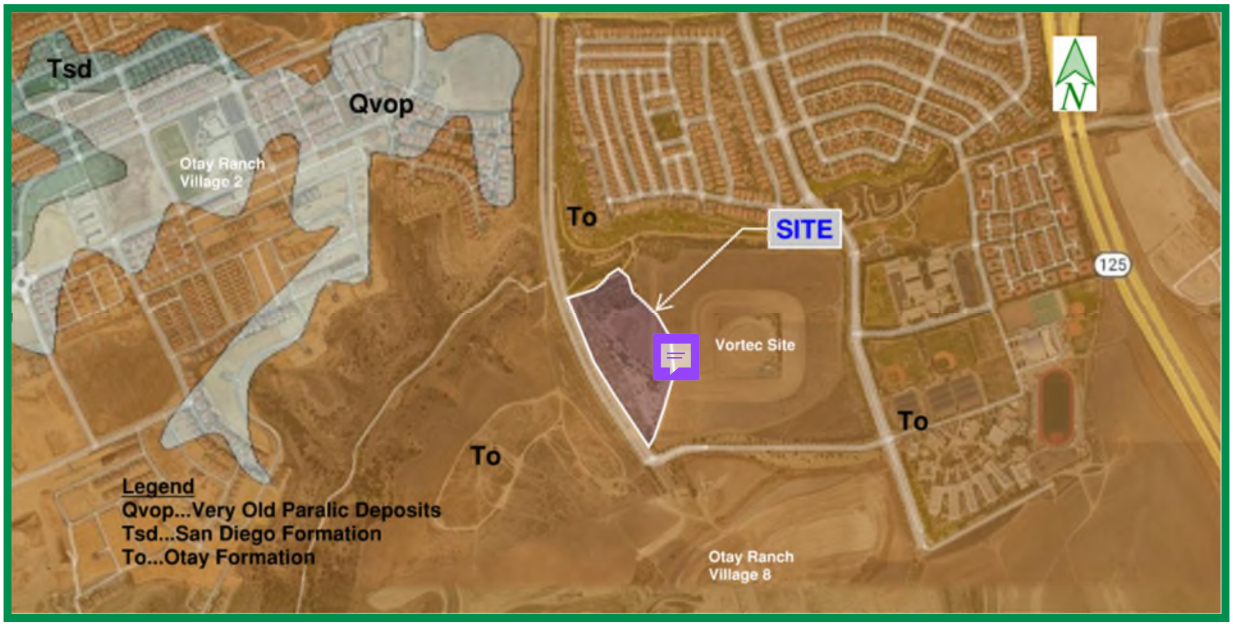
The locations, site descriptions, and proposed development are based on our site reconnaissance, review of published geologic literature, field investigations, and discussions with project personnel. If development plans differ from those described herein, Geocon Incorporated should be contacted for review of the plans and possible revisions to this report.

3. PREVIOUS GRADING

The site is located within an area that was previously graded as part of the Otay Ranch Village 7, Neighborhood R-2 development in 2005 and 2006. The central portion of the site is situated over a previous shallow canyon that was filled in during grading operations. The previous grading operations consisted of removing unsuitable materials (i.e. surficial soil and vegetation) prior to the placement of compacted fill. We provided the testing and observation services during grading operations that consisted of performing laboratory and compaction testing. The field density test results indicate that the fill soil was placed at a dry density of at least 90 percent of the laboratory maximum dry density. The results of the density tests are shown in the referenced report dated April 12, 2006. Fill within the canyon ranges between 3 and 15 feet in depth. Due to the presence of an approximately 1-foot thick layer of bentonite claystone, the constructed slopes were graded at inclinations of about 4:1 (horizontal to vertical) to achieve a suitable stability rather than constructing stability fill buttress slopes at steeper inclinations.

4. GEOLOGIC SETTING

The site is located in the eastern portion of the coastal plain within the southern portion of the Peninsular Ranges Geomorphic Province of southern California. The Peninsular Ranges is a geologic and geomorphic province that extends from the Imperial Valley to the Pacific Ocean and from the Transverse Ranges to the north and into Baja California to the south. The coastal plain of San Diego County is underlain by a thick sequence of relatively undisturbed and non-conformable sedimentary rocks that thicken to the west and range in age from Upper Cretaceous through the Pleistocene with intermittent deposition. The sedimentary units are deposited on bedrock Cretaceous to Jurassic age igneous and metavolcanic rocks. Geomorphically, the coastal plain is characterized by a series of 21, stair-stepped marine terraces (younger to the west) that have been dissected by west flowing rivers. The coastal plain is a relatively stable block that is dissected by relatively few faults consisting of the potentially active La Nacion Fault Zone and the active Rose Canyon Fault Zone. The Peninsular Ranges Province is also dissected by the Elsinore Fault Zone that is associated with and sub-parallel to the San Andreas Fault Zone, which is the plate boundary between the Pacific and North American Plates. The site consists of Oligocene-age (Tertiary) Otay Formation that generally consists of sandstones with interbeds of claystones and siltstones with a reported maximum thickness of roughly 400 feet. The Otay Formation contains multiple layers of bentonitic claystone that is highly expansive and has low shear strength. The Regional Geologic Map shows the geologic units in the area of the site. During our investigation, we encountered previously placed fill (Qpf) in the western and central portion of the site as shown on the . Geologic Map, Figure 1. The following Regional Geologic Map is an excerpt from published geologic maps and is not updated to show the current conditions.



Regional Geologic Map

5. SOIL AND GEOLOGIC CONDITIONS

We encountered two surficial soil units (consisting of topsoil and previously placed fill) and one formational unit (consisting of the Otay Formation). The occurrence, distribution, and description of each unit encountered is shown on the Geologic Map, Figure 1 and on the boring logs in Appendix A. The Geologic Cross-Sections, Figure 2, show the approximate subsurface relationship between the geologic units. We prepared the geologic cross-sections using interpolation between exploratory excavations and observations; therefore, actual geotechnical conditions may vary from those illustrated and should be considered approximate. The surficial soil and geologic units are described herein in order of increasing age.

5.1 Topsoil (Unmapped)

We encountered topsoil in Boring B-2 to a depth of approximately 3½ feet. In general, topsoil is located at the surface overlying the Otay Formation and consists of loose to medium dense, damp to moist, clayey sand and possesses a “very low” to “medium” expansion index (expansion index of 90 or less). We expect the thickest areas of topsoil are located within the higher portions of the site that were left as natural ground during previous grading. The topsoil is not considered suitable in its current condition for the support of foundations or structural fill, and remedial grading will be required.

5.2 Previously Placed Fill (Qpf)

We encountered previously placed compacted fill in Borings B-1 and B-3 ranging from about 3 to 5½ feet thick. Based on the referenced grading report, the previously placed fill at the site ranges up to 15 feet thick. In general, the fill consists of loose (near the surface) to medium dense, damp to moist, clayey sand and stiff sandy clay and possesses a “very low” to “medium” expansion index (expansion index of 90 or less). The upper portions of the previously placed fill is not considered suitable in its current condition for the support of foundations or structural fill and remedial grading will required. The previously placed fill can be reused for new compacted fill during grading operations provided it is generally free of roots and debris.

5.3 San Diego Formation (Unmapped)

Tertiary-age San Diego Formation was previously mapped in the eastern portion of the site. The San Diego Formation rests unconformably above the Otay Formation when present and generally consists of weakly to well-cemented cemented, micaceous, moist to wet, light gray to light yellowish brown, brownish yellow, and strong brown, fine- to medium-grained sandstone and siltstone with zones of cemented gravel and cobble beds. This unit was not encountered during our field investigation. If present and or/encountered in the eastern cut slope during construction, a stability fill will likely need to be constructed due to the relatively cohesionless nature of the unit. The San Diego Formation possesses a "very low" to "low" expansion potential (expansion index of 50 or less). The San Diego Formation is considered suitable for support of structural loads but may require stabilization if encountered in proposed cut slopes.

5.4 Otay Formation (To, Tob)

Tertiary-age Otay Formation (To) is exposed across the site and located below the topsoil and previously placed fill. This unit consists of interbeds of dense to very dense, slightly cemented, silty to clayey sandstone and hard, siltstone and claystone layers. Excavations will generally be possible with heavy-duty grading equipment with heavy effort; however, moderately to highly cemented zones may create very difficult ripping and generate oversize cemented cobbles and boulders. The Otay Formation is suitable for the support of proposed fill and structural loads.

In addition, a laterally extensive bed of bentonite claystone (Tob) with a variable thickness of approximately 1 to 1½ feet exists at an approximate elevation of 440 to 443 feet MSL. Bentonite layers have been mapped as underlying the majority of Otay Ranch and its occurrence is well documented in the geologic literature (Cleveland, 1960). The bentonitic claystone beds consist of highly expansive clays (expansion index greater than 130), which typically exhibit low shear strength. The Geologic Map, Figure 1, shows the approximate elevation of the bentonite layer as encountered in the borings and previous grading operations. The bentonite claystone will require slope stabilization at the locations shown on the Geologic Map.

6. GEOLOGIC STRUCTURE

Bedding attitudes observed within formational materials encountered during the investigation are nearly horizontal to slightly dipping toward the northwest and southwest. The regional dip of sedimentary units in the eastern Chula Vista area is generally 1 to 5 degrees toward the southwest. The granular portions of the formational units are typically massive with bedding not discernible. Shear zones create a possibility for slope instability and, where encountered during grading, should be evaluated for the necessity of remedial grading. High-angle contacts between formational units are not uncommon; however, it is our opinion that adverse geologic structure does not present a significant geologic hazard to the proposed development of the site if the recommendations of this report are incorporated into design and construction.

7. GROUNDWATER

We did not encounter groundwater during our site investigation; however, we did encounter minor seepage in Boring B-3. It is not uncommon for seepage conditions to develop where none previously existed when sites are irrigated or infiltration is implemented. Seepage is dependent on seasonal precipitation, irrigation, land use, among other factors, and varies as a result. Proper surface drainage will be important to future performance of the project. We expect groundwater is deeper than about 100 feet below existing grade. We do not expect groundwater to be encountered during construction of the proposed development.

8. GEOLOGIC HAZARDS

8.1 Regional Faulting and Seismicity

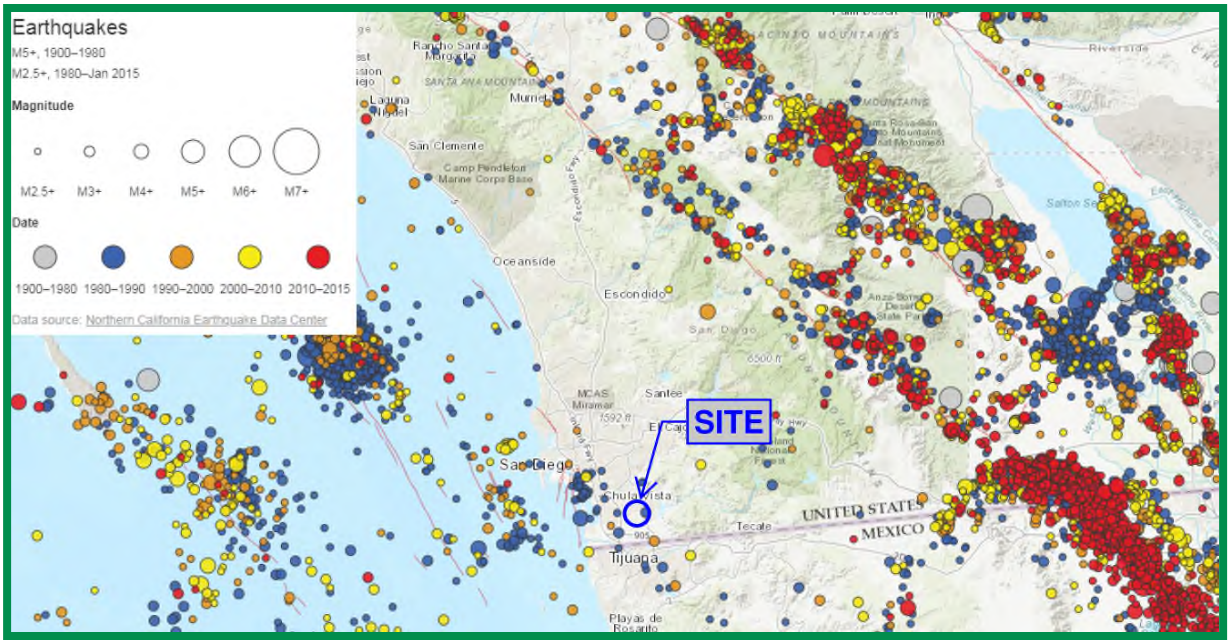
A review of the referenced geologic materials and our knowledge of the general area indicate that the site is not underlain by active, potentially active, or inactive faults. An active fault is defined by the California Geological Survey (CGS) as a fault showing evidence for activity within the last 11,700 years. The site is not located within a State of California Earthquake Fault Zone.

The USGS has developed a program to evaluate the approximate location of faulting in the area of properties. The following figure shows the location of the existing faulting in the San Diego County and Southern California region. The fault traces are shown as solid, dashed and dotted that represent well-constrained, moderately constrained and inferred, respectively. The fault line colors represent faults with ages less than 150 years (red), 15,000 years (orange), 130,000 years (green), 750,000 years (blue) and 1.6 million years (black).

Faults in Southern California

The San Diego County and Southern California region is seismically active. The following figure presents the occurrence of earthquakes with a magnitude greater than 2.5 from the period of 1900 through 2015 according to the Bay Area Earthquake Alliance website.

Considerations important in seismic design include the frequency and duration of motion and the soil conditions underlying the site. Seismic design of structures should be evaluated in accordance with the California Building Code (CBC) guidelines currently adopted by the local agency.



Earthquakes in Southern California

8.2 Ground Rupture

Ground surface rupture occurs when movement along a fault is sufficient to cause a gap or rupture where the upper edge of the fault zone intersects the ground surface. The potential for ground rupture is considered to be very low due to the absence of active faults at the subject site.

8.3 Liquefaction

Liquefaction typically occurs when a site is located in a zone with seismic activity, onsite soils are cohesionless or silt/clay with low plasticity, groundwater is encountered within 50 feet of the surface and soil densities are less than about 70 percent of the maximum dry densities. If the four previous criteria are met, a seismic event could result in a rapid pore water pressure increase from the earthquake-generated ground accelerations. Due to the lack of a permanent, near-surface groundwater table and the very dense nature of the underlying fill and Otay Formation, liquefaction potential for the site is considered very low.

8.4 Storm Surge, Tsunamis, and Seiches

Storm surges are large ocean waves that sweep across coastal areas when storms make landfall. Storm surges can cause inundation, severe erosion and backwater flooding along the water front. The site is located greater than 9 miles from the Pacific Ocean and is at an elevation of about 400 feet or greater above Mean Sea Level (MSL). Therefore, the potential of storm surges affecting the site is considered very low.

A tsunami is a series of long period waves generated in the ocean by a sudden displacement of large volumes of water. Causes of tsunamis include underwater earthquakes, volcanic eruptions, or offshore slope failures. The potential for the site to be affected by a tsunami is negligible due to the distance from the Pacific Ocean and the site elevation.

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement. The site is not located in the vicinity of or downstream from such bodies of water. Therefore, the risk of seiches affecting the site is negligible.

8.5 Landslides

While a bentonite claystone layer is present at the site, we did not observe evidence of previous or incipient slope instability at the site during previous grading or this study. Published geologic mapping indicates landslides are not present on or adjacent to the site. Therefore, we opine the potential for future landsliding adversely affecting the proposed improvements is low provided the grading recommendations presented herein are followed.

8.6 Slope Stability

We evaluated the maximum proposed cut and fill slope heights, as depicted on the Geologic Map, Figure 1, to evaluate both surficial and global stability based on the current geologic information. The portions of the site planned for grading are generally underlain by Tertiary-age Otay Formation. The unit most likely to be subject to slope instability is the bentonitic claystone layer within the Otay Formation encountered at the site. Slope stability analyses using the two-dimensional computer program *GeoStudio 2018* developed by Geo-Slope International Ltd. Are presented in Appendix C. The proposed slopes have calculated factors of safety greater than 1.5 for global and shallow sloughing conditions provided our recommendations for grading and drainage are incorporated into the design and construction of the proposed slopes.

In general, permanently graded fill slopes constructed of granular soil and cut slopes excavated within the Otay Formation at the site with gradients of 2:1 (horizontal to vertical) or flatter will possess Factors of Safety of 1.5 or greater. However, buttress and stability fills will be required during grading operations on the northern and western portion of the site where bentonite is located within the slope zone (see Geologic Cross-Sections, Figure 2). The eastern slope should be buttresses due to the existing claystones within the Otay Formation. The buttress widths will range from 15 feet to 20 feet, as calculated. The Geologic Map and Cross-Sections provide the required width of the buttresses for the planned development.

Grading of cut and fill slopes should be designed in accordance with the requirements of the local building codes of the City of Chula Vista and the 2022 California Building Code (CBC). Mitigation of unstable cut slopes can be achieved by the use of drained stability or buttress fills.

Table 8.6 presents the surficial slope stability analysis for the proposed sloping conditions.

TABLE 8.6
SURFICIAL SLOPE STABILITY EVALUATION

Parameter	Value	
Slope Condition	Cut Slope	Fill Slope
Slope Height, H	∞	∞
Vertical Depth of Saturation, Z	3 Feet	3 Feet
Slope Inclination, I (Horizontal to Vertical)	2:1 (26.6 Degrees)	2:1 (26.6 Degrees)
Total Soil Unit Weight, γ	125 pcf	125 pcf
Water Unit Weight, γ_W	62.4 pcf	62.4 pcf
Friction Angle, ϕ	33 Degrees	28 Degrees
Cohesion, C	325 psf	300 psf
Factor of Safety = $(C+(\gamma+\gamma_W)Z\cos^2 I \tan\phi)/(\gamma Z \sin I \cos I)$	2.8	2.5

Slopes should be landscaped with drought-tolerant vegetation having variable root depths and requiring minimal landscape irrigation. In addition, slopes should be drained and properly maintained to reduce erosion.

8.7 Erosion

The site is relatively gently sloping to west but is not located adjacent to the Pacific Ocean coast or a free-flowing drainage where active erosion is occurring. The site is located south of an existing controlled drainage area. Provided the engineering recommendations herein are followed and the project civil engineer prepares the grading plans in accordance with generally-accepted regional standards, we do not expect erosion to be a major impact to site development. In addition, we expect the proposed development would not increase the potential for erosion if properly designed.

9. CONCLUSIONS AND RECOMMENDATIONS

9.1 General

9.1.1 We did not encounter soil or geologic conditions during our exploration or previous grading operations that would preclude the proposed development, provided the recommendations presented herein are followed and implemented during design and construction. We will provide supplemental recommendations if we observe variable or undesirable conditions during construction, or if the proposed construction will differ from that anticipated herein.

9.1.2 With the exception of possible moderate to strong seismic shaking and slope instability, we did not observe or know of significant geologic hazards to exist on the site that would adversely affect the proposed project.

9.1.3 The topsoil, upper weathered portions of the Otay Formation, and upper portions of the previously placed fill are potentially compressible and unsuitable in their present condition for the support of compacted fill or settlement-sensitive improvements. Remedial grading of these materials should be performed as discussed herein. The dense portions of the previously placed fill and Otay Formation are considered suitable for the support of proposed fill and structural loads.

9.1.4 The bentonite claystone layer at the site should be stabilized using designed buttresses, as shown on the Geologic Map and Geologic Cross-Sections, Figures 1 and 2, respectively.

9.1.5 We did not encounter groundwater during our subsurface exploration and we do not expect it to be a constraint to project development. However, seepage within surficial soils and within the formational materials was observed during our investigation and may be encountered during grading operations, especially during the rainy season.

9.1.6 Excavation of the topsoil, previously placed fill and the Otay Formation should generally be possible with moderate to very heavy effort using conventional, heavy-duty equipment during grading and trenching operations. We expect possible refusal in localized areas for excavations into strongly cemented portions of the Otay Formation.

9.1.7 Proper drainage should be maintained in order to preserve the engineering properties of the fill in both the building pads and slope areas. Recommendations for site drainage are provided herein.

9.1.8 Based on our review of the project plans, we opine the planned development can be constructed in accordance with our recommendations provided herein. We do not expect the planned development will destabilize or result in settlement of adjacent properties if properly constructed.

9.1.9 Surface settlement monuments and canyon subdrains will not be required on this project.

9.2 Excavation and Soil Characteristics

9.2.1 Excavations of the in-situ soil should be possible with moderate to heavy effort using conventional heavy-duty equipment. Excavation of the formational materials will require very heavy effort and may generate oversized material using conventional heavy-duty equipment during the grading operations. Oversized rock (rocks greater than 12-inches in maximum dimension) generated during excavation of The Otay Formation can be incorporated within deep compacted fill areas, if available.

9.2.2 The soil encountered during previous grading operations is considered to be non-expansive" and "expansive" (expansion index [EI] of 20 or less and greater than 20, respectively) as defined by 2022 California Building Code (CBC) Section 1803.5.3. We expect a majority of the soil encountered possess a "very low" to "medium" expansion potential (EI of 90 or less) in accordance with ASTM D 4829. The bentonitic claystone possesses a "high" to "very high" expansion potential (EI greater than 90) in accordance with ASTM D 4829. Table 9.2 presents soil classifications based on the expansion index.

TABLE 9.2
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	ASTM D 4829 Expansion Classification	2022 CBC Expansion Classification
0 – 20	Very Low	Non-Expansive
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

9.2.3 Laboratory tests performed during previous grading operations indicate the on-site materials at the locations tested possess "S0" sulfate exposure to concrete structures as defined by 2022 CBC Section 1904 and ACI 318-19 Chapter 19. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

9.2.4 Geocon Incorporated does not practice in the field of corrosion engineering. Therefore, further evaluation by a corrosion engineer may be performed if improvements susceptible to corrosion are planned.

9.3 Grading

9.3.1 Grading should be performed in accordance with the recommendations provided in this report, the Recommended Grading Specifications contained in Appendix D and the City of Chula Vista's Grading Ordinance. Geocon Incorporated should observe the grading operations on a full-time basis and provide testing during the fill placement.

9.3.2 Prior to commencing grading, a preconstruction conference should be held at the site with the regulatory agency, developer, grading/underground contractors, civil engineer and geotechnical engineer in attendance. Special soil handling and/or the grading plans can be discussed at that time.

9.3.3 Site preparation should begin with the removal of deleterious material, debris and vegetation. The depth of vegetation removal should be such that material exposed in cut areas or soil to be used as fill is relatively free of organic matter. Material generated during stripping and/or site demolition should be exported from the site. Asphalt and concrete should not be mixed with the fill soil unless approved by the Geotechnical Engineer.

9.3.4 Topsoil, the upper portions of the formational materials, and the upper 2 to 3 feet of the previously placed fill within the limits of grading should be removed to expose firm, competent material. The actual depth of removal should be evaluated in the field during grading operations.

9.3.5 We do not expect to observe bentonitic claystone near the proposed finish pad grade elevations for the proposed development. Bentonitic claystone layers that occur within 5 feet of finish grade, if observed, should be removed and replaced with properly compacted fill that possesses a "very low" to "medium" expansion potential (EI of 90 or less). The undercut within the building pads should be sloped at least 2 percent toward the adjacent street or deep fill area.

9.3.6 Bentonitic claystone layers encountered during the normal excavation or undercutting of building pads, streets, or slopes should be mixed with granular materials in a ratio of at least two parts sand to one part bentonite clay and compacted to a dry density of at least 90 percent of the laboratory maximum dry density at or slightly above optimum moisture. The mixed bentonite clay should be placed at least 5 feet below finish grade, at least 15 feet from the face of a fill slope, and not within buttress or stability fill slopes.

9.3.7 The upper 3 feet of cut and cut/fill transition lots should be over excavated and replaced with properly compacted fill due to the very dense and cemented nature of the formational materials. The bottom of the excavations should be sloped at least one percent toward the adjacent deeper fill areas or adjacent roadways to reduce the potential for subsurface water to saturate fill materials.

9.3.8 The City of Chula Vista requires additional removals and grading requirements within the street and right-of-way areas. Based on the City of Chula Vista, the upper 5 feet of fill and upper 3 feet of formational materials within the public right of way areas should possess an expansion index of 90 or less. Additional removals of formational materials may be required if the expansion index is greater than 90.

9.3.9 We should observe the grading operations and the removal bottoms to check the exposure of the formational materials prior to the placement of compacted fill. Deeper excavations may be required if highly weathered formational materials are present at the base of the removals. Table 9.3.1 provides a summary of the grading recommendations.

TABLE 9.3.1
SUMMARY OF GRADING RECOMMENDATIONS

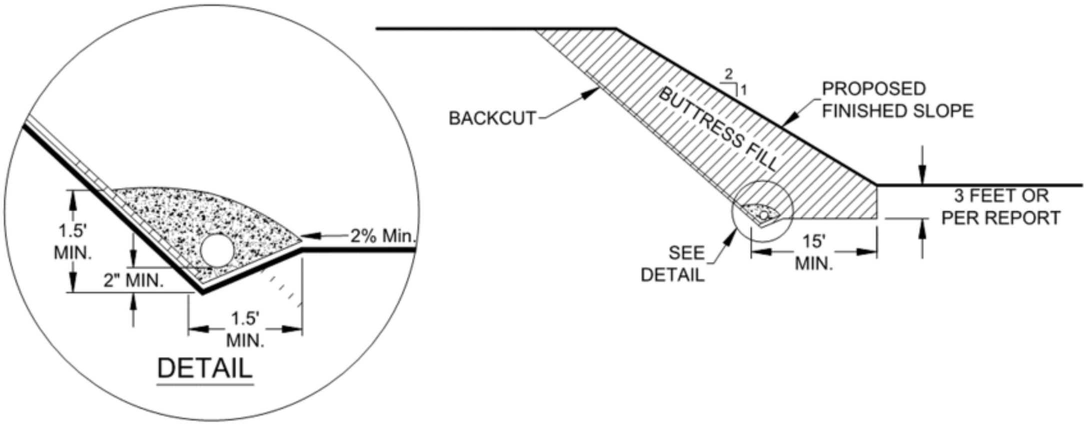
Area	Removal Requirements
Site Development	Remove to Competent Formational Materials or Competent Previously Placed Fill
	Undercut 3 Feet Below Finish Grade Cut or Cut/Fill Transitions
Street and Right-Of-Way	Upper 5 Feet of Fill/3 Feet of Formation
	Expansion Index of 90 or Less
Exposed Bottoms of Remedial Grading	Scarify Upper 12 Inches
	Slope 1 Percent to Adjacent Street or Deepest Fill

9.3.10 Some areas of overly wet and saturated soil could be encountered due to the existing landscape and pavement areas. The saturated soil would require additional effort prior to placement of compacted fill or additional improvements. Stabilization of the soil would include scarifying and air-drying, removing and replacement with drier soil, use of stabilization fabric (e.g. Tensar TX7 or other approved fabric), or chemical treating (i.e. cement or lime treatment).

9.3.11 Slope stability analysis utilizing drained direct shear strength parameters based on our experience with similar soil types in nearby areas and laboratory test results indicates that the proposed fill slopes, constructed of on-site materials, should have calculated factors of safety of at least 1.5 under static conditions for both deep-seated failure and shallow sloughing conditions. Cut slopes that are not impacted by bentonitic clay layers were also found to possess a calculated factor of safety in excess of 1.5 for a deep-seated failure condition. However, some of the proposed slopes will require buttressing to obtain a factor of safety of at least 1.5. These slopes are shown on the Geologic Map (Figure 1) and should be graded with buttresses varying from approximately 15 to 20 feet wide. The design buttress widths are shown on the Geologic Map.

9.3.12 Due to the expansive and blocky nature of the Otay Formation, we recommend that a stability fill is constructed along the face of the descending cut slope on the east side of the site.

9.3.13 The Typical Buttress and Stability Fill Detail should be used for design and construction of slope buttresses, where required. The backcut for the buttress should commence at least 10 feet from the top of the proposed finish-graded slope and should extend at least 3 feet below adjacent pad grade or below the bentonite layer. The base of the key should be slopes at least 5 percent to the drain, into slope. Stability fills may also be required on cut slopes that expose the San Diego Formation where cohesionless sand is encountered.



Typical Buttress and Stability Fill Detail

9.3.14 The slope backcut should be a 1:1 and in accordance with OSHA requirements. Chimney drains should be installed along the backcut that are 4 feet wide, 20-foot on center and provide dual-sided drainage. Closer spacing may be required where seepage is encountered. The collector pipe at the base of the backcut should consists of a minimum 4-inch diameter, perforated, Schedule 40 PVC pipe drained at a minimum of 1 percent. The pipe should be surrounded by ¾-inch gravel wrapped in an approved filter fabric (Mirafi 140N or equivalent).

9.3.15 Cut slope excavations including buttresses and shear keys should be observed during grading operations to check that soil and geologic conditions do not differ significantly from those expected. During the construction of buttresses, there is a risk that the temporary backcut slopes will become unstable. This risk can be reduced by grading the buttress fill in short segments and/or flattening the inclination of the temporary slope.

9.3.16 The site should then be brought to final subgrade elevations with fill compacted in layers. In general, soil native to the site is suitable for use from a geotechnical engineering standpoint as fill if relatively free from vegetation, debris and other deleterious material. Layers of fill should be about 6 to 8 inches in loose thickness and no thicker than will allow for adequate bonding and compaction. Fill, including backfill and scarified ground surfaces, should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM Test Procedure D 1557. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill. The upper 12 inches of subgrade soil underlying vehicular pavement should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content shortly before paving operations.

9.3.17 Import fill (if necessary) should consist of the characteristics presented in Table 9.3.2. Geocon Incorporated should be notified of the import soil source and should perform laboratory testing of import soil prior to its arrival at the site to determine its suitability as fill material.

TABLE 9.3.2
SUMMARY OF IMPORT FILL RECOMMENDATIONS

Soil Characteristic	Values
Expansion Potential	"Very Low" to "Medium" (Expansion Index of 90 or less)
Particle Size	Maximum Dimension Less Than 3 Inches
	Generally Free of Debris

9.4 Earthwork Grading Factors

9.4.1 Estimates of shrink-swell factors are based on comparing laboratory compaction tests with the density of the material in its natural state and experience with similar soil types. Variations in natural soil density and compacted fill render shrinkage value estimates very approximate. As an example, the contractor can compact fill to a density of 90 percent or higher of the laboratory maximum dry density. Thus, the contractor has at least a 10 percent range of control over the fill volume. Based on the work performed to date in the Otay Ranch area and considering the discussion herein, the earthwork factors in Table 9.4 may be used as a general basis for estimating how much the on-site soils may shrink or swell when removed from their natural state and placed as compacted fill.

TABLE 9.4
SHRINKAGE AND BULK FACTORS

Soil Unit	Shrink/Bulk Factor
Topsoil (unmapped)	10% to 15% Shrink
Previously Placed Fill (Qpf)	2% Shrink to 1% Bulk
Otay Formation (To)	4% to 8% Bulk

9.5 Slope Stability Analyses

9.5.1 We performed slope stability analyses using the two-dimensional computer program GeoStudio created by Geo-Slope International Ltd. We calculated the factor of safety for the planned slopes for rotational-mode and block-mode analyses using the Spencer’s method. Output of the computer program including the calculated factor of safety and the failure surface is presented in Appendix C.

9.5.2 We used average drained direct shear strength parameters based on laboratory tests and our experience with similar soil types in nearby areas for the slope stability analyses. Our calculations indicate the proposed slopes, constructed of on-site materials, should have calculated factors of safety (FOS) of at least 1.5 under static conditions, for both deep-seated failure and shallow sloughing conditions when the recommendations of this report are followed.

9.5.3 We selected Cross-Sections A-A’, B-B’, C-C’ and D-D’ to perform the slope stability analyses. Appendix C presents the results of the slope stability analyses.

9.5.4 Among the slopes analyzed for acceptable calculated factors of safety, Cross-Sections A-A’ and C-C’ will require buttresses due to the presence of bentonite claystone layers. Buttress designs have assumed a 1:1 (horizontal to vertical) frontcut and backcut extending down through the critical bentonite claystone layers.

9.5.5 Due to the very light loads expected from the planned homes and improvements, the loads are considered negligible with no appreciable impact to the slope stability analyses and, therefore were not incorporated into the analyses.

9.5.6 Planned buttress keys and proposed subdrains should be surveyed during construction with their approximate locations depicted on the Geologic Map using the 40-scale grading plans (Figure 1). The buttresses will require the construction of a subdrain located at the heel of the buttress (toe of the backcut) and should be as-built and surveyed by the project civil engineer.

9.5.7 Excavations including buttresses, shear keys, and stability fills should be observed during grading by an engineering geologist with Geocon to evaluate whether soil and geologic conditions do not differ significantly from those expected or identified in this report.

9.5.8 We performed the slope stability analyses based on the interpretation of geologic conditions encountered during our field investigation. We should evaluate the geologic conditions during the grading operations to check if the conditions observed during grading are consistent with our interpretations. Additional slope stability analyses and modifications to the proposed buttresses may be required during the grading operations.

9.5.9 The buttress excavations are not planned adjacent to existing improvements or residences. If excavation failures were to occur, the failures would be limited to within the property limits and outside improvements/structures would not be affected. In addition, the grading contractor would be required to remove the volume of soil that failed and evaluate the additional excavation procedures.

9.5.10 Slopes should be landscaped with drought-tolerant vegetation having variable root depths and requiring minimal landscape irrigation. In addition, slopes should be drained and properly maintained to reduce erosion.

9.6 Temporary Excavations

9.6.1 The recommendations included herein are provided for stable excavations. It is the responsibility of the contractor and their competent person to ensure all excavations, temporary slopes and trenches are properly constructed and maintained in accordance with applicable OSHA guidelines in order to maintain safety and the stability of the excavations and adjacent improvements. These excavations should not be allowed to become saturated or to dry out. Surcharge loads should not be permitted to a distance equal to the height of the excavation from the top of the excavation. The top of the excavation should be a minimum of 15 feet from the edge of existing improvements. Excavations steeper than those recommended or closer than 15 feet from an existing surface improvement should be stored in accordance with applicable OSHA codes and regulations.

9.6.2 The stability of the excavations is dependent on the design and construction of the shoring system and site conditions. Therefore, Geocon Incorporated cannot be responsible for site safety and the stability of the proposed excavations.

9.7 Seismic Design Criteria

9.7.1 Table 9.7.1 summarizes site-specific design criteria obtained from the 2022 California Building Code (CBC; Based on the 2021 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. We used the computer program *U.S. Seismic Design Maps*, provided by the Structural Engineers Association (SEA) to calculate the seismic design parameters. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2022 CBC and Table 20.3-1 of ASCE 7-16. The buildings and improvements should be designed using a Site Class C where the fill thickness is 20 feet or less or a Site Class D where the fill is thicker than 20 feet. The values presented herein are for the risk-targeted maximum considered earthquake (MCE$_R$). Sites designated as Site Class D, E and F may require additional analyses if requested by the project structural engineer and client.

TABLE 9.7.1
2022 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value		2022 CBC Reference
Site Class	C	D	Section 1613.2.2
Fill Thickness, T (feet)	T<20	T≥20	--
MCE$_R$ Ground Motion Spectral Response Acceleration – Class B (short), S$_S$	0.766g	0.766g	Figure 1613.2.1(1)
MCE$_R$ Ground Motion Spectral Response Acceleration – Class B (1 sec), S$_1$	0.278g	0.278g	Figure 1613.2.1(2)
Site Coefficient, F$_A$	1.200	1.200	Table 1613.2.3(1)
Site Coefficient, F$_V$	1.500*	2.044*	Table 1613.2.3(2)
Site Class Modified MCE$_R$ Spectral Response Acceleration (short), S$_{MS}$	0.920g	1.920g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE$_R$ Spectral Response Acceleration – (1 sec), S$_{M1}$	0.417g*	0.568g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S$_{DS}$	0.613g	0.613g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S$_{D1}$	0.278g*	0.379g*	Section 1613.2.4 (Eqn 16-39)

*See following paragraph..

9.7.2 Using the code-based values presented in the previous table, in lieu of performing a ground motion hazard analysis, requires the exceptions outlined in ASCE 7-16 Section 11.4.8 be followed by the project structural engineer. Per Section 11.4.8 of ASCE/SEI 7-16, a ground motion hazard analysis should be performed for projects for Site Class “D” sites with S1 greater than 0.2g. Section 11.4.8 also provides exceptions which indicate that the ground motion hazard analysis may be waived provided the exceptions are followed. Supplement 3 of ASCE 7-16 provides an exception stating that that the GMHA may be waived provided that the parameter S_{M1} is increased by 50% for all applications of S_{M1}. The values for parameters S_{M1} and S_{D1} presented herein above have **not** been increased in accordance with Supplement 3 of ASCE 7-16.

9.7.3 Table 9.7.2 presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-16.

TABLE 9.7.2
2022 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value		ASCE 7-16
Site Class	C	D	--
Fill Thickness, T (Feet)	T≤20	T>20	--
Mapped MCE_G Peak Ground Acceleration, PGA	0.333g	0.333g	Figure 22-9
Site Coefficient, F_{PGA}	1.200	1.267	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.400g	0.422g	Section 11.8.3 (Eqn 11.8-1)

9.7.4 Conformance to the criteria in Tables 9.7.1 and 9.7.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur in the event of a large earthquake. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

9.7.5 The project structural engineer and architect should evaluate the appropriate Risk Category and Seismic Design Category for the planned structures. The values presented herein assume a Risk Category of II and resulting in a Seismic Design Category D. Table 9.7.3 presents a summary of the risk categories in accordance with ASCE 7-16.

TABLE 9.7.3
ASCE 7-16 RISK CATEGORIES

Risk Category	Building Use	Examples
I	Low risk to Human Life at Failure	Barn, Storage Shelter
II	Nominal Risk to Human Life at Failure (Buildings Not Designated as I, III or IV)	Residential, Commercial and Industrial Buildings
III	Substantial Risk to Human Life at Failure	Theaters, Lecture Halls, Dining Halls, Schools, Prisons, Small Healthcare Facilities, Infrastructure Plants, Storage for Explosives/Toxins
IV	Essential Facilities	Hazardous Material Facilities, Hospitals, Fire and Rescue, Emergency Shelters, Police Stations, Power Stations, Aviation Control Facilities, National Defense, Water Storage

9.8 Foundation and Concrete Slabs-On-Grade Recommendations

9.8.1 The foundation recommendations herein are for proposed residential structures. The foundation recommendations have been separated into three categories based on either the maximum and differential fill thickness or expansion index. The foundation category criteria are presented in Table 9.8.1. We will provide final foundation categories for each building or lot after finish pad grades have been achieved and we perform laboratory testing of the subgrade soil.

TABLE 9.8.1
FOUNDATION CATEGORY CRITERIA

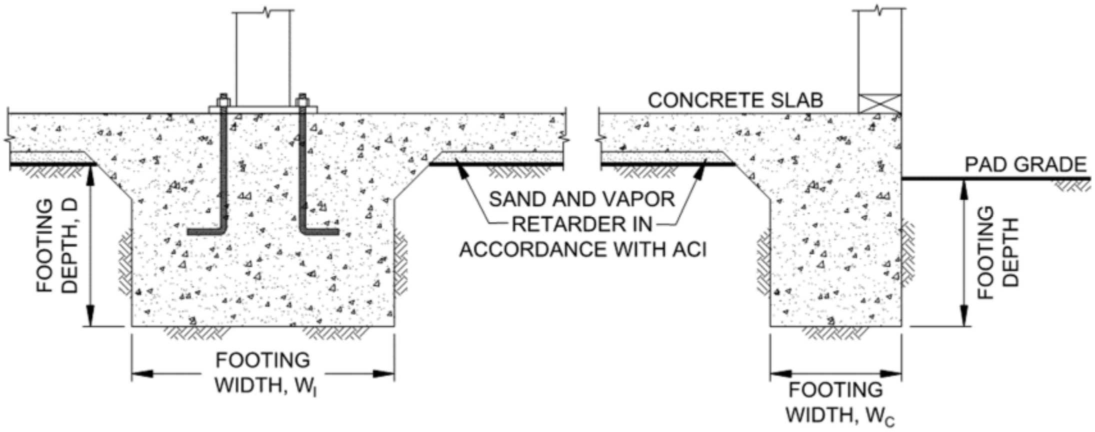
Foundation Category	Maximum Fill Thickness, T (Feet)	Differential Fill Thickness, D (Feet)	Expansion Index (EI)
I	T<20	--	EI≤50
II	20≤T<50	10≤D<20	50<EI≤90
III	T≥50	D≥20	90<EI≤130

9.8.2 Table 9.8.2 presents minimum foundation and interior concrete slab design criteria for conventional foundation systems.

TABLE 9.8.2
CONVENTIONAL FOUNDATION RECOMMENDATIONS BY CATEGORY

Foundation Category	Minimum Footing Embedment Depth, D (inches)	Minimum Continuous Footing Reinforcement	Minimum Footing Width (Inches)
I	12	Two No. 4 bars, one top and one bottom	12 – Continuous, W_C 24 – Isolated, W_I
II	18	Four No. 4 bars, two top and two bottom	
III	24	Four No. 5 bars, two top and two bottom	

9.8.3 The foundations should be embedded in accordance with the recommendations herein and the Wall/Column Footing Dimension Detail. The embedment depths should be measured from the lowest adjacent pad grade for both interior and exterior footings. Footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope (unless designed with a post-tensioned foundation system as discussed herein).



Wall/Column Footing Dimension Detail

9.8.4 The proposed structures can be supported on a shallow foundation system founded in the compacted fill/formational materials. Table 9.8.3 provides a summary of the foundation design recommendations.

TABLE 9.8.3
SUMMARY OF FOUNDATION RECOMMENDATIONS

Parameter	Value
Allowable Bearing Capacity	2,000 psf
Estimated Total Settlement: Foundation Loads	1 Inch
Estimated Differential Settlement: Foundation Loads	½ Inch in 40 Feet

9.8.5 The bearing capacity values presented herein are for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces.

9.8.6 The concrete slab-on-grades should be a designed in accordance with Table 9.8.4.

TABLE 9.8.4
CONVENTIONAL SLAB-ON-GRADE RECOMMENDATIONS BY CATEGORY

Foundation Category	Minimum Concrete Slab Thickness (inches)	Interior Slab Reinforcement	Typical Slab Underlayment
I	4	6 x 6 – 10/10 welded wire mesh at slab mid-point	3 to 4 Inches of Sand/Gravel/Base
II	4	No. 3 bars at 24 inches on center, both directions	
III	5	No. 3 bars at 18 inches on center, both directions	

9.8.7 Slabs that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder. The vapor retarder design should be consistent with the guidelines presented in the American Concrete Institute's (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06). The vapor retarder used should be specified by the project architect or developer based on the type of floor covering that will be installed and if the structure will possess a humidity controlled environment.

9.8.8 The bedding sand thickness should be determined by the project foundation engineer, architect, and/or developer. However, we should be contacted to provide recommendations if the bedding sand is thicker than 6 inches. It is common to see 3 inches and 4 inches of sand below the concrete slab-on-grade for 5-inch and 4-inch thick slabs, respectively, in the southern California area. The foundation design engineer should provide appropriate concrete mix design criteria and curing measures to assure proper curing of the slab by reducing the potential for rapid moisture loss and subsequent cracking and/or slab curl. We suggest that the foundation design engineer present the concrete mix design and proper curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plans.

9.8.9 As an alternative to the conventional foundation recommendations, consideration should be given to the use of post-tensioned concrete slab and foundation systems for the support of the proposed structures. The post-tensioned systems (foundation dimensions and embedment depths, slab thickness and steel placement) should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI) DC 10.5-12 *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils* or *WRI/CRSI Design of Slab-on-Ground Foundations*, as required by the 2022 California Building Code (CBC Section 1808.6.2). Although this procedure was developed for expansive soil conditions, it can also be used to reduce the potential for foundation distress due to differential fill settlement. The post-tensioned design should incorporate the geotechnical parameters presented in Table 9.8.3 for the particular Foundation Category designated. The parameters presented in Table 9.8.5 are based on the guidelines presented in the PTI DC 10.5 design manual.

TABLE 9.8.5
POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS

Post-Tensioning Institute (PTI) DC10.5 Design Parameters	Foundation Category		
	I	II	III
Thornthwaite Index	-20	-20	-20
Equilibrium Suction	3.9	3.9	3.9
Edge Lift Moisture Variation Distance, e_M (Feet)	5.3	5.1	4.9
Edge Lift, y_M (Inches)	0.61	1.10	1.58
Center Lift Moisture Variation Distance, e_M (Feet)	9.0	9.0	9.0
Center Lift, y_M (Inches)	0.30	0.47	0.66

9.8.10 The foundations for the post-tensioned slabs should be embedded in accordance with the recommendations of the structural engineer. If a post-tensioned mat foundation system is planned, the slab should possess a thickened edge with a minimum width of 12 inches and extend below the clean sand or crushed rock layer.

9.8.11 If the structural engineer proposes a post-tensioned foundation design method other than PTI, DC 10.5:

- The deflection criteria presented in Table 9.8.5 are still applicable.

- Interior stiffener beams should be used for Foundation Categories II and III.
- The width of the perimeter foundations should be at least 12 inches.
- The perimeter footing embedment depths should be at least 12 inches, 18 inches and 24 inches for foundation categories I, II, and III, respectively. The embedment depths should be measured from the lowest adjacent pad grade.

9.8.12 Our experience indicates post-tensioned slabs may be susceptible to excessive edge lift from tensioning, regardless of the underlying soil conditions. Placing reinforcing steel at the bottom of the perimeter footings and the interior stiffener beams may mitigate this potential. The structural engineer should design the foundation system to reduce the potential of edge lift occurring for the proposed structures.

9.8.13 During the construction of the post-tension foundation system, the concrete should be placed monolithically. Under no circumstances should cold joints form between the footings/grade beams and the slab during the construction of the post-tension foundation system unless designed by the structural engineer.

9.8.14 Isolated footings outside of the slab area, if present, should have the minimum embedment depth and width recommended for conventional foundations for a particular Foundation Category. The use of isolated footings, which are located beyond the perimeter of the building and support structural elements connected to the building, are not recommended for Category III. Where this condition cannot be avoided, the isolated footings should be connected to the building foundation system with grade beams in both directions. In addition, consideration should be given to connecting patio slabs, which exceed 5 feet in width, to the building foundation to reduce the potential for future separation to occur.

9.8.15 Interior stiffening beams should be incorporated into the design of the foundation system in accordance with the PTI design procedures.

9.8.16 Special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be moisture conditioned, as necessary, to maintain a moist condition as would be expected in any such concrete placement.

9.8.17 Where buildings or other improvements are planned near the top of a slope 3:1 (horizontal to vertical) or steeper, special foundation and/or design considerations are recommended due to the tendency for lateral soil movement to occur.

- For fill slopes less than 20 feet high or cut slopes regardless of height, footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

- When located next to a descending 3:1 (horizontal to vertical) fill slope or steeper, the foundations should be extended to a depth where the minimum horizontal distance is equal to H/3 (where H equals the vertical distance from the top of the fill slope to the base of the fill soil) with a minimum of 7 feet but need not exceed 40 feet. The horizontal distance is measured from the outer, deepest edge of the footing to the face of the slope. A post-tensioned slab and foundation system or mat foundation system can be used to reduce the potential for distress in the structures associated with strain softening and lateral fill extension. Specific design parameters or recommendations for either of these alternatives can be provided once the building location and fill slope geometry have been determined.

- If swimming pools are planned, Geocon Incorporated should be contacted for a review of specific site conditions.

- Swimming pools located within 7 feet of the top of cut or fill slopes are not recommended. Where such a condition cannot be avoided, the portion of the swimming pool wall within 7 feet of the slope face be designed assuming that the adjacent soil provides no lateral support. This recommendation applies to fill slopes up to 30 feet in height and cut slopes regardless of height. For swimming pools located near the top of fill slopes greater than 30 feet in height, additional recommendations may be required and Geocon Incorporated should be contacted for a review of specific site conditions.

- Although other improvements, which are relatively rigid or brittle, such as concrete flatwork or masonry walls, may experience some distress if located near the top of a slope, it is generally not economical to mitigate this potential. It may be possible, however, to incorporate design measures which would permit some lateral soil movement without causing extensive distress. Geocon Incorporated should be consulted for specific recommendations.

9.8.18 The recommendations of this report are intended to reduce the potential for cracking of slabs and foundations due to expansive soil (if present), differential settlement of fill soil with varying thicknesses. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade placed on such conditions may still exhibit some cracking due to soil movement and/or shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

9.8.19 Concrete slabs should be provided with adequate crack-control joints, construction joints and/or expansion joints to reduce unsightly shrinkage cracking. The design of joints should consider criteria of the American Concrete Institute when establishing crack-control spacing. Additional steel reinforcing, concrete admixtures and/or closer crack control joint spacing should be considered where concrete-exposed finished floors are planned.

9.8.20 Geocon Incorporated should be consulted to provide additional design parameters as required by the structural engineer.

9.8.21 We should observe the foundation excavations prior to the placement of reinforcing steel to check that the exposed soil conditions are similar to those expected and that they have been extended to the appropriate bearing strata. If unexpected soil conditions are encountered, foundation modifications may be required.

9.9 Exterior Concrete Flatwork

9.9.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations presented in Table 9.9. The recommended steel reinforcement would help reduce the potential for cracking.

TABLE 9.9
MINIMUM CONCRETE FLATWORK RECOMMENDATIONS

Expansion Index, EI	Minimum Steel Reinforcement* Options	Minimum Thickness
EI ≤ 90	6x6-W2.9/W2.9 (6x6-6/6) welded wire mesh	4 Inches
	No. 3 Bars 18 inches on center, Both Directions	

*In excess of 8 feet square.

9.9.2 The subgrade soil should be properly moisturized and compacted prior to the placement of steel and concrete. The subgrade soil should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM D 1557.

9.9.3 Even with the incorporation of the recommendations of this report, the exterior concrete flatwork has a potential to experience some uplift due to expansive soil beneath grade. The steel reinforcement should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork. Additionally, flatwork should be structurally connected to the curbs, where possible, to reduce the potential for offsets between the curbs and the flatwork.

9.9.4 Concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior to concrete placement. Subgrade soil should be properly compacted and the moisture content of subgrade soil should be verified prior to placing concrete. Base materials will not be required below concrete improvements.

9.9.5 Where exterior flatwork abuts the structure at entrant or exit points, the exterior slab should be dowelled into the structure's foundation stemwall. This recommendation is intended to reduce the potential for differential elevations that could result from differential settlement or minor heave of the flatwork. Dowelling details should be designed by the project structural engineer.

9.9.6 The recommendations presented herein are intended to reduce the potential for cracking of exterior slabs as a result of differential movement. However, even with the incorporation of the recommendations presented herein, slabs-on-grade will still crack. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, the use of crack control joints and proper concrete placement and curing. Crack control joints should be spaced at intervals no greater than 12 feet. Literature provided by the Portland Concrete Association (PCA) and American Concrete Institute (ACI) present recommendations for proper concrete mix, construction, and curing practices, and should be incorporated into project construction.

9.10 Retaining Walls

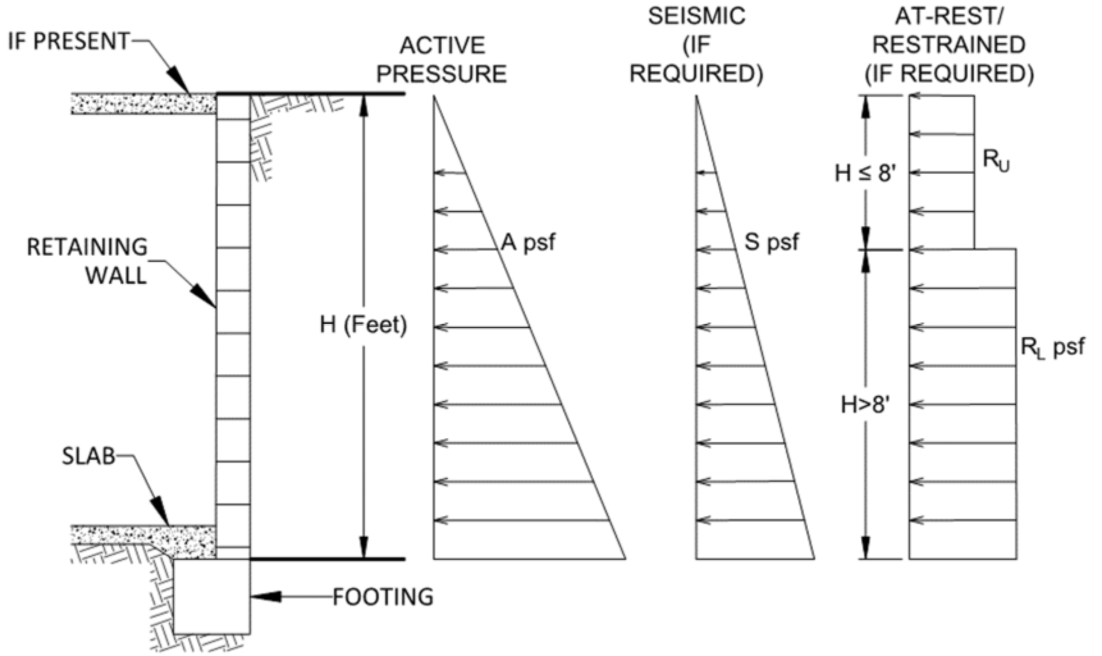
9.10.1 Retaining walls should be designed using the values presented in Table 9.10.1. Soil with an expansion index (EI) of greater than 90 should not be used as backfill material behind retaining walls.

TABLE 9.10.1
RETAINING WALL DESIGN RECOMMENDATIONS

Parameter	Value
Active Soil Pressure, A (Fluid Density, Level Backfill)	40 pcf
Active Soil Pressure, A (Fluid Density, 2:1 Sloping Backfill)	55 pcf
Seismic Pressure, S	12H psf
At-Rest/Restrained Walls Additional Uniform Pressure (0 to 8 Feet High)	7H psf
At-Rest/Restrained Walls Additional Uniform Pressure (8+ Feet High)	13H psf
Expected Expansion Index for the Subject Property	EI≤90

H equals the height of the retaining portion of the wall

9.10.2 The project retaining walls should be designed as shown in the Retaining Wall Loading Diagram.



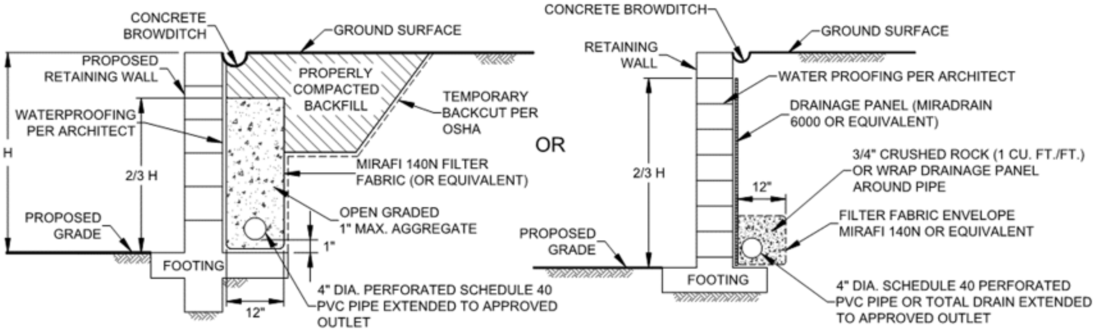
Retaining Wall Loading Diagram

9.10.3 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall) at the top of the wall. Where walls are restrained from movement at the top (at-rest condition), an additional uniform pressure should be applied to the wall. For retaining walls subject to vehicular loads within a horizontal distance equal to two-thirds the wall height, a surcharge equivalent to 2 feet of fill soil should be added.

9.10.4 The structural engineer should determine the Seismic Design Category for the project in accordance with Section 1613.3.5 of the 2022 CBC or Section 11.6 of ASCE 7-10. For structures assigned to Seismic Design Category of D, E, or F, retaining walls that support more than 6 feet of backfill should be designed with seismic lateral pressure in accordance with Section 1803.5.12 of the 2022 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall.

9.10.5 Retaining walls should be designed to ensure stability against overturning sliding, and excessive foundation pressure. Where a keyway is extended below the wall base with the intent to engage passive pressure and enhance sliding stability, it is not necessary to consider active pressure on the keyway.

9.10.6 Drainage openings through the base of the wall (weep holes) should not be used where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted granular (EI of 90 or less) free-draining backfill material with no hydrostatic forces or imposed surcharge load. The retaining wall should be properly drained as shown in the Typical Retaining Wall Drainage Detail. If conditions different than those described are expected, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.



Typical Retaining Wall Drainage Detail

9.10.7 The retaining walls may be designed using either the active and restrained (at-rest) loading condition or the active and seismic loading condition as suggested by the structural engineer. Typically, it appears the design of the restrained condition for retaining wall loading may be adequate for the seismic design of the retaining walls. However, the active earth pressure combined with the seismic design load should be reviewed and also considered in the design of the retaining walls.

9.10.8 In general, wall foundations should be designed in accordance with Table 9.10.2. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure. Therefore, retaining wall foundations should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

TABLE 9.10.2
SUMMARY OF RETAINING WALL FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Retaining Wall Foundation Width	12 inches
Minimum Retaining Wall Foundation Depth	12 Inches
Minimum Steel Reinforcement	Per Structural Engineer
Allowable Bearing Capacity	2,000 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 40 Feet

9.10.9 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls. In the event that other types of walls (such as mechanically stabilized earth [MSE] walls, soil nail walls, or soldier pile walls) are planned, Geocon Incorporated should be consulted for additional recommendations.

9.10.10 It is common to see retaining walls constructed in the areas of the elevator pits. The retaining walls should be properly drained and designed in accordance with the recommendations presented herein. If the elevator pit walls are not drained, the walls should be designed with an increased active pressure with an equivalent fluid density of 90 pcf. It is also common to see seepage and water collection within the elevator pit. The pit should be designed and properly waterproofed to prevent seepage and water migration into the elevator pit.

9.10.11 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependent on the wall height, the type of soil used for backfill, and loads acting on the wall. The retaining walls and improvements above the retaining walls should be designed to incorporate an appropriate amount of lateral deflection as determined by the structural engineer.

9.10.12 Soil contemplated for use as retaining wall backfill, including import materials, should be identified in the field prior to backfill. At that time, Geocon Incorporated should obtain samples for laboratory testing to evaluate its suitability. Modified lateral earth pressures may be necessary if the backfill soil does not meet the required expansion index or shear strength. City or regional standard wall designs, if used, are based on a specific active lateral earth pressure and/or soil friction angle. In this regard, on-site soil to be used as backfill may or may not meet the values for standard wall designs. Geocon Incorporated should be consulted to assess the suitability of the on-site soil for use as wall backfill if standard wall designs will be used.

9.11 Lateral Loading

9.11.1 Table 9.11 should be used to help design the proposed structures and improvements to resist lateral loads for the design of footings or shear keys. The allowable passive pressure assumes a horizontal surface extending at least 5 feet, or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

TABLE 9.11
SUMMARY OF LATERAL LOAD DESIGN RECOMMENDATIONS

Parameter	Value
Passive Pressure Fluid Density	350 pcf
Coefficient of Friction (Concrete and Soil)	0.35
Coefficient of Friction (Along Vapor Barrier)	0.2 to 0.25*

*Per manufacturer's recommendations.

9.11.2 The passive and frictional resistant loads can be combined for design purposes. The lateral passive pressures may be increased by one-third when considering transient loads due to wind or seismic forces.

9.12 Preliminary Pavement Recommendations

9.12.1 We calculated the flexible pavement sections in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4) using an estimated Traffic Index (TI) of 5.0, 5.5, 6.0, and 7.0 for parking stalls, driveways, medium truck traffic areas, and heavy truck traffic areas, respectively. The project civil engineer and owner should review the pavement designations to determine appropriate locations for pavement thickness. The final pavement sections for the parking lot should be based on the R-Value of the subgrade soil encountered at final subgrade elevation. We have assumed an R-Value of 10 (based on our experience in the area) and 78 for the subgrade soil and base materials, respectively, for the purposes of this preliminary analysis. Table 9.12.1 presents the preliminary flexible pavement sections.

TABLE 9.12.1
PRELIMINARY FLEXIBLE PAVEMENT SECTION

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Parking stalls for automobiles and light-duty vehicles	5.0	10	3	9
Driveways for automobiles and light-duty vehicles	5.5	10	3	11
Medium truck traffic areas	6.0	10	3.5	12
Driveways for heavy truck traffic	7.0	10	4	15

9.12.2 Prior to placing base materials, the upper 12 inches of the subgrade soil should be scarified, moisture conditioned as necessary, and recompacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM D 1557. Similarly, the base material should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Asphalt concrete should be compacted to a density of at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

9.12.3 Base materials should conform to Section 26-1.028 of the *Standard Specifications for The State of California Department of Transportation (Caltrans)* with a ¾-inch maximum size aggregate. The asphalt concrete should conform to Section 203-6 of the *Standard Specifications for Public Works Construction (Greenbook).*

9.12.4 The base thickness can be reduced if a reinforcement geogrid is used during the installation of the pavement. Geocon should be contact for additional recommendations, if required.

9.12.5 A rigid Portland cement concrete (PCC) pavement section should be placed in roadway aprons and cross gutters. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330-21 *Commercial Concrete Parking Lots and Site Paving Design and Construction – Guide.* Table 9.12.2 provides the traffic categories and design parameters used for the calculations for 20-year design life.

TABLE 9.12.2
TRAFFIC CATEGORIES

Traffic Category	Description	Reliability (%)	Slabs Cracked at End of Design Life (%)
A	Car Parking Areas and Access Lanes	60	15
B	Entrance and Truck Service Lanes	60	15
E	Garbage or Fire Truck Lane	75	15

9.12.6 We used the parameters presented in Table 9.12.3 to calculate the pavement design sections. We should be contacted to provide updated design sections, if necessary.

TABLE 9.12.3
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	100 pci
Modulus of rupture for concrete, M_R	500 psi
Concrete Compressive Strength	3,000 psi
Concrete Modulus of Elasticity, E	3,150,000

9.12.7 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 9.12.4.

TABLE 9.12.4
RIGID VEHICULAR PAVEMENT RECOMMENDATIONS

Traffic Category	Trucks Per Day	Portland Cement Concrete, T (Inches)
A = Car Parking Areas and Access Lanes	10	5½
B = Entrance and Truck Service Lanes	10	6
	50	6½
E = Garbage or Fire Truck Lanes	5	6½
	10	7

9.12.8 The PCC vehicular pavement should be placed over subgrade soil that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Base materials will not be required below concrete improvements.

9.12.9 Adequate joint spacing should be incorporated into the design and construction of the rigid pavement in accordance with Table 9.12.5.

TABLE 9.12.5
MAXIMUM JOINT SPACING

Pavement Thickness, T (Inches)	Maximum Joint Spacing (Feet)
$4<T<5$	10
$5\leq T<6$	12.5
$6\leq T$	15

9.12.10 The rigid pavement should also be designed and constructed incorporating the parameters presented in Table 9.12.6.

TABLE 9.12.6
ADDITIONAL RIGID PAVEMENT RECOMMENDATIONS

Subject	Value
Thickened Edge	1.2 Times Slab Thickness Adjacent to Structures
	1.5 Times Slab Thickness Adjacent to Soil
	Minimum Increase of 2 Inches
	4 Feet Wide
Crack Control Joint Depth	Early Entry Sawn = T/6 to T/5, 1.25 Inch Minimum
	Conventional (Tooled or Conventional Sawing) = T/4 to T/3
Crack Control Joint Width	¼-Inch for Sealed Joints and Per Sealer Manufacturer's Recommendations
	$^{1}/_{16}$- to $^{1}/_{4}$-Inch is Common for Unsealed Joints

9.12.11 Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed herein.

9.12.12 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. The depth of the crack-control joints should be in accordance with the referenced ACI guide.

9.12.13 To provide load transfer between adjacent pavement slab sections, a butt-type construction joint should be constructed. The butt-type joint should be thickened by at least 20 percent at the edge and taper back at least 4 feet from the face of the slab.

9.12.14 Concrete curb/gutter should be placed on soil subgrade compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Cross-gutters that receives vehicular should be placed on subgrade soil compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Base materials should not be placed below the curb/gutter, or cross-gutters so water is not able to migrate from the adjacent parkways to the pavement sections. Where flatwork is located directly adjacent to the curb/gutter, the concrete flatwork should be structurally connected to the curbs to help reduce the potential for offsets between the curbs and the flatwork.

9.13 Site Drainage and Moisture Protection

9.13.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2022 CBC 1804.4 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

9.13.2 In the case of basement walls or building walls retaining landscaping areas, a water-proofing system should be used on the wall and joints, and a Miradrain drainage panel (or similar) should be placed over the waterproofing. The project architect or civil engineer should provide detailed specifications on the plans for all waterproofing and drainage.

9.13.3 Underground utilities should be leak free. Utility and irrigation lines should be checked periodically for leaks, and detected leaks should be repaired promptly. Detrimental soil movement could occur if water is allowed to infiltrate the soil for prolonged periods of time.

9.13.4 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement’s subgrade and base course. Area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes can be used. In addition, where landscaping is planned adjacent to the pavement, construction of a cutoff wall along the edge of the pavement that extends at least 6 inches below the bottom of the base material should be considered.

9.13.5 We should prepare a storm water infiltration feasibility report of storm water management devices are planned.

9.14 Grading and Foundation Plan Review

9.14.1 Geocon Incorporated should review the grading and building foundation plans for the project prior to final design submittal to evaluate if additional analyses and/or recommendations are required.

9.15 Testing and Observation Services During Construction

9.15.1 Geocon Incorporated should provide geotechnical testing and observation services during the grading operations, foundation construction, utility installation, retaining wall backfill and pavement installation. Table 9.15 presents the typical geotechnical observations we would expect for the proposed improvements.

TABLE 9.15
EXPECTED GEOTECHNICAL TESTING AND OBSERVATION SERVICES

Construction Phase	Observations	Expected Time Frame
Grading	Base of Removal	Part Time
	Geologic Logging	Part Time to Full Time
	Fill Placement and Soil Compaction	Full Time
	Buttress Construction	Full Time
Foundations	Foundation Excavation Observations	Part Time to Full Time
Utility Backfill	Fill Placement and Soil Compaction	Part Time to Full Time
Retaining Wall Backfill	Fill Placement and Soil Compaction	Part Time to Full Time
Subgrade for Sidewalks, Curb/Gutter and Pavement	Soil Compaction	Part Time
Pavement Construction	Base Placement and Compaction	Part Time
	Asphalt Concrete Placement and Compaction	Full Time

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

2. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Incorporated should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon Incorporated.

3. This report is issued with the understanding that it is the responsibility of the owner or his representative to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

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

GEOCON LEGEND

Qpf	........PREVIOUSLY PLACED FILL (2005-2006)
Qal	........ALLUVIUM (Dotted Where Buried)
To	........OTAY FORMATION (Dotted Where Buried)
B-3 ⊕	........APPROX. LOCATION OF GEOTECHNICAL BORING
540	........APPROX. ELEVATION AT BASE OF FILL (In Feet, MSL)
	........APPROX. LOCATION OF GEOLOGIC CONTACT
•••••	........APPROX. SURFACE EXPOSURE OF BENTONITE CLAYSTONE BED
∘∘∘∘∘	........APPROX. AREA OF BURIED BENTONITE CLAYSTONE BED
D D'	........APPROX. LOCATION OF GEOLOGIC CROSS-SECTION
W=20'	........APPROX. WIDTH OF BUTTRESS (In Feet)
🟨	........APPROX. LOCATION OF PROPOSED BUTTRESS
🟩	........APPROX. LOCATION OF PROPOSED STABILITY FILL

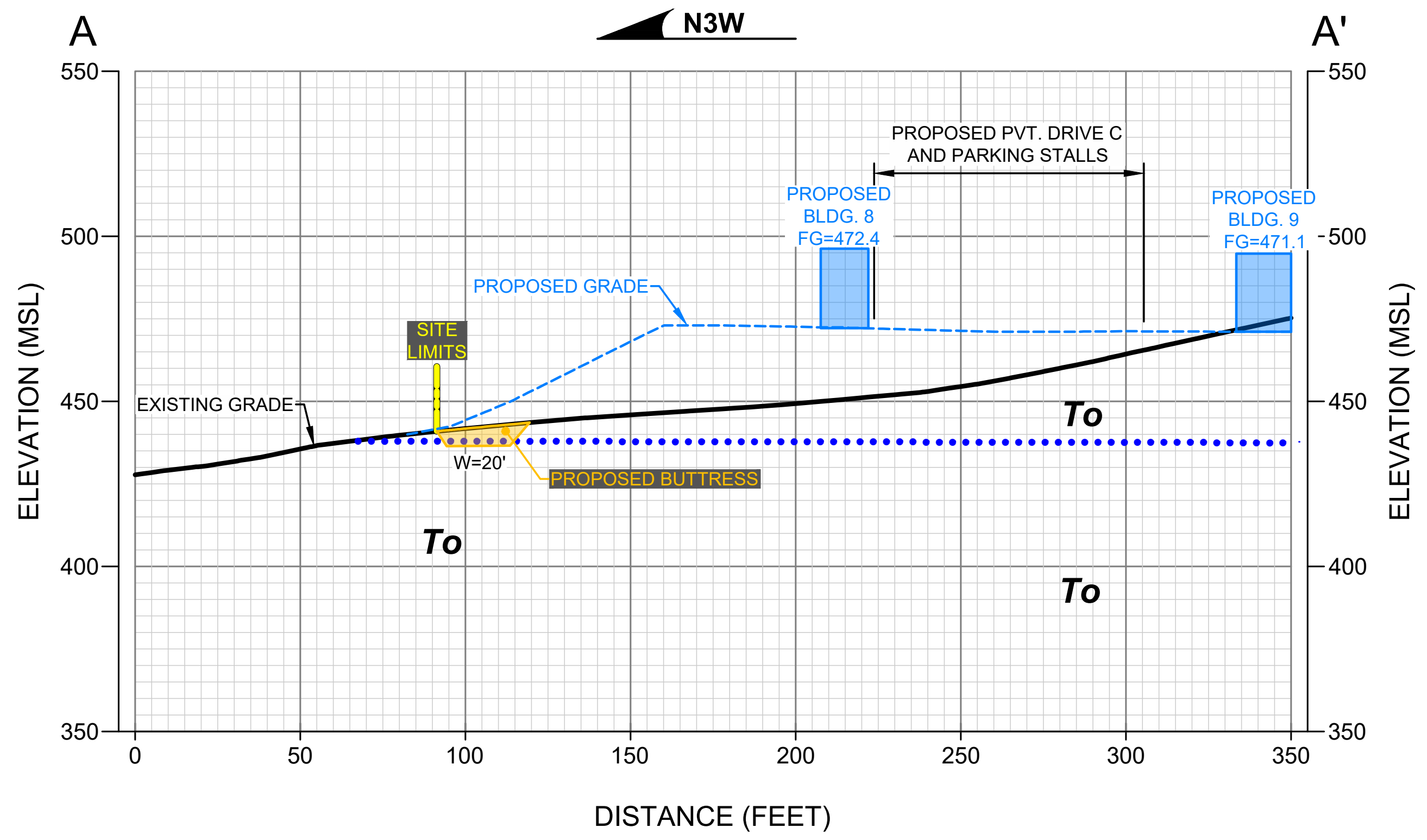
GEOLOGIC MAP

OTAY RANCH VILLAGE 7 - SOUTH
CHULA VISTA, CALIFORNIA

GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

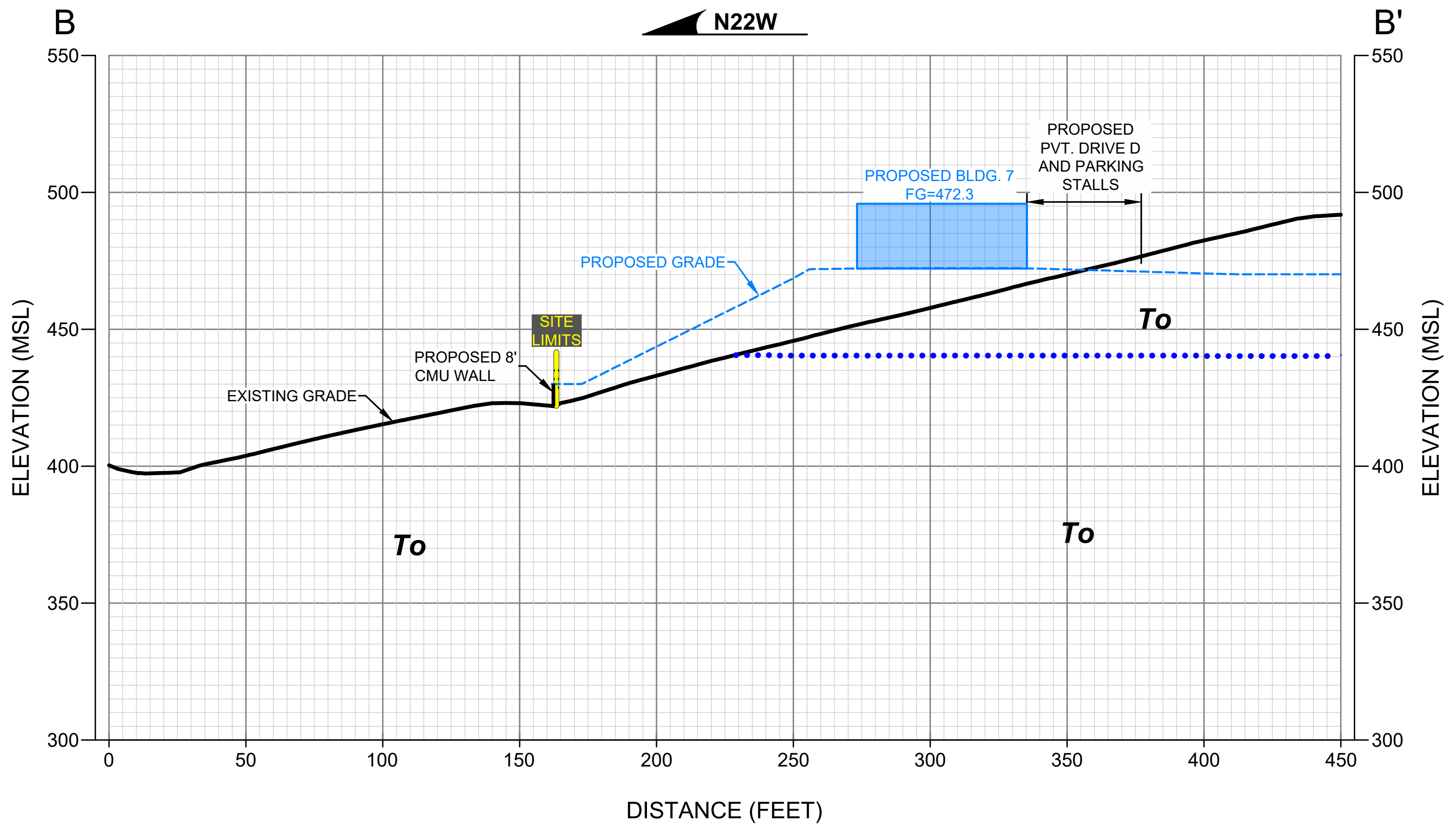
SCALE 1" = 50'
DATE 05 - 09 - 2024
PROJECT NO. 06862 - 52 - 67
SHEET 1 OF 1
FIGURE 1

Plotted:05/08/2024 2:37PM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\06862-52-67 Otay Ranch Village 7 - Vortex\SHEETS\06862-52-67 GeologicMAP.dwg



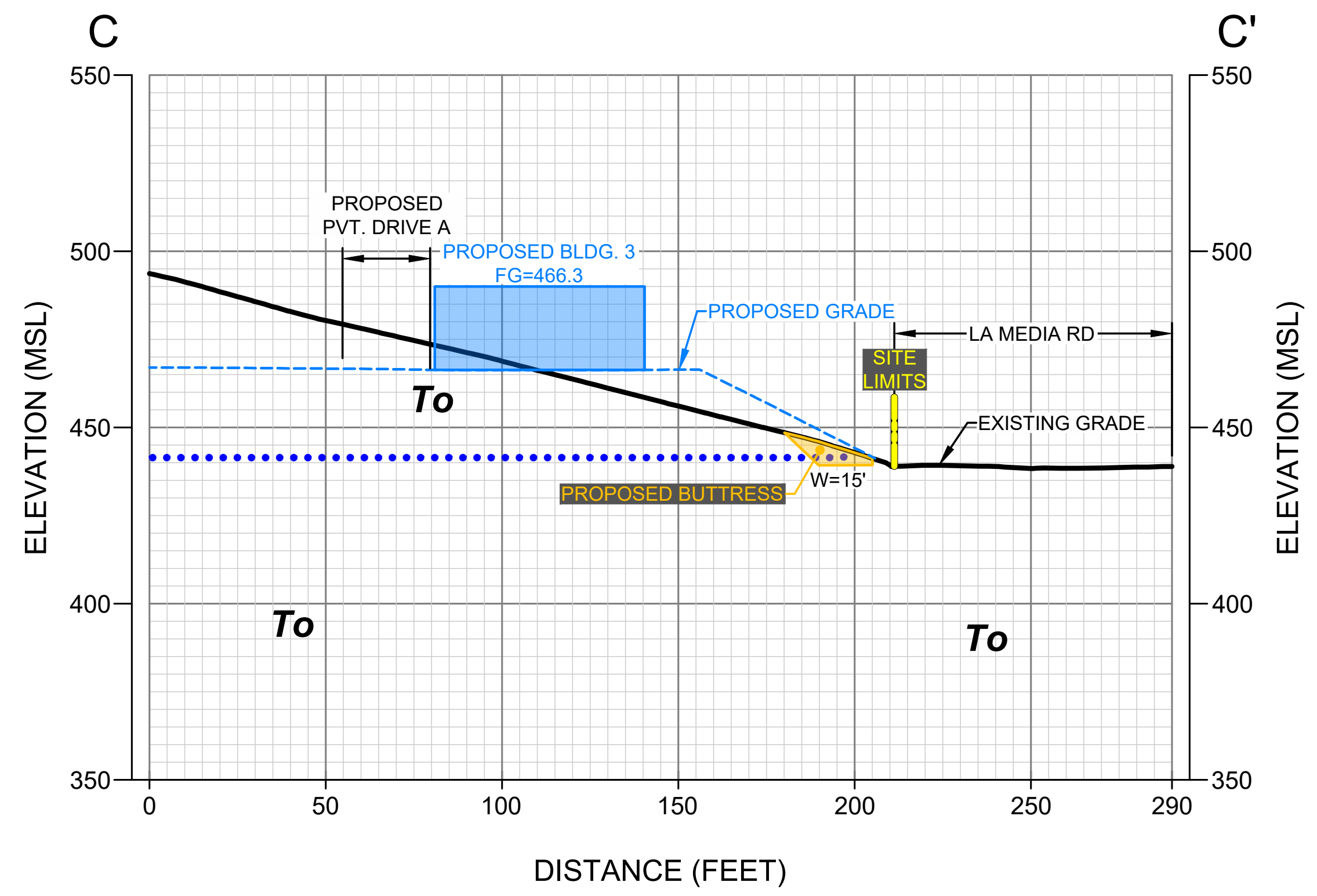
GEOLOGIC CROSS-SECTION A-A'

SCALE: 1" = 50' (Vert. = Horiz.)



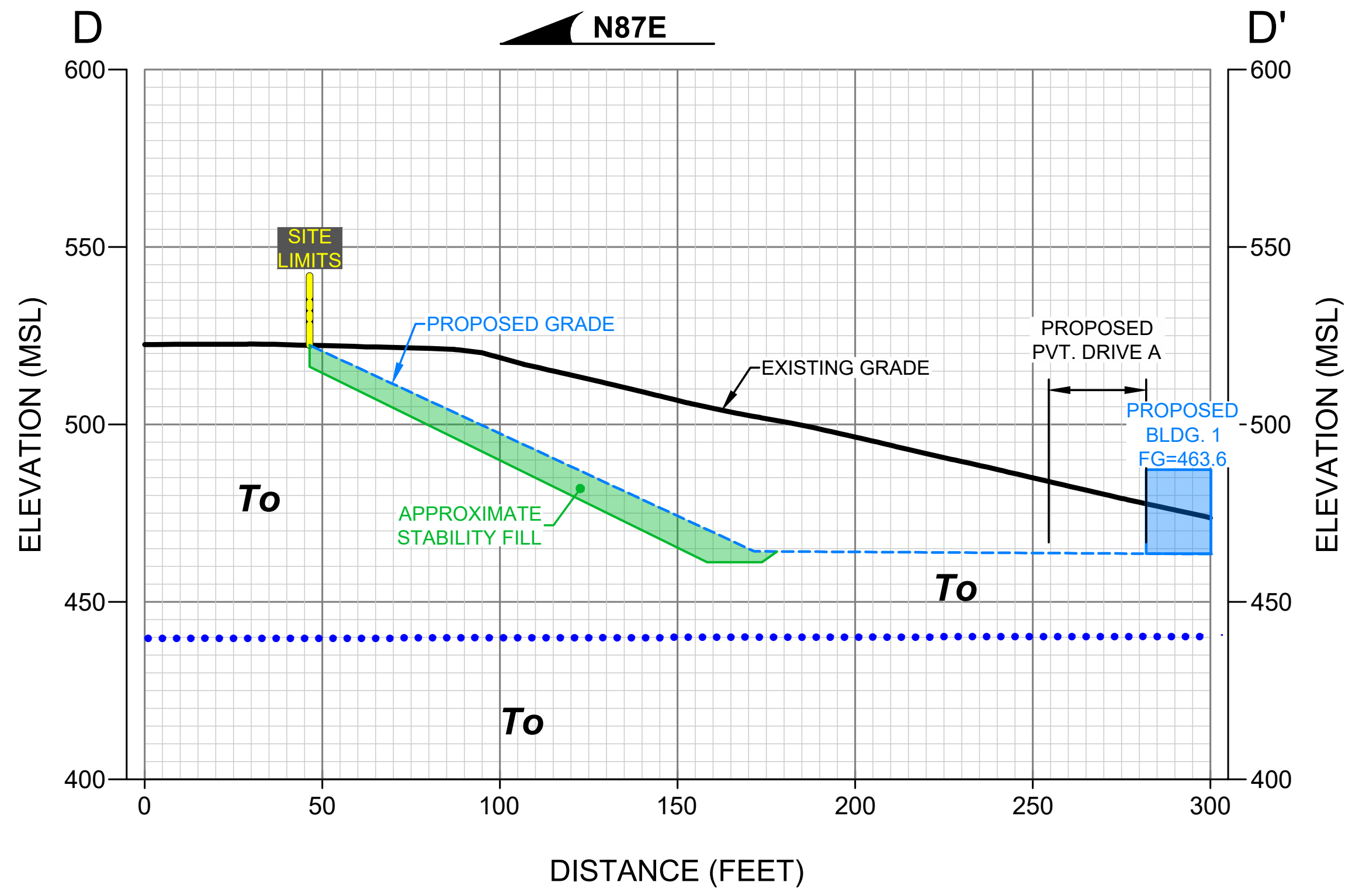
GEOLOGIC CROSS-SECTION B-B'

SCALE: 1" = 50' (Vert. = Horiz.)



GEOLOGIC CROSS-SECTION C-C'

SCALE: 1" = 50' (Vert. = Horiz.)



GEOLOGIC CROSS-SECTION D-D'

SCALE: 1" = 50' (Vert. = Horiz.)

GEOCON LEGEND

To........OTAY FORMATION

••••••........APPROX. ELEVATION OF BENTONITE CLAYSTONE

GEOLOGIC CROSS - SECTIONS

OTAY RANCH VILLAGE 7 - SOUTH
CHULA VISTA, CALIFORNIA

GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

SCALE 1" = 50'
DATE 05 - 09 - 2024
PROJECT NO. 06862 - 52 - 67
FIGURE 2
SHEET 1 OF 1

Plotted:05/09/2024 7:42AM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\06862-52-67 Otay Ranch Village 7 - Vortex\SHEETS\06862-52-67 Geologic Cross-Section.dwg

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

We performed the drilling operations on October 7 and 8, 2021 with Dave’s Drilling using an EZ Bore 120 drill rig equipped with a 30-inch diameter bucket-auger. Borings extended to maximum depth of approximately 60 feet. The locations of the current exploratory borings are shown on the Geologic Map, Figure 1. The boring logs are presented in this Appendix. We located the borings in the field using a measuring tape and existing reference points; therefore, actual boring locations may deviate slightly.

We obtained samples during our subsurface exploration in the borings using a California sampler that is composed of steel and is driven to obtain ring samples. The California sampler has an inside diameter of 2.5 inches and an outside diameter of 3 inches. Up to 18 rings are placed inside the sampler that is 2.4 inches in diameter and 1 inch in height. We obtained ring samples at appropriate intervals, placed them in moisture-tight containers, and transported them to the laboratory for testing. The type of sample is noted on the exploratory boring logs.

The large-diameter boring sampler was driven up to 12 inches into the bottom of the excavation with the use of a telescoping Kelly bar. The weight of the Kelly bar (4,500 pounds maximum) drives the sampler and varies in weight and depth. The height of drop is usually 18 inches. Blow counts are recorded for every 12 inches the sampler is driven. The penetration resistance values shown on the boring logs are shown in terms of blow per foot. These values are not be taken as N-values and adjustments have not been applied. We obtained ring samples at appropriate intervals, placed them in moisture-tight containers, and transported them to the laboratory for testing.

We visually examined, classified, and logged the soil encountered in the borings in general accordance with American Society for Testing and Materials (ASTM) practice for Description and Identification of Soils (Visual-Manual Procedure D 2488). The logs depict the soil and geologic conditions observed and the depth at which samples were obtained.

PROJECT NO. 06862-52-67

BORING B 1

ELEV. (MSL.) 486' DATE COMPLETED 10-07-2021

EQUIPMENT EZ-BORE W/ 30" BUCKET AUGER BY: K. HAASE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SC	**PREVIOUSLY PLACED FILL (Qpf)** Loose to medium dense, damp to moist, Clayey, fine to medium SAND			
				CL	Stiff, moist, dark brown, fine to medium, Sandy CLAY			
6				ML	**OTAY FORMATION (To)** Hard, moist, light olive to light gray, Sandy SILTSTONE; micaceous, weakly cemented			
10	B1-1 ■				-Less olive	6	104.2	19.6
					-Becomes moderately cemented			
16				CL	Hard, moist, olive brown, Sandy CLAYSTONE			
				SM	Very dense, moist, light olive gray, Silty, fine- grained SANDSTONE; moderately cemented, massive			
20	B1-2 ■					7	114.6	15.2
				CL	Hard, moist, olive brown, fine- grained, Sandy CLAYSTONE			
				SM	Very dense, moist, olive, Silty, fine- to medium- grained SANDSTONE; massive, moderately cemented			
24					-Becomes light gray, finer grained, weakly cemented			

Figure A-1,
Log of Boring B 1, Page 1 of 3

06862-52-67.GPJ

SAMPLE SYMBOLS			
	▢ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	⊠ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR ▽ ... SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

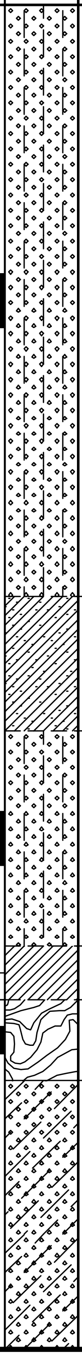
GEOCON

PROJECT NO. 06862-52-67

BORING B 1

ELEV. (MSL.) 486' DATE COMPLETED 10-07-2021

EQUIPMENT EZ-BORE W/ 30" BUCKET AUGER BY: K. HAASE



DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
26						
28						
30	B1-3			15	116.2	14.7
32						
34						
			-Becomes coarse grained			
36		CL	Hard, moist, olive, Sandy CLAYSTONE; abrupt contact with unit above, N55E, 2°NW			
38						
		SM	Very dense, moist, light gray, Silty, fine- grained SANDSTONE; poorly cemented, micaceous			
40	B1-4			15	117.7	12.1
42						
	B1-5	CL	Hard, moist, olive, CLAYSTONE			
44	B1-6 B1-7	CH	43.4 feet to 45 feet: BENTONITE CLAYSTONE; gray to white to pink, blocky, intact. Layer is continuous around hole; N60E at 3°NW	15/6"	119.9	12.8
		SC	Very dense, moist, olive brown, Clayey, fine- to medium- grained SANDSTONE; very well cemented; massive			
46						
48						

Figure A-1,
Log of Boring B 1, Page 2 of 3

06862-52-67.GPJ

SAMPLE SYMBOLS

▢ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
▩ ... DISTURBED OR BAG SAMPLE
◣ ... CHUNK SAMPLE
▼ ... WATER TABLE OR ▽ ... SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. 06862-52-67

BORING B 1

ELEV. (MSL.) **486'** DATE COMPLETED **10-07-2021**

EQUIPMENT **EZ-BORE W/ 30" BUCKET AUGER** BY: **K. HAASE**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
50								
52								
					-Becomes coarser grained			
54								
56								
58								
60					BOTTOM OF HOLE AT 60 FEET No groundwater or seepage encountered No caving			

Figure A-1,
Log of Boring B 1, Page 3 of 3

06862-52-67.GPJ

SAMPLE SYMBOLS

- ▢ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ☒ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR ▽ ... SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. 06862-52-67

BORING B 2

ELEV. (MSL.) **512'** DATE COMPLETED **10-07-2021**

EQUIPMENT **EZ-BORE W/ 30" BUCKET AUGER** BY: **K. HAASE**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SC	**TOPSOIL** Loose to medium dense, damp to moist, dark brown, Clayey, fine to medium SAND; little roots			
2								
4				ML	**OTAY FORMATION (To)** Very dense, damp, light gray, Sandy SILTSTONE; highly weathered			
				CL	Hard, moist, olive to reddish brown CLAYSTONE			
6				SM	Very dense, moist, olive brown, Silty, fine- grained SANDSTONE; interbedded thin claystone layers, moderately cemented			
8								
10					-Becomes light grayish brown, well cemented			
12								
14								
16								
				CL	Hard, moist, olive, Sandy CLAYSTONE			
18				SM	Very dense, moist, light gray, Silty, fine- grained SANDSTONE; well cemented; massive			
20								
22								
24								

Figure A-2,
Log of Boring B 2, Page 1 of 2

06862-52-67.GPJ

SAMPLE SYMBOLS

▒ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
⊠ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR ▽ ... SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. 06862-52-67

BORING B 2

ELEV. (MSL.) **512'** DATE COMPLETED **10-07-2021**

EQUIPMENT **EZ-BORE W/ 30" BUCKET AUGER** BY: **K. HAASE**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
26								
				CL	Hard, moist, brown, CLAYSTONE			
28								
				SM	Very dense, moist, light olive to light gray, Silty, fine- grained SANDSTONE			
30								
32								
34								
					BOTTOM OF HOLE AT 35 FEET No groundwater or seepage encountered No caving			

Figure A-2,
Log of Boring B 2, Page 2 of 2

SAMPLE SYMBOLS
- ▢ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR ▽ ... SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. 06862-52-67

BORING B 3

ELEV. (MSL.) 461' DATE COMPLETED 10-08-2021

EQUIPMENT EZ-BORE W/ 30" BUCKET AUGER BY: K. HAASE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				CL	**PREVIOUSLY PLACED FILL (Qpf)** Loose, damp, brown, Sandy CLAY, trace of organics			
2								
				SM	**OTAY FORMATION (To)** Dense, damp to moist, light gray, Silty, fine- to medium- grained SANDSTONE; friable, moderately cemented, massive			
4								
6				CL	Stiff, moist, brown, Sandy CLAYSTONE; highly fractured (bedding at N47E at 4°NW)			
8				SC	Dense, moist, light gray, Clayey, fine- to medium- grained SANDSTONE; moderately cemented			
10	B3-1	■				6	117.6	10.8
12								
					-Becomes coarser grained			
14								
	B3-2	■			-Becomes olive gray	10	111.0	17.7
16	B3-3	⊠						
18								
20	B3-4	■				10	107.9	19.0
	B3-5	■		CH	21.2 feet to 22.3 feet: BENTONITE CLAYSTONE; gray to white to pink, blocky, fractured, waxy. Layer is continuous around hole; N70E at 4°NW			
22				SC	Dense, moist, olive gray, Clayey, fine- to coarse- grained SANDSTONE; trace gravel, moderately cemented, massive			
24								

Figure A-3,
Log of Boring B 3, Page 1 of 3

06862-52-67.GPJ

SAMPLE SYMBOLS			
	▢ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	⊠ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR ▽ ... SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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BORING B 3

ELEV. (MSL.) **461'** DATE COMPLETED **10-08-2021**

EQUIPMENT **EZ-BORE W/ 30" BUCKET AUGER** BY: **K. HAASE**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
	B3-6	■				12	124.1	10.2
26								
28								
					-Becomes very dense, difficult drilling			
30	B3-7	■				30/6"	128.2	7.7
32								
34								
36								
					-Becomes very coarse grained, gravelly			
38								
			▽		-Minor seepage			
40	B3-8	■				30/6"	129.6	9.0
42								
44					-Becomes finer grained			
46								
			▽		-Minor seepage			
48								

Figure A-3,
Log of Boring B 3, Page 2 of 3

06862-52-67.GPJ

SAMPLE SYMBOLS
- ▢ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ☒ ... DISTURBED OR BAG SAMPLE
- ◩ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR ▽ ... SEEPAGE

BORING B 3

ELEV. (MSL.) **461'** DATE COMPLETED **10-08-2021**

EQUIPMENT **EZ-BORE W/ 30" BUCKET AUGER** BY: **K. HAASE**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
50	B3-9 ■	▨			-No recovery	30/2"		
52		▨						
54		▨			-Difficult drilling			

BORING TERMINATED AT 55 FEET
No groundwater encountered. Minor seepage encountered at 39 and 47 feet
No caving

Figure A-3,
Log of Boring B 3, Page 3 of 3

06862-52-67.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE	◣ ... CHUNK SAMPLE	▼ ... WATER TABLE OR ▽ ... SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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APPENDIX B

APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. We tested selected soil samples for in-place dry density/moisture content, plasticity index, unconfined compressive strength, gradation and residual shear strength. The results of our current laboratory tests are presented herein. The in-place dry density and moisture content of the samples tested are presented on the boring logs in Appendix A.

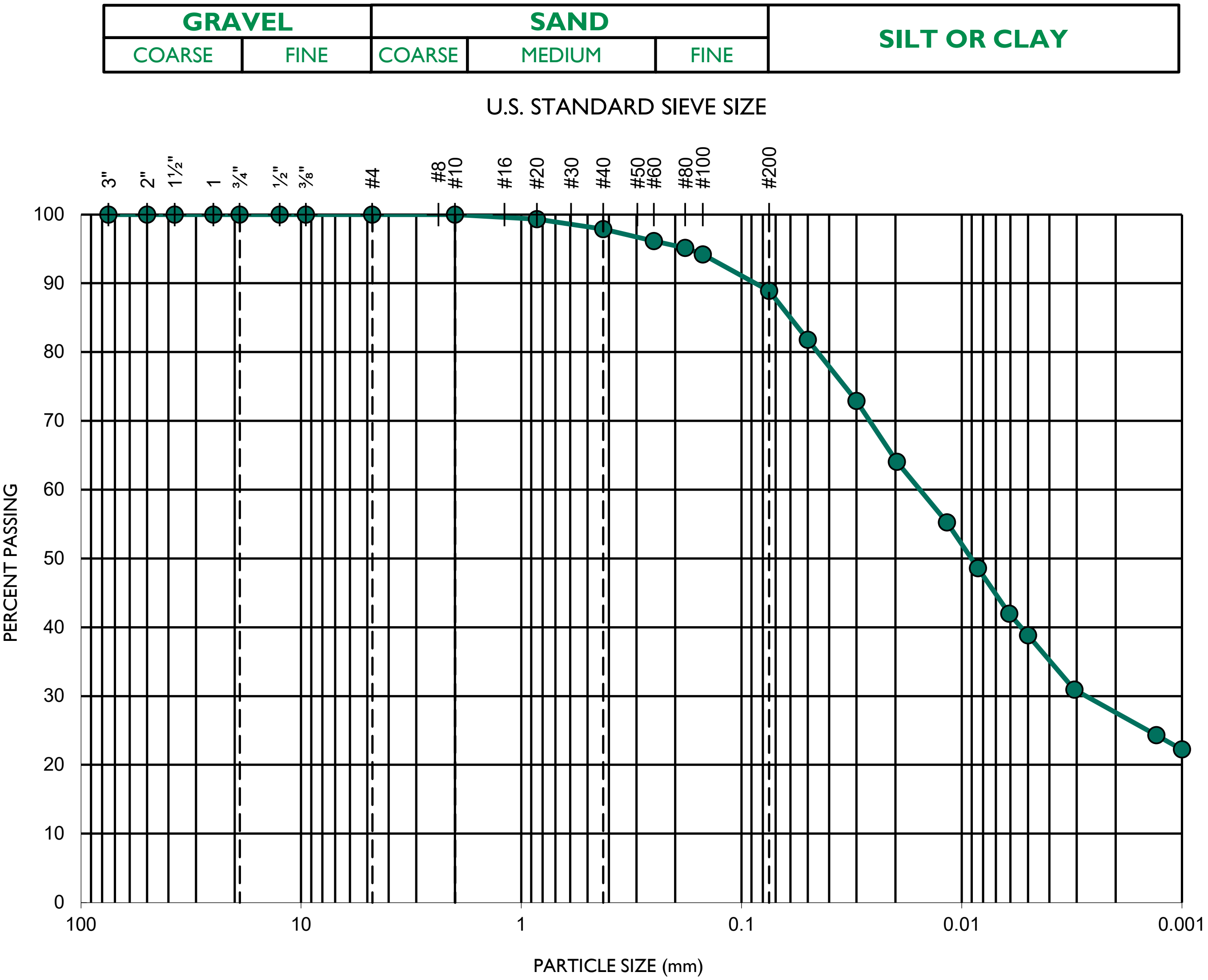
SUMMARY OF LABORATORY UNCONFINED COMPRESSIVE STRENGTH TEST RESULTS
ASTM D 1558

Sample No.	Depth (feet)	Geologic Unit	Hand Penetrometer Reading/Unconfined Compression Strength (tsf) and Undrained Shear Strength (ksf)
B1-1	10	To	4.5+
B1-2	20	To	4.5+
B1-3	30	To	4.5+
B1-4	40	To	4.5+
B1-7	44	To	4.5+
B3-1	10	To	4.5+
B3-2	15	To	4.5+
B3-4	20	To	4.5+
B3-6	25	To	4.5+
B3-7	30	To	4.5+
B3-8	40	To	4.5+

PREVIOUS GRADING LABORATORY TEST RESULTS
SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
3	Grayish brown, fine to medium, Sandy SILT	106.9	18.9
10	Light olive brown, fine, Sandy CLAY	112.2	15.8
15	Light brown, Silty SAND	115.2	14.3

SAMPLE NO.: **B3-5**
SAMPLE DEPTH (FT.): **21'**
GEOLOGIC UNIT: **Tob**



TEST DATA					
D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	C_c	C_u	SOIL DESCRIPTION
--	0.00282	0.01598	--	--	CLAY

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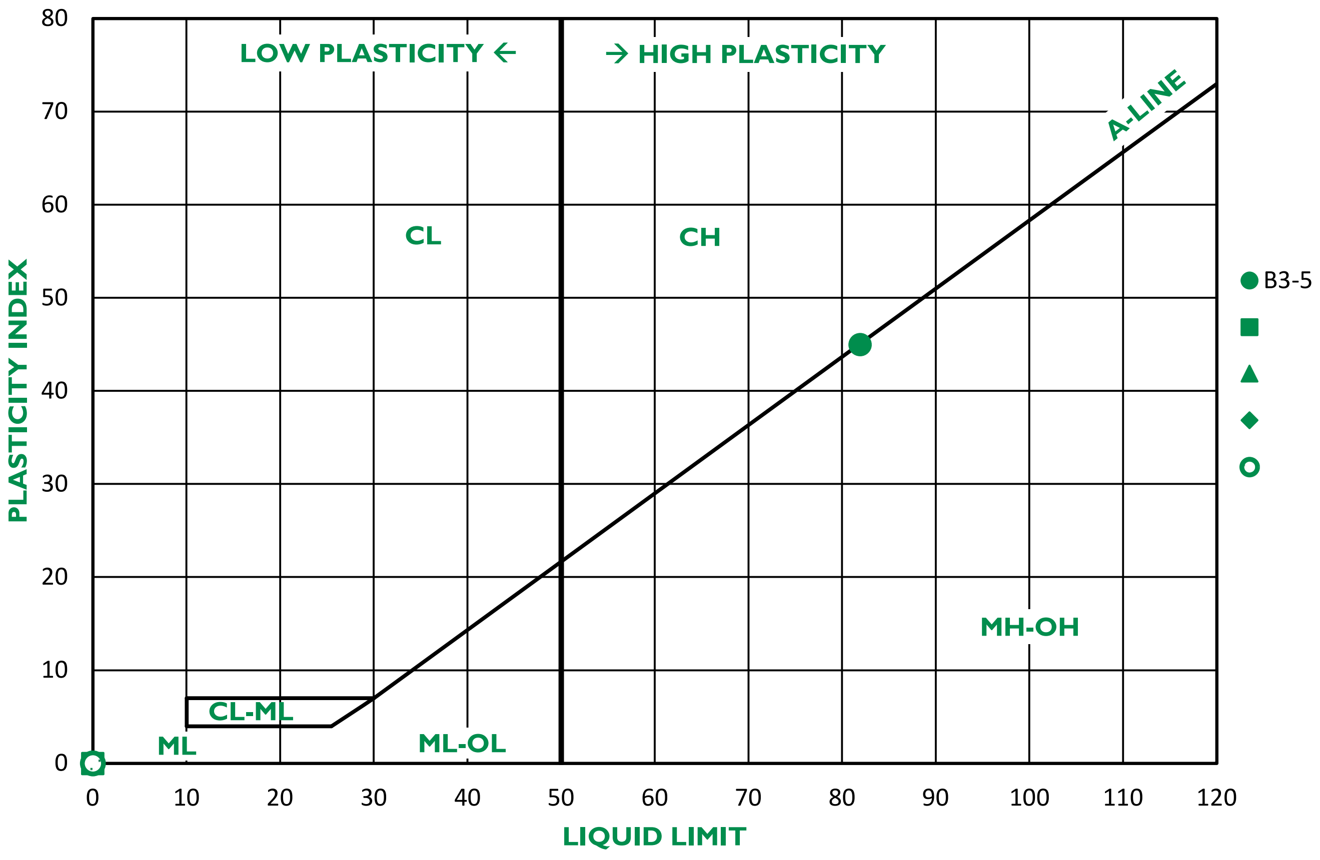
PHONE 858 558-6900 - FAX 858 558-6159

SIEVE ANALYSES - ASTM D 135 & D 422

OTAY RANCH VILLAGE 7, R-8

PROJECT NO.: 06862-52-67

TEST RESULTS					
SAMPLE NO.	GEOLOGIC UNIT	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	SOIL TYPE
B3-5	Tob @ 21'	82	37	45	CH



SOIL TYPE DESCRIPTION	
CH	High-Plasticity Clay
CL	Low-Plasticity Clay
ML	Low-Plasticity Silt
CL-ML	Low-Plasticity Clay to Low-Plasticity Silt
MH-OH	High-Plasticity Silt to High-Plasticity, Organic Silt
ML-OL	Low-Plasticity Silt to Low-Plasticity, Organic Silt

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PLASTICITY INDEX - ASTM D 4318

OTAY RANCH VILLAGE 7, R-8

PROJECT NO.: 06862-52-67

SAMPLE NO.: **B3-5**
GEOLOGIC UNIT: **Tob**
SAMPLE DEPTH (FT): **21**
NATURAL/REMOLDED: **N**

INITIAL CONDITIONS

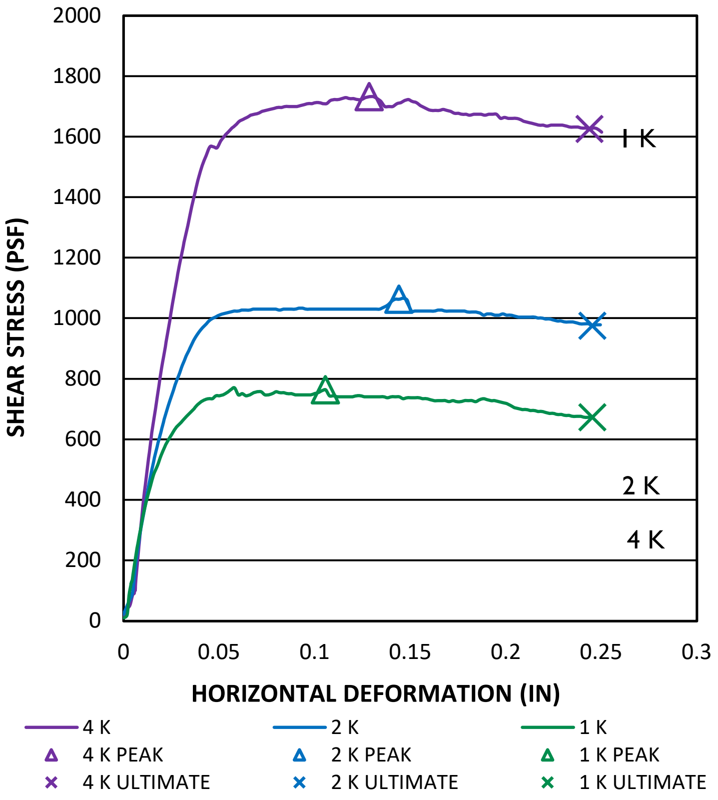
NORMAL STRESS TEST LOAD	4 K	2 K	1 K	AVERAGE
ACTUAL NORMAL STRESS (PSF):	4000	2000	1000	--
WATER CONTENT (%):	44.3	44.3	44.3	44.3
DRY DENSITY (PCF):	68.3	68.3	68.3	68.3

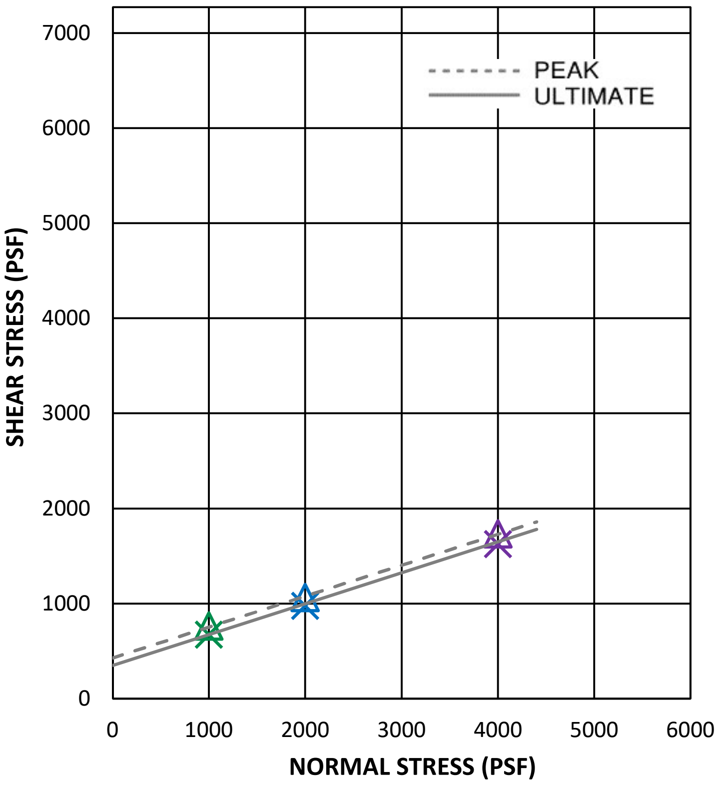
AFTER TEST CONDITIONS

NORMAL STRESS TEST LOAD	4 K	2 K	1 K	AVERAGE
WATER CONTENT (%):	53.7	53.7	53.7	53.7
PEAK SHEAR STRESS (PSF):	1732	1063	764	--
ULT.-E.O.T. SHEAR STRESS (PSF):	1625	975	673	--

RESULTS

PEAK	COHESION, C (PSF)	430
	FRICTION ANGLE (DEGREES)	18
ULTIMATE	COHESION, C (PSF)	350
	FRICTION ANGLE (DEGREES)	18





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DIRECT SHEAR - ASTM D 3080

OTAY RANCH VILLAGE 7, R-8

PROJECT NO.: 06862-52-67

APPENDIX C

APPENDIX C

SLOPE STABILITY ANALYSIS

We performed slope stability analyses using a two-dimensional computer software *GeoStudio 2018* developed by Geo-Slope International Ltd. We analyzed the critical modes of potential slip surfaces including rotational-mode and block-mode based on Spencer’s method. The soil parameters used, case conditions, and the calculated factors of safety were presented herein. Plots of analyses results, including the soil stratigraphy, potential failure surfaces, and calculated Factors of Safety, are included in this appendix.

Shear strength characters of the existing geologic units were estimated based on laboratory direct shear tests on samples obtained during our field investigation in accordance with ASTM D 3080 (see Appendix B) and based on empirical data obtained from the referenced geotechnical literature. Table C-I presents the soil parameters used for the stability analyses.

TABLE C-I
SUMMARY OF SOIL PROPERTIES USED FOR SLOPE STABILITY ANALYSES

Geologic Unit/Material	Density (pcf)	Cohesion (psf)	Friction Angle (degrees)
Compacted Fill (Qcf)	130	300	28
Otay Formation (To)	130	325	33
Otay Formation Bentonite (Tob)	130	50	10

We selected Geologic Cross-Sections A-A’ through D-D’ to perform the slope stability analyses. Table C-II provides a summary of cases analyzed and calculated factors of safety. A minimum Factor of Safety of 1.5 under static conditions is currently required by the City of Chula Vista for slope stability. Results of slope stability analyses generated by *GeoStudio 2018* are plotted herein. As discussed herein, we encountered claystone layers in several of the exploratory borings within the Otay Formation. The claystone possesses relatively low shear strengths and may be prone to slope instability if exposed in near cut and natural slopes. Surficial slope stability calculations are presented herein.

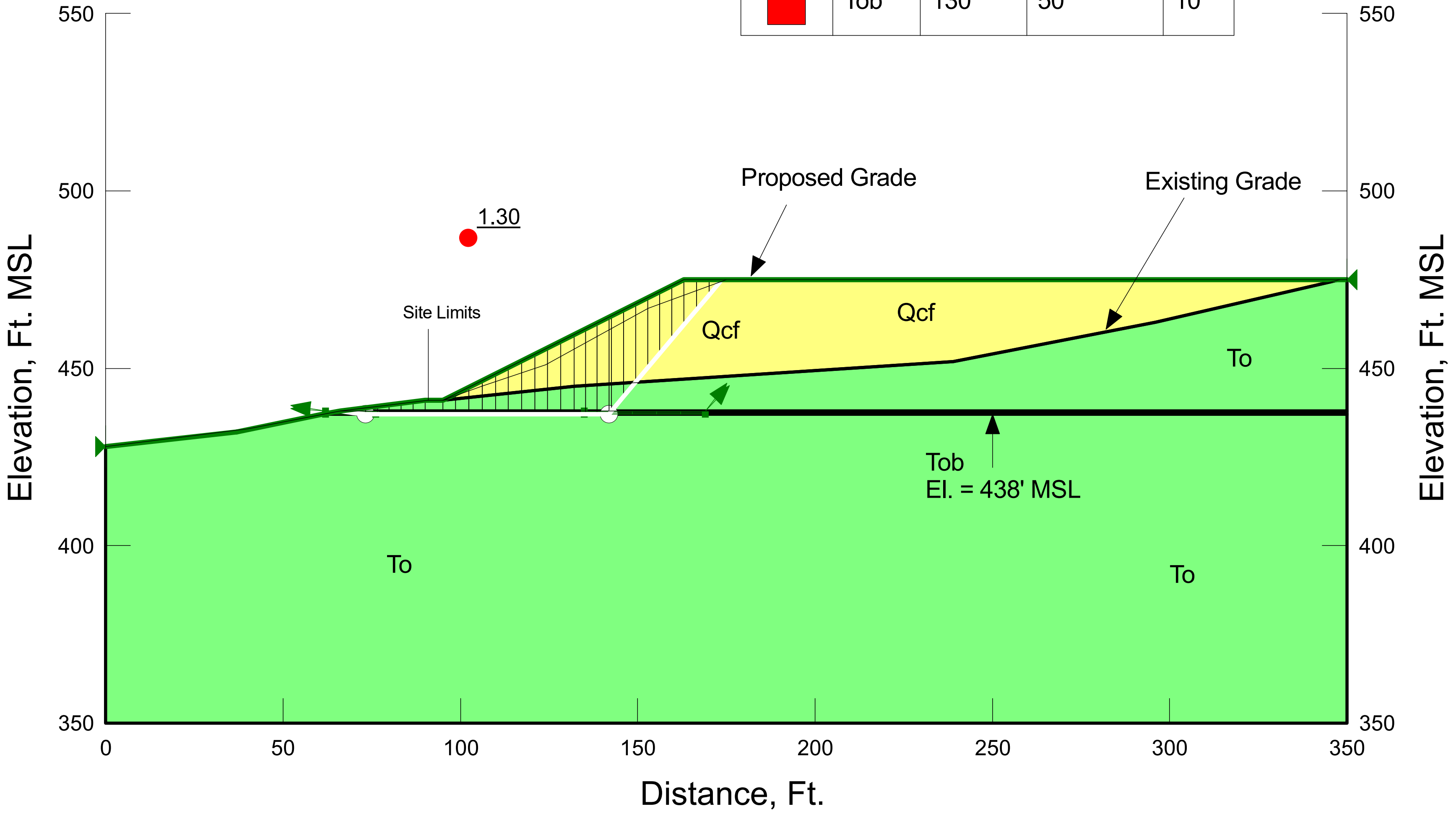
TABLE C-II
SUMMARY OF SLOPE STABILITY ANALYSES

Cross Section	File Name	Condition of Slope Stability Analyses	Calculated Factor of Safety
A-A’	A-A_Case 1	Minimum FOS, Tob Plane, Block–Mode Analysis, Static Condition	1.30
	A-A_Case 2	Minimum FOS, Tob Plane, Block–Mode Analysis, Static Condition – 20-foot buttress	1.57
	A-A_Case 2	Minimum Factor of Safety (FOS), Rotational–Mode Analysis, Static Condition	2.00
B-B’	B-B_Case 1	Minimum FOS, Tob Plane, Block–Mode Analysis, Static Condition	1.53
	B-B_Case 1	Minimum Factor of Safety (FOS), Rotational–Mode Analysis, Static Condition	1.81
C-C’	C-C_Case 1	Minimum FOS, Tob Plane, Block–Mode Analysis, Static Condition	1.42
	C-C_Case 2	Minimum FOS, Tob Plane, Block–Mode Analysis, Static Condition – 15-Foot Buttress	1.55
	C-C_Case 2	Minimum Factor of Safety (FOS), Rotational–Mode Analysis, Static Condition	2.04
D-D’	D-D_Case 1	Minimum FOS, Tob Plane, Block–Mode Analysis, Static Condition	1.90
	D-D_Case 1	Minimum Factor of Safety (FOS), Rotational–Mode Analysis, Static Condition	2.08

Otay Ranch V7 - R-8
Project No. 06862-52-67
Section A-A'
Name: A-A_Case 1.gsz
Date: 11/09/2021 Time: 07:52:06 AM

Material Properties:

Color	**Name**	**Unit Weight (pcf)**	**Cohesion' (psf)**	**Phi' (°)**
🟨	Qcf	130	300	28
🟩	To	130	325	33
🟥	Tob	130	50	10



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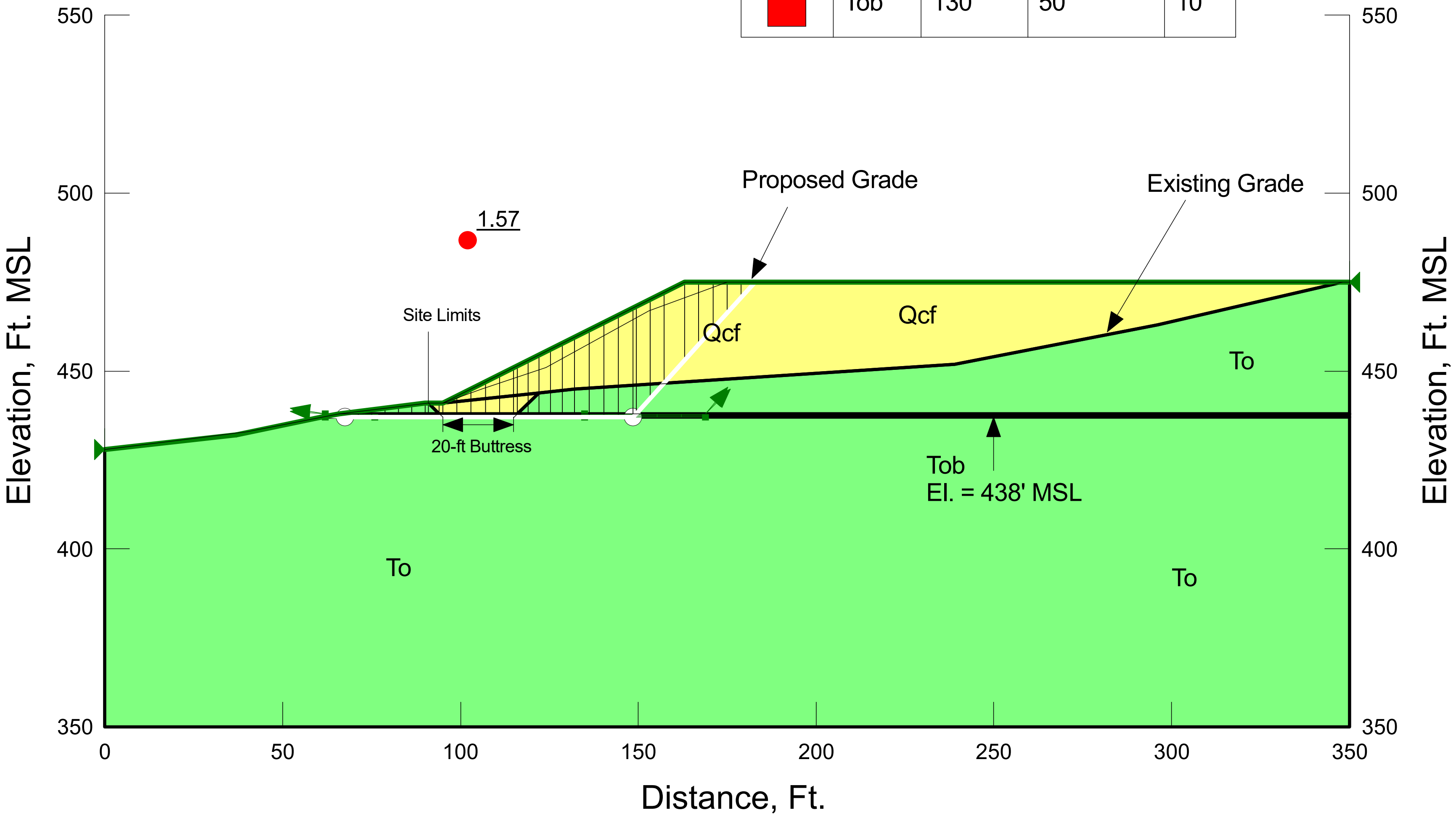
Otay Ranch V7 - R-8
Project No. 06862-52-67
Section A-A'
Name: A-A_Case 2.gsz
Date: 11/09/2021 Time: 07:58:31 AM

Material Properties:

Color	**Name**	**Unit Weight (pcf)**	**Cohesion' (psf)**	**Phi' (°)**
🟨	Qcf	130	300	28
🟩	To	130	325	33
🟥	Tob	130	50	10



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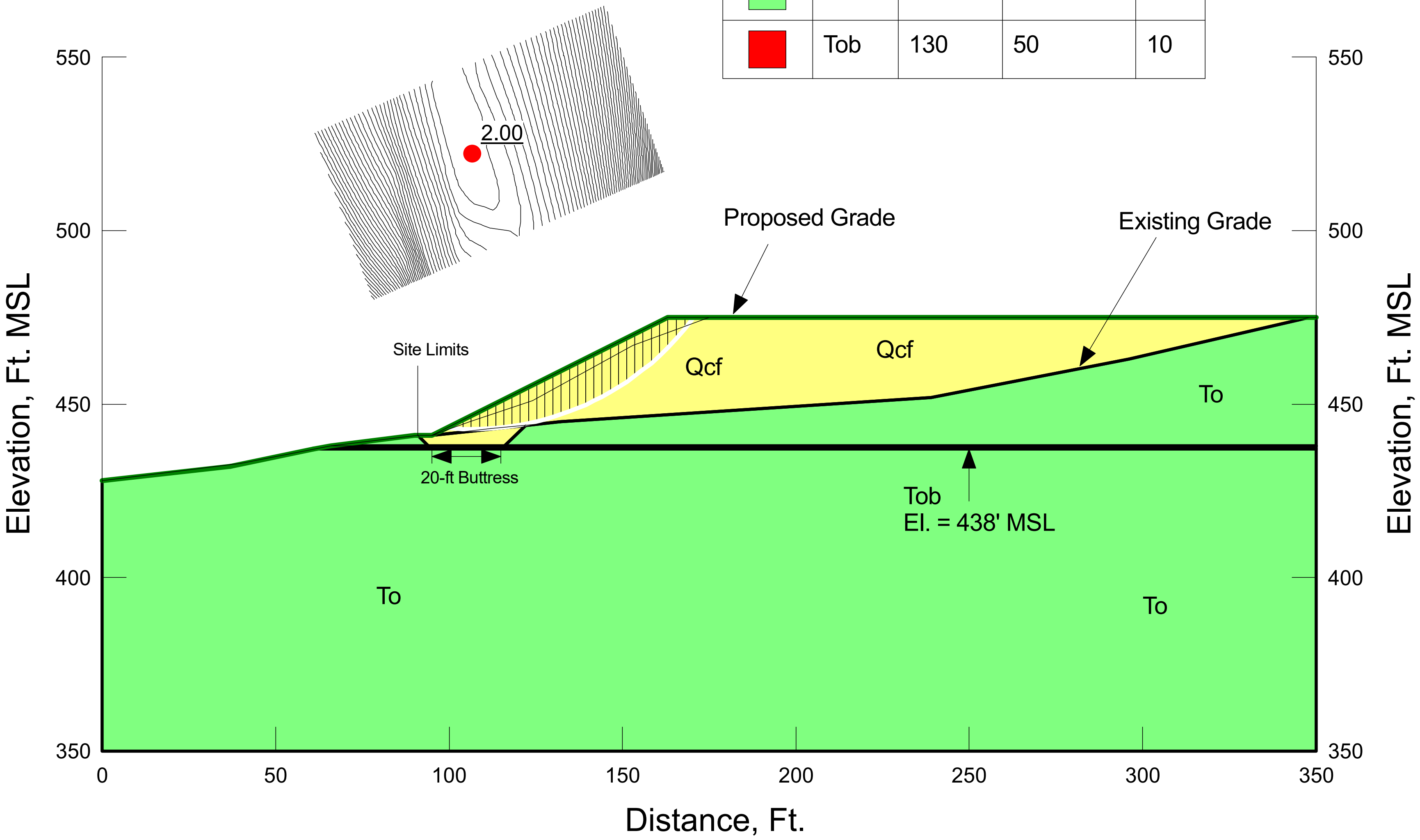
Otay Ranch V7 - R-8
Project No. 06862-52-67
Section A-A'
Name: A-A_Case 2.gsz
Date: 11/09/2021 Time: 08:30:28 AM

Material Properties:

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟨	Qcf	130	300	28
🟩	To	130	325	33
🟥	Tob	130	50	10



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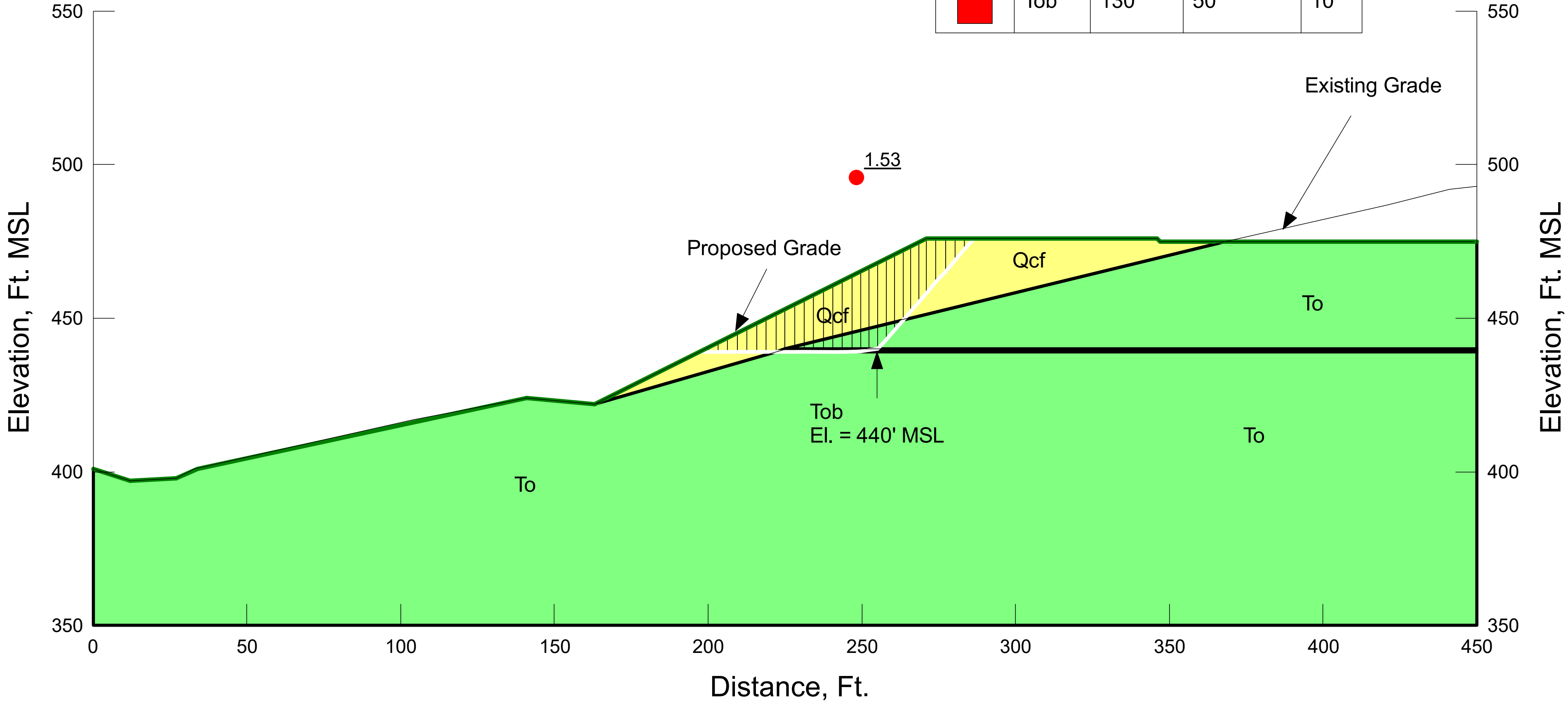
Otay Ranch V7 - R-8
Project No. 06862-52-67
Section B-B'
Name: B-B_Case 1.gsz
Date: 11/09/2021 Time: 08:37:20 AM

Material Properties:

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟨	Qcf	130	300	28
🟩	To	130	325	33
🟥	Tob	130	50	10



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Otay Ranch V7 - R-8
Project No. 06862-52-67
Section B-B'
Name: B-B_Case 1.gsz
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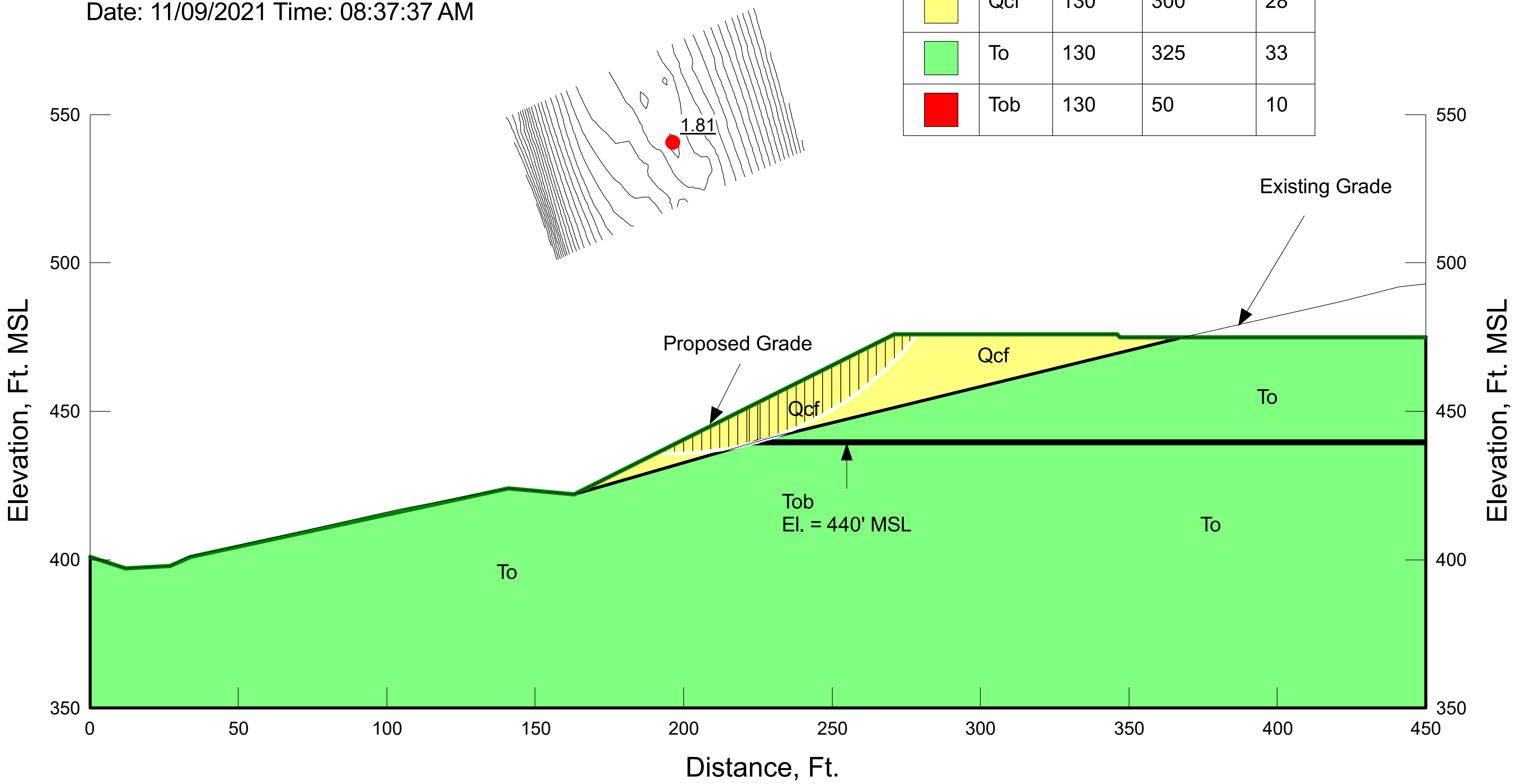
Material Properties:

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟨	Qcf	130	300	28
🟩	To	130	325	33
🟥	Tob	130	50	10



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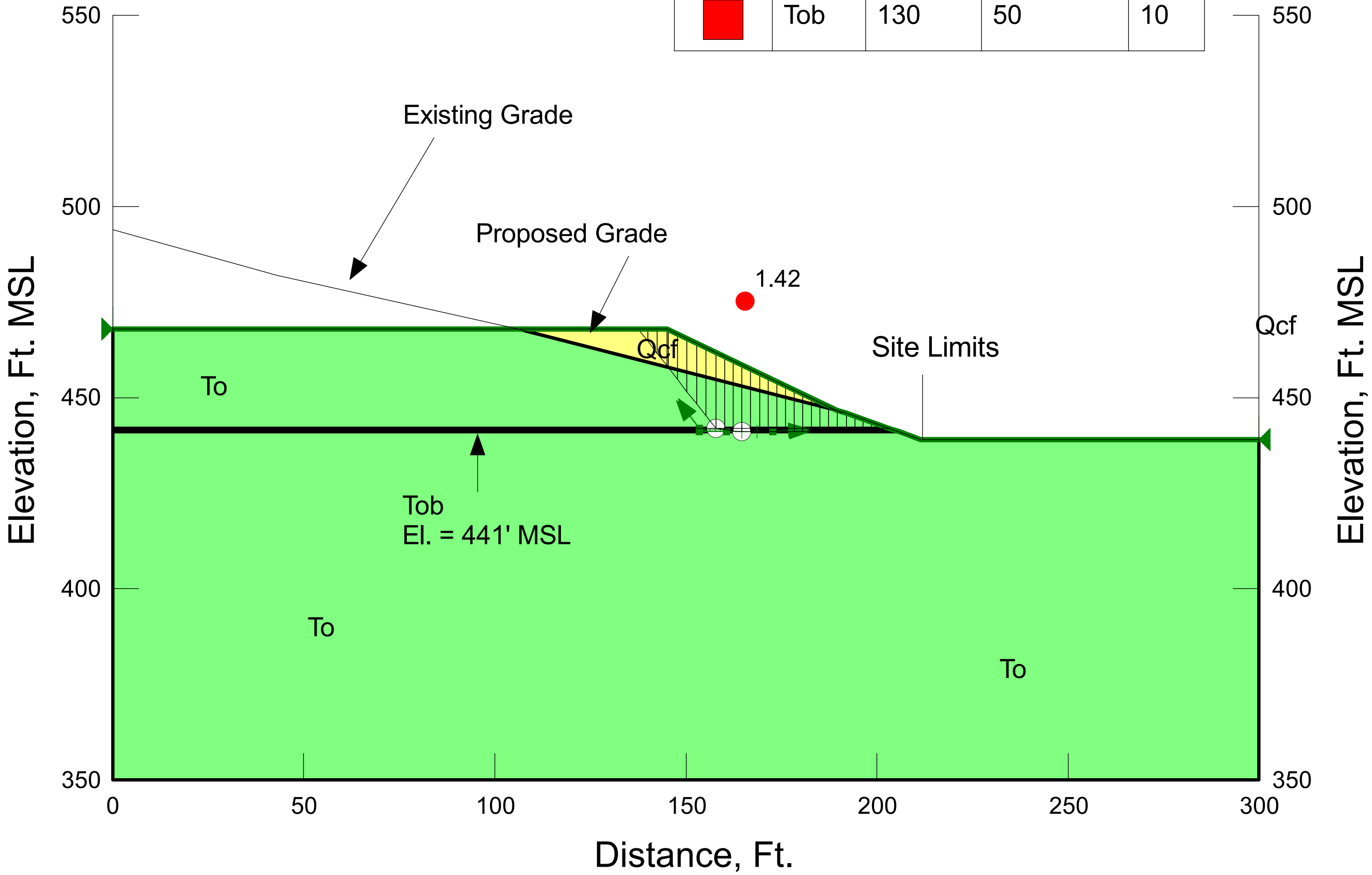
Otay Ranch V7 - R-8
Project No. 06862-52-67
Section C-C'
Name: C-C_Case 1.gsz
Date: 11/10/2021 Time: 09:17:02 AM

Material Properties:

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟨	Qcf	130	300	28
🟩	To	130	325	33
🟥	Tob	130	50	10



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Otay Ranch V7 - South
Project No. 06862-52-67
Section C-C'
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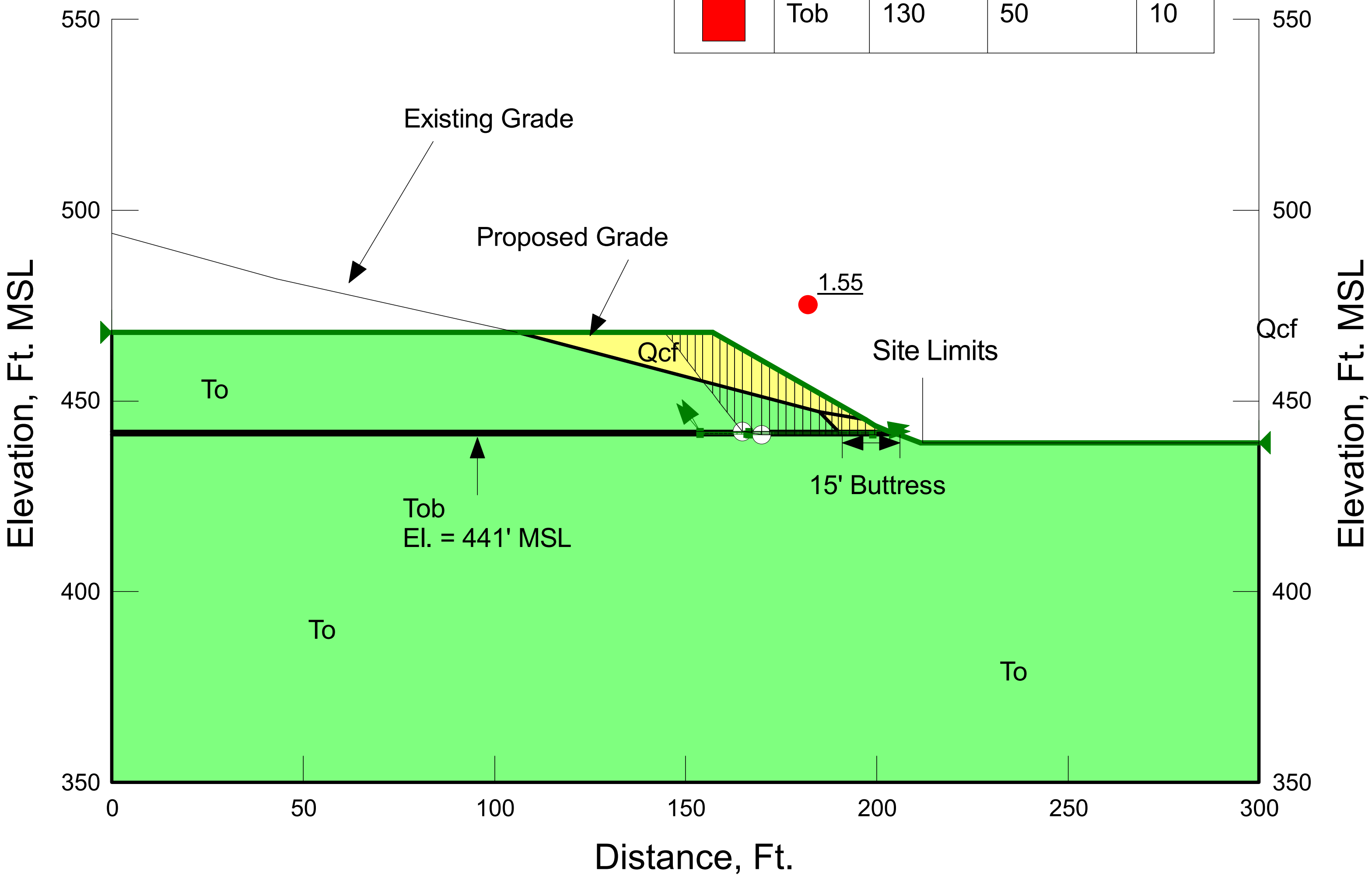
Date: 05/07/2024 Time: 02:47:57 PM

Material Properties:

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟨	Qcf	130	300	28
🟩	To	130	325	33
🟥	Tob	130	50	10



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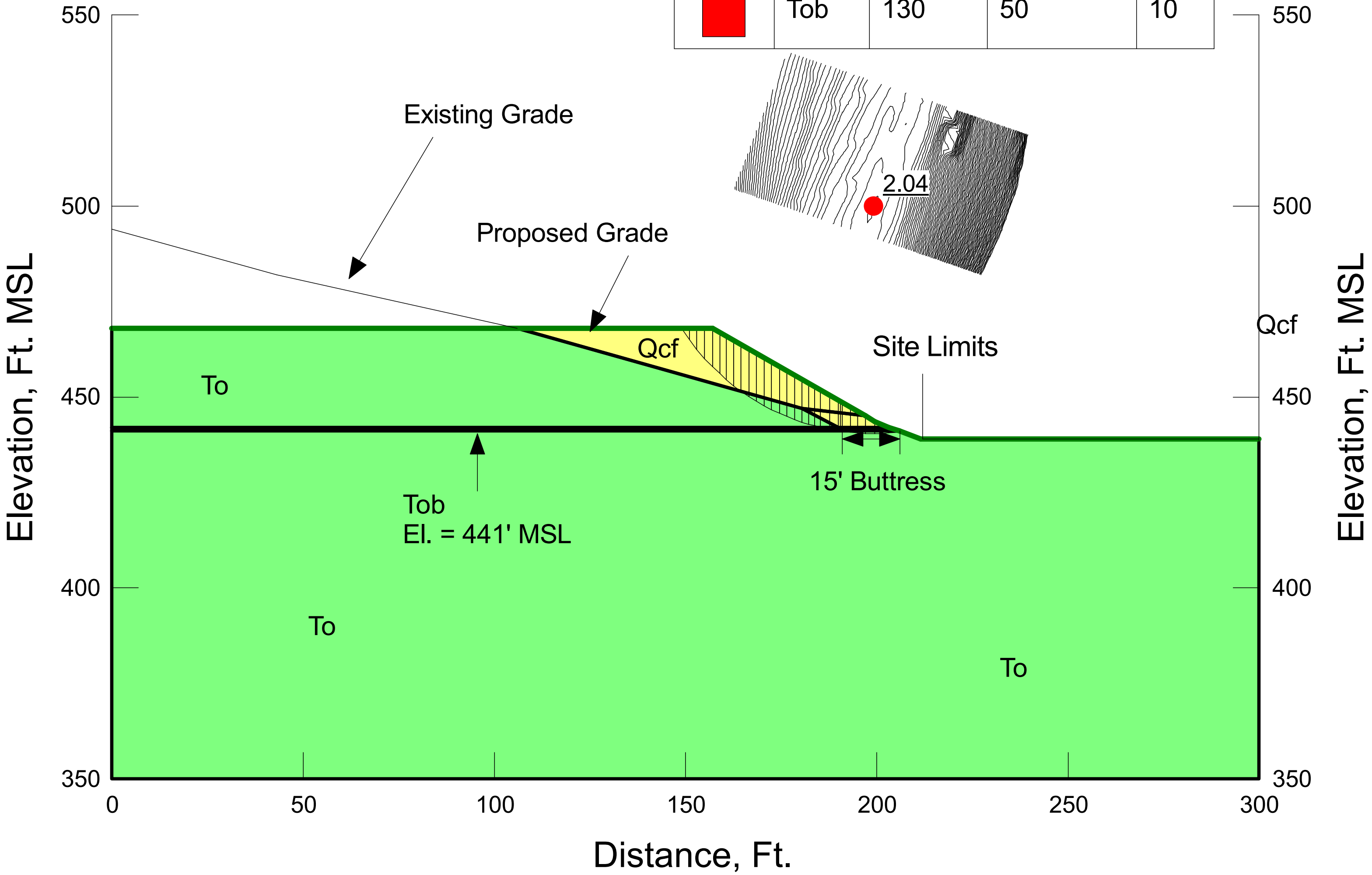
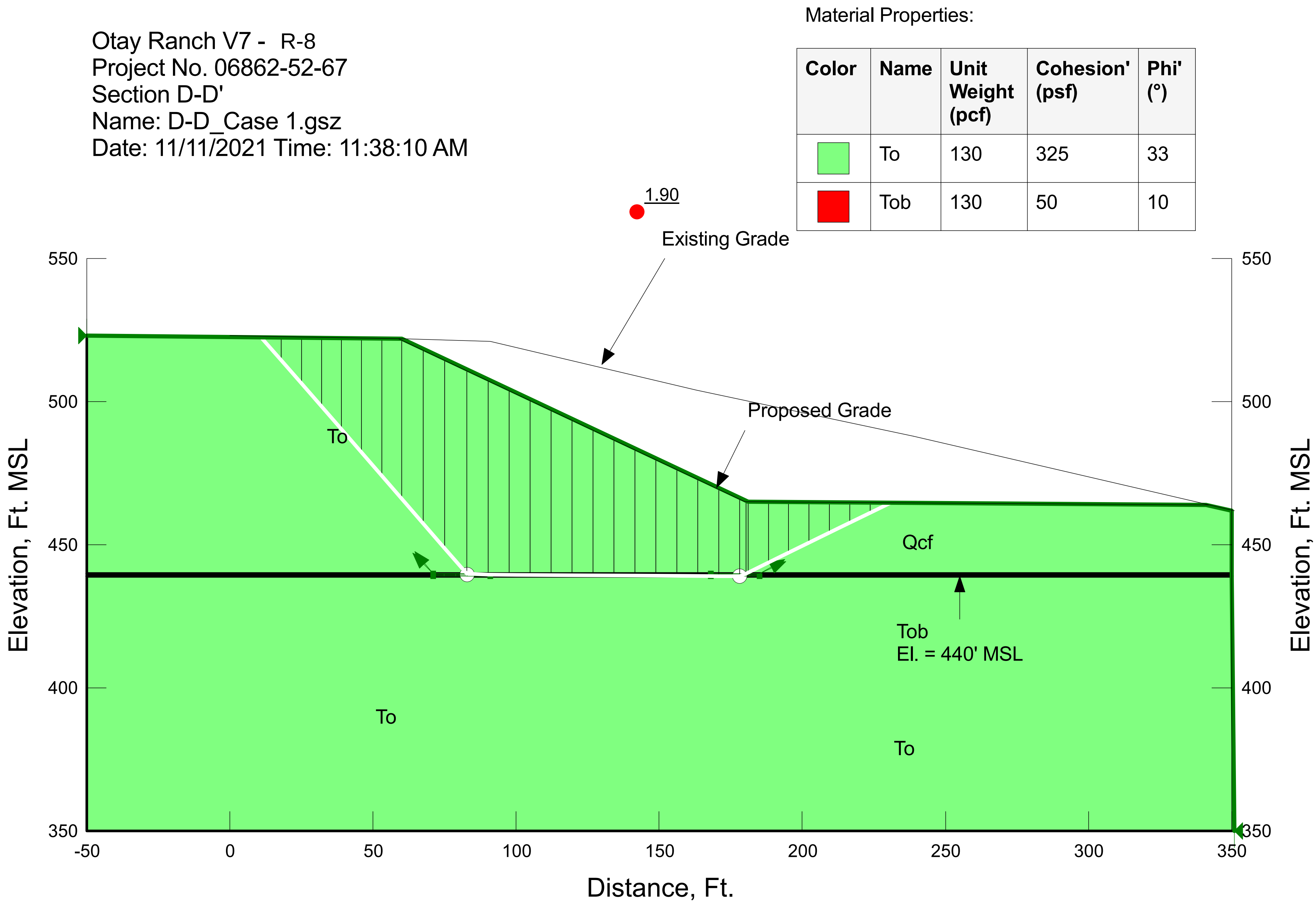
Otay Ranch V7 - South
Project No. 06862-52-67
Section C-C'
Name: C-C_Case 3-Proposed-Cir.gsz
Date: 05/07/2024 Time: 02:11:05 PM

Material Properties:

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟨	Qcf	130	300	28
🟩	To	130	325	33
🟥	Tob	130	50	10





Otay Ranch V7 - R-8
Project No. 06862-52-67
Section D-D'
Name: D-D_Case 1.gsz
Date: 11/11/2021 Time: 11:38:10 AM

Material Properties:

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟩	To	130	325	33
🟥	Tob	130	50	10

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Otay Ranch V7 - R-8
Project No. 06862-52-67
Section D-D'
Name: D-D_Case 1.gsz
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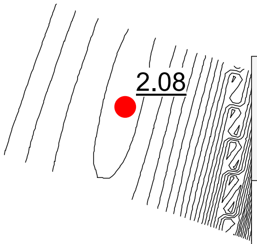
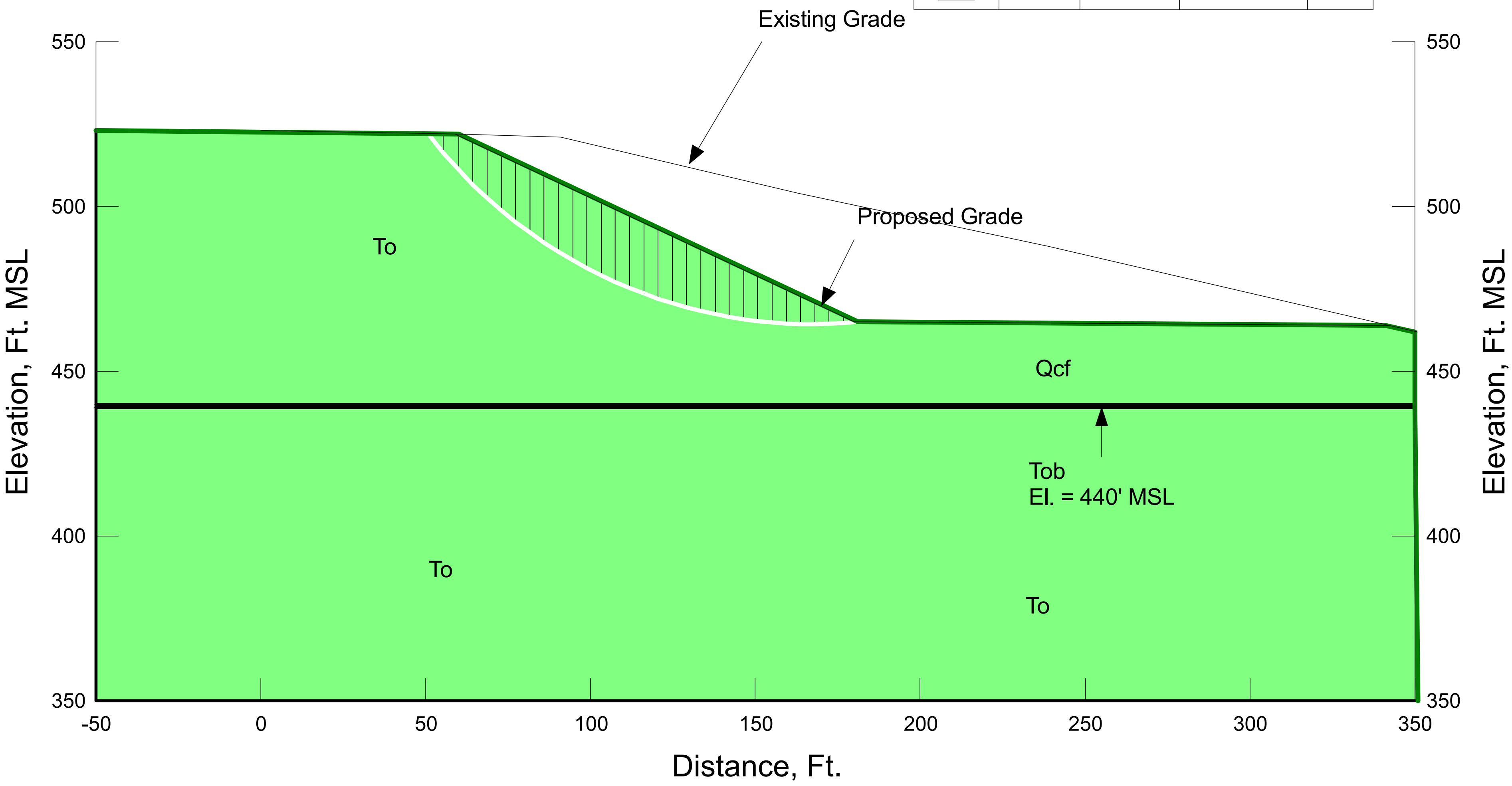


Material Properties:

Color	**Name**	**Unit Weight (pcf)**	**Cohesion' (psf)**	**Phi' (°)**
🟩	To	130	325	33
🟥	Tob	130	50	10



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APPENDIX D

APPENDIX D

RECOMMENDED GRADING SPECIFICATIONS

FOR

OTAY RANCH VILLAGE 7 SOUTH
CHULA VISTA, CALIFORNIA

PROJECT NO. 06862-52-67

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

1.1 These Recommended Grading Specifications shall be used in conjunction with the Geotechnical Report for the project prepared by Geocon. The recommendations contained in the text of the Geotechnical Report are a part of the earthwork and grading specifications and shall supersede the provisions contained hereinafter in the case of conflict.

1.2 Prior to the commencement of grading, a geotechnical consultant (Consultant) shall be employed for the purpose of observing earthwork procedures and testing the fills for substantial conformance with the recommendations of the Geotechnical Report and these specifications. The Consultant should provide adequate testing and observation services so that they may assess whether, in their opinion, the work was performed in substantial conformance with these specifications. It shall be the responsibility of the Contractor to assist the Consultant and keep them apprised of work schedules and changes so that personnel may be scheduled accordingly.

1.3 It shall be the sole responsibility of the Contractor to provide adequate equipment and methods to accomplish the work in accordance with applicable grading codes or agency ordinances, these specifications and the approved grading plans. If, in the opinion of the Consultant, unsatisfactory conditions such as questionable soil materials, poor moisture condition, inadequate compaction, and/or adverse weather result in a quality of work not in conformance with these specifications, the Consultant will be empowered to reject the work and recommend to the Owner that grading be stopped until the unacceptable conditions are corrected.

2. DEFINITIONS

2.1 **Owner** shall refer to the owner of the property or the entity on whose behalf the grading work is being performed and who has contracted with the Contractor to have grading performed.

2.2 **Contractor** shall refer to the Contractor performing the site grading work.

2.3 **Civil Engineer** or **Engineer of Work** shall refer to the California licensed Civil Engineer or consulting firm responsible for preparation of the grading plans, surveying and verifying as-graded topography.

2.4 **Consultant** shall refer to the soil engineering and engineering geology consulting firm retained to provide geotechnical services for the project.

2.5 **Soil Engineer** shall refer to a California licensed Civil Engineer retained by the Owner, who is experienced in the practice of geotechnical engineering. The Soil Engineer shall be responsible for having qualified representatives on-site to observe and test the Contractor's work for conformance with these specifications.

2.6 **Engineering Geologist** shall refer to a California licensed Engineering Geologist retained by the Owner to provide geologic observations and recommendations during the site grading.

2.7 **Geotechnical Report** shall refer to a soil report (including all addenda) which may include a geologic reconnaissance or geologic investigation that was prepared specifically for the development of the project for which these Recommended Grading Specifications are intended to apply.

3. MATERIALS

3.1 Materials for compacted fill shall consist of any soil excavated from the cut areas or imported to the site that, in the opinion of the Consultant, is suitable for use in construction of fills. In general, fill materials can be classified as *soil* fills, *soil-rock* fills or *rock* fills, as defined below.

3.1.1 **Soil fills** are defined as fills containing no rocks or hard lumps greater than 12 inches in maximum dimension and containing at least 40 percent by weight of material smaller than ¾ inch in size.

3.1.2 **Soil-rock fills** are defined as fills containing no rocks or hard lumps larger than 4 feet in maximum dimension and containing a sufficient matrix of soil fill to allow for proper compaction of soil fill around the rock fragments or hard lumps as specified in Paragraph 6.2. **Oversize rock** is defined as material greater than 12 inches.

3.1.3 **Rock fills** are defined as fills containing no rocks or hard lumps larger than 3 feet in maximum dimension and containing little or no fines. Fines are defined as material smaller than ¾ inch in maximum dimension. The quantity of fines shall be less than approximately 20 percent of the rock fill quantity.

3.2 Material of a perishable, spongy, or otherwise unsuitable nature as determined by the Consultant shall not be used in fills.

3.3 Materials used for fill, either imported or on-site, shall not contain hazardous materials as defined by the California Code of Regulations, Title 22, Division 4, Chapter 30, Articles 9 and 10; 40CFR; and any other applicable local, state or federal laws. The Consultant shall not be responsible for the identification or analysis of the potential presence of hazardous materials. However, if observations, odors or soil discoloration cause Consultant to suspect the presence of hazardous materials, the Consultant may request from the Owner the termination of grading operations within the affected area. Prior to resuming grading operations, the Owner shall provide a written report to the Consultant indicating that the suspected materials are not hazardous as defined by applicable laws and regulations.

3.4 The outer 15 feet of *soil-rock* fill slopes, measured horizontally, should be composed of properly compacted *soil* fill materials approved by the Consultant. *Rock* fill may extend to the slope face, provided that the slope is not steeper than 2:1 (horizontal:vertical) and a soil layer no thicker than 12 inches is track-walked onto the face for landscaping purposes. This procedure may be utilized provided it is acceptable to the governing agency, Owner and Consultant.

3.5 Samples of soil materials to be used for fill should be tested in the laboratory by the Consultant to determine the maximum density, optimum moisture content, and, where appropriate, shear strength, expansion, and gradation characteristics of the soil.

3.6 During grading, soil or groundwater conditions other than those identified in the Geotechnical Report may be encountered by the Contractor. The Consultant shall be notified immediately to evaluate the significance of the unanticipated condition.

4. CLEARING AND PREPARING AREAS TO BE FILLED

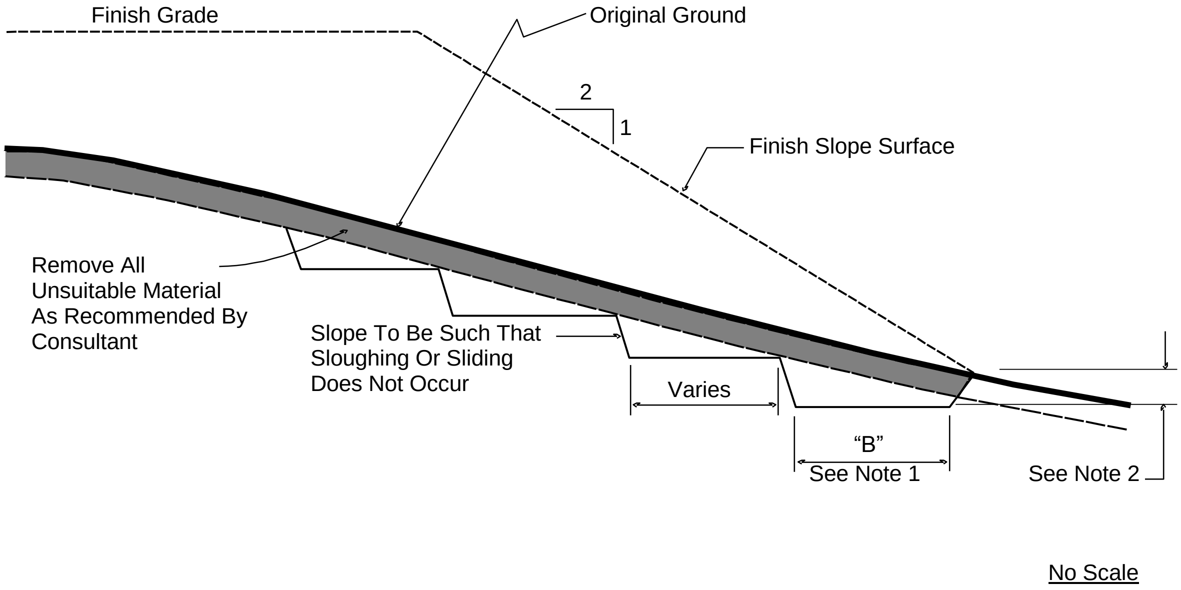
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material and shall be performed in areas to be graded. Roots and other projections exceeding 1½ inches in diameter shall be removed to a depth of 3 feet below the surface of the ground. Borrow areas shall be grubbed to the extent necessary to provide suitable fill materials.

4.2 Asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility or in an acceptable area of the project evaluated by Geocon and the property owner. Concrete fragments that are free of reinforcing steel may be placed in fills, provided they are placed in accordance with Section 6.2 or 6.3 of this document.

4.3 After clearing and grubbing of organic matter and other unsuitable material, loose or porous soils shall be removed to the depth recommended in the Geotechnical Report. The depth of removal and compaction should be observed and approved by a representative of the Consultant. The exposed surface shall then be plowed or scarified to a minimum depth of 6 inches and until the surface is free from uneven features that would tend to prevent uniform compaction by the equipment to be used.

4.4 Where the slope ratio of the original ground is steeper than 5:1 (horizontal:vertical), or where recommended by the Consultant, the original ground should be benched in accordance with the following illustration.

TYPICAL BENCHING DETAIL



DETAIL NOTES: (1) Key width "B" should be a minimum of 10 feet, or sufficiently wide to permit complete coverage with the compaction equipment used. The base of the key should be graded horizontal, or inclined slightly into the natural slope.

(2) The outside of the key should be below the topsoil or unsuitable surficial material and at least 2 feet into dense formational material. Where hard rock is exposed in the bottom of the key, the depth and configuration of the key may be modified as approved by the Consultant.

4.5 After areas to receive fill have been cleared and scarified, the surface should be moisture conditioned to achieve the proper moisture content, and compacted as recommended in Section 6 of these specifications.

5. COMPACTION EQUIPMENT

5.1 Compaction of *soil* or *soil-rock* fill shall be accomplished by sheepsfoot or segmented-steel wheeled rollers, vibratory rollers, multiple-wheel pneumatic-tired rollers, or other types of acceptable compaction equipment. Equipment shall be of such a design that it will be capable of compacting the *soil* or *soil-rock* fill to the specified relative compaction at the specified moisture content.

5.2 Compaction of *rock* fills shall be performed in accordance with Section 6.3.

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

6.1 *Soil* fill, as defined in Paragraph 3.1.1, shall be placed by the Contractor in accordance with the following recommendations:

6.1.1 *Soil* fill shall be placed by the Contractor in layers that, when compacted, should generally not exceed 8 inches. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to obtain uniformity of material and moisture in each layer. The entire fill shall be constructed as a unit in nearly level lifts. Rock materials greater than 12 inches in maximum dimension shall be placed in accordance with Section 6.2 or 6.3 of these specifications.

6.1.2 In general, the *soil* fill shall be compacted at a moisture content at or above the optimum moisture content as determined by ASTM D 1557.

6.1.3 When the moisture content of *soil* fill is below that specified by the Consultant, water shall be added by the Contractor until the moisture content is in the range specified.

6.1.4 When the moisture content of the *soil* fill is above the range specified by the Consultant or too wet to achieve proper compaction, the *soil* fill shall be aerated by the Contractor by blading/mixing, or other satisfactory methods until the moisture content is within the range specified.

6.1.5 After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted by the Contractor to a relative compaction of at least 90 percent. Relative compaction is defined as the ratio (expressed in percent) of the in-place dry density of the compacted fill to the maximum laboratory dry density as determined in accordance with ASTM D 1557. Compaction shall be continuous over the entire area, and compaction equipment shall make sufficient passes so that the specified minimum relative compaction has been achieved throughout the entire fill.

6.1.6 Where practical, soils having an Expansion Index greater than 50 should be placed at least 3 feet below finish pad grade and should be compacted at a moisture content generally 2 to 4 percent greater than the optimum moisture content for the material.

6.1.7 Properly compacted *soil* fill shall extend to the design surface of fill slopes. To achieve proper compaction, it is recommended that fill slopes be over-built by at least 3 feet and then cut to the design grade. This procedure is considered preferable to track-walking of slopes, as described in the following paragraph.

6.1.8 As an alternative to over-building of slopes, slope faces may be back-rolled with a heavy-duty loaded sheepsfoot or vibratory roller at maximum 4-foot fill height intervals. Upon completion, slopes should then be track-walked with a D-8 dozer or similar equipment, such that a dozer track covers all slope surfaces at least twice.

6.2 *Soil-rock* fill, as defined in Paragraph 3.1.2, shall be placed by the Contractor in accordance with the following recommendations:

6.2.1 Rocks larger than 12 inches but less than 4 feet in maximum dimension may be incorporated into the compacted *soil* fill, but shall be limited to the area measured 15 feet minimum horizontally from the slope face and 5 feet below finish grade or 3 feet below the deepest utility, whichever is deeper.

6.2.2 Rocks or rock fragments up to 4 feet in maximum dimension may either be individually placed or placed in windrows. Under certain conditions, rocks or rock fragments up to 10 feet in maximum dimension may be placed using similar methods. The acceptability of placing rock materials greater than 4 feet in maximum dimension shall be evaluated during grading as specific cases arise and shall be approved by the Consultant prior to placement.

6.2.3 For individual placement, sufficient space shall be provided between rocks to allow for passage of compaction equipment.

6.2.4 For windrow placement, the rocks should be placed in trenches excavated in properly compacted *soil* fill. Trenches should be approximately 5 feet wide and 4 feet deep in maximum dimension. The voids around and beneath rocks should be filled with approved granular soil having a Sand Equivalent of 30 or greater and should be compacted by flooding. Windrows may also be placed utilizing an "open-face" method in lieu of the trench procedure, however, this method should first be approved by the Consultant.

6.2.5 Windrows should generally be parallel to each other and may be placed either parallel to or perpendicular to the face of the slope depending on the site geometry. The minimum horizontal spacing for windrows shall be 12 feet center-to-center with a 5-foot stagger or offset from lower courses to next overlying course. The minimum vertical spacing between windrow courses shall be 2 feet from the top of a lower windrow to the bottom of the next higher windrow.

6.2.6 Rock placement, fill placement and flooding of approved granular soil in the windrows should be continuously observed by the Consultant.

6.3 *Rock* fills, as defined in Section 3.1.3, shall be placed by the Contractor in accordance with the following recommendations:

6.3.1 The base of the *rock* fill shall be placed on a sloping surface (minimum slope of 2 percent). The surface shall slope toward suitable subdrainage outlet facilities. The *rock* fills shall be provided with subdrains during construction so that a hydrostatic pressure buildup does not develop. The subdrains shall be permanently connected to controlled drainage facilities to control post-construction infiltration of water.

6.3.2 *Rock* fills shall be placed in lifts not exceeding 3 feet. Placement shall be by rock trucks traversing previously placed lifts and dumping at the edge of the currently placed lift. Spreading of the *rock* fill shall be by dozer to facilitate *seating* of the rock. The *rock* fill shall be watered heavily during placement. Watering shall consist of water trucks traversing in front of the current rock lift face and spraying water continuously during rock placement. Compaction equipment with compactive energy comparable to or greater than that of a 20-ton steel vibratory roller or other compaction equipment providing suitable energy to achieve the required compaction or deflection as recommended in Paragraph 6.3.3 shall be utilized. The number of passes to be made should be determined as described in Paragraph 6.3.3. Once a *rock* fill lift has been covered with *soil* fill, no additional *rock* fill lifts will be permitted over the *soil* fill.

6.3.3 Plate bearing tests, in accordance with ASTM D 1196, may be performed in both the compacted *soil* fill and in the *rock* fill to aid in determining the required minimum number of passes of the compaction equipment. If performed, a minimum of three plate bearing tests should be performed in the properly compacted *soil* fill (minimum relative compaction of 90 percent). Plate bearing tests shall then be performed on areas of *rock* fill having two passes, four passes and six passes of the compaction equipment, respectively. The number of passes required for the *rock* fill shall be determined by comparing the results of the plate bearing tests for the *soil* fill and the *rock* fill and by evaluating the deflection variation with number of passes. The required number of passes of the compaction equipment will be performed as necessary until the plate bearing deflections are equal to or less than that determined for the properly compacted *soil* fill. In no case will the required number of passes be less than two.

6.3.4 A representative of the Consultant should be present during *rock* fill operations to observe that the minimum number of "passes" have been obtained, that water is being properly applied and that specified procedures are being followed. The actual number of plate bearing tests will be determined by the Consultant during grading.

6.3.5 Test pits shall be excavated by the Contractor so that the Consultant can state that, in their opinion, sufficient water is present and that voids between large rocks are properly filled with smaller rock material. In-place density testing will not be required in the *rock* fills.

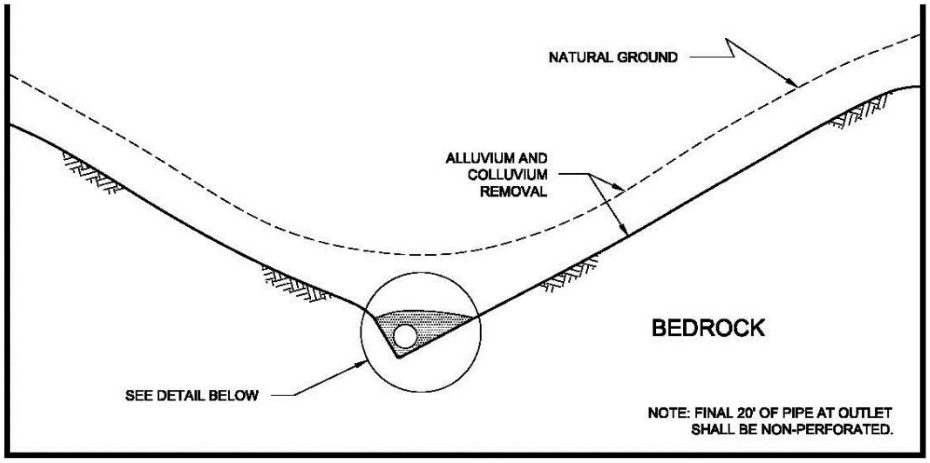
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant prior to commencing grading. The gradation of the graded filter material will be determined at the time the *rock* fill is being excavated. Materials typical of the *rock* fill should be submitted to the Consultant in a timely manner, to allow design of the graded filter prior to the commencement of *rock* fill placement.

6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

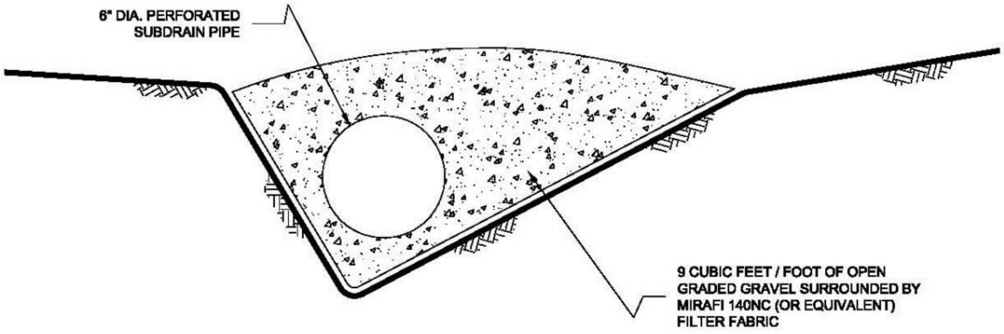
7. SUBDRAINS

7.1 The geologic units on the site may have permeability characteristics and/or fracture systems that could be susceptible under certain conditions to seepage. The use of canyon subdrains may be necessary to mitigate the potential for adverse impacts associated with seepage conditions. Canyon subdrains with lengths in excess of 500 feet or extensions of existing offsite subdrains should use 8-inch-diameter pipes. Canyon subdrains less than 500 feet in length should use 6-inch-diameter pipes.

TYPICAL CANYON DRAIN DETAIL



NOTE: FINAL 20' OF PIPE AT OUTLET SHALL BE NON-PERFORATED.



NOTES:

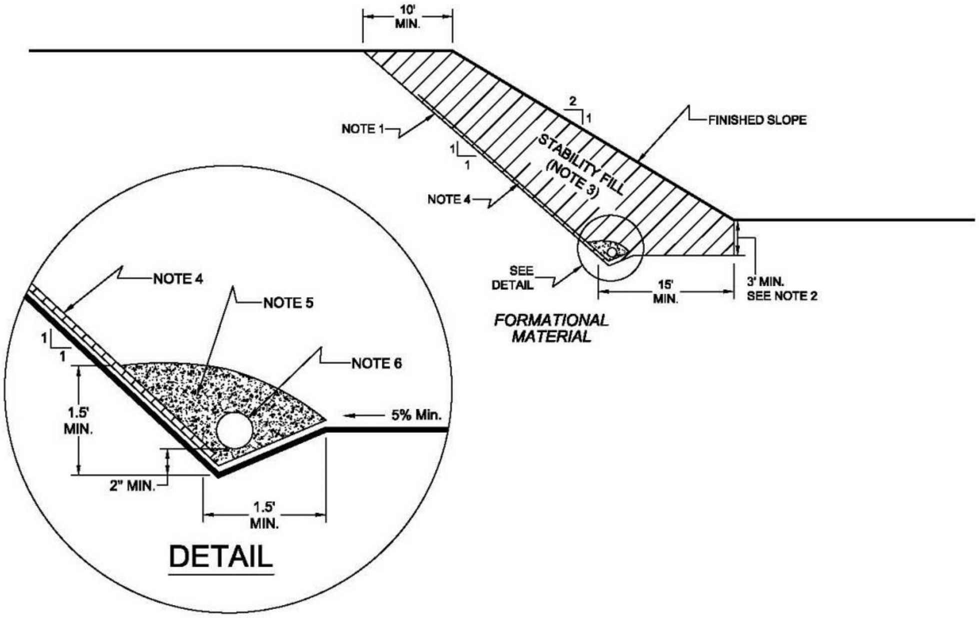
1......6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

2......6-INCH DIAMETER, SCHEDULE 40 PVC PERFORATED PIPE FOR FILLS LESS THAN 100-FEET IN DEPTH OR A PIPE LENGTH SHORTER THAN 500 FEET.

NO SCALE

7.2 Slope drains within stability fill keyways should use 4-inch-diameter (or lager) pipes.

TYPICAL STABILITY FILL DETAIL



NOTES:

1.....EXCAVATE BACKCUT AT 1:1 INCLINATION (UNLESS OTHERWISE NOTED).

2.....BASE OF STABILITY FILL TO BE 3 FEET INTO FORMATIONAL MATERIAL, SLOPING A MINIMUM 5% INTO SLOPE.

3.....STABILITY FILL TO BE COMPOSED OF PROPERLY COMPACTED GRANULAR SOIL.

4.....CHIMNEY DRAINS TO BE APPROVED PREFABRICATED CHIMNEY DRAIN PANELS (MIRADRAIN G200N OR EQUIVALENT) SPACED APPROXIMATELY 20 FEET CENTER TO CENTER AND 4 FEET WIDE. CLOSER SPACING MAY BE REQUIRED IF SEEPAGE IS ENCOUNTERED.

5.....FILTER MATERIAL TO BE 3/4-INCH, OPEN-GRADED CRUSHED ROCK ENCLOSED IN APPROVED FILTER FABRIC (MIRAFI 140NC).

6.....COLLECTOR PIPE TO BE 4-INCH MINIMUM DIAMETER, PERFORATED, THICK-WALLED PVC SCHEDULE 40 OR EQUIVALENT, AND SLOPED TO DRAIN AT 1 PERCENT MINIMUM TO APPROVED OUTLET.

NO SCALE

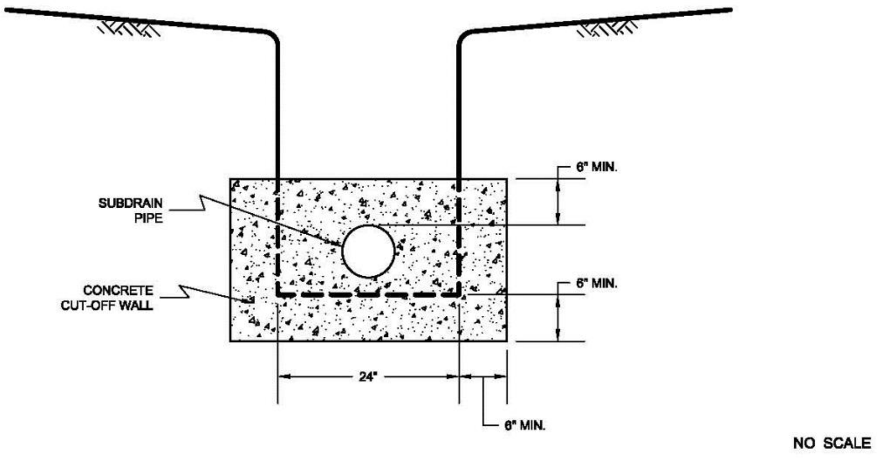
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing 40-scale grading plans.

7.4 *Rock* fill or *soil-rock* fill areas may require subdrains along their down-slope perimeters to mitigate the potential for buildup of water from construction or landscape irrigation. The subdrains should be at least 6-inch-diameter pipes encapsulated in gravel and filter fabric. *Rock* fill drains should be constructed using the same requirements as canyon subdrains.

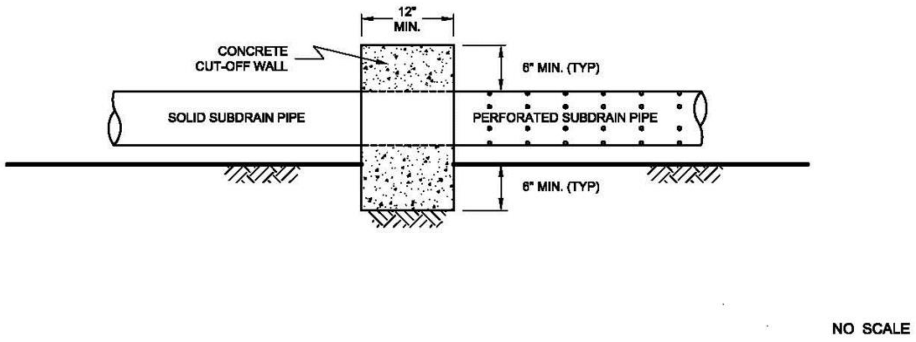
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW

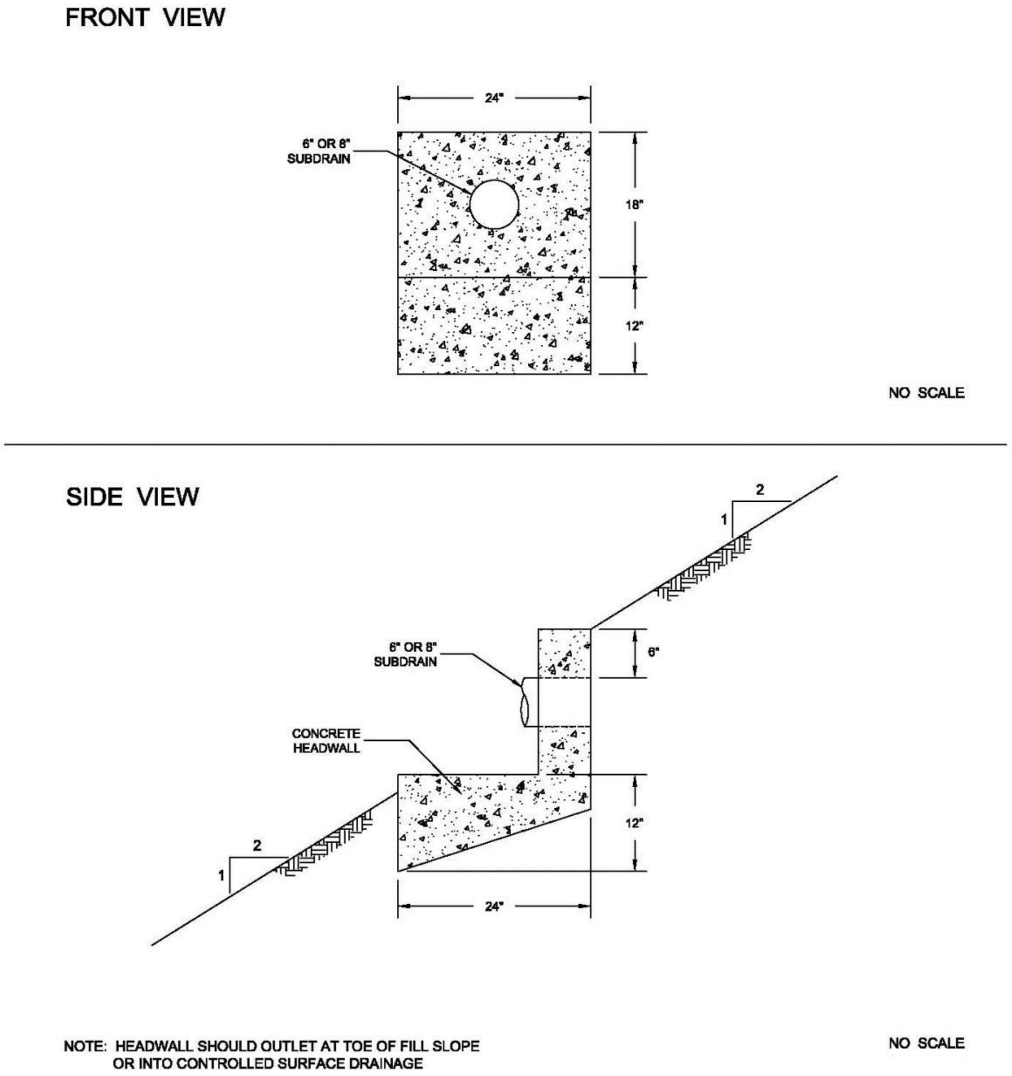


SIDE VIEW



7.6 Subdrains that discharge into a natural drainage course or open space area should be provided with a permanent headwall structure.

TYPICAL HEADWALL DETAIL



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

7.7 The final grading plans should show the location of the proposed subdrains. After completion of remedial excavations and subdrain installation, the project civil engineer should survey the drain locations and prepare an "as-built" map showing the drain locations. The final outlet and connection locations should be determined during grading operations. Subdrains that will be extended on adjacent projects after grading can be placed on formational material and a vertical riser should be placed at the end of the subdrain. The grading contractor should consider videoing the subdrains shortly after burial to check proper installation and functionality. The contractor is responsible for the performance of the drains.

8. OBSERVATION AND TESTING

8.1 The Consultant shall be the Owner's representative to observe and perform tests during clearing, grubbing, filling, and compaction operations. In general, no more than 2 feet in vertical elevation of *soil* or *soil-rock* fill should be placed without at least one field density test being performed within that interval. In addition, a minimum of one field density test should be performed for every 2,000 cubic yards of *soil* or *soil-rock* fill placed and compacted.

8.2 The Consultant should perform a sufficient distribution of field density tests of the compacted *soil* or *soil-rock* fill to provide a basis for expressing an opinion whether the fill material is compacted as specified. Density tests shall be performed in the compacted materials below any disturbed surface. When these tests indicate that the density of any layer of fill or portion thereof is below that specified, the particular layer or areas represented by the test shall be reworked until the specified density has been achieved.

8.3 During placement of *rock* fill, the Consultant should observe that the minimum number of passes have been obtained per the criteria discussed in Section 6.3.3. The Consultant should request the excavation of observation pits and may perform plate bearing tests on the placed *rock* fills. The observation pits will be excavated to provide a basis for expressing an opinion as to whether the *rock* fill is properly seated and sufficient moisture has been applied to the material. When observations indicate that a layer of *rock* fill or any portion thereof is below that specified, the affected layer or area shall be reworked until the *rock* fill has been adequately seated and sufficient moisture applied.

8.4 A settlement monitoring program designed by the Consultant may be conducted in areas of *rock* fill placement. The specific design of the monitoring program shall be as recommended in the Conclusions and Recommendations section of the project Geotechnical Report or in the final report of testing and observation services performed during grading.

8.5 We should observe the placement of subdrains, to check that the drainage devices have been placed and constructed in substantial conformance with project specifications.

8.6 Testing procedures shall conform to the following Standards as appropriate:

8.6.1 Soil and Soil-Rock Fills:

8.6.1.1 Field Density Test, ASTM D 1556, *Density of Soil In-Place By the Sand-Cone Method.*

8.6.1.2 Field Density Test, Nuclear Method, ASTM D 6938, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth).*

8.6.1.3 Laboratory Compaction Test, ASTM D 1557, *Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-Pound Hammer and 18-Inch Drop*.

8.6.1.4. Expansion Index Test, ASTM D 4829, *Expansion Index Test*.

9. PROTECTION OF WORK

9.1 During construction, the Contractor shall properly grade all excavated surfaces to provide positive drainage and prevent ponding of water. Drainage of surface water shall be controlled to avoid damage to adjoining properties or to finished work on the site. The Contractor shall take remedial measures to prevent erosion of freshly graded areas until such time as permanent drainage and erosion control features have been installed. Areas subjected to erosion or sedimentation shall be properly prepared in accordance with the Specifications prior to placing additional fill or structures.

9.2 After completion of grading as observed and tested by the Consultant, no further excavation or filling shall be conducted except in conjunction with the services of the Consultant.

10. CERTIFICATIONS AND FINAL REPORTS

10.1 Upon completion of the work, Contractor shall furnish Owner a certification by the Civil Engineer stating that the lots and/or building pads are graded to within 0.1 foot vertically of elevations shown on the grading plan and that all tops and toes of slopes are within 0.5 foot horizontally of the positions shown on the grading plans. After installation of a section of subdrain, the project Civil Engineer should survey its location and prepare an *as-built* plan of the subdrain location. The project Civil Engineer should verify the proper outlet for the subdrains and the Contractor should ensure that the drain system is free of obstructions.

10.2 The Owner is responsible for furnishing a final as-graded soil and geologic report satisfactory to the appropriate governing or accepting agencies. The as-graded report should be prepared and signed by a California licensed Civil Engineer experienced in geotechnical engineering and by a California Certified Engineering Geologist, indicating that the geotechnical aspects of the grading were performed in substantial conformance with the Specifications or approved changes to the Specifications.

LIST OF REFERENCES

1. *2022 California Building Code, California Code of Regulations, Title 24, Part 2, based on the 2021 International Building Code*, prepared by California Building Standards Commission, dated July 2022.

2. American Concrete Institute, *ACI 318-11, Building Code Requirements for Structural Concrete and Commentary*, dated August, 2011.

3. *ACI 330-21, Commercial Concrete Parking Lots and Site Paving Design and Construction*, prepared by the American Concrete Institute, dated May 2021.

4. American Society of Civil Engineers (ASCE), *ASCE 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, 2017.

5. California Department of Conservation, Division of Mines and Geology, *Probabilistic Seismic Hazard Assessment for the State of California*, Open File Report 96-08, 1996.

6. County of San Diego, *San Diego County Multi Jurisdiction Hazard Mitigation Plan, San Diego, California – Final Draft*, dated 2017.

7. Historical Aerial Photos. *http://www.historicaerials.com*

8. Jennings, C. W., 1994, California Division of Mines and Geology, *Fault Activity Map of California and Adjacent Areas*, California Geologic Data Map Series Map No. 6.

9. Todd, V. R., 2004, *Preliminary Geologic Map of the El Cajon 30'x60' Quadrangle, Southern California*, Version 1.0, Open-File Report 2004-1361 Scale 1:100,000.

10. Special Publication 117A, *Guidelines For Evaluating and Mitigating Seismic Hazards in California 2008*, California Geological Survey, Revised and Re-adopted September 11, 2008.

11. Unpublished reports, aerial photographs, and maps on file with Geocon Incorporated.

12. USGS computer program, Seismic Hazard Curves and Uniform Hazard Response Spectra, *http://geohazards.usgs.gov/designmaps/us/application.php.*