PRELIMINARY GEOTECHNICAL EVALUATION
PROPOSED RESIDENTIAL AND EQUESTRIAN DEVELOPMENT
ASSESSOR'S PARCEL NUMBER (APN) 264-110-30-00
RANCHO SANTA FE, SAN DIEGO COUNTY, CALIFORNIA 92067

FOR

MR. JOHN HONARVAR
1521 MOUNTAIN PASS CIRCLE
VISTA, CALIFORNIA 92081

W.O. 8543-A-SC APRIL 18, 2023

GeoSoils, Inc.

Geotechnical • Geologic • Coastal • Environmental

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April 18, 2023

W.O. 8543-A-SC

Mr. John Honarvar
1521 Mountain Pass Circle
Vista, California 92081

Subject: Preliminary Geotechnical Evaluation, Proposed Residential and Equestrian Development, Assessor's Parcel Number (APN) 264-110-30-00, Rancho Santa Fe, San Diego County, California 92067

Dear Mr. Honarvar:

In accordance with your request and authorization, GeoSoils, Inc. (GSI) is pleased to present the results of our preliminary geotechnical evaluation of the subject parcel, relative to the proposed residential and equestrian development thereon. The purpose of our study was to evaluate the onsite geologic and geotechnical conditions in order to develop preliminary conclusions regarding the technical feasibility of the proposed development and initial recommendations for earthwork and the design of foundations, concrete slab-on-grade floors, retaining walls, pavements/hardscape, and other improvements possibly associated with the project.

EXECUTIVE SUMMARY

Based upon our document reviews, field exploration, laboratory testing, and geologic, and geotechnical engineering analyses, the proposed residential and equestrian development at the subject parcel is considered technically feasible from both geotechnical and geologic viewpoints, provided that the recommendations presented in the text of this report are properly incorporated into the design and construction phases of the project. The most significant elements of our study are summarized below:

- All existing undocumented artificial fill, Quaternary-age colluvium (topsoil), and weathered portions of the Tertiary-age Friars Formation and Mesozoic-age metavolcanic bedrock (referred to hereinafter as "metavolcanics") are considered potentially compressible in their existing state and may settle appreciably under loads imparted by the proposed improvements and new planned fills. Unweathered Friars Formation and metavolcanics are considered suitable bearing materials within the subject property. Remedial earthwork is recommended to manage settlement.

- Laboratory testing indicates that the onsite soils exhibit very low to medium expansion potential (expansion index [E.I.] range of 0 to 90). Based on our past work experience on nearby sites, soils with high expansion potential (E.I. range of 91 to 130) may also exist locally. Earthwork or structural mitigation is recommended to temper distortions to the proposed improvements from shrinking and swelling soils. The unweathered metavolcanics are generally inert as it relates to expansion potential.

- We encountered non-productive excavation (i.e., practical refusal) in the unweathered metavolcanics, exposed in 3 of the 4 test pits using a rubber-tired backhoe equipped with tiger teeth on the digging bucket. Non-productive excavation was realized at depths ranging between 2½ and 5½ feet below the existing grades. Based on this finding, we anticipate that rock breaking equipment (i.e., hoe ram), a non-explosive demolition agent (i.e., Dexpan), or drill-and-shoot blasting may be necessary during the planned excavations and overexcavations (undercuts) extending into the unweathered metavolcanics.

- Excavations into the unweathered metavolcanics will produce an abundance of oversized rock fragments that will require special handling and placement techniques when incorporating into compacted fills.

- Infiltrating storm water into the onsite earth materials to meet permanent post-construction storm water best management (BMP) objectives is not recommended from a geotechnical standpoint.

- Adverse geologic structures that would preclude the proposed development were not encountered.

- Due to the site's location within a seismically active region, the proposed development is subject to moderate to strong ground shaking caused by earthquakes.

The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to contact our office.

Respectfully submitted,

GeoSoils, Inc.

John P. Franklin
Engineering Geologist, CEG 1340

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Geotechnical Engineer, GE 2057

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RBB/JPF/SJC/sh

Distribution: (1) Addressee (PDF via email)

TABLE OF CONTENTS

PRELIMINARY GEOTECHNICAL EVALUATION
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SCOPE OF SERVICES

The scope of our services has included the following:

1. Reviews of readily available published geologic literature and maps, soil data, and aerial photographs, and site-specific geotechnical reports previously prepared by another consultant (see Appendix A).

2. Site reconnaissance mapping and the excavation and geologic logging of four (4) exploratory test pits completed with a rubber-tired backhoe to evaluate the near-surface soil/geologic profiles and to sample the onsite earth materials (see Appendix B).

3. General areal geologic and seismic hazards evaluations (see Appendix C).

4. Appropriate laboratory testing of representative bulk and relatively undisturbed soil samples collected during our subsurface exploration program (see Appendix D).

5. An evaluation of storm water infiltration feasibility (see Appendix E).

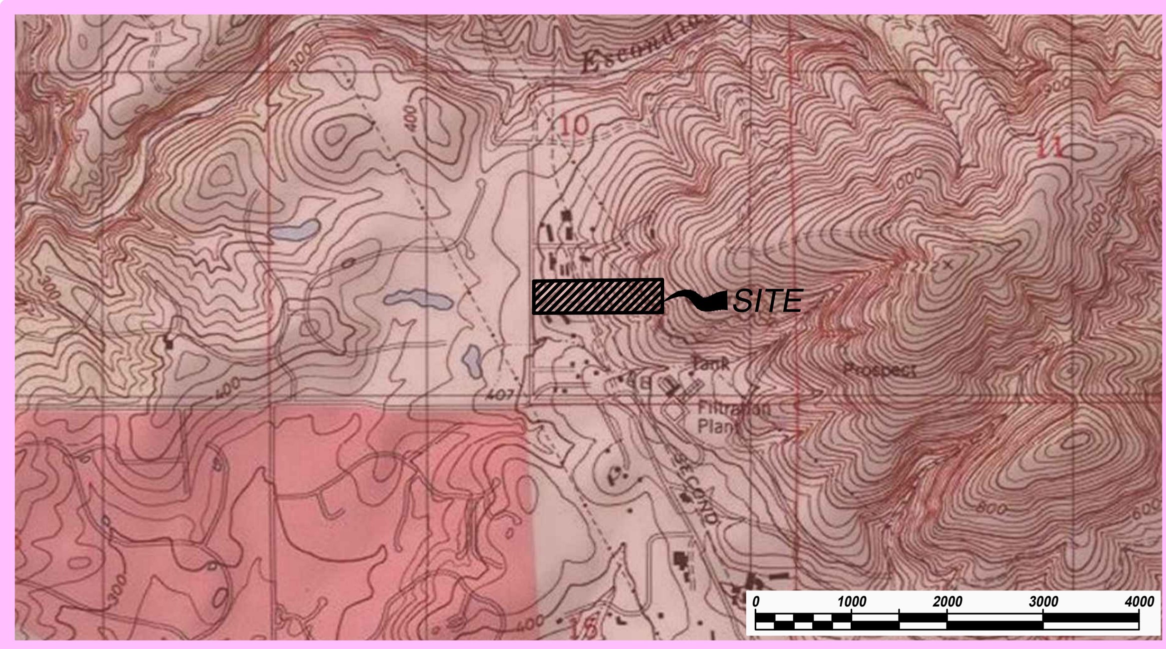
6. Engineering analysis of the field and laboratory data.

7. The preparation of this summary report and accompaniments.

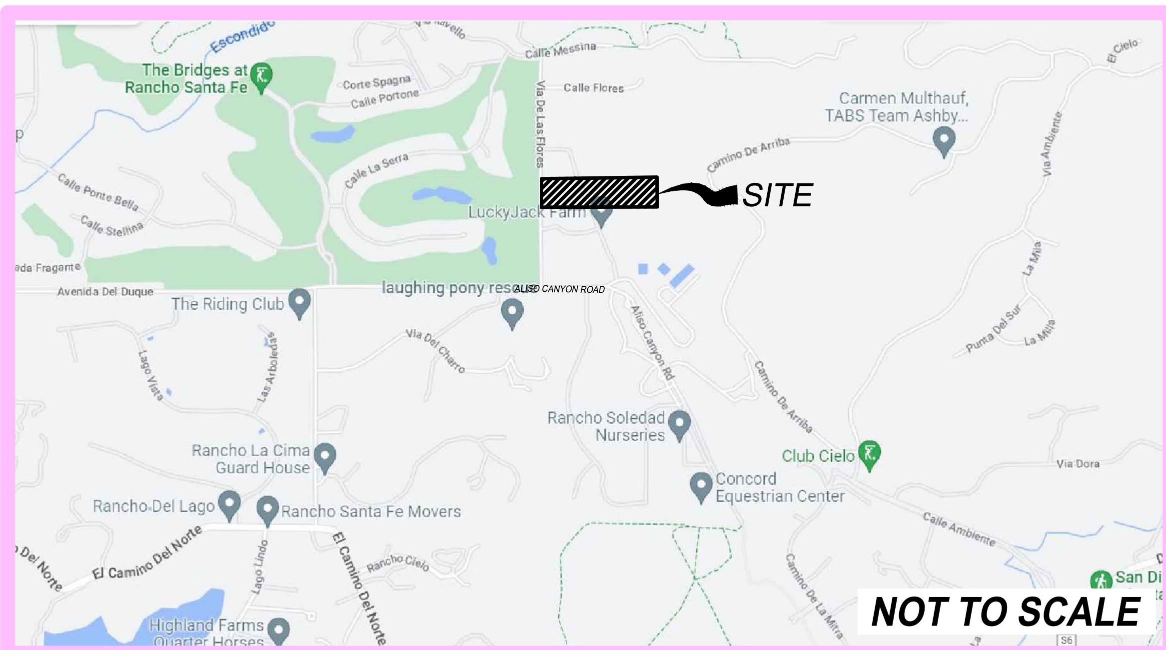
SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The subject site consists of a vacant, quadrilateral-shaped parcel of land located along the eastern side of Via De Las Flores Drive in Rancho Santa Fe, San Diego County, California 92067 (see Figure 1, Site Location Map). The latitude and longitude of the approximate geographic centroid of the parcel are 33.054195° North and -117.176944° West. The site is bounded by La Flores Drive to the west, by developed residential and equestrian properties to the north and south, and by open space to the remaining quadrant.

The subject site consists of relatively level to locally steep terrain within an area of rolling hills. According to the undated grading plans prepared by Snipes-Dye Associates ([SDA], undated), existing site elevations range between approximately 391 and 653 feet above National Geodetic Vertical Datum of 1929 (NGVD29), for an overall relief of approximately 262 feet. The property generally slopes toward the southwest and northwest at gradients on the order of 1.4:1 (horizontal:vertical [h:v]) or flatter. Surface drainage appears to be controlled by sheet flow runoff that follows the site topography toward the southwest and



Base Map: TOPO!® ©2003 National Geographic, U.S.G.S Rancho Santa Fe Quadrangle, California -- San Diego Co., 7.5 Minute, dated 1996, current, 2021.



