Appendix G3. Geotechnical Investigation for Off-Site Improvements to Cuyamaca Street

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GEOTECHNICAL INVESTIGATION

FANITA RANCH
OFF-SITE IMPROVEMENT TO
CUYAMACA STREET
SANTEE, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

HOMEFED CORPORATION
CARLSBAD, CALIFORNIA

APRIL 17, 2020
PROJECT NO. 05254-32-18A

Project No. 05254-32-18A
April 17, 2020

HomeFed Corporation
1903 Wright Place, Suite 220
Carlsbad, California 92008

Attention: Mr. Tom Blessent

Subject: GEOTECHNICAL INVESTIGATION
FANITA RANCH
OFF-SITE IMPROVEMENT TO CUYAMACA STREET
SANTEE, CALIFORNIA

Dear Mr. Blessent:

In accordance with your request, we have prepared this geotechnical investigation report for the future alignment of Cuyamaca Street from Silver Country Estates to the southern boundary of Orchard Village in the proposed Fanita Ranch project located in Santee, California.

The accompanying report presents the findings of our study and our recommendations relative to the geotechnical aspects of grading the roadway as presently proposed. This information was also presented in 1997 during submittal of a previous Tentative Map and Specific Plan.

Should you have any 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 a geotechnical investigation for the proposed off-site improvements to Cuyamaca Street as part of the Fanita Ranch project located in Santee, California (see Vicinity Map, Figure 1). This information was previously presented in our report entitled *Geotechnical Investigation, Fanita Ranch, Off-site Improvement to Cuyamaca Street, Santee, California*, dated June 11, 1997, during submittal of a previous Tentative Map and Environmental Impact Report.

The purpose of this study was to investigate the soil and geologic conditions along the alignment as well as the geotechnical constraints that may impact the construction of Cuyamaca Street from Silver Country Estates northward to the southerly boundary of the Orchard Village of Fanita Ranch. Aerial photographs, readily available published and unpublished geologic literature and previous geotechnical reports pertaining to the site were reviewed (see *List of References*). Please note, for continuity, the List of References is considered a “master list” applicable to all of our Fanita Ranch investigation reports The scope also included performing a field investigation, laboratory testing to identify physical soil properties, engineering analyses, and preparation of this report.

The geotechnical investigation for the roadway alignment was performed in conjunction with our overall study of Fanita Ranch and off-site Fanita Parkway. Field operations for the overall study were conducted intermittently between February 6, 1995, and November 20, 1996, and consisted of a site reconnaissance by an engineering geologist, excavation of 85 large-diameter borings, 207 backhoe trenches, and performance of 19 seismic refraction traverses. The emphasis of this study, which consisted of performing 6 seismic refraction traverses and excavating 4 exploratory borings, was placed in the south end of the alignment where the potential for slope instability was suspected. The seismic refraction survey was conducted to evaluate rippability in areas where excavations in granitic rock and Stadium Conglomerate are proposed. Details of the field investigation, as well as descriptive boring logs and the results of the seismic refraction survey, are presented in Appendix A.

Laboratory tests were performed on selected representative soil samples obtained during the field investigation for the overall project to evaluate the pertinent physical properties of the soils encountered. The laboratory information was used in engineering analyses and to assist in providing recommendations for site grading and development. Details of the laboratory tests and a summary of the test results are presented in Appendix B.

The base map used to depict the soil and geologic conditions consisted of AutoCAD files of the proposed improvements entitled *Fanita Ranch – Vesting Tentative Map/Preliminary Grading Plan*, Sheets 17, 18 and 21, prepared by Hunsaker & Associates San Diego, Inc., Revision 5 dated

March 27, 2020 (see Geologic Map, Figure 2, map pocket). The geologic map depicts the proposed roadway alignment and grading, existing topography, mapped geologic contacts, and the approximate locations of the exploratory excavations and seismic traverses. The conclusions and recommendations presented herein are based on an analysis of the data obtained from the exploratory field investigation, laboratory tests, and our experience with similar soil and geologic conditions.

2. SITE AND PROJECT DESCRIPTION

The proposed off-site Cuyamaca Street alignment is approximately 4,600 feet long and will traverse undeveloped property in the City of Santee. As indicated on the Geologic Map, the roadway is relatively straight and will ascend from a low elevation of 570 feet Mean Sea Level (MSL) at its southern end to a high of 790 feet MSL at the boundary of Fanita Ranch.

Topographically, Cuyamaca Street will generally parallel the natural contours of the east facing hillsides which form the eastern boundary of open space south of Orchard Village within the Fanita Ranch Specific Plan. Natural slope gradients along the alignment vary from approximately 6:1 (horizontal to vertical) to 2.5:1. The roadway will cross at least three easterly draining ravines. Cut slopes are proposed for a maximum height of 85 feet at an inclination of 1.5:1. All fill slopes are planned at 2:1 with a maximum height of approximately 50 feet. It is anticipated that the proposed embankments will be constructed from materials excavated from the roadway cut areas.

The locations and descriptions of the site and proposed roadway improvements are based on a site reconnaissance, a review of the available plans, and our understanding of the project. If project details vary significantly from those described, Geocon Incorporated should be consulted to provide additional recommendations and/or analysis.

3. SOIL AND GEOLOGIC CONDITIONS

Six surficial soil types and three geologic formations were encountered during the field investigation. The surficial soil deposits consist of undocumented fill, fill placed during construction of Princess Joann Road, topsoil/colluvium, alluvium, debris flows, and landslide deposits. Formational units include the Eocene-age Stadium Conglomerate and Friars Formation, and Cretaceous-age granitic rock. Each of the surficial soil types and geologic units encountered is described below in order of increasing age.

3.1 Undocumented Fill (Qudf)

Several relatively small areas of undocumented end-dumped fill were mapped within the proposed roadway alignment. Although not specifically explored, these fills likely contain vegetation and debris unsuitable for use in properly compacted fill. Where encountered during grading of the roadway, such fills should be cleaned of debris and deleterious matter, removed and properly compacted or exported from the site.

3.2 Princess Joann Road Fill (Qaf)

The condition of this fill is unknown, however, it is suspected that it was placed in conjunction with compaction testing and observation services of a geotechnical engineer. Only a minor portion of this fill will be impacted by the proposed alignment of Cuyamaca Street. The need to remove a portion or all of the impacted fill area should be determined during grading of the site.

3.3 Topsoil/Colluvium (Unmapped)

Topsoils and colluvium blanket the majority of the site and range in thickness from approximately 1 to 4 feet where encountered in the exploratory excavations. The topsoils are characterized as loose/soft to medium dense, reddish brown to dark brown, silty/clayey, fine to medium sands and sandy clays. Topsoils which overlie the Stadium Conglomerate and granitic rock are generally thinner, and have a greater percentage of gravel and cobble fragments. These deposits are considered unsuitable in their present condition and will require removal and compaction in areas planned to receive structural fill and/or settlement sensitive structures.

3.4 Alluvium (Qal)

Alluvial soils are generally limited to the bottom of the three ravines that cross the proposed roadway. As encountered elsewhere on the Fanita Ranch property, these deposits consist of relatively loose/soft, silty/clayey sands and sandy clays with varying amounts of gravel and cobble derived from the Stadium Conglomerate and weathered granitic rock. The alluvial deposits are poorly consolidated and will require remedial grading. The estimated maximum depth of removal is on the order of 5 feet.

3.5 Debris Flow Deposit (Qdf)

The downslope terminus of a debris flow deposit extends on to the mid-length of proposed Cuyamaca Street. Previous excavations in these deposits revealed a relatively unconsolidated cobbly/clayey sand mixture similar to that of nearby alluvial deposits. These deposits will also require removal and compaction prior to placing additional fill.

3.6 Landslide Deposits (Qls)

A landslide was encountered in Boring No. 71 at the southerly end of the proposed Cuyamaca Street alignment. The estimated extent of the landslide is depicted on the Geologic Map and is based on topographic features as well as subsurface explorations performed by others for the Silver Country Estates subdivision to the south. The outcropping of granitic rock along the northeast margins of the landslide appears to have formed a natural buttress to deep-seated movement in an easterly direction. The previous studies by Southern California Soil and Testing (Reference 12) indicate that the slide is considerably deeper to the south of Boring No. 71. Subsequent studies and analyses by Pacific Soils Engineers Inc. (References 10, 11, and 16) concluded that the grading proposed for Silver Country Estates (i.e. placement of an embankment along the toe) will provide a factor of safety of at least 1.5 for the suspected landslide. Due to the heterogeneous characteristics of the landslide deposit, removal, and compaction within the roadway, as well as the construction of a stability fill for that portion of the proposed cut slope that exposes landslide materials, will be required. Cross Section A-A' (Figure 3) depicts the inferred configuration of the landslide and the general extent of remedial grading required.

3.7 Stadium Conglomerate (Tst)

The Eocene-age Stadium Conglomerate unconformably overlies the granitic rock at varying elevations and comprises the majority of the proposed roadway excavations above elevation 700 MSL. Geomorphically the Stadium Conglomerate forms characteristic dissected, lobate ridges within the upper elevations of the site with some conical peaks. Localized, steeply eroded scars occur within this formation where debris flows originated at the head of tributary canyons and ravines. As encountered in exploratory excavations, this deposit generally consists of very dense, light brown to orange-brown, sandy to clayey, gravel and cobble conglomerate.

Moderately heavy to heavy ripping should be anticipated during grading within the Stadium Conglomerate due to randomly occurring highly cemented zones. Excavations performed during the overall Fanita Ranch study confirmed the presence of these zones. Cut or fill slopes composed of the Stadium Conglomerate generally possess good slope stability characteristics.

3.8 Friars Formation (Tf)

The Eocene-age Friars Formation was deposited on an irregular erosional surface formed on the crystalline basement rock of the Southern California Batholith. The Friars Formation consists of relatively flat-lying lagoonal and alluvial claystone, sandstone, and conglomerate units. Specifically, weak, waxy claystone, and thinly laminated siltstone/claystone and sandstone occur at the site below an approximate elevation of 650 MSL. With the possible exception of remedial grading for the landslide deposit, the current alignment for Cuyamaca Street is not expected to expose soils of the Friars Formation.

3.9 Granitic Rock (Kgr)

Cretaceous-age granitic rock of the Southern California Batholith are exposed along the majority of the Cuyamaca Street alignment. Granitic rocks are the oldest geologic units in the region and are believed to underlie the entire roadway and the Fanita Ranch project at depth. Granitic rock materials generally exhibit excellent bearing characteristics in both a natural or properly compacted condition.

Cut slopes excavated in granitic rocks with an inclination of 1.5:1 or flatter should be stable if free from adversely oriented fractures and/or joints.

It is anticipated that the majority of the proposed cuts below elevation 700 MSL will encounter granitic rock. To evaluate the rippability characteristics of the rock, a geophysical survey consisting of 6 seismic refraction traverses was performed. The traverses were conducted with an EG&G Geometrics 1225-model, 12-channel seismograph unit. The traverses were 100 feet long and were performed in both a forward and reverse direction. Typically, the depth evaluated by a seismic survey is on the order of one-third of the traverse length which generally correlates to 30 feet for a 100-foot traverse. The approximate locations of the seismic refraction traverses are shown on Figure 2. Results of the seismic interpretation are included in Appendix A, Tables A-I and A-II.

Based on a review of the geophysical data, it appears that the depth to non-rippable material is variable. Excavations beyond the depths indicated on Table A-II, at those locations, will likely require blasting to efficiently excavate the rock materials.

4. GROUNDWATER/SEEPAGE

No groundwater or seepage was observed or encountered in the exploratory borings or during a reconnaissance of the site. It is possible that areas of localized seepage, perched groundwater, or wet soil may be encountered after periods of heavy rainfall, particularly within the ravines which cross the proposed roadway alignment.

5. GEOLOGIC STRUCTURE

The Stadium Conglomerate is presumed to have been deposited unconformably on an irregular crystalline bedrock surface yielding variable contact geometry. Due to this irregularity, planned cuts above the contact may encounter granitic rock at a shallower depth than anticipated based on the mapped contact elevation. The Stadium Conglomerate observed during the investigation of Fanita Ranch was massive and lacked structure. Previous studies suggest that a regional dip of the unit of 3 to 10 degrees to the south-southwest may exist.

6. GEOLOGIC HAZARDS

6.1 Ancient Landslides

The potentially compressible portions of the landslide deposit mapped within the roadway alignment can be mitigated using generally accepted remedial grading techniques. The techniques consist of partial or complete removal and compaction of the deposits. A stability fill is recommended where weak claystone beds within the landslide are exposed in the proposed slope.

6.2 Debris Flow Deposits

These deposits are limited to a small area along the roadway alignment and generally consist of an accumulation of topsoil, colluvium and debris derived from formational "parent material" near the base of moderate to steep slopes that have resulted from rapid flow of saturated near-surface soils. High rainfall, steep slopes, loss of vegetation cover, and thick overburden are the suspected main factors contributing to the occurrence of debris flows. The primary difference, in terms of the potential for activation, between ancient landslides and debris flows is that, by definition, debris flows do not possess a basal slip surface. Thus, they are much less likely to become reactivated by grading than ancient landslides.

The existing debris flow deposits will be removed below the proposed roadway embankment and the roadway will be elevated above the deposit. Should reactivation of the debris flow occur, it is unlikely that the roadway embankment would be breached by the flow. In areas of proposed development, mitigation of debris flow deposits will be similar to that for alluvium and colluvium, and the presence of these materials is not likely to impact the improvements.

6.3 Faulting and Seismicity

Based on our reconnaissance and a review of published geologic maps and reports, the site is not located on any known “active,” “potentially active” or “inactive” fault traces as defined by the California Geological Survey (CGS).

The Newport-Inglewood Fault and Rose Canyon Fault Zone, located approximately 14 miles west of the site, are the closest known active faults. The CGS considers a fault seismically active when evidence suggests seismic activity within roughly the last 11,000 years. The CGS has included portions of the Rose Canyon Fault Zone within an Alquist-Priolo Earthquake Fault Zone.

6.4 Seismicity-Deterministic Analysis

We used the computer program *EZ-FRISK* (Version 7.65) to determine the distance of known faults to the site and to estimate ground accelerations at the site for the maximum anticipated seismic event.

According to the results of the computer program *EZ-FRISK* (Version 7.65), 8 known active faults are located within a search radius of 50 miles from the property. We used acceleration attenuation relationships developed by Boore-Atkinson (2008) NGA USGS2008, Campbell-Bozorgnia (2008) NGA USGS, and Chiou-Youngs (2008) NGA in our analysis. The nearest known active faults are the Newport-Inglewood and Rose Canyon Fault Zones, located approximately 14 miles west of the site, respectively, and are the dominant sources of potential ground motion. Table 6.4 lists the estimated maximum earthquake magnitudes and PGA’s for the most dominant faults for the site location calculated for Site Class D as defined by Table 1613.3.2 of the 2016 California Building Code (CBC).

TABLE 6.4
DETERMINISTIC SPECTRA SITE PARAMETERS

Fault Name	Distance from Site (miles)	Maximum Earthquake Magnitude (Mw)	Peak Ground Acceleration		
			Boore-Atkinson 2008 (g)	Campbell-Bozorgnia 2008 (g)	Chiou-Youngs 2008 (g)
Newport-Inglewood	14	7.5	0.23	0.17	0.22
Rose Canyon	14	6.9	0.19	0.15	0.16
Elsinore	27	7.85	0.19	0.12	0.16
Palos Verdes Connected	26	7.7	0.18	0.12	0.15
Coronado Bank	27	7.4	0.16	0.11	0.12
Palos Verdes Connected	27	7.7	0.18	0.12	0.15
Earthquake Valley	31	6.8	0.12	0.08	0.07
San Jacinto	48	7.88	0.13	0.08	0.10

6.5 Seismicity-Probabilistic Analysis

We used the computer program *EZ-FRISK* (version 7.65) to perform a probabilistic seismic hazard analysis. *EZ-FRISK* operates under the assumption that the occurrence rate of earthquakes on each mapped Quaternary fault is proportional to the fault slip rate. The program accounts for earthquake magnitude as a function of rupture length. Site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program also accounts for uncertainty in each of following: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from considered earthquake sources, the program calculates the total average annual expected number of occurrences of site acceleration greater than a specified value. We utilized acceleration-attenuation relationships suggested by Boore-Atkinson (2008) NGA USGS 2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2008) NGA USGS 2008 in the analysis. Table 6.5 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence for Site Class D.

TABLE 6.5
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedence	Peak Ground Acceleration		
	Boore-Atkinson, 2008 (g)	Campbell-Bozorgnia, 2008 (g)	Chiou-Youngs, 2008 (g)
2% in a 50 Year Period	0.44	0.36	0.42
5% in a 50 Year Period	0.34	0.27	0.30
10% in a 50 Year Period	0.27	0.22	0.23

While listing peak accelerations is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including frequency and duration of motion and soil conditions underlying the site. Seismic design of the structures should be evaluated in accordance with the California Building Code (CBC) or City of Santee guidelines.

6.6. Liquefaction

Liquefaction typically occurs when a site is located in a zone with seismic activity, onsite soils are cohesionless, groundwater is encountered within 50 feet of the surface, and soil relative density is less than about 70 percent. If all four criteria are met, a seismic event could result in a rapid increase in pore water pressure from the earthquake-generated ground accelerations. The potential for liquefaction at the site is considered low due to the dense formational material encountered, remedial grading recommended, and lack of significant deposits of saturated soils that could be susceptible to liquefaction.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 No soil or geologic conditions were encountered that would preclude the construction of Cuyamaca Street, as presently planned, provided the recommendations of this report are followed.

7.1.2 The surficial soils (topsoil, colluvium, undocumented fill, alluvium, debris flow deposits, and landslide deposits) are not considered suitable for the support of fill or structural loads in their present condition and will require remedial grading.

7.1.3 A stability fill in landslide material will be required along the southern portion of the alignment. Since the slope ratio is planned at 1.5:1 (H:V), geogrid reinforcement will be necessary to construct the over-steepened fill slope. Typically, a layer of geogrid, such as Miragrid 5XT, placed every 2 vertical feet and approximately 10 feet long will be required. A specific design will be provided as grading plans progress.

7.2 Soil and Excavation Characteristics

7.2.1 The soil conditions encountered vary from low expansive, sandy gravel and cobble conglomerate and silty sands to highly expansive, clayey topsoils. The Stadium Conglomerate will likely require moderately heavy to heavy ripping due to the random occurrence of highly cemented zones. Excavating within the granitic materials will generally vary in difficulty with the depth of excavation. Blasting will likely be required for most excavations deeper than 20 feet. Oversize, cemented chunks of conglomerate and oversize rocks will likely be generated and require special handling and placement in fill areas.

7.3 Terrace Drains

7.3.1 The use of terrace drains on cut or fill slopes exceeding 30 feet in height is not considered necessary to maintain gross stability of the slopes. Based on past experience with similar projects, properly-constructed and maintained terrace drains may reduce slope erosion, particularly on fill slopes. However, improperly-maintained terrace drains can result in significant slope erosion and possible slope distress. Terrace drains that are allowed to fill with debris may concentrate surface runoff down the slope face, resulting in deep, extensive erosion gullies. It is therefore recommended that the use of terrace drains planned for cut or fill slopes on the project be kept to a minimum, consistent with the general guidelines which follow.

7.3.2 For cut or fill slopes above developed lots, a terrace drain should be provided no higher than 30 feet above the toe of slope or alternatively a lined surface drain may be located along the toe of slope.

7.3.3 For cut or fill slopes above streets or non-building areas, terrace drains are not required.

7.3.4 All terrace drains should direct the flow of water into storm drains or other suitable drainage facilities. For "daylight" canyon fills, down-drains should be provided at the contact between fill and natural materials, to reduce erosion along the contact.

7.3.5 The above recommendations are presented as general guidelines only; other considerations may dictate the design of slope terrace drains. All terrace drains should be sized to accommodate the maximum flow of water anticipated from the drainage area above, under the design rainfall event.

7.3.6 It is recommended that terrace drains be constructed at a drainage gradient of at least 2 percent, and steeper, where practical. In addition, a maintenance program should be devised and followed, which clearly designates the persons or agencies responsible for maintaining terrace drains within specific areas.

7.4 Grading

7.4.1 All grading should be performed in accordance with the attached Recommended Grading Specifications (Appendix D). Where the recommendations of this section conflict with Appendix D, the recommendations of this section take precedence. All earthwork should be observed and all fills tested for proper compaction by Geocon Incorporated.

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

7.4.3 Site preparation should begin with the removal of all deleterious material and vegetation. The depth of removal should be such that material exposed in cut areas or soils to be used as fill is relatively free of organic matter. Material generated during stripping and/or site demolition can remain on-site and be used for ecological restoration. Trash or any other objectionable materials not suitable for fills should be hauled off-site.

7.4.4 All potentially compressible surficial soils within areas of planned grading should be removed to firm natural ground and properly compacted prior to placing additional fill and/or structural loads. The upper 10 feet of landslide deposits within the paved roadway area should be removed and properly compacted. The actual extent of unsuitable soil removals should be determined in the field by the soil engineer and/or engineering geologist. Overly wet, surficial materials will require drying and/or mixing with drier soils to facilitate proper compaction.

7.4.5 The site should then be brought to final subgrade elevations with structural fill compacted in layers. In general, soils native to the site are suitable for re-use as fill if free from vegetation, debris and other deleterious material. Layers of fill should be no thicker than will allow for adequate bonding and compaction. All fill, including backfill and scarified ground surfaces, should be compacted to at least 90 percent of maximum dry density at above optimum moisture content, as determined in accordance with ASTM Test Procedure D 1557-12. Fill materials near and/or below optimum moisture content will require additional moisture conditioning prior to placing additional fill.

7.4.6 Where practical, the upper 2 feet of subgrade in pavement areas should be composed of properly compacted or undisturbed formational "very low" to "low" expansive soils. The more highly expansive fill soils should be placed in the deeper fill areas and properly compacted. "Very low" to "low" expansive soils are defined as those soils that have an Expansion Index of 50 or less as defined by 2016 California Building Code (CBC) Section 1803.5.3. Rock or concretions greater than 12 inches in maximum dimension should not be placed within 10 feet of finish grade or 3 feet of the deepest utility.

7.4.7. Where granitic rock requiring blasting to excavate is encountered at subgrade elevation, consideration should be given to overexcavating all or a portion of the subgrade a sufficient depth to facilitate subsequent excavations for planned utility lines.

7.5 Slope Stability Evaluation

7.5.1 A slope stability analysis was performed on the proposed 1.5:1 (horizontal:vertical), approximately 20-foot-high cut slope located at the south end of the alignment. The proposed excavations will expose an incipient landslide which will require mitigation in the form of a geogrid reinforced stability fill in the slope zone and removal and compaction of landslide material within the roadway alignment. We understand the stability of the overall slide mass has been mitigated during grading of the Silver Country Estates project to the south. The analysis utilized the computer software program GeoStudio 2007 to evaluate the factor of safety against deep-seated failure using Spencer’s Method. A summary of the static slope stability analyses performed is shown on Table 7.5.2. Although not encountered during the field exploration, groundwater was conservatively incorporated into the analysis approximately 5 feet above the landslide basal slip surface.

7.5.2 Laboratory tests were performed on relatively undisturbed samples of the prevailing soil and geologic units and the results are presented in Appendix B. Table 7.5.1 presents the soil strength parameters that were utilized in the slope stability analyses.

TABLE 7.5.1
SOIL STRENGTH PARAMETERS

Soil Condition	Angle of Internal Friction ϕ (degrees)	Cohesion c (psf)
Compacted Fill	35	300
Landslide Debris	20	200
Basal Slip Surface	7	150
Granitic Rock	35	500
Friars Formation	33	500
Stadium Conglomerate	35	500

TABLE 7.5.2
STATIC SLOPE STABILITY SUMMARY

Section	Figure Number	Condition Analyzed	Factor Of Safety
A-A	C-1	Block-Type Failure through BPS and Qcf	1.6

7.5.3 Based on the results of the slope stability analysis, the proposed 1.5:1 geogrid reinforced fill slope exhibits a factor of safety of at least 1.5 under static conditions and is considered stable. The output files and calculated factor of safety for the proposed 1.5:1 slope are presented in Appendix C.

7.6 Slope Stability-General

7.6.1. Slope stability analysis utilizing average drained direct shear strength parameters based on laboratory tests and experience with similar soil types in nearby areas 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. Generalized slope stability calculations for deep-seated and surficial slope stability are presented on Figures 4 through 6.

7.6.2. It is recommended that all cut slope excavations be observed during grading by an engineering geologist to verify that soil and geologic conditions do not differ significantly from those anticipated.

7.6.3. Slope stabilization measures will be required where cut slopes are planned in areas of thick surficial deposits, such as colluvium or landslide debris, or if fractured claystones and/or groundwater seepage are observed during the grading operations. Observation and testing during grading may be necessary to evaluate the suitability of these materials in slopes. Drained stabilization fills are recommended where these conditions are encountered (e.g. south end of proposed roadway). A typical stability fill detail is shown as Figure 7.

7.6.4. The outer 15 feet (or a distance equal to the height of the slope, whichever is less) of fill slopes should be composed of properly compacted granular "soil" fill to reduce the potential for surficial sloughing. In general, soils with an Expansion Index of less than 90 or at least 35 percent sand size particles should be acceptable as "granular" fill. Soils of questionable strength to satisfy surficial stability should be tested in the laboratory for acceptable drained shear strength. Slopes should be compacted by backrolling with a loaded sheepsfoot roller at vertical intervals not to exceed 4 feet and should be track-walked at the completion of each slope such that the fill soils are uniformly compacted to at least 90 percent relative compaction to the face of the finished sloped.

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

7.7 Pavement Design (Cuyamaca Street)

7.7.1 The determination of the appropriate pavement section for Cuyamaca Street will be dependent on the R-Value of the subgrade soils after grading and the design Traffic Index. Where practical, consideration should be given to using decomposed granite excavated from proposed cut areas to plate the upper two feet of the roadway subgrade. Decomposed granite typically possesses a high R-Value and, hence, should result in a reduced pavement section.

7.8 Site Drainage

7.8.1 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 2016 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.

7.8.2 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.

7.9 Grading Plan Review

7.9.1 Geocon Incorporated should review the grading plans for the project prior to final design submittal to determine if additional analysis and/or recommendations are required.

7.9.2 The proposed stability fill and recommended landslide deposit removal subdrains should be shown on the final grading plans.

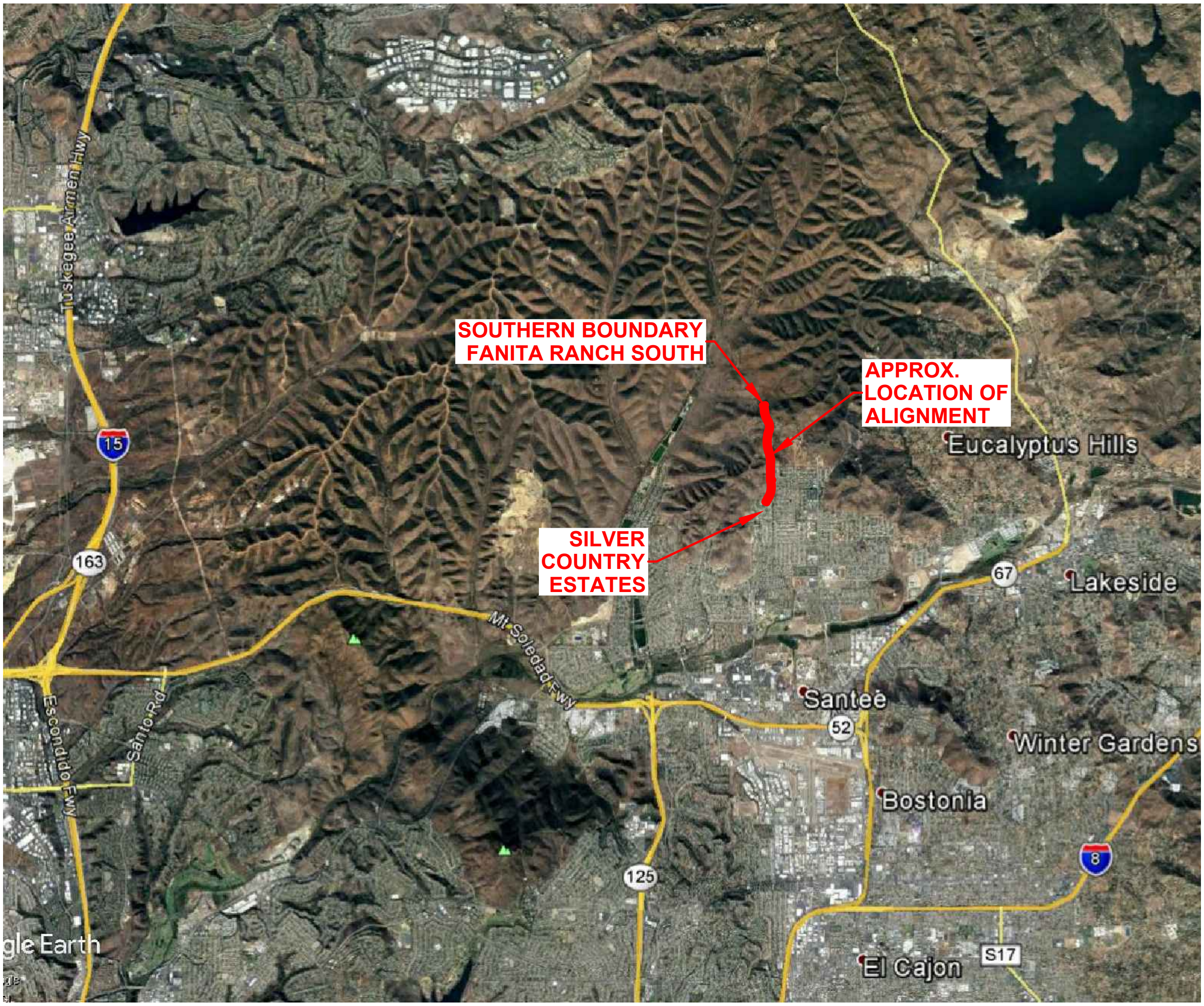
LIMITATIONS AND UNIFORMITY OF CONDITIONS

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.



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N

NO SCALE

VICINITY MAP

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FANITA RANCH
OFF-SITE IMPROVEMENT TO CUYAMACA STREET
SANTEE, CALIFORNIA

DE / RA		DSK/GTYPD	DATE 04 - 17 - 2020	PROJECT NO. 05254 - 32 - 18A	FIG. 1

Plotted:04/14/2020 2:55PM | By:RUBEN AGUILAR | File Location:Y:\PROJECTS\05254-32-18A Fanita Ranch-Improvement Cuyamaca St\DETAILS\05254-32-18A VicinityMap.dwg

GEOLOGIC MAP

FANITA RANCH
OFF-SITE IMPROVEMENT TO CUYAMACA STREET
SANTEE, CALIFORNIA

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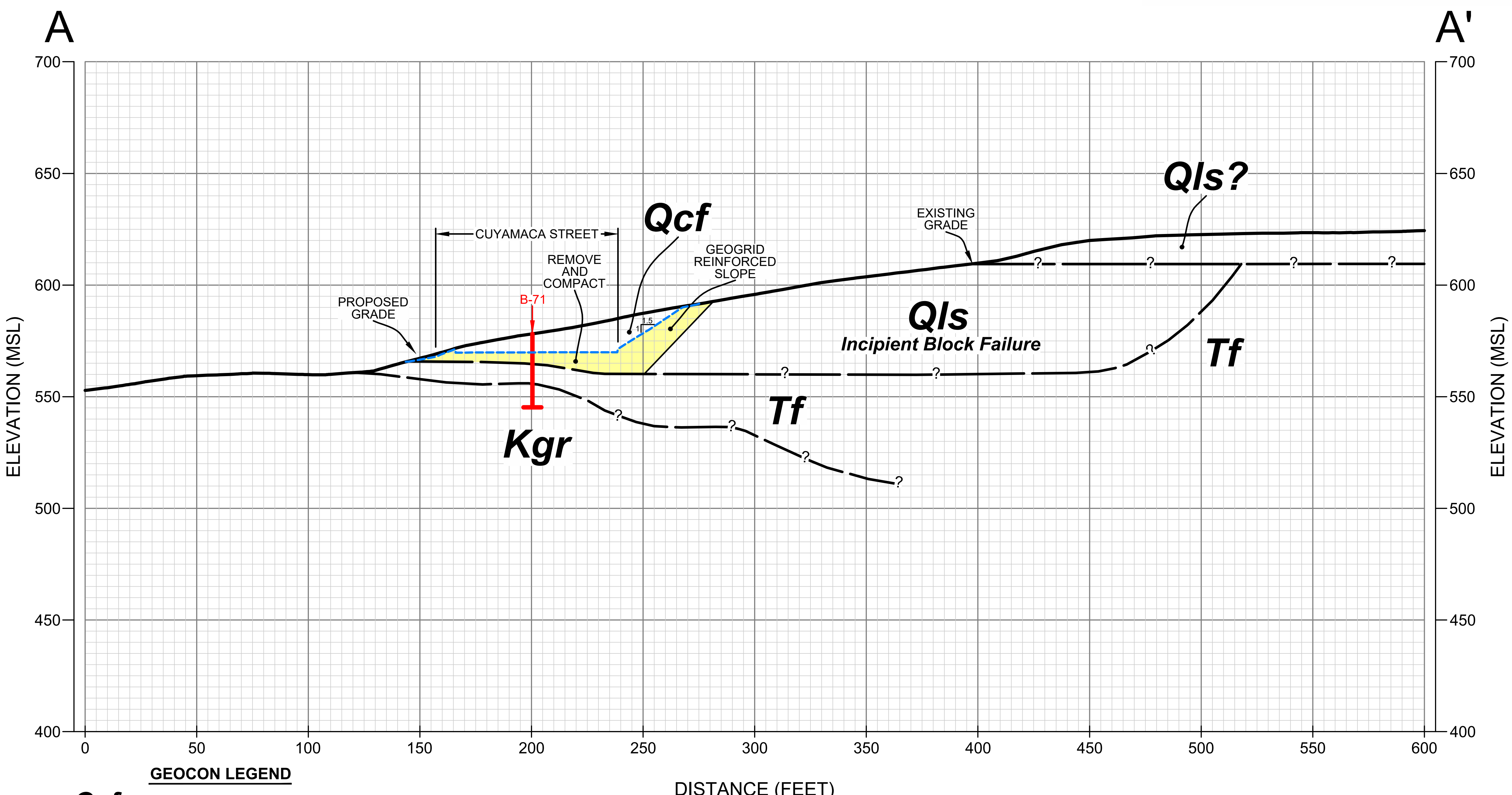
SCALE 1" = 100'	DATE 04 - 17 - 2020		FIGURE
PROJECT NO. 05254 - 32 - 18A			2
SHEET 1 OF 1			

Plotted:04/15/2020 2:20PM | By:RUBEN AGUILAR | File Location:Y:\PROJECTS\05254-32-18A Fanita Ranch-Improvement Cuyamaca St\SHEETS\05254-32-18A GeoMap-ImprvmntsCuyamacaSt.dwg

FANITA RANCH
OFF-SITE IMPROVEMENT TO CUYAMACA STREET
SANTEE, CALIFORNIA



GEOCON LEGEND

Qcf ....COMPACTED FILL
Qls ....LANDSLIDE DEPOSITS
Tf ....FRIARS FORMATION
Kgr ....GRANITIC ROCK
B-71 ....APPROX. LOCATION OF EXPLORATORY BORING
? ....APPROX. LOCATION OF GEOLOGIC CONTACT
(Queried Where Uncertain)

GEOLOGIC CROSS-SECTION A-A'

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

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GEOLOGIC CROSS - SECTION
PROJECT NO. 05254 - 32 - 18A
FIGURE 3
DATE 04 - 17 - 2020

Plotted:04/14/2020 3:04PM | By:RUBEN AGUILAR | File Location:Y:\PROJECTS\05254-32-18A Fanita Ranch-Improvement Cuyamaca St\SHEETS\05254-32-18A Cross-Section.dwg

ASSUMED CONDITIONS :

SLOPE HEIGHT	H	= Infinite
DEPTH OF SATURATION	Z	= 3 feet
SLOPE INCLINATION	2 : 1 (Horizontal : Vertical)	
SLOPE ANGLE	i	= 26.6 degrees
UNIT WEIGHT OF WATER	γ_w	= 62.4 pounds per cubic foot
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 130 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 35 degrees
APPARENT COHESION	C	= 300 pounds per square foot

SLOPE SATURATED TO VERTICAL DEPTH Z BELOW SLOPE FACE

SEEPAGE FORCES PARALLEL TO SLOPE FACE

ANALYSIS :

$$FS = \frac{C + (\gamma_t - \gamma_w)\ Z \cos^2 i \tan \phi}{\gamma_t\ Z \sin i \cos i} = 2.6$$

REFERENCES :

1......Haefeli, R. *The Stability of Slopes Acted Upon by Parallel Seepage*, Proc. Second International Conference, SMFE, Rotterdam, 1948, 1, 57-62

2......Skempton, A. W., and F.A. Delory, *Stability of Natural Slopes in London Clay*, Proc. Fourth International Conference, SMFE, London, 1957, 2, 378-81

SURFICIAL SLOPE STABILITY ANALYSIS

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FANITA RANCH
OFF-SITE IMPROVEMENT TO CUYAMACA STREET
SANTEE, CALIFORNIA

DE / RA		DSK/GTYPD	DATE 04 - 17 - 2020	PROJECT NO. 05254 - 32 - 18A	FIG. 4

Plotted:04/14/2020 3:04PM | By:RUBEN AGUILAR | File Location:Y:\PROJECTS\05254-32-18A Fanita Ranch-Improvement Cuyamaca St\DETAILS\Slope Stability Analyses-Surficial (SFSSA).dwg

ASSUMED CONDITIONS :

SLOPE HEIGHT	H	= 150 feet
SLOPE INCLINATION	2 : 1 (Horizontal : Vertical)	
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 125 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 35 degrees
APPARENT COHESION	C	= 300 pounds per square foot
NO SEEPAGE FORCES		

ANALYSIS :

$$\gamma_{c\phi} = \frac{\gamma_t H \tan\phi}{C}$$ EQUATION (3-3), REFERENCE 1

$$FS = \frac{N_{cf} C}{\gamma_t H}$$ EQUATION (3-2), REFERENCE 1

$\gamma_{c\phi}$ = 44 CALCULATED USING EQ. (3-3)

N_{cf} = 100 DETERMINED USING FIGURE 10, REFERENCE 2

FS = 1.6 FACTOR OF SAFETY CALCULATED USING EQ. (3-2)

REFERENCES :

1......Janbu, N., Stability Analysis of Slopes with Dimensionless Parameters, Harvard Soil Mechanics, Series No. 46, 1954

2......Janbu, N., Discussion of J.M. Bell, Dimensionless Parameters for Homogeneous Earth Slopes, Journal of Soil Mechanics and Foundation Design, No. SM6, November 1967.

SLOPE STABILITY ANALYSIS - FILL SLOPES

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FANITA RANCH
OFF-SITE IMPROVEMENT TO CUYAMACA STREET
SANTEE, CALIFORNIA

DE / RA		DSK/GTYPD	DATE 04 - 17 - 2020	PROJECT NO. 05254 - 32 - 18A	FIG. 5

ASSUMED CONDITIONS :

SLOPE HEIGHT	H	= 85 feet
SLOPE INCLINATION	1.5 : 1	(Horizontal : Vertical)
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 130 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 35 degrees
APPARENT COHESION	C	= 500 pounds per square foot
NO SEEPAGE FORCES		

ANALYSIS :

$\gamma_{c\phi} = \dfrac{\gamma_t H \tan\phi}{C}$		EQUATION (3-3), REFERENCE 1
$FS = \dfrac{N_{cf} C}{\gamma_t H}$		EQUATION (3-2), REFERENCE 1
$\gamma_{c\phi}$	= 15.5	CALCULATED USING EQ. (3-3)
Ncf	= 35	DETERMINED USING FIGURE 10, REFERENCE 2
FS	= 1.6	FACTOR OF SAFETY CALCULATED USING EQ. (3-2)

REFERENCES :

1......Janbu, N., Stability Analysis of Slopes with Dimensionless Parameters, Harvard Soil Mechanics, Series No. 46, 1954

2......Janbu, N., Discussion of J.M. Bell, Dimensionless Parameters for Homogeneous Earth Slopes, Journal of Soil Mechanics and Foundation Design, No. SM6, November 1967.

SLOPE STABILITY ANALYSIS - CUT SLOPES

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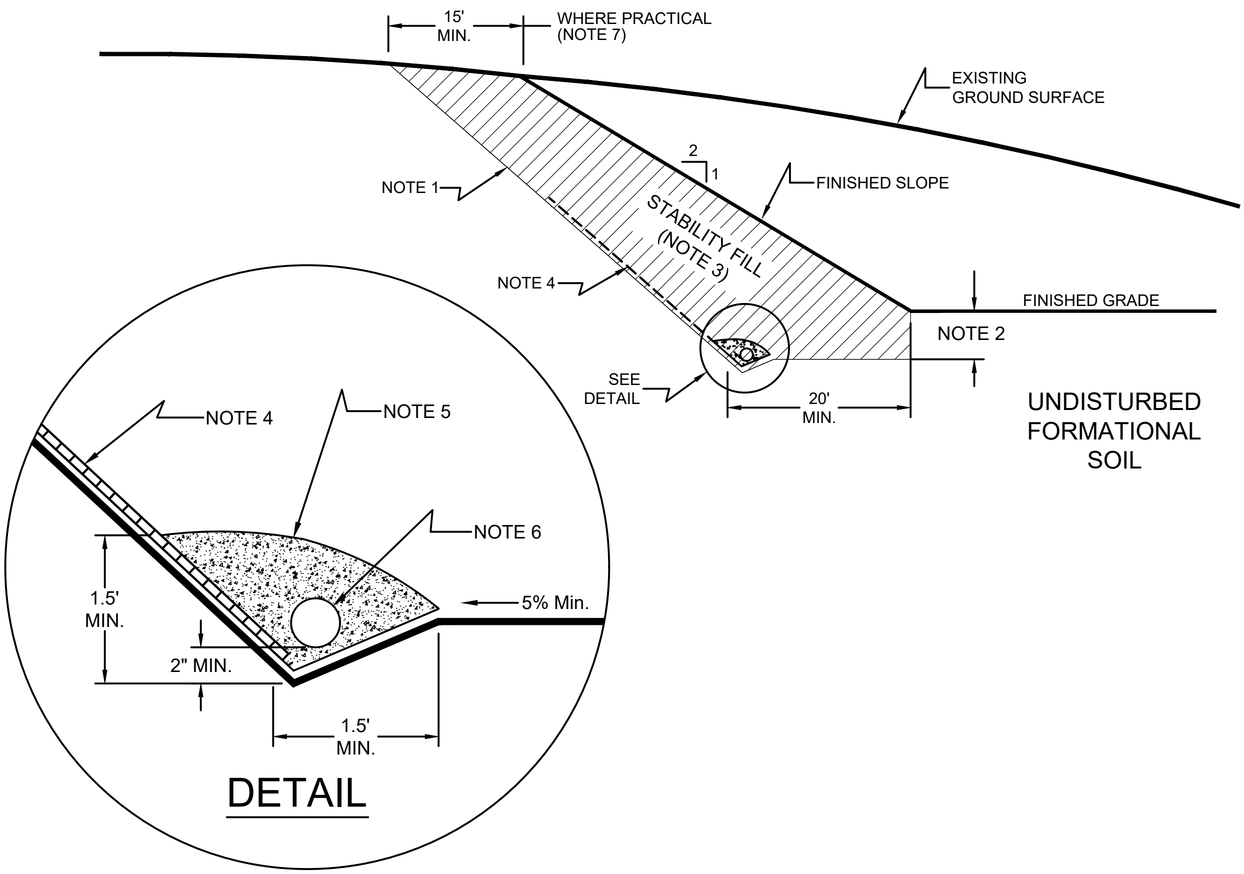
FANITA RANCH
OFF-SITE IMPROVEMENT TO CUYAMACA STREET
SANTEE, CALIFORNIA

DE / RA		DSK/GTYPD	DATE 04 - 17 - 2020	PROJECT NO. 05254 - 32 - 18A	FIG. 6



NOTES:

1.....EXCAVATE BACKCUT IN ACCORDANCE WITH GEOTECHNICAL CONSULTANTS RECOMMENDATION.

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

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

4.....CHIMNEY DRAINS TO BE APPROVED, PREFABRICATED CHIMNEY DRAIN PANELS (MIRADRAIN, TENSAR, OR EQUIVALENT) SPACED APPROXIMATELY 30 FEET CENTER TO CENTER. ADDITIONAL DRAINS WILL BE REQUIRED WHERE AREAS OF SEEPAGE ARE ENCOUNTERED.

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

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.

7.....IF HORIZONTAL, EXTENT OF GRADING IS CONSTRAINED (e.g., THE PRESENCE OF A PROPERTY LINE), THE SLOPE SHOULD BE OVERBUILT, AT LEAST 4 FEET, AND TRIMMED BACK.

NO SCALE

TYPICAL STABILITY FILL DETAIL

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FANITA RANCH
OFF-SITE IMPROVEMENT TO CUYAMACA STREET
SANTEE, CALIFORNIA

DE / RA		DSK/GTYPD	DATE 04 - 17 - 2020	PROJECT NO. 05254 - 32 - 18A	FIG. 7

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The original field investigation for the overall project was performed intermittently between February 6, 1995 and November 20, 1996 and consisted of a visual site reconnaissance, excavation of 85 large-diameter borings, 207 backhoe trenches, and performance of 19 seismic refraction traverses. The study for off-site Cuyamaca Street consisted of excavating 4 exploratory borings on May 12, 1995 and performing 6 seismic refraction traverses at a later date. The approximate locations of the exploratory trenches and seismic traverses that were performed for Cuyamaca Street are shown on Figure 2.

The large-diameter borings were advanced to a depth of 6 to 31 feet using an Easy Bore 120 truck-mounted drill rig equipped with a 30-inch-diameter bucket auger. Relatively undisturbed samples were obtained by driving a 3-inch split-tube samples 12 inches into the undisturbed soil mass with blows from a telescoping Kelly bar varying in weight from 1,800 to 4,500 pounds. The sampler was equipped with six 1-inch by 2.5-inch brass sampler rings to facilitate removal and testing. The soils encountered in the borings were visually examined, classified, and logged. Logs of borings are presented on Figures A-1 through A-4. The logs depict the soil and geologic conditions encountered.

The seismic traverses were performed with an EG&G Geometrics 1225-model, 12-channel seismograph unit. The traverses were 100 feet long and were performed in both a forward and reverse direction. The results of each seismic traverse are summarized on Table A-I. Table A-II presents our interpretation of rippable thickness of the rock based on the date obtained.

TABLE A-I
SEISMIC TRAVERSES

Seismic Traverse No.	Average Velocity (ft./sec.) V_1	V_2	V_3	Average Depth (ft.) D_1	D_2	D_3	Length of Traverse (ft.)	Approximate Maximum Depth Explored (ft.)
S-14	3300	5100	-	4	>30	-	100	30
S-15	1500	4300	-	7	>30	-	100	30
S-16	1500	3000	7900	3	16	>30	100	30
S-17	1500	3800	5400	6	16	>30	100	30
S-18	1700	3100	5700	5	17	>30	100	30
S-19	1200	2700	5800	6	22	>30	100	30

V_1 = Velocity in feet per second of first layer of materials
V_2 = Second layer velocities
V_3 = Third layer velocities
D_1 = Depth in feet to base of first layer
D_2 = Depth to base of second layer
D_3 = Depth to base of third layer

NOTE:
For mass grading, materials with velocities of less than 4500 fps are generally rippable with a D9 Caterpillar Tractor equipped with a single shank hydraulic ripper. Velocities of 4500 to 5500 fps indicate marginal ripping and blasting. Velocities greater than 5500 fps generally require pre-blasting. For trenching, materials with velocities less than 3800 fps are generally rippable depending upon the degree of fracturing and the presence or absence of boulders. Velocities between 3800 and 4300 fps generally indicate marginal ripping, and velocities greater than 4300 fps generally indicate non-rippable conditions. The above velocities are based on a Kohring 505.
The reported velocities represent average velocities over the length of each traverse, and should not generally be used for subsurface interpretation greater than 100 feet from a traverse.

TABLE A-II
APPROXIMATE THICKNESS OF RIPPABLE ROCK

Traverse No.	Approximate Thickness (ft.)*
S-14	30 (marginally rippable at 4 feet)
S-15	30**
S-16	16
S-17	30 (marginally rippable at 16 feet)
S-18	17
S-19	22

*****Assumes D9 Caterpillar Dozer.
******Possible erratic data.

PROJECT NO. 05254-52-02

BORING B 71

ELEV. (MSL.) 574 DATE COMPLETED 5/12/95

EQUIPMENT E-120

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0–4				CL/CH	**COLLUVIUM** Stiff, very moist, grayish brown, Silty CLAY with some fine to medium SAND			
4–6.5	B71-1			CL	**LANDSLIDE DEBRIS(Incipient block failure)** Stiff to very stiff, moist, light green, Silty CLAY -Several discontinuous, olive-grey clay seams along fractures at 4 feet, no discernible shearing or remolding, abundant calcium carbonate -Gradational contact at 6.5 feet	PUSH	95.0	28.5
6.5–8				CL	Hard, damp, light green, Silty CLAYSTONE; slightly blocky -Gradational contact at 8 feet			
8–10.5	B71-2			ML/CL	Dense, damp, light green, fine Sandy SILTSTONE with clay; massive and structureless **BASAL SLIP PLANE from 10.4 to 11.5 feet** (24,N65W); 1/4 to 3/4 inch thick, moist, olive-grey, slightly remolded plastic clay, irregular thickness, continuous and well developed, minor disturbance above plane	~~2~~	~~110.3~~	~~19.8~~
10.5–22	B71-3			ML/CL	**FRIARS FORMATION** Dense, damp, light green, fine Sandy SILTSTONE with clay	2	95.9	29.1
					-Becomes sandy and slightly micaceous at 19 feet			
	B71-4					3	97.4	28.0
					-Gradational contact at 22 feet			
22–31	B71-5				**GRANITIC ROCK** Moderately to highly weathered, light green to grey, strong, GRANITIC ROCK; relic feldspar crystals, becomes less weathered with depth -Steeply dipping fracture infilled with 1/4 inch thick clay (solution mineral ?) (36,S40W)	8/8"	128.9	8.6
					-Becomes slightly weathered, dark grey, and very strong at 29 feet			
					REFUSAL AT 31 FEET			

Figure A-1 Log of Boring B 71, page 1 of 1

FRNC1

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. 05254-52-02

BORING B 72

ELEV. (MSL.) 568 DATE COMPLETED 5/12/95

EQUIPMENT E-120

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	**TOPSOIL** Stiff, moist, orange-brown, fine to coarse Sandy CLAY			
2					**GRANITIC ROCK** Moderately weathered, orange-brown, strong, GRANITIC ROCK; excavates to silty, fine to coarse sand			
4								
6					-Becomes light greyish brown in color at 6 feet			
8					-Becomes slightly weathered, grey and very strong at 8 feet			
10								
					PRACTICAL REFUSAL AT 11 FEET			

Figure A-2 Log of Boring B 72, page 1 of 1

FRNC1

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. 05254-52-02

BORING B 73

ELEV. (MSL.) 622 DATE COMPLETED 5/12/95

EQUIPMENT E-120

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	**TOPSOIL** Stiff, moist, orange-brown, fine to coarse Sandy CLAY			
2					**GRANITIC ROCK** Moderately weathered, orange-brown, strong, GRANITIC ROCK; excavates to silty, fine to very coarse sand			
4								
6					-Becomes slightly weathered, dark grey, and very strong at 5 feet			
					PRACTICAL REFUSAL AT 6 FEET			

Figure A-3 **Log of Boring B 73, page 1 of 1**

FRNC1

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. 05254-52-02

BORING B 74

ELEV. (MSL.) 645 DATE COMPLETED 5/12/95

EQUIPMENT E-120

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0–3.5				CL	**COLLUVIUM** Stiff, moist to very moist, dark brown, Silty CLAY with some fine to coarse sand and gravel			
3.5–8				ML	**FRIARS FORMATION** Dense, damp, light green, fine to coarse Sandy SILTSTONE -Moderately weathered with heavy calcium carbonate from 3.5 to 4.5 feet -Highly fractured and blocky to 7 feet with some 1/8 inch open fractures -Gradation contact at 8 feet			
8–24				SM-ML	Very dense, damp, light green, Silty, fine to medium SANDSTONE/fine to medium Sandy SILTSTONE grades back and forth -Some 2-inch- to 4-inch-diameter clasts of highly weathered granitic rock observed in cuttings from approximately 16 feet -Becomes highly cemented at 22 feet			
24					REFUSAL AT 24 FEET			

Figure A-4 Log of Boring B 74, page 1 of 1

FRNC1

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.

APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in general accordance with the test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected, relatively undisturbed drive samples were tested for their in-place dry density, moisture content, shear strength, and consolidation characteristics. Grain size distribution, maximum dry density and optimum moisture content, Expansion Index, pH/resistivity, and plasticity index of selected bulk samples were determined. Portions of the bulk samples were then remolded to selected densities and subjected to drained direct shear tests.

The results of our laboratory tests are presented in tabular and graphical forms hereinafter. The in-place density and moisture characteristics are presented on the logs of the exploratory borings.

TABLE B-I
SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557-12

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
B3-7	Brown, Sandy CLAY	108.6	18.8
B3-16	Light green, Silty, fine SAND	114.2	15.4
B7-2	Light brown, Silty, fine SAND	106.1	18.8
B11-2	Medium brown, Silty CLAY	113.9	16.5
B16-2	Light grey, Silty, fine SAND	114.2	15.3
B16-10	Medium green, Clayey Silty SAND	112.4	16.8
B20-1	Light brown, Gravelly Silty, SAND with cobbles	122.8	12.5
B21-5	Light grey, Silty, fine SAND	123.3	11.4
B26-2	Light brown, Clayey, fine to medium SAND	119.9	13.2
B27-1	Light brown, Gravelly CLAY with cobbles	123.0	11.0
B29-4	Light green-grey, Silty, fine SAND	102.5	22.7
B34-1	Light brown, Gravelly Clayey SAND with cobbles	130.1	9.1
B37-1	Light green-grey, Clayey SAND	116.1	15.4
B43-1	Light brown, Gravelly Clayey SAND with cobbles	128.7	10.6
B45-5	Dark green, Silty CLAY	112.0	17.8
B50-7	Light grey, Silty SAND	122.8	12.0
B51-2	Grey-green, Gravelly Clayey Silty SAND	121.5	13.8
B55-3	Brown, Sandy Silty CLAY	113.8	15.3

TABLE B-II
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829-11

Sample No.	Moisture Content		Dry Density (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
B27-1	11.0	27.7	107.0	57
B28-2	11.6	28.4	103.4	73
B34-1	8.2	23.0	117.0	37
B35-6	14.4	40.6	94.2	115
B45-9	10.9	34.3	104.8	76
B51-2	10.8	28.3	108.2	57

TABLE B-III
SUMMARY OF DIRECT SHEAR TEST RESULTS
ASTM D 3080-11

Sample No.	Dry Density (pcf)	Moisture Content (%)	Unit Cohesion (psf)	Angle of Shear Resistance (degrees)
B3-7*	98.4	18.1	525	29
B3-8	106.3	21.2	390	37
B3-14	116.5	12.6	420	43
B3-16*	103.3	15.0	800	32
B5-8**	89.3	35.0	240	7
B8-3	101.1	22.2	700	19
B8-10	105.7	21.9	2200	21
B11-2*	102.6	16.6	1000	7
B11-6	97.0	26.9	600	44
B11-11	101.0	25.9	270	38
B16-2*	103.3	14.9	425	30
B16-9	106.4	21.7	1375	20
B16-10*	101.5	16.4	940	30
B19-3	104.1	23.2	450	37
B19-7	104.0	22.8	375	28
B20-1*	110.4	12.8	900	25
B21-5*	111.0	11.3	950	36
B24-2	106.9	20.2	1000	36
B26-1	118.9	13.6	1350	39
B26-2*	107.8	13.3	900	38
B26-5	118.1	15.9	1940	33
B27-1*	109.9	11.8	450	34
B29-4*	93.9	20.8	975	32
B29-8	121.7	14.5	1500	45
B29-12	117.1	17.4	900	45

Sample No.	Dry Density (pcf)	Moisture Content (%)	Unit Cohesion (psf)	Angle of Shear Resistance (degrees)
B34-1*	117.1	9.3	775	27
B35-3	111.5	17.4	880	41
B35-4	101.3	25.0	600	24
B37-1*	104.8	15.2	400	30
B43-1*	115.4	10.8	890	30
B43-2	108.8	20.0	760	30
B43-4	127.4	12.3	700	45
B44-1	99.3	24.5	650	37
B44-5	117.2	15.5	1400	40
B45-4	113.3	19.0	1500	30
B45-5*	101.1	17.4	1070	30
B50-2	101.5	24.4	1000	30
B50-6	127.2	11.8	1600	45
B50-7*	110.7	11.8	750	36
B51-2*	109.0	14.2	1050	23
B52-1	119.1	14.3	350	45
B55-2	110.9	17.3	590	34
B55-3*	101.9	15.9	.385	31
B55-4	115.4	20.2	790	44
B60-1**	83.5	38.8	300	14
B68-6	105.6	22.2	800	34
B68-10**	87.9	35.5	100	12
B75-1**	106.4	21.7	695	18
B75-3	124.1	13.0	1340	29

*Sample remolded to approximately 90 percent of maximum dry density at near optimum moisture content.
**Residual shear.

TABLE B-IV
SUMMARY OF LABORATORY POTENTIAL OF
HYDROGEN (PH) AND RESISTIVITY TEST RESULTS
CALIFORNIA TEST METHOD 643

Sample No.	pH	Resistivity (ohm centimeters)
B3-3	9.9	704
B55-5	8.3	484

TABLE B-V
SUMMARY OF LABORATORY PLASTICITY INDEX TEST RESULTS
ASTM D 4318-10

Sample No.	Description	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Unified Soil Classification (Group Symbol)
B7-7	Olive-tan, Silty CLAY with trace of sand	70	28	42	CH
B11-2	Brown, fine to medium, Sandy CLAY	78	22	56	CH

APPENDIX C

APPENDIX C

SLOPE STABILITY ANALYSES

FOR

FANITA RANCH
OFF-SITE IMPROVEMENT TO
CUYAMACA STREET
SANTEE, CALIFORNIA

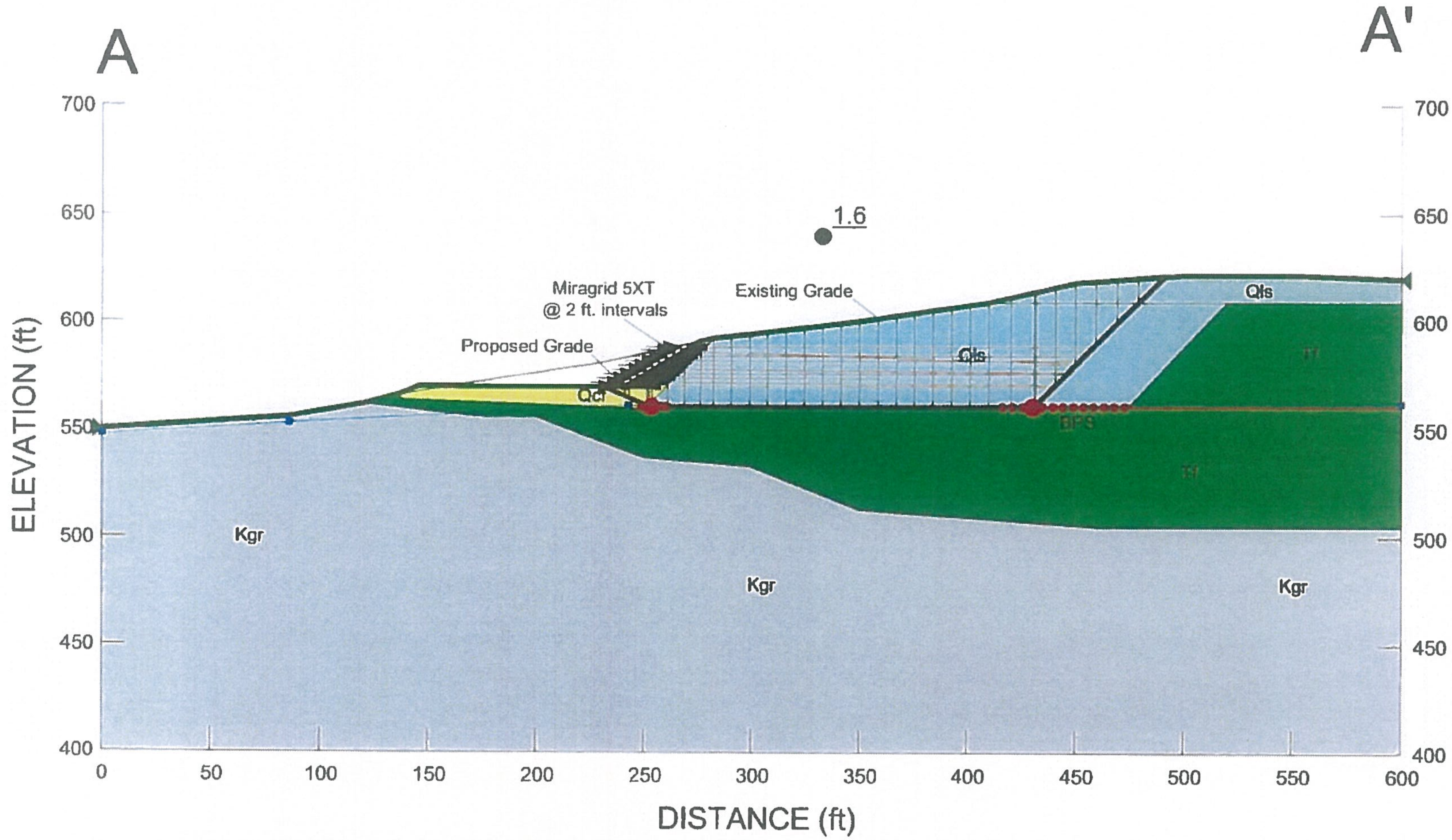
PROJECT NO. 05254-02-18A

Fanita Ranch - Cuyamaca Street
Project No. 05254-32-18
Section A-A'
Name: AA-Case1.gsz
Date: 8/10/2017 Time: 11:21:53 AM

Proposed 1.5:1 Geogrid Reinforced Fill Slope
and Landslide Mitigation

Static Condition

Name: Compacted Fill (Qcf) Unit Weight: 125 pcf Cohesion: 300 psf Phi: 35 °
Name: Landslide Debris (Qls) Unit Weight: 125 pcf Cohesion: 200 psf Phi: 20 °
Name: Friars Formation (Tf) Unit Weight: 130 pcf Cohesion: 500 psf Phi: 33 °
Name: Granitic Rock (Kgr) Unit Weight: 130 pcf Cohesion: 500 psf Phi: 35 °
Name: Bedding Plane Shear (BPS) Unit Weight: 120 pcf Cohesion: 150 psf Phi: 7 °



X:\Engineering and Geology\ENGINEER PROGRAMS, GUIDES, ETC\EngrgPrg\GEO-SLOPE2007\Projects\05254-32-18 Fanita - Cuyamaca Street\

Figure C-1

APPENDIX D

APPENDIX D

RECOMMENDED GRADING SPECIFICATIONS

FOR

FANITA RANCH
OFF-SITE IMPROVEMENT TO
CUYAMACA STREET
SANTEE, CALIFORNIA

PROJECT NO. 05254-02-18A

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

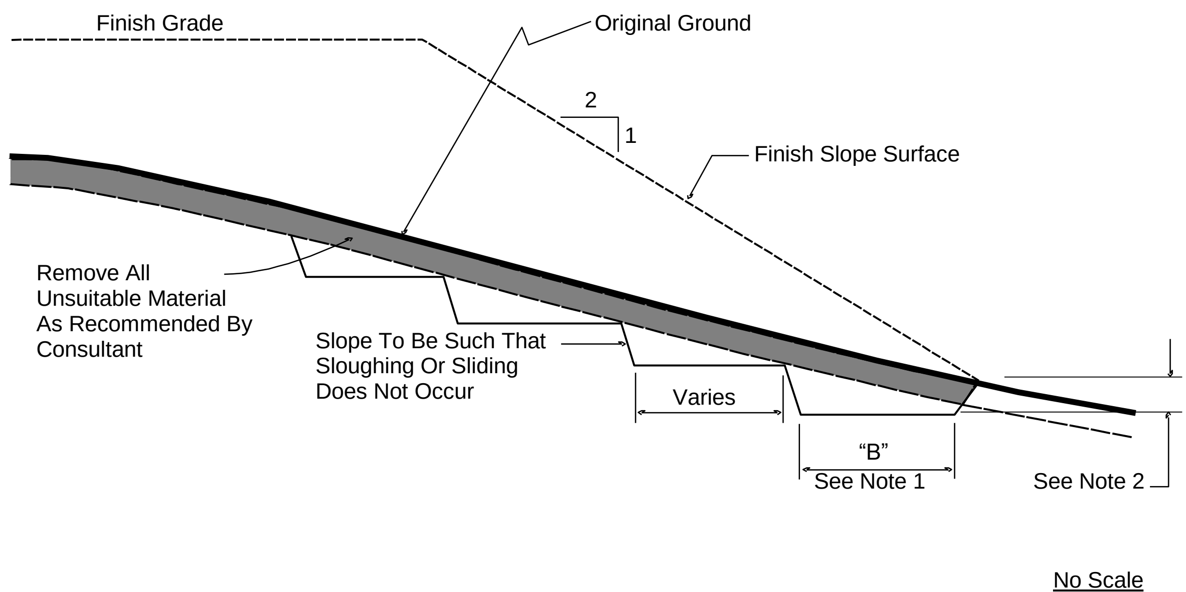
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material 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.

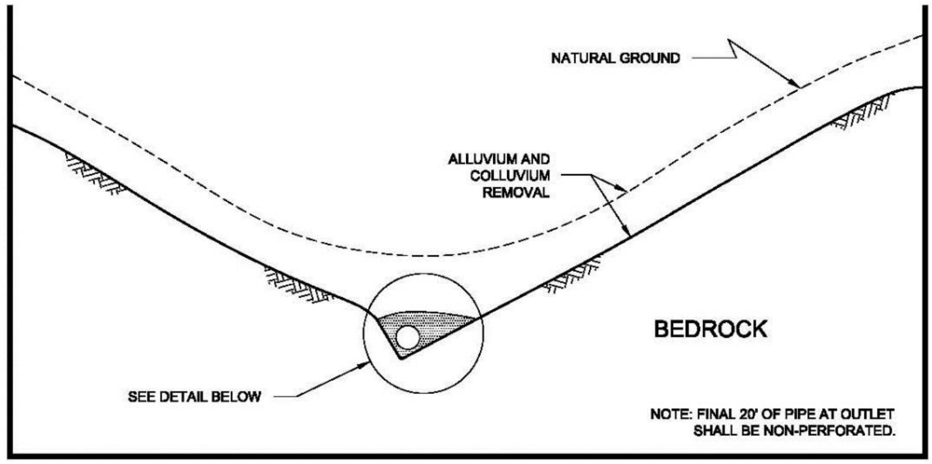
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant 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.

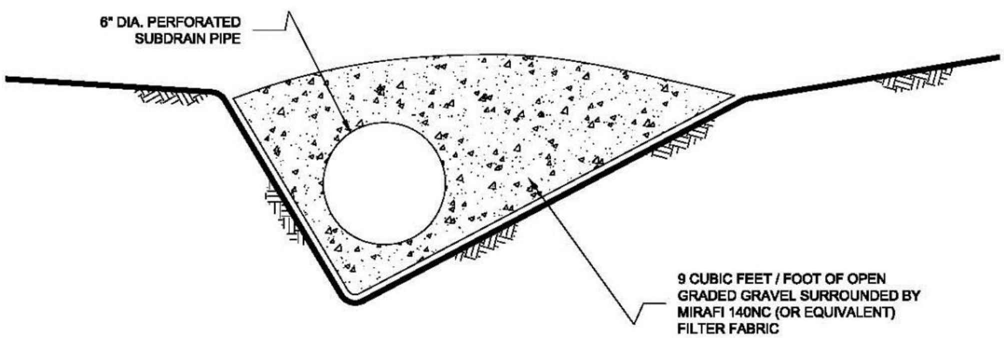
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

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





NOTES:

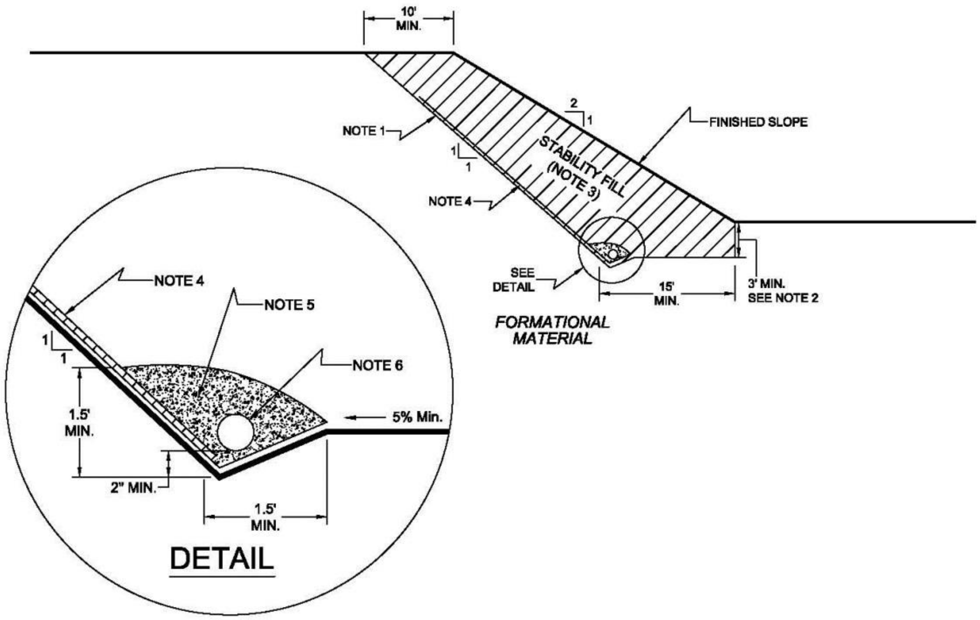
1......6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

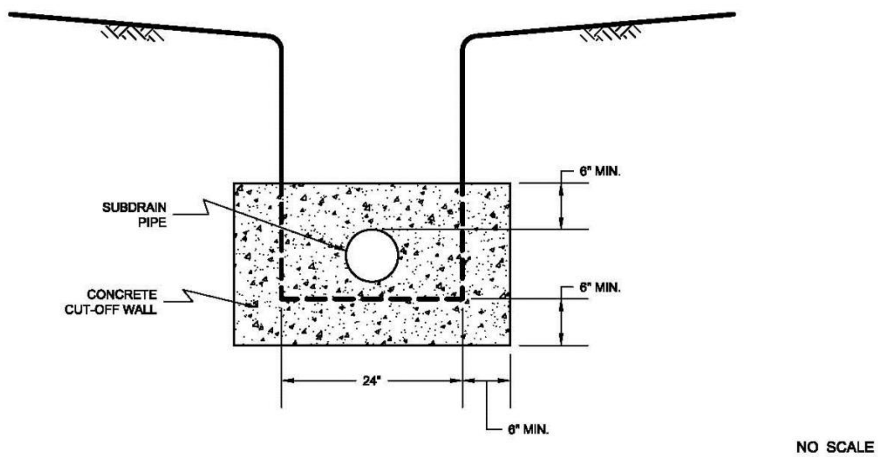
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

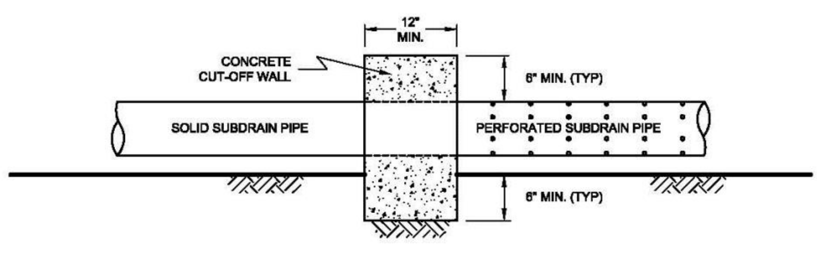
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW

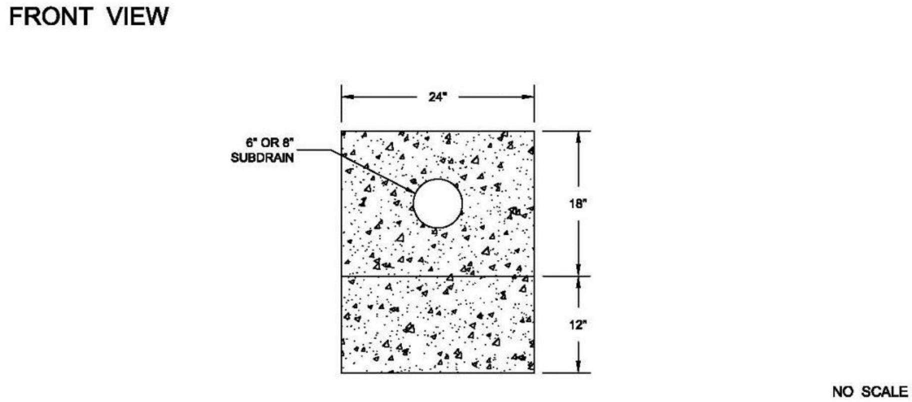


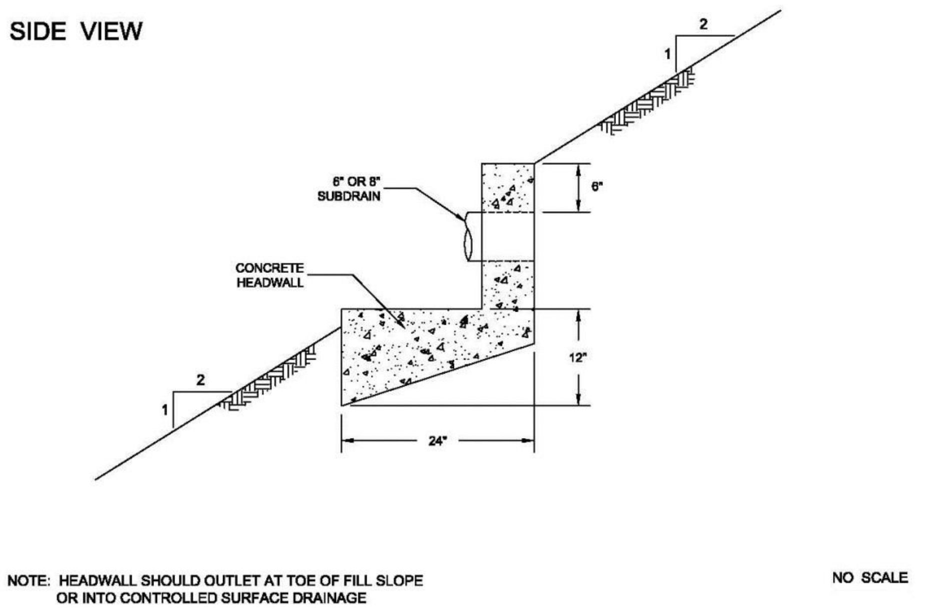
SIDE VIEW



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





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.

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