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SITE LOCATION MAP

Figure 1

N

northwest. Utility easements maintained by the San Diego County Water Authority (SDCWA), Olivenhain Municipal Water District (OMWD), and San Diego Gas and Electric (SDG&E) traverse the parcel. Existing vegetation predominately consists of a mixture of grasses and scrub. A localized cluster of trees occurs in the southwestern quadrant of the parcel.

Based on our review of SDA (undated), GSI understands that the proposed development includes preparing the parcel to receive a new single-family residence and associated underground utilities and pavements (vehicular and pedestrian). An equestrian paddock is proposed west of the SDCWA easement. SDA (undated) indicates that cut and fill grading will be necessary to achieve the proposed graded configuration, with maximum planned cuts and fills on the order of 26 feet and 19 feet, respectively. Grade transitions will primarily be accommodated by the construction of cut and fill slopes with maximum heights on the order of 52 feet and 35 feet, respectively. SDA (undated) shows that the gradients of the planned cut and fill slopes will generally be 2:1 (h:v); however, the proposed cut slope along the eastern margin of the project and a portion of the proposed northwest-facing fill slope that ascends to the planned driveway is currently intended to be constructed at a 1.5:1 (h:v) gradient. GSI anticipates that the proposed residential structure will be supported by shallow foundations with a concrete slab-on-grade floor. Building loads are currently unavailable but assumed typical of similar, light residential development.

PREVIOUS GEOTECHNICAL STUDIES

Based on our review of an existing geotechnical report provided by the Client, GSI understands that the subject parcel was previously investigated by Coast Geotechnical (CG) in early 2018 (CG, 2018) with respect to previous uncontrolled grading along the western portion of the property. The study included reviews of regional geologic maps and the former site plan; the excavation and geologic logging of three (3) exploratory test pits completed with a rubber-tired backhoe; laboratory testing of collected soil samples; data analysis; and the preparation of CG (2018).

CG identified undocumented artificial fill at the surface in all their test pits. The thickness of the fill varied between approximately 7 and 10 feet. CG encountered topsoil with thicknesses ranging between approximately 10 and 36 inches underlying the undocumented artificial fill. Sedimentary bedrock, belonging to the Tertiary Friars Formation, was encountered beneath the topsoil near the western margin of the site. CG encountered Mesozoic-age metavolcanics underlying the topsoil further to the east. CG concluded that the expansion potential of the evaluated earth materials was in the medium to high range.

CG (2018) included a discussion of geologic and seismic hazards that could potentially impact the subject parcel. Other than the potential for the site to experience strong ground shaking from the design earthquake occurring along the nearby Rose Canyon fault, CG judged that the susceptibility of the site to be affected by surface rupture from fault displacement, landslides, liquefaction, and flooding from tsunami was low.

CG (2018) concluded that the artificial fill was placed inappropriately for use in an engineering application and indicated that remedial grading (i.e., the removal and recompaction of the fill materials with adequate keying and benching) would be required to develop the site. They further surmised that although no geologic hazards were found that would preclude the formerly proposed development, the costs for remedial earthwork may present an economic concern.

A copy of CG (2018) is provided in Appendix A.

RECENT FIELD STUDIES

GSI evaluated the surficial and subsurface conditions within the subject parcel in late February 2023. The field exploration consisted of reconnaissance geologic mapping and the excavation and geologic logging of four (4) exploratory test pits completed with a rubber-tired backhoe. The test pits were logged by a representative of this firm, who also collected representative bulk and relatively undisturbed soil samples for appropriate laboratory testing. The logs of the test pits are presented in Appendix B. Site geology and the location of the recent test pits and the previous trenches by CG (2018) are shown in plan view on the Geotechnical Map (see Plates 1 and 2), which uses SDA (undated) as a base. Site geology in proximity to the proposed residential structure is also shown in profile view on Plate 3 (Geologic Cross Section A-A').

PHYSIOGRAPHIC AND REGIONAL GEOLOGIC SETTINGS

Physiographic Setting

The site is located near the boundary between the coastal plain and central mountain-valley physiographic sections of San Diego County. The coastal plain section is characterized by pronounced marine wave-cut terraces intermittently dissected by stream channels that convey water from the eastern highlands to the Pacific Ocean. The central mountain-valley section consists of ridges and intermontane basins, with the basins and valleys ranging between 500 and 5,000 feet in elevation.

Regional Geologic Setting

San Diego County lies within the Peninsular Ranges Geomorphic Province of Southern California. This province is characterized by elongated mountain ranges and valleys that trend northwesterly (Norris and Webb, 1990). This geomorphic province extends from the base of the east-west aligned Santa Monica - San Gabriel Mountains, and continues south into Baja California, Mexico. The mountain ranges within this province are underlain by basement rocks consisting of pre-Cretaceous metasedimentary rocks, Jurassic metavolcanic rocks, and Cretaceous plutonic (granitic) rocks.

The San Diego County region was originally a broad area composed of pre-batholithic rocks that were subsequently subjected to tectonism and metamorphism. In the late Cretaceous Period, the southern California Batholith was emplaced causing the aforementioned metamorphism of pre-batholithic rocks. Many separate magmatic injections originating from this body occurred along zones of structural weakness.

Following batholith emplacement, uplift occurred, resulting in the removal of the overlying rocks by erosion. Erosion continued until the area was that of low relief and highly weathered. The eroded materials were deposited along the sea margins. Sedimentation also occurred during the late Cretaceous Period. However, subsequent erosion has removed much of this evidence. In the early Tertiary Period, terrestrial sedimentation occurred on a low-relief land surface. In Eocene time, previously fluctuating sea levels stabilized and marine deposition occurred. In the late Eocene, regional uplift produced erosion and thick deposition of terrestrial sediments. In the middle Miocene, the submergence of the Los Angeles Basin resulted in the deposition of thick marine beds in the northwestern portion of San Diego County. During the Pliocene, marine sedimentation was more discontinuous and generally occurred within shallow marine embayments. The Pleistocene saw regressive and transgressive sea levels that fluctuated with prograding and recessive glaciation. The changes in sea level had a significant effect on coastal topography and resultant wave erosion, and deposition formed many terraces along the coastal plain. In the mid-Pleistocene, regional faulting separated highland erosional surfaces into major blocks lying at varying elevations. A later rise in sea level, during the late Pleistocene, caused the deposition of thick alluvial deposits within the coastal river channels. In recent geologic time, crystalline rocks have weathered to form soil residuum, highland areas have eroded, and the deposition of river, lake, lagoonal, and beach sediments has occurred.

Regional geologic mapping by Kennedy and Tan (2007) indicates that the site is underlain by undivided, Mesozoic-age metamorphosed and unmetamorphosed volcanic and sedimentary bedrock. This unit includes basaltic andesite, andesite, dacite, rhyolite, volcaniclastic breccia, welded tuff, and epiclastic rocks.

SITE GEOLOGIC UNITS

General

The earth material units encountered within the subject parcel during the CG (2018) investigation and our recent field studies consisted of undocumented artificial fill, Quaternary-age colluvium (topsoil), sedimentary bedrock belonging to the Tertiary-age Friars Formation, and Mesozoic-age metavolcanics. A general description of each earth material type is presented as follows, from youngest to oldest. The general distribution of these units across the site is presented on Plate 1.

Undocumented Artificial Fill (Map Symbol - Afu)

Undocumented artificial fill was encountered at the surface in all of the test pits excavated in preparation of CG (2018). It was also encountered in our Test Pits TP-101 and TP-104. CG (2018) described the undocumented fill exposed in their test pits as tan, fine- to medium-grained sand with pebbles and dark-brown silty fine-grained sandy clay and clayey sand with gravel-, cobble-, and boulder-sized clasts and fragments of concrete, asphaltic concrete, piping, bricks, wood, wire mesh, and rope that extended to depths ranging between approximately 7 and 10 feet below the existing grades. The undocumented fill in our Test Pits TP-101 and TP-104 consisted of variegated light gray and dark-brownish-gray silty sand with traces of clay and angular and subangular gravels and cobbles; variegated brown and light gray sandy clay with a trace of angular and subangular gravels and cobbles; and brown sandy clay with a trace of angular and subangular gravels and cobbles that extended to depths on the order of ½-foot to 2½ feet below the existing grades. The undocumented fill is considered potentially compressible in its existing state. It was also observed overlying colluvium (see below) in the CG (2018) test pits and our Test Pit TP-101. The colluvium is also considered prone to settlement under load.

Quaternary Colluvium (Not Mapped)

Quaternary colluvium (topsoil) was encountered directly beneath the undocumented fill in the CG (2018) test pits and our Test Pit TP-101. It was also observed at the surface in our Test Pits TP-102 and directly below a thin compost layer in our Test Pit TP-103. CG (2018) described the colluvium as an approximately 10- to 36-inch thick section of black, fine- to coarse-grained silty clayey sand that graded into a reddish-brown clay. However, the reddish-brown clay CG (2018) identified as topsoil is believed to be a weathering horizon developed upon the metavolcanics.

The colluvium observed in our test pits consisted dark-grayish-brown, wet to saturated, and loose clayey sand with localized traces of angular and subangular gravels and cobbles. The thickness of the colluvium encountered in our test pits varied between approximately ½ foot and 1¼ feet. As previously stated, the colluvium is considered potentially compressible in its existing state and may settle appreciably under improvement and fill loads.

Tertiary Friars Formation (Map Symbol - Tf)

CG (2018) encountered the Friars Formation in their Test Pit TP-1 at an approximate depth of 10 feet below the existing grade. They described these sediments as gray and brown clay and pale tan-green silty clay that was damp to moist and stiff. GSI did not encounter the Friars Formation during our subsurface exploration. However, based on our past experience with the Friars Formation, it is generally suitable for supporting engineered improvements and fills where weathering is absent. It does often contain expansive soils and can be susceptible to landslides where shearing occurs along bedding planes of weak claystone and siltstone.

Mesozoic Undifferentiated Metavolcanics (Map Symbol - Mzu)

Metavolcanics were encountered beneath the surficial earth materials in the CG (2018) Test Pits TP-2 and TP-3 at depths on the order of 8 to 13 feet below the existing grades. CG (2018) characterizes the metavolcanics as tan-brown and gray silty sand and fine-grained sand that are slightly moist (damp) and dense.

Metavolcanics were observed in all of our test pits at depths of approximately ½-foot to 3¾ feet below the existing grades. However, the upper approximately 1¼ to 3 feet of the metavolcanics in our Test Pits TP-101 through TP-103 exhibited varying degrees of weathering, and consisted of brown, reddish-brown, and olive-brown sandy clay and light-reddish-brown clayey sand with localized traces of angular and subangular gravels. The weathered metavolcanics were generally moist to wet and stiff/medium dense.

The unweathered metavolcanics in our Test Pits TP-102 through TP-104 occurred at depths of approximately ½-foot to 4 feet below the existing grades. The metavolcanics varied between dark-gray and olive-brown, fractured, altered, aphanitic volcanic rock and gray most massive, altered aphanitic volcanic rock. The metavolcanics in these test pits disintegrated to angular and subangular gravels, cobbles, and boulders upon excavation. The boulder-sized clasts of the excavated metavolcanics were up to approximately 3½ feet in the longest dimension.

The unweathered metavolcanics encountered in our Test Pit TP-101 occurred at a depth of approximately 5¼ feet below the existing grade and consisted of angular and subangular gravels in a slightly lithified matrix of sandy clay (diamictite). This subunit may represent the distal extent of an ancient lahar or debris avalanche.

STRUCTURAL GEOLOGY

Kennedy and Tan (2007) show metavolcanic foliation inclined 85 degrees to the northwest. The Friars Formation is generally gently inclined to the west-northwest; however, it is not anticipated to be exposed during grading. We observed joints within the metavolcanics oriented between N 75° E and N 77° W. The joints generally exhibited moderate to vertical dip planes. However, we recorded a joint oriented N 41° E and inclined 34 degrees to the northwest in the southwestern quadrant of the property. A regional joint trend was not identified.

Based on our review of Kennedy and Tan (2007) and our observations, the onsite structural geology is generally not considered adverse with respect to the proposed development shown on SDA (undated). However, intersecting joint planes have the potential to result in wedge failures in planned and temporary excavations. Thus, all excavations should be observed by the geotechnical consultant.

ONSITE SOILS AND STORM WATER INFILTRATION FEASIBILITY

According to the United States Department of Agriculture/ Natural Resources Conservation Service's (USDA/NRCS's) "Web Soil Survey" website (http://websoilsurvey.sc.egov.usda.gov), the onsite soils consist of the Huerhuero loam, 2 to 9 percent slopes and the San Miguel - Exchequer rocky silt loams, 9 to 70 percent slopes. Mapping by the USDA/NRCS shows the Huerhuero loam, 2 to 9 percent slopes near the western margin of the parcel and the San Miguel - Exchequer rocky silt loams, 9 to 70 percent slopes throughout the remainder of the property. The attributes of these soils are summarized in the following table:

SOIL TYPE (MAP UNIT)	LANDFORM SETTING	DRAINAGE CLASS	RUNOFF CLASS	CAPACITY OF THE MOST LIMITING LAYER TO TRANSMIT WATER (Ksat)	HYDROLOGIC SOIL GROUP
Huerhuero loam, 2 to 9 percent slopes (Map symbol - HrC)	Marine terraces	Moderately well drained	Very high	Very low to moderately low (0.00 to 0.06 inches per hour)	D
San Miguel - Exchequer rocky silt loams, 9 to 70 percent slopes	Mountain slopes	Well drained	Very high	Very low to moderately low (0.00 to 0.06 inches per hour)	D

While the USDA/NRCS soil data suggest that infiltration of storm water into the onsite soils to meet permanent post-construction storm water BMP objectives may be feasible, albeit at a relatively low rate, it is our opinion that infiltration of storm water into the onsite earth materials, in any volume, has a high potential to result in perched groundwater accumulating upon the Friars Formation, metavolcanics, and clay lifts within the planned and remedial fills. The perched groundwater seepage we observed in our test pits, emanating above the weathered metavolcanics and from joints in the metavolcanics (see "Groundwater" section below), and the HSG "D" designation assigned to the onsite soils by the USDA/NRCS support this conclusion.

Lateral migration of perched groundwater could induce settlement of low density soils and activate swelling of expansive soils within the subject parcel and the adjacent properties. Both have the potential to adversely affect the proposed onsite improvements and the existing development on the adjacent properties and the Via De Las Flores right-of-way. Thus, the infiltration of storm water into the onsite soils is <u>not</u> considered sound engineering practice and is not recommended, from a geotechnical perspective. The completed Table B.2-1 of the County of San Diego's "BMP Design Manual" (County of San Diego, 2020) is included in Appendix E.

GROUNDWATER

Perched groundwater was observed seeping from the sidewalls of Test Pits TP-101, TP-102, and TP-103. The seeps were emanating above the weathered metavolcanics in Test Pits TP-101 and 103 and from joints within the metavolcanics in Test Pit TP-102. The source of the perched groundwater is believed to be infiltrated precipitation from rain events that preceded our field exploration. We estimate that the regional groundwater table is most likely within a few feet of sea level or greater than 500 feet below the lowest existing site elevation. The local groundwater conditions are subject to change due to meteorological or climatic factors, excessive irrigation, or other circumstances that were not obvious, at the time of our site exploration.

Based on the available subsurface data and our understanding of the proposed site development, groundwater is not anticipated to be a significant geotechnical factor, provided that the recommendations contained in this report are properly incorporated into final design and construction. Perched groundwater seepage from infiltrated precipitation and irrigation waters may be encountered in planned and remedial grading excavations and along the planned cut slopes. This may require the use of submersible dewatering pumps within excavations and subdrain systems along the toes of the planned cut slopes (i.e., toe drains).

Due to the potential for post-development perched groundwater to manifest near the surface, owing to as-graded permeability/density contrasts, more rigorous concrete slab-on-grade floor design is recommended (State of California, 2023). Recommendations for reducing the amount of moisture or water vapor through concrete slab-on-grade floors are provided in the "Soil Moisture Transmission Considerations" section of this report.

The perched groundwater seepage encountered in Test Pit TP-101 resulted in caving to an approximate depth of 2 feet below the existing grade. Thus, caving of excavation sidewalls should be anticipated if perched groundwater is present.

BEDROCK HARDNESS AND RIPPABILITY/TRENCHABILITY

We encountered non-productive excavation (i.e., practical refusal) within the unweathered metavolcanics exposed in Test Pits TP-102, TP-103, and TP-104 using a John Deere 410G rubber-tired backhoe equipped with tiger teeth on a 24-inch wide bucket. Non-productive excavation occurred at approximate depths of 5½ feet, 4½ feet, and 2½ feet in Test Pits TP-102, TP-103, and TP-104, respectively. Thus, excavations extending below these depths, using similar trenching equipment, will likely require the use of hydraulic breakers (i.e., hoe rams), larger excavation or ripping equipment, non-explosive demolition agents, or possibly drill-and-shoot blasting techniques. Excavation productivity within the metavolcanics will be controlled by joint spacing and the degree of decomposition.

Excavation equipment should be in good-working condition and be appropriately sized and powered for the required excavation task. Reduced excavation productivity may be realized if smaller equipment, such as rubber-tired backhoes and mini-excavators, are used. If additional information regarding the excavation characteristics of the onsite earth materials is needed, this office can perform seismic refraction studies.

GEOLOGIC HAZARDS EVALUATION

GSI has evaluated the site relative to geologic and seismic hazards that could affect the proposed development. These hazards and their potential effects on the proposed development are summarized below.

Mass Wasting/Landslide Susceptibility

Mass wasting refers to the various processes by which earth materials are moved down slope in response to the force of gravity. Examples of these processes include slope creep, surficial failures, and deep-seated landslides. Creep is the slowest form of mass wasting and generally involves the outer 5 to 10 feet of a slope surface. During heavy rains, such as those in El Niño years, creep-affected materials may become saturated, resulting in a more rapid form of downslope movement (i.e., landslides or surficial failures).

According to regional landslide susceptibility mapping by Tan and Giffen (1995), the portion of the site located above approximate elevations 600 to 620 feet above Mean Sea Level (MSL) is located within Relative Landslide Susceptibility Subarea 3-2. The remainder of the parcel is located within Relative Landslide Susceptibility Subarea 3-1. Tan and Giffen (1995) report that slopes within Subarea 3-1 are at or close to their stability limits due to a combination of the presence of weak earth materials and steep topography. Although most slopes within Subarea 3-1 do not contain landslides, they are locally susceptible to failure when adversely modified. Subarea 3-2 includes steeper and higher slopes which have greater predisposition to landslides and other slope failures.

Our review of Kennedy and Tan (2007), Tan and Giffen (1995), and the California Department of Conservation, California Geological Survey's "Landslide Inventory" website application (https://maps.conservation.ca.gov/cgs/lsi/app/) did not reveal the presence of landslides within the subject property. In addition, we did not observe evidence of landslides within the parcel during our field investigation. Moreover, geomorphic features, indicative of past mass wasting events (i.e., scarps, hummocky terrain, debris cones, arcuate drainage patterns, etc.), were not identified during our review of stereoscopic aerial photographs (Park Aerial Surveys, Inc., 1953) nor during our field evaluation. Lastly, much of the steep topography at the site is underlain in the near-surface by metavolcanics with high shear strength. Based on the above, GSI concludes that the potential for the proposed development to be adversely affected by deep-seated slope instability is relatively low.

The onsite soils are, however, considered erodible, as evidenced by gully erosion in the previously graded areas. Properly designed and regularly maintained surface drainage is recommended to mitigate erosion and improve surficial slope stability.

Faulting and Regional Seismicity

Regional Faults

Our review indicates that there are no known Holocene-active faults (i.e., faults that have ruptured in the last 11,700 years) crossing the subject property (Jennings and Bryant, 2010), and the site is not located within an Alquist-Priolo Earthquake Fault Zone (California Department of Conservation, California Geological Survey [CGS], 2018). However, the site is situated in a region subject to periodic earthquakes along Holocene-active faults. According to Blake (2000a), the Rose Canyon fault (part of the Newport-Inglewood - Rose Canyon fault zone [NIRCFZ]) is the closest known Holocene-active fault to the site, located at a distance of approximately 9.1 miles (14.7 kilometers) to the southwest. This fault should have the greatest effect on the site in the form of strong ground shaking, should the design earthquake occur. Cao, et al. (2003) indicate the slip rate on the Rose Canyon fault is 1.5 ($\pm$0.5) millimeters per year (mm/yr) and the fault is capable of a maximum magnitude 7.2 earthquake. The location of the Rose Canyon fault and other major faults within 100 kilometers of the site are shown on the "California Fault Map" in Appendix C. The possibility of ground acceleration, or shaking at the site, may be considered as approximately similar to the southern California region as a whole.

Local Faulting

A review of available regional geologic maps (Kennedy and Tan, 2007; Jennings and Bryant, 2010, and CGS, 2018) does not indicate the presence of faults, Holocene-active or otherwise, crossing the subject parcel.

Surface Rupture

Owing to the lack of known Holocene-active faults crossing the site, the potential for the proposed development to be adversely affected by surface rupture from fault displacement is considered low.

Seismicity

The acceleration-attenuation relation of Bozorgnia, Campbell, and Niazi (1999) has been incorporated into EQFAULT (Blake, 2000a). EQFAULT is a computer program developed by Thomas F. Blake (2000a), which performs deterministic seismic hazard analyses using digitized California faults as earthquake sources.

The program estimates the closest distance between each fault and a given site. If a fault is found to be within a user-selected radius, the program estimates peak horizontal ground acceleration that may occur at the site from an upper bound (formerly “maximum credible earthquake”), on that fault. Upper bound refers to the maximum expected ground acceleration produced from a given fault. Site acceleration (g) was computed by one user-selected acceleration-attenuation relation that is contained in EQFAULT. Based on the EQFAULT program, a peak horizontal ground acceleration at the site from an upper bound event on the Rose Canyon fault may be on the order of 0.41 g. The computer printouts of pertinent portions of the EQFAULT program are included within Appendix C.

Historical site seismicity was evaluated with the acceleration-attenuation relation of Bozorgnia, Campbell, and Niazi (1999), and the computer program EQSEARCH (Blake, 2000b, updated to May 8, 2021). This program performs a search of the historical earthquake records for magnitude 5.0 to 9.0 seismic events within a 100-kilometer radius, between the years 1800 through May 8, 2021. Based on the selected acceleration-attenuation relationship, a peak horizontal ground acceleration is estimated, which may have affected the site during the specific time frame. Based on the available data and the attenuation relationship used, the estimated maximum (peak) site acceleration during the period 1800 through May 8, 2021 was about 0.30 g. A historic earthquake epicenter map and a seismic recurrence curve was also estimated/generated from the historical data. Computer printouts of the EQSEARCH program are presented in Appendix C.

Seismic Shaking Parameters

The following table summarizes the site-specific seismic design criteria obtained from the 2022 California Building Code (CBC), Chapter 16 Structural Design, Section 1613, Earthquake Loads (California Building Standards Commission [CBSC], 2022) and American Society of Civil Engineers (ASCE 7-16 [ASCE, 2017]). The short spectral response uses a period of 0.2 seconds. Based on the findings from our onsite subsurface exploration and our experience with other similar sites, it is our opinion that Site Class “C” conditions are applicable to the proposed development.

2022 CBC SEISMIC DESIGN PARAMETERS		
PARAMETER	MAPPED VALUE PER ASCE 7-16	2022 CBC or REFERENCE
Risk Category*	I, II, or III	Table 1604.5
Site Class	C	Section 1613.2.2/Chap. 20 ASCE 7-16 (p. 203-204)
Spectral Response - (0.2 sec), S_s	0.888 g	Section 1613.2.1 Figure 1613.2.1(1)
Spectral Response - (1 sec), S_1	0.326 g	Section 1613.2.1 Figure 1613.2.1(2)
Site Coefficient, F_a	1.2	Table 1613.2.3(1)
Site Coefficient, F_v	1.5	Table 1613.2.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec), S_{MS}	1.065 g	Section 1613.2.3 (Eqn 16-36)
Maximum Considered Earthquake Spectral Response Acceleration (1 sec),S_{M1}	0.489 g	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	0.710 g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.326 g	Section 1613.2.4 (Eqn 16-39)
PGA_M - Probabilistic Vertical Ground Acceleration may be assumed as about 50% of this value.	0.462 g	ASCE 7-16 (Eqn 11.8.1)
Seismic Design Category	D	Section 1613.2.5/ASCE 7-16 (p. 85: Table 11.6-1 or 11.6-2)
* - Risk Category to be confirmed by the project architect or structural engineer.		

GENERAL SEISMIC PARAMETERS	
PARAMETER	VALUE
Distance to Seismic Source (Rose Canyon fault)	9.1 mi (14.7 km)[(1)]
Upper Bound Earthquake (Rose Canyon fault)	M_w = 7.2[(2)]
[(1)] - Blake (2000a) [(2)] - Cao, et al. (2003)	

Conformance to the criteria above 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 eliminate all damage, since such design may be economically prohibitive. Cumulative effects of seismic events are not addressed in the 2022 CBC (CBSC, 2022) and regular maintenance and repair following locally significant seismic events (i.e., M_w5.5) will likely be necessary, as is the case in all of southern California.

Secondary Seismic Hazards

Liquefaction

Liquefaction describes a phenomenon in which cyclic stresses, produced by earthquake-induced ground motion, create excess pore pressures in relatively cohesionless soils. These soils may thereby acquire a high degree of mobility, which can lead to vertical deformation, lateral movement, lurching, sliding, and as a result of seismic loading, volumetric strain and manifestation in surface settlement of loose sediments, sand boils and other damaging deformations. This phenomenon occurs only below the water table, but after liquefaction has developed, it can propagate upward into overlying non-saturated soil as excess pore water dissipates.

One of the primary factors controlling the potential for liquefaction is the depth to groundwater. Typically, liquefaction has a relatively low potential at depths greater than 50 feet and is unlikely or will produce vertical strains well below 1 percent for depths greater than 60 feet, when relative densities are 40 to 60 percent and effective overburden pressures are two or more atmospheres (i.e., 4,232 pounds per square foot [Seed, 2005]).

The condition of liquefaction has two principal effects. One is the consolidation of loose sediments with resultant settlement of the ground surface. The other effect is lateral sliding. Significant permanent lateral movement generally occurs only when there is significant differential loading, such as fill or natural ground slopes within susceptible materials. No such loading conditions exist at the site.

Liquefaction susceptibility is related to numerous factors and the following five conditions should be concurrently present for liquefaction to occur: 1) sediments must be relatively young in age and not have developed a large amount of cementation; 2) sediments must generally consist of fine- to medium-grained, relatively cohesionless sands; 3) the sediments must have low relative density; 4) free groundwater must be present in the sediment; and 5) the site must experience a seismic event of a sufficient duration and magnitude, to induce straining of soil particles. Only about one to perhaps two of these five necessary conditions have the potential to affect the site, concurrently.

It is the opinion of GSI that the susceptibility of the site to experience damaging deformations from seismically-induced liquefaction is relatively low owing to the dense, nature of the Friars Formation and metavolcanics that underlie the site in the near-surface and the depth to the regional groundwater. In addition, the recommendations for remedial earthwork and foundations would further reduce any significant liquefaction potential.

Other Geologic/Secondary Seismic Hazards

The following list includes other geologic-/seismic-related hazards that have been considered during our evaluation of the site. The hazards listed are considered negligible or mitigated as a result of site location, soil characteristics, and typical site development procedures:

- Hydroconsolidation/Hydrocollapse
- Subsidence due to fluid withdrawal
- Coseismic deformation (Ground lurching or shallow ground rupture)
- Tsunami
- Seiche

LABORATORY TESTING

Laboratory tests were performed on relatively undisturbed and representative bulk samples of the site earth materials collected during our subsurface exploration in order to evaluate their physical characteristics. The test procedures used and the results obtained are presented below.

Classification

Soils were visually classified with respect to the Unified Soil Classification System (U.S.C.S.) developed by Sowers and Sowers (1979). Visual classification was conducted in general accordance with ASTM D 2487 and D 2488. The soil classifications of the onsite earth materials, encountered in our test pits, are included on the Test Pit Logs in Appendix B.

Moisture Content

The field moisture contents were evaluated for representative bulk samples of the onsite earth materials in the laboratory. Testing was performed in general accordance with ASTM D 2216-19. The field moisture content was reported as a percentage of the dry weight. The results of these tests are shown on the Test Pit Logs in Appendix B.

Moisture-Density Relations

The field moisture content and dry unit weight were evaluated for relatively undisturbed samples of the onsite earth materials in the laboratory. Testing was performed in general accordance with ASTM D 2937 and ASTM D 2216. The dry unit weight was reported in pounds per cubic foot (pcf), and the field moisture content was reported as a percentage of the dry weight. The results of these tests are shown on the Test Pit Logs in Appendix B.

Laboratory Standard

The maximum density and optimum moisture content was evaluated for a bulk sample of the undocumented artificial fill collected from Test Pit TP-101. Testing was performed in general accordance with ASTM D 1557-12 Method “B”. The moisture-density relationship obtained for this soil sample is shown in the following table:

SAMPLE LOCATION AND DEPTH (FT)	MAXIMUM DENSITY (PCF)	OPTIMUM MOISTURE CONTENT (%)
TP-101 @ ¾-2½	121.2	10.7
TP-101 @ ¾-2½	124.9*	9.6*
* - Corrected for 11.3 percent retained on the ⅜-inch sieve (rock correction)		

Atterberg Limits

Atterberg limits testing was performed on a composite sample of the weathered metavolcanics, obtained from Test Pits TP-1 and TP-3, to evaluate the sample’s liquid limit (L.L.), plastic limit (P.L.), and plasticity index (P.I.). The Atterberg limits test was performed in general accordance with ASTM D 4318. The test results are presented in the following table and in Appendix D. Testing indicates that the sample is subject to plastic deformation. The Unified Soils Classification System (USCS) symbol for the sample is CH.

SAMPLE LOCATION AND DEPTH (FT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
TP-101 @ 3¾-4½ & TP-103 @ 1½-2	52	16	36

Particle-Size Analysis

Particle-size analysis was performed on a bulk sample of the weathered metavolcanics encountered in Test Pit TP-101 between depths of approximately 3¾ and 4¼ feet below the existing grade. The particle-size analysis was conducted in general accordance with ASTM D 422-63. The testing was used to classify the sample in accordance with the Unified Soil Classification System (USCS). The results of the particle-size analysis indicated that the sample consists of 0.3 percent gravel, 31 percent sand and 68.7 percent fines (silt and clay). Per the USCS, the tested sample is classified as a sandy fat clay (USCS symbol CH). The grain-size distribution curve is presented in Appendix D.

Direct Shear

Shear testing was performed on a remolded sample of the undocumented artificial fill collected from Test Pit TP-101. Shear testing was performed in general accordance with ASTM Test Method D 3080 in a Direct Shear Machine of the strain control type. Prior to testing the sample was remolded to 90 percent of the laboratory standard (ASTM D 1557) at optimum moisture content. The shear test results are presented in the table below and in Appendix D:

SAMPLE LOCATION AND ELEVATION (FT [NGVD29])	PRIMARY		RESIDUAL	
	COHESION (PSF)	FRICTION ANGLE (DEGREES)	COHESION (PSF)	FRICTION ANGLE (DEGREES)
TP-101 @ ¾-2½	166	35	147	34

Saturated Resistivity, pH, and Soluble Sulfates, and Chlorides

Testing was performed on a bulk sample of the undocumented artificial fill from Test Pit TP-101 for evaluations of general soil corrosivity and soluble sulfates, and chlorides. The test results are summarized below and presented in Appendix D:

SAMPLE LOCATION AND DEPTH (FT)	pH	SATURATED RESISTIVITY (ohm-cm)	SOLUBLE SULFATES (% by weight)	SOLUBLE CHLORIDES (mg/kg)
TP-101 @ ¾-2½	6.5	1,000	0.004	53

Corrosion Summary

The laboratory testing indicates that the tested sample is slightly acid with respect to soil acidity/alkalinity; is corrosive to exposed, buried metals when moist; presents negligible sulfate exposure to concrete (i.e., Exposure Class S0 per Table 19.3.2.1 of American Concrete Institute [ACI] 318-14 [ACI, 2014]); and has relatively low concentrations of soluble chlorides that are below action levels. GSI does not consult in the field of corrosion engineering. Therefore, additional comments and recommendations may be obtained from a qualified corrosion engineer based on the level of corrosion protection required for the project, as determined by other members of the project design team.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on our map, literature, and geotechnical report reviews, field exploration, laboratory testing, and geotechnical engineering analysis, it is GSI's opinion that the subject parcel is suitable to receive the proposed residential and equestrian development, from both geotechnical engineering and geologic viewpoints, provided that the recommendations presented in the following sections are incorporated into the design and construction phases of the project. The primary geotechnical concerns with respect to the proposed development are:

- Earth material characteristics and depth to suitable bearing materials below the existing grades.
- The on-going expansion and corrosion potentials of the onsite soils.
- The potential for perched groundwater to manifest both during and following site development.
- The excavation characteristics of the onsite earth materials.
- The quality of the excavated earth materials for reuse as compacted fill and the possible need for special handling and placement of oversized rock constituents in compacted fills.
- The potential for storm water infiltration into the onsite earth materials to adversely affect the proposed improvements and existing development on the adjacent properties.
- Perimeter conditions and planned improvements near the property boundaries.
- Uniform support of the proposed residential structure.
- Permanent and temporary slope stability.
- Erodibility of the onsite earth materials.
- Regional seismic activity.

The aforementioned geotechnical factors are further discussed herein.

The recommendations presented herein consider these as well as other aspects of the site. The engineering analyses performed concerning site preparation and the recommendations presented herein recognize the development information provided to us and the data obtained during our evaluation.

If any significant changes are made to the proposed site development, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and the recommendations of this report verified or modified in writing by this office. Foundation design parameters are considered preliminary until the foundation design, layout, and structural loads are provided to this office for review.

1. Geotechnical observation and testing services should be provided during grading to aid the contractor in removing unsuitable bearing soils and in their effort to compact the fill.

2. Geologic observations should be performed during earthwork and foundation construction to further evaluate the onsite geologic conditions. Although unlikely, if adverse geologic conditions or structures are encountered, supplemental recommendations and earthwork may be warranted.

3. All undocumented artificial fill, colluvium (topsoil), and any weathered portions of the Friars Formation and metavolcanics are considered potentially compressible in their existing state and unsuitable for the support of the proposed settlement-sensitive improvements (i.e., the residential structure, underground utilities, vehicular and pedestrian pavements, etc.) and new planned fills. If not extracted during the planned excavations, unsuitable soils within the influence of proposed settlement-sensitive improvements and new planned fills should be removed to expose suitable, unweathered Friars Formation or metavolcanics and then be reused as properly compacted fill. Proposed improvements and planned fills underlain by unmitigated earth materials may be subjected to damaging settlements.

4. The results of E.I. testing performed on representative samples of the onsite earth materials indicate expansion indices of 10 and 52. This corresponds to very low and medium expansion potentials, respectively. Atterberg limits testing performed on a composite sample of the weathered metavolcanics demonstrated that the specimen has a plasticity index (P.I.) of 36. Based on our experience and professional judgement, this P.I. correlates to high expansion potential (E.I. range of 91 to 130). Based on our observations and the laboratory test results, we conclude that earth materials meeting the criteria of expansive soils, as defined in Section 1803.5.3 of the 2022 CBC. Thus, selective grading or specialized structural design is recommended to reduce the adverse effects of shrink/swell deformations on the proposed improvements.

5. General corrosivity, soluble sulfates, and soluble chlorides testing performed on a sample of the undocumented fill from Test Pit TP-101 indicates that the tested sample is slightly acid with respect to soil acidity/alkalinity; is corrosive to exposed, buried metals when moist; presents negligible sulfate exposure to concrete (i.e., Exposure Class S0, per Table 19.3.1.1 of American Concrete Institute [ACI] 318-14);

and contains relatively low concentrations of soluble chlorides that are below action levels. GSI does not consult in the field of corrosion engineering. Therefore, additional comments and recommendations may be obtained from a qualified corrosion engineer based on the level of corrosion protection required for the project, as determined by other members of the project design team.

6. No evidence of the regional groundwater table was observed during subsurface exploration. However, perched groundwater was observed seeping from the sidewalls of our Test Pits TP-101, TP-102, and TP-103 above the weathered metavolcanics and from joints within the metavolcanics. The source of the perched groundwater is likely infiltrated precipitation from rain events that preceded our field study. The occurrence of perched groundwater may require submersible dewatering pumps during the planned and remedial excavations and subdrainage systems along the toes of the planned cut slope (i.e., toe drains).

7. Based on data obtained during subsurface exploration and our past work experience with sites underlain by metavolcanics, GSI anticipates that non-productive ripping and trenching may be realized during the planned excavations and overexcavations (undercuts). This may require the use of rock-breaking equipment (i.e., hoe ram), a non-explosive demolition agent (i.e., Dexpan), or drill-and-shoot blasting techniques. If additional information regarding the excavation characteristics of the onsite earth materials is needed, this office can perform seismic refraction studies.

8. Based on our observations, we anticipate that the onsite earth materials incorporated into compacted fills will include an abundance of oversized rock fragments that will require special handling and placement during grading. Recommendations for the placement of oversized rock fragments in compacted fills are provided herein.

9. Based on our observations during subsurface exploration, and our past work experience with sites underlain by Friars Formation or metavolcanics, there is increased potential that the infiltration of storm water into the onsite earth materials to meet permanent post-construction storm water BMP objectives would result in perched groundwater conditions (i.e., groundwater mounding). Perched groundwater would likely migrate laterally in the subsurface and adversely affect the proposed onsite improvements as well as the existing development on the adjacent properties. The infiltration of storm water into the onsite earth materials also has the potential to induce swelling of the onsite and offsite expansive soils and settlement of the onsite undocumented fill. Thus, the infiltration of storm water into the onsite earth materials is not recommended from a geotechnical perspective.

10. The removal and recompaction of potentially compressible undocumented fill, colluvium, and weathered Friars Formation or metavolcanics below a 1:1 (h:v) plane

projected down from the bottom, outboard edge of the proposed settlement-sensitive improvements and the limits of new planned fill, along the perimeter of the site and adjacent to existing utility easements, may be restricted due to boundary conditions. Existing onsite or offsite improvements that are to remain in service may also constrain the lateral extent of remedial grading. As such, any proposed settlement-sensitive improvement located above a 1:1 (h:v) plane projected up and into the project area from the bottom outboard edge of the remedial grading excavations at the property boundaries, utility easements, or from existing onsite or offsite improvements that need to remain in service would require deepened foundations below this plane, additional reinforcement, or would retain some potential for distress. If not properly supported and designed, these improvements may experience a reduced service life. On a preliminary basis, any proposed settlement-sensitive improvement or planned fill located within a horizontal distance of approximately ½ foot to 13 feet from the property boundaries, utility easements, or existing onsite, or offsite improvements that are to remain in service would be affected by perimeter conditions. This should be considered during project planning and design. Slot grading/slot excavation may be performed to extend remedial earthwork excavations to the property and utility easement boundaries or the aforementioned existing improvements. Recommendations for slot grading/slot excavations are included in this report.

11. Provided the recommendations for graded slope construction, surficial drainage, and landscaping in this report are properly incorporated into the project, it is our opinion that the permanent graded slopes will be grossly and surficially stable. On a preliminary basis, temporary slopes for excavations with overall heights of 20 feet or less should be constructed in accordance with CAL/OSHA guidelines for Type "B" soils, provided groundwater, seepage, or running sands are <u>absent</u>. Otherwise, the temporary slopes should conform to CAL/OSHA guidelines for Type "C" soils. All temporary slopes should be evaluated by a licensed engineering geologist or engineer, prior to entry by an unprotected worker. Should adverse conditions be identified, the temporary slope may need to be laid back to a flatter gradient or require the use of shoring. If the recommended temporary slopes conflict with property lines, utility easements, or existing improvements that need to remain in service, slot grading/slot excavation or shoring may be necessary. Additional geotechnical analyses should be performed for temporary slopes greater than 20 feet in overall height or those that would receive surcharge from structures, traffic, or stockpiles.

12. Site soils are considered erodible. Therefore, site drainage should be designed to eliminate the potential for concentrated flows along the ground surface. Positive surface drainage away from foundations and tops of slopes is recommended. Temporary erosion control measures should be implemented until vegetative covering is well established. Proper surface drainage should be maintained over the life of the project.

13. The site is subject to moderate to strong ground shaking should an earthquake occur along any of a number of the regional, Holocene-active fault splays. The seismic acceleration values and design parameters, provided herein, should be considered during the design of the proposed development. The adverse effects of seismic shaking on the proposed improvements will likely be wall/pool shell cracks, some foundation/slab distress, and some seismic settlement. This potential should be disclosed to any owners and all interested/affected parties.

14. General Earthwork and Grading Guidelines are provided at the end of this report as Appendix F. Specific recommendations are provided below.

EARTHWORK CONSTRUCTION RECOMMENDATIONS

General

All earthwork should conform to the guidelines presented in Appendix Chapter "J" of the 2022 CBC (CBSC, 2022), the requirements of the County of San Diego, and the General Earthwork and Grading Guidelines presented in Appendix F, except where specifically superceded in the text of this report. Prior to the start of earthwork, a pre-construction meeting should be held with a GSI representative in attendance to provide additional earthwork guidelines, if needed, and to review the earthwork schedule.

This firm should be notified in advance of any excavation, fill placement, supplemental regrading of the site, or backfilling underground utility trenches and retaining walls after rough earthwork has been completed. This includes grading for driveway approaches, driveways, and any exterior pedestrian hardscape.

During earthwork construction, all site preparation and the general grading procedures of the contractor should be observed and the fill selectively tested by a representative(s) of GSI. If unusual or unexpected conditions are exposed in the field, they should be reviewed by this firm, and if warranted, modified, or additional recommendations will be offered. All applicable requirements of local and national construction and general industry safety orders, the Occupational Safety and Health Act (OSHA), and the Construction Safety Act should be met. It is the onsite general contractor's and individual subcontractors' responsibility to provide a safe working environment for our field staff who are onsite. GSI does not consult in the area of safety engineering.

Site Preparation

Any existing non-essential improvements, vegetation (including root systems) and deleterious debris located within the limits of the planned and remedial earthwork should be removed from the site prior to the start of construction. Any remaining cavities should be observed by the geotechnical consultant. Mitigation of cavities would likely include removing any potentially compressible soils to expose suitable, unweathered Friars Formation or metavolcanics and then backfilling the excavation with a controlled engineered fill or soils that have been uniformly moisture conditioned to at least 2 to 3 percent above the soil's optimum moisture content and compacted to a minimum relative density of 90 percent of the laboratory standard (per ASTM D 1557).

Although not anticipated, should an onsite sewage disposal system (seepage pits, leach lines, etc.), cisterns, or water wells be exhumed during earthwork construction, this office should be contacted to provide recommendations for the removal and disposal of such. If discovered, abandoned water wells should be destroyed in accordance with County of San Diego Department of Environmental Health and Quality (DEHQ) guidelines.

Removal and Recompaction of Potentially Compressible Earth Materials

Unless extracted by the planned excavations, potentially compressible undocumented artificial fill, colluvium (topsoil), and any weathered Friars Formation or metavolcanics, within the influence of the proposed settlement-sensitive improvements and new planned fills, should be removed to expose suitable, unweathered Friars Formation or metavolcanics. Following removal, these earth materials should be cleaned of any vegetation, organic matter, and deleterious debris, uniformly moisture conditioned to at least 2 to 3 percent above the soil's optimum moisture content, and then be recompacted to a minimum relative density of 90 percent of the laboratory standard (per ASTM D 1557). Based on the available subsurface data, excavations necessary to remove unsuitable soils are anticipated to extend to depths ranging between approximately ½ foot and 13 feet below the existing grades. Variations are possible and the potential for remedial grading excavations to extend to greater depths than stated above, cannot be precluded and should be anticipated.

Potentially compressible earth materials should be removed below a 1:1 (h:v) plane projected down and away from the bottom, outboard edge of any proposed settlement-sensitive improvement and the limits of new planned fills, where not restricted by property lines, utility easements, and existing onsite, or offsite improvements that need to remain in service. Remedial grading excavations should be observed by the geotechnical consultant prior to scarification and fill placement. Once observed and approved, the bottoms of the remedial grading excavations should be scarified at least 6 to 8 inches, uniformly moisture conditioned to at least 2 to 3 percent above the soil's optimum moisture content, and then be recompacted to a minimum relative density of 90 percent of the laboratory standard (per ASTM D 1557).

Overexcavation

In order to provide uniform foundation support and reduce the potential for ponding of surface water, or the accumulation of perched groundwater within the proposed building pad, any unweathered Friars Formation or metavolcanics located within the greater of either 48 inches of the building pad grade or 24 inches below the lowest foundation element should be overexcavated (undercut) and replaced with compacted fill, prepared and compacted in accordance with the recommendations in the “Compacted Fill Placement” section of this report. The maximum to minimum fill thickness beneath the proposed building should not exceed a ratio of 3:1 (maximum:minimum). The bottom of the overexcavation should be sloped such that any accumulated perched groundwater will migrate away from the building pad. At a minimum, overexcavation should be completed for a horizontal distance of at least 5 feet beyond the perimeter building foundations, including those supporting exterior columns.

Slot Grading/Slot Excavations

As an alternative to shoring, slot grading/slot excavations may be performed for remedial excavation adjacent to the property boundaries, utility easements, or existing improvements that need to remain in service. The slot excavations may be conducted in an “A,” “B,” and “C” sequence, and should be a maximum of 6 feet in width. Multiple slots may be simultaneously excavated provided that open slots are separated by at least 12 feet of tested and approved compacted fill or undisturbed soils. The actual number and widths of the slot excavations should not cause the allowable bearing capacity of any existing, adjacent foundation to increase by more than 2.0 times the allowable bearing. This will require proper sequencing during construction. Pre-construction surveys and survey monitoring should be performed in conjunction with slot grading/slot excavations.

Perimeter Conditions

The 2022 CBC (CBSC, 2022) indicates that the removal of potentially compressible soils be performed across all areas to be graded, under the purview of a grading permit, and not just within the influence of the proposed residential structure. Relatively deep removals may also necessitate a special zone of consideration, on perimeter/confining areas. This zone would be approximately equal to the depth of the remedial grading excavations, if remedial grading cannot be performed onsite or offsite. Based on the available subsurface data, any proposed improvement or planned fill located within approximately ½ foot to 13 feet from the property boundaries, utility easements, or existing onsite, or offsite improvements that are to remain in service, would be affected by perimeter conditions.

Earthwork Mitigation of Expansive Soils

As an alternative to designing the proposed improvements, particularly the foundation and concrete slab-on-grade floor of the residential structure, to resist on-going shrink/swell deformations from the onsite expansive soils, onsite earth materials with an expansion index greater than 20 and a plasticity index greater than 14 may be removed and replaced with soils possessing an expansion index of 20 or less and a plasticity index of 14 or less, such that the effective plasticity index of the upper 15 feet of the soil profile is less than 15. This may require a substantial volume of export and import. Therefore, additional geotechnical studies should be performed to evaluate the cost feasibility of the mitigation of expansive soils through selective grading.

Compacted Fill Placement

Following scarification of the bottom of the remedial grading excavations and overexcavations, the reused onsite soils and import (if necessary) should be cleaned of any vegetation, organic matter, and deleterious debris, placed in approximately 6- to 8-inch thick lifts, uniformly moisture conditioned to at least 2 to 3 percent above the soil's optimum moisture content, and compacted to achieve a minimum relative density of 90 percent of the laboratory standard (ASTM D 1557). Benching should be performed when placing compacted fills on surfaces with slope gradients steeper than 5:1 (h:v). Fill placement and compaction should be observed and tested by the geotechnical consultant.

Rock Placement Guidelines

GSI anticipates that some of the onsite earth materials to be used as compacted fill for the subject project may contain oversized rock fragments. Appropriately, the need for oversized rock disposal may be necessary during grading operations on the site. From a geotechnical standpoint, the depth of any rocks, rock fills, or rock blankets, should be a sufficient distance from pad grade and the faces of fill slopes. This depth within the building pad area is generally the same as any overexcavation due to cut/fill transitions in hard rock areas, and generally facilitates the excavation of structural footings and substructures. Should deeper excavations be proposed (i.e., deepened footings, utility trenching, swimming pools, spas, etc.), the Client may consider increasing the hold-down depth of any rock fills to be placed, as appropriate. In addition, some agencies/jurisdictions mandate a specific hold-down depth for oversized materials placed in fills. The hold-down depth, and potential to encounter oversize rock, both within fills, and in occurring in cut or natural areas, would need to be disclosed to all interested/affected parties. Once approved by the governing agency, the hold-down depth for oversized rock (i.e., greater than 12 inches) in fills on this project is provided as 10 feet. The reuse of oversized materials around pools (next to or below) is not recommended.

General

Generally for the purpose of this report, the materials may be described as either 8 inches or less and greater than 8 and less than 24 inches. These two categories set the basic dimensions for where and how the materials are to be placed. However, the volume and hold down requirements for placement of materials > 12 inches in size may be difficult to achieve, and should also be part of the value engineering assessment.

Materials 8 Inches in Dimension or Less

Since rock fragments along with the overburden materials are anticipated to be a part of the compacted fill used in the grading of the site, a criteria is needed to facilitate the placement of these materials within guidelines which would be workable during the rough grading, post-grading improvements, and serve as acceptable compacted fill.

1. Fines and rock fragments 8 inches or less in dimension may be placed as compacted fill cap materials within the building pad, slopes, and driveway areas as described below. The mixture of rock fragments and fines should be brought to at least 2 to 3 percent above optimum moisture content and compacted to a minimum relative compaction of 90 percent of the laboratory standard.

2. The purpose for the 8-inch dimension limit is to allow reasonably sized rock fragments into the fill under selected conditions (optimum moisture or above) surrounded with compacted fines. The 8-inch dimension size also allows a greater volume of the rock fragments to be handled during grading, while staying in reasonable limits for later onsite excavation equipment (backhoes and trenchers) to excavate footings and utility line trenches.

3. Fill materials 8 inches or less in dimension should be placed (but not limited to) within the hold-down depth on the proposed building pad, the upper 5 feet of overexcavated cut areas of cut/fill transition pads, and the entire driveway and utility corridor widths, including the proposed overexcavated areas and replacement fill areas, from the depth of the lowest underground utility (within the driveway and building pad), to subgrade, or to the hold-down depth below pad grade. Overexcavation is discussed earlier in this report.

Materials Greater Than 8 inches and Less Than 24 Inches in Dimension

1. During the process of bedrock excavation, a significant amount of rock fragments or constituents larger than 8 inches in dimension may be generated. These significant amounts of oversized materials, greater than 8 and less than 24 inches in dimension, may be incorporated into the compacted fills using a series of rock blankets.

2. Each rock blanket should consist of rock fragments of approximately greater than 8 and less than 24 inches in dimension along with fines generated from the proposed cuts and overburden materials from the remedial grading excavations. The blankets should be limited to 24 inches in thickness and should be placed with granular fines which are flooded into and around the rock fragments.

3. Rock blankets should be restricted to areas which are at least 1 foot below the lowest underground utility invert, at least the hold-down depth below pad grade, and a minimum of 20 horizontal feet from the faces of fill slopes, and outside of any underground utility laterals or under pools/spas.

4. Compaction may be achieved by using wheel rolling methods with scrapers and water trucks, track-walking by bulldozers, and sheepsfoot tampers.

5. Each rock blanket should be completed with its surface compacted prior to placement of any subsequent rock blanket or rock windrows.

6. Minor amounts of rock material in this size range may also be placed in rock windrows.

7. Rock fragments greater than 24 inches in dimension may be placed at the discretion of the geotechnical consultant on a case-by-case basis. Rocks of this size are typically disposed in rock windrows or within individual rock disposal pits, and surrounded by granular fines that are flooded in place.

Substructures Placed in the Hold-down Depth Zone

Disclosure to any interested/affected parties regarding the proximity of oversize materials, excavation difficulties, hard rock, etc., that may potentially impact future improvements is recommended. The cap above the hold-down distance is only intended to support shallow foundations of the residence, appurtenant structures, and certain specified improvements. Utility poles, pools, spas, or similar improvements that penetrate or nearly penetrate the fill cap should have a site-specific subsurface investigation, and review by the geotechnical consultant, prior to planning, design, and construction.

Import Soils

If import fill is necessary, a sample of the soil import should be evaluated by this office prior to importing, in order to assure compatibility with the onsite soils and the recommendations presented in this report. If non-manufactured materials are used, environmental documentation for the export site should be provided for GSI review. At least three (3) business days of lead time should be allowed by builders or contractors for proposed import submittals. This lead time will allow for environmental document reviews, particle-size analysis, laboratory standard, expansion index testing, and an evaluation of the blended import/native characteristics, as deemed necessary. Import soils should have an E.I. and P.I. that do not exceed those exhibited by the onsite soils. The use of subdrains at the bottom of the fill cap may be necessary, and may be recommended based on the import soil's compatibility with the onsite soils.

Subdrains

Subdrains should be installed along the axes of natural drainage courses, following the removal of potentially compressible earth materials, where the as-graded fill thickness will exceed 10 feet. At a minimum, the subdrain should consist of a 4-inch diameter Schedule 40 or SDR 35 drain pipe (with perforations oriented down) that is encased in at least

1 cubic foot of clean, crushed ¾ inch to 1½-inch gravels entirely wrapped in filter fabric (Mirafi 140N or approved equivalent). The subdrain should gravity flow (minimum 1 percent slope) toward an approved drainage facility identified by the project civil engineer. The subdrain should be connected to the approved discharge point using a solid Schedule 40 or SDR 35 drain pipe that is at least 4 inches in diameter. A concrete cutoff collar should be installed at the connection between solid and perforated drain pipes. The cutoff collar should be at least 6 inches long in plan view and extend at least 12 inches beyond the outer wall of the pipe coupler in all directions (360 degrees).

Graded Slope Construction

General

The following recommendations should be followed during the construction of the planned cut and fill slopes associated with the proposed project.

Fill Slopes

Graded fill slopes should be properly keyed and benched, and be compacted to a minimum relative density of 90 percent of the laboratory standard (ASTM D 1557), throughout, including the slope face. Compaction at the slope face may be achieved by either overbuilding and trimming back fill slopes or back-rolling fill slopes with compaction equipment every 4 vertical feet. The minimum width of keyway excavations should be H/2, where "H" equals the slope height, but not be less than 15 feet. The toe of the keyway should extend at least 2 feet into suitable, unweathered Friars Formation or metavolcanics. The bottom of the keyway excavation should be sloped at a minimum of 2 percent from the toe to the heel. Benching should be performed on all surfaces with slope gradients steeper than 5:1 (h:v), prior to fill placement. Fill slopes should not be constructed at gradients steeper than 2:1 (h:v) without prior approval by the geotechnical consultant and the County of San Diego.

SDA (undated) indicates the construction of a planned fill slope with a 1.5:1 (h:v) gradient along a portion of the proposed driveway. In order to improve surficial stability and reduce creep and lateral fill extension along this slope, GSI recommends that Solmax Miragrid 8XT geogrid (or equivalent) be placed at 2-foot vertical spacings within the outer 7 feet of the slope. Fill materials should be placed above each layer of geogrid, uniformly moisture conditioned to at least 2 to 3 percent above the soil's optimum moisture content, and compacted to a minimum relative density of 90 percent of the laboratory standard (ASTM D 1557).

Cut Slopes

The planned cut slopes should be constructed into the Friars Formation or metavolcanics at gradients no steeper than 1.5:1 (h:v). We generally recorded high-angle joints where the metavolcanics were exposed in our test pits and in outcrop. Thus, the potential for plane and wedge failures along the planned cut slopes is considered low on a preliminary basis.

All cut slopes should be observed by a representative of this firm during grading to evaluate the presence of adverse geologic conditions (i.e., exposures of significant undocumented fill, colluvium, or highly weathered Friars Formation or metavolcanics) or adverse structures (i.e., out-of-slope bedding planes or daylighted foliation, joint, or fracture planes). Should adverse conditions be exposed in cut slopes, mitigation measures would be recommended. Mitigation measures may include, but not necessarily be limited to inclining cut slopes to flatter gradients, stabilization fills, etc. If there are concerns of establishing vegetative covering on the planned cut slopes, the construction of stability fills would be recommended. A generalized detail illustrating the recommended construction of stabilization fills is provided on Plate F-9 in Appendix F.

Fill-Over-Cut Slopes

In order to reduce the potential for seepage along the slope face near the as-built daylight line, the cut portions of fill-over-cut slopes should be overexcavated and reconstructed as a fill slope per the recommendations in the “Fill Slopes” section.

Other Considerations Regarding Graded Slopes

- Graded slopes should receive a deep-rooted, drought tolerant vegetative covering immediately following construction. In the interim between construction and the establishment of landscape cover, the graded slopes should receive County of San Diego-approved erosion control devices.

- In order to reduce erosion and help maintain surficial stability, the project landscape architect should consider the use of drip-system irrigation with moisture sensors on all graded slopes.

- Planting on graded cut slopes exposing Friars Formation or metavolcanics may be problematic. Soil amendments may be necessary.

- Due to the presence of expansive soils, proposed improvements located near the tops of slopes should be designed and constructed to reduce the adverse effects of lateral fill extension and creep. Recommendations for mitigation are included herein.

- Surface drainage should be directed away from the tops of graded slopes using berms or concrete-lined drainage swales. Conveyance of surface runoff along the toes of slopes should be avoided or transported in concrete-lined swales or through piping. The storage or infiltration of surface runoff along the toes and tops of slopes is not recommended.

- The condition of graded slopes should be periodically reviewed by the property owner throughout the life of the development. Any observed deficiencies should be corrected as soon as possible. If requested, this office can provide additional consultation regarding the maintenance of graded slopes.

Temporary Slopes

Temporary slopes for excavations extending to depths less than or equal to 20 feet should conform to CAL/OSHA or OSHA requirements for Type “B” soils (i.e., 1:1 [h:v] gradient), provided groundwater, seepage, or running sands are not present. Otherwise, the temporary slopes should be constructed in accordance with CAL/OSHA guidelines for Type “C” soils (i.e., 1.5:1 [h:v] gradient). Temporary slopes greater than 20 feet in overall height should receive additional geotechnical evaluation prior to construction. Construction materials and soil stockpiles, and heavy equipment storage/traffic should not occur within “H” of the top of any temporary slope where “H” equals the height of the temporary slope. All temporary slopes should be observed by a licensed engineering geologist or engineer prior to unprotected worker entry into the excavation. If adverse conditions are exposed in temporary slopes, they may need to be inclined to flatter gradients or require the use of shoring. If temporary slopes conflict with property boundaries, or existing improvements that need to remain in service, shoring or slot grading/slot excavation may be necessary. The need for shoring or slot grading/slot excavation could be further evaluated during the grading plan review stage and during site earthwork.

We encountered caving between the ground surface and an approximate depth of 2 feet below the existing grade in TP-101. The caving was due to perched groundwater seepage and saturated soil conditions from the rain events that preceded our field work. Thus, it temporary slopes may need to be inclined to flatter gradients or require shoring if excavations are conducted shortly following rainfall.

Excavation Observation and Monitoring (All Excavations)

When excavations are made adjacent to an existing improvement (i.e., underground utility, wall, road, building, etc.) there is a risk of some damage even if a well-designed system of excavation is planned and executed. We therefore recommend that a systematic program of observations be made before, during, and after construction to determine the effects (if any) of the excavation on existing improvements.

We believe that this is necessary for two reasons. First, if excessive movements (i.e., more than ½-inch) are detected early enough, remedial measures can be undertaken which could possibly prevent serious damage to existing improvements. Second, the responsibility for damage to the existing improvement can be evaluated more equitably if the cause and extent of the damage can be determined more precisely.

Monitoring should include the measurement of any horizontal and vertical movements of the existing structures/improvements. Locations and types of monitoring devices should be selected prior to the start of construction. The program of monitoring should be agreed upon between the project team, the site surveyor and the geotechnical consultant, prior to excavation.

Reference points should be provided on existing walls, buildings, and other settlement-sensitive improvements. These points should be placed as low as possible on the walls and buildings adjacent to the excavation. Exact locations may be dictated by critical points, such as bearing walls or columns for buildings; and surface points on roadways or curbs, near the top of the excavation.

For a survey monitoring system, an accuracy of a least 0.01 foot should be required. Reference points should be installed and read initially prior to excavation. The readings should continue until all construction below ground has been completed and the permanent backfill has been brought to finish grade.

The frequency of readings will depend upon the results of previous readings and the rate of construction. Weekly readings could be assumed throughout the duration of construction with daily readings during rapid excavation near the bottom of the excavation. The readings should be plotted by the project surveyor/civil engineer and then reviewed by the geotechnical consultant. In addition to the monitoring system, it would be prudent for the geotechnical consultant and the contractor to make a complete inspection of the existing structures and improvements both before and after construction. The inspection should be directed toward detecting any signs of damage, particularly those caused by settlement. Notes should be made and pictures should be taken where necessary.

Observation

All excavations should be observed by a licensed engineering geologist or engineer. Should the observation reveal any unforseen hazard, the engineering geologist or engineer will recommend treatment. Please inform GSI at least 24 hours prior to any required site observation.

Earthwork Balance (Shrinkage/Bulking)

The volume change of excavated materials, upon compaction as engineered fill, is anticipated to vary with material type and location. The overall earthwork shrinkage and bulking of the earth materials anticipated to be encountered during earthwork may be approximated by using the following parameters:

Undocumented Fill and Colluvium . . . . 5% to 15% shrinkage
Friars Formation and Metavolcanics. . . . . 5% shrinkage or bulking

The above factors are estimates only, based on preliminary data. The undocumented fill, colluvium, and weathered Friars Formation and metavolcanics may achieve higher shrinkage if organics or clay content is higher than anticipated, if a high degree of porosity is encountered, or if compaction averages more than 90 percent of the laboratory standard (per ASTM D 1557). In addition, extensive rodent burrowing may result in higher shrinkage. Final earthwork balance factors could vary. In this regard, it is recommended that balance areas be reserved where grades could be adjusted up or down near the completion of grading in order to accommodate any yardage imbalance for the project.

PRELIMINARY RECOMMENDATIONS - BUILDING FOUNDATIONS AND CONCRETE SLAB-ON-GRADE FLOORS

Preliminary recommendations for building foundation and concrete slab-on-grade floor design and construction are provided in the following sections. These preliminary recommendations have been developed from our understanding of the currently proposed site development and our site observations, subsurface exploration, laboratory testing, and engineering analyses. Foundation design should be re-evaluated at the conclusion of site grading/remedial earthwork for the as-graded soil conditions. Although not anticipated, revisions to these recommendations may be necessary. If the information concerning the proposed development plan is not correct, or any changes in the design, location or loading conditions of the proposed residential structure are made, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report are modified or approved in writing by this office.

Absent selective grading, we anticipate that the compacted fill located near pad grade for the proposed residential structure will be expansive (E.I. greater than 20 and P.I. greater than 14). Thus, we are providing preliminary recommendations for the design and construction of post-tensioned slab and mat-slab foundations in the following sections to resist distortions caused by shrink/swell deformations of the onsite expansive soils, as required in Section 1808.6.2 of the 2022 CBC. However, the size and configuration of the proposed residential building footprint may not be compatible with post-tensioned slab foundations. To that end, the project architect and structural engineer should select the appropriate foundation type.

The information and recommendations presented in this section are not meant to supercede design by the project structural engineer or civil engineer specializing in structural design. Upon request, GSI could provide additional input/consultation regarding soil parameters, as related to foundation design.

The preliminary geotechnical data indicates the subject site is underlain by soils very low to high in expansion potential (E.I. $\leq$ 130). The preliminary recommendations for post-tensioned slab and mat-slab foundations consider these soil conditions. The foundation system should bear entirely on approved compacted fill, observed and tested by this office, which is directly underlain by suitable, unweathered Friars Formation or metavolcanics.

POST-TENSIONED SLAB FOUNDATION SYSTEMS

Post-tensioned (PT) slab foundation systems may be used to mitigate the damaging shrink/swell effects of the onsite expansive soils on the proposed building foundation and slab-on-grade floor. The post-tensioned slab foundation designer may elect to exceed the minimum recommendations, provided herein, in order to increase slab stiffness performance. Post-tensioned slab foundation design may be either ribbed or mat-type. The former uses reinforced internal, concrete beams to assist with rigidity. The latter is also referred to as uniform thickness foundations (UTFs). The use of a UTF is an alternative to the traditional ribbed-type. The UTF offers a reduction in the quantity or surface area of the internal concrete beams. That is to say a UTF typically uses a single perimeter grade beam and "shovel" footings for hold-downs, but has a thicker slab than the ribbed-type.

The information and recommendations presented in this section are not meant to supercede design by a registered structural engineer or civil engineer qualified to perform post-tensioned slab foundation design. Post-tensioned slab foundations should be designed using sound engineering practice and be in accordance with local building codes, 2019 CBC requirements, and Post Tensioning Institute (PTI) methodologies (PTI; 2004, 2008, 2012, 2013, and 2014). Upon request, GSI can provide additional data/consultation regarding soil parameters, as they relate to post-tensioned slab foundation design.

From a soil expansion/shrinkage standpoint, a common contributing factor to distress of structures using post-tensioned slab foundations is a "dishing" or "arching" of the slabs. This is caused by the fluctuation of moisture content in the soils below the perimeter of the slab, primarily due to onsite and offsite irrigation practices, climatic and seasonal changes, and the presence of expansive soils. When the soil environment surrounding the exterior of the slab has a higher moisture content than the area beneath the slab, moisture tends to migrate inward, underneath the slab edges to a distance beyond the slab edges referred to as the moisture variation distance. When this migration of water occurs, the volume of the soils beneath the slab edges expands and causes the slab edges to lift in response. This is referred to as an edge-lift condition. Conversely, when the outside soil environment is drier, the moisture transmission regime is reversed and the soils underneath the slab edges lose their moisture and shrink. This process leads to dropping of the slab at the edges, which results in what is commonly referred to as the center lift condition. A well-designed, post-tensioned slab foundation having sufficient stiffness and rigidity provides a resistance to excessive bending that results from non-uniform swelling and shrinking slab subgrade soils, particularly within the moisture variation distance, near the slab edges. Other mitigation techniques typically used in conjunction with post-tensioned slab foundations consist of a combination of specific soil pre-saturation and the construction of a perimeter "cut-off" wall/grade beam. Soil pre-saturation consists of moisture conditioning the slab subgrade soils prior to the post-tensioned slab foundation construction. This effectively reduces soil moisture migration from the area located outside the building toward the soils underlying the post-tensioned slab foundation. Perimeter cut-off walls are thickened edges of the concrete slab that impede both outward and inward soil moisture migration.

Slab Subgrade Pre-Soaking

Pre-moistening of the slab subgrade soil is recommended to reduce the potential for post-construction soil heave. The moisture content of the subgrade soils should be greater than optimum moisture to a depth equivalent to the perimeter grade beam or cut-off wall depth in the slab areas (typically 12, 18, or 24 inches deep for soils that are low, medium, or high in expansion potential, respectively).

Pre-moistening or pre-soaking should be evaluated by the geotechnical consultant 72 hours prior to vapor retarder placement. In summary:

EXPANSION POTENTIAL	PAD SOIL MOISTURE	CONSTRUCTION METHOD	SOIL MOISTURE RETENTION
Low (E.I. = 21-50)	Upper 12 inches of pad grade soil moisture 2 percent over optimum	Wetting or reprocessing	Periodically wet or cover with plastic after trenching. Evaluation within 72 hours prior to placement of vapor retarder and underlayment section.
Medium (E.I. = 51-90)	Upper 18 inches of pad grade soil moisture 2 percent over optimum	Berm and flood <u>or</u> wetting and reprocessing	Periodically wet or cover with plastic after trenching. Evaluation within 72 hours prior to placement of vapor retarder and underlayment section.
High (E.I. = 91-130)	Upper 24 inches of pad grade soil moisture 3 percent over optimum	Berm and flood <u>or</u> wetting and reprocessing	Periodically wet or cover with plastic after trenching. Evaluation within 72 hours prior to placement of vapor retarder and underlayment section.

Perimeter Cut-Off Walls

Perimeter cut-off walls should be at least 12, 18, or 24 inches deep for soils that are low, medium, or high in expansion potential, respectively. The cut-off walls may be integrated into the post-tensioned slab foundation or independent of the foundation. The cut-off walls should be a minimum of 6 inches thick (wide). The bottom of the perimeter cut-off wall should be designed to resist tension, using cable or steel reinforcement per the project structural engineer.

Post-Tensioned Slab Foundation Design

The following recommendations for the design of post-tensioned slab foundations have been prepared in general conformance with the requirements of the recent Post Tensioning Institute's (PTI's) publication titled "Design of Post-Tensioned Slabs on Ground, Third Edition" (PTI, 2004), together with it's subsequent addendums and errata (PTI; 2008, 2012, 2013, and 2014).

Post-Tensioned Slab Foundation Soil Support Parameters

The recommendations for soil support parameters have been provided based on the typical soil index properties for soils that are low to very high in expansion potential. The soil index properties are typically the upper bound values based on our experience and practice in the southern California area. Additional testing is recommended either during or following grading, and prior to foundation construction to further evaluate the soil conditions within the upper 7 to 15 feet of pad grade. The following table presents suggested minimum coefficients to be used in the Post-Tensioning Institute design method:

Thornthwaite Moisture Index	-20 inches/year
Correction Factor for Irrigation	20 inches/year
Depth to Constant Soil Suction	7 feet or overexcavation depth to bedrock
Constant soil Suction (pf)	3.6
Moisture Velocity	0.7 inches/month
Effective Plasticity Index (P.I.)*	20-40
* - The weighted plasticity index should be evaluated for the upper 15 feet of foundation soils either during or following rough grading and prior to foundation construction.	

Based on the above, the recommended post-tensioned slab foundation soil support parameters are tabulated below:

DESIGN PARAMETERS	LOW EXPANSION (E.I. = 21-50)	MEDIUM EXPANSION (E.I. = 51-90)	HIGH EXPANSION (E.I. = 91-130)
e_m center lift	9.0 feet	8.7 feet	8.5 feet
e_m edge lift	5.2 feet	4.5 feet	3.75 feet
y_m center lift	0.4 inches	0.66 inches	0.75 inches
y_m edge lift	0.7 inch	1.3 inch	1.7 inches
Bearing Value(1)	1,000 psf	1,000 psf	1,000 psf
Lateral Pressure(2)	150 psf	100 psf	100 psf
Subgrade Modulus (k)	100 pci/inch	85 pci/inch	70 pci/inch
Lateral Sliding Resistance(3)	0.35 (Coefficient of Friction)	130 psf (Cohesion)(4)	130 psf (Cohesion)(4)
Minimum Perimeter Footing Embedment(5)	12 inches	18 inches	24 inches

(1) Internal bearing values within the perimeter of the post-tension slab may be increased to 1,200 psf for a minimum embedment of 12 inches, then by 20 percent for each additional foot of embedment to a maximum of 1,440 psf.
(2) The upper 6 inches of passive pressure should be neglected if not confined by slabs or pavement.
(3) Concrete to soil contact when multiplied by the dead load. When combining passive pressure and lateral sliding resistance, the passive pressure component should be reduced by one-third.
(4) Cohesion value to be multiplied by the contact area, as limited by Section 1806.3.2 of the 2022 CBC.
(5) As measured below the lowest adjacent compacted subgrade surface without landscape layer or sand underlayment.
Note: The use of open bottomed raised planters adjacent to foundations will require more onerous design parameters.

The parameters are considered minimums and may not be adequate to represent all expansive soils and site conditions such as adverse drainage or improper landscaping and maintenance. The above parameters are applicable provided the grades around the proposed residential structure provide positive drainage that is maintained away from the building foundation. In addition, no trees with significant root systems should be planted within 15 feet of the foundation perimeter. Therefore, it is important that information regarding drainage, site maintenance, trees, settlements, and effects of expansive soils be disclosed to all interested/affected parties. The values tabulated above may not be appropriate to account for possible differential settlement of the slab due to other factors, such as excessive settlements. If a stiffer slab is desired, alternative Post-Tensioning Institute ([PTI] third edition) parameters may be recommended.

All exterior columns not integrated into the post-tensioned slab foundation should be supported by isolated spread footings that are a minimum of 24 inches square in dimension and extend at least 24 inches into approved compacted fill overlying dense, unweathered Friars Formation or metavolcanics. Exterior column footings should be tied to the perimeter of the post-tensioned slab foundation with 12 square inch, reinforced grade beams in at least two directions.

MAT-SLAB FOUNDATION

A mat-slab foundation may also be used to support the proposed residential structure, in light of the onsite expansive soil conditions. The project structural engineer may supercede the following recommendations based on the planned building loads and use. Wire Reinforcement Institute (WRI) methodologies (Snowden, 1981, 1996) may be used in the mat-slab foundation design.

For a mat-slab foundation bearing uniformly on approved compacted fill that has been placed directly upon dense, unweathered Friars Formation or metavolcanics, a maximum allowable net bearing capacity of 1,000 psf is recommended. Internal bearing values along the perimeter of the mat-slab foundation may be increased to 1,200 psf if the mat-slab foundation incorporates a perimeter grade beam with a minimum width of 12 inches and a minimum embedment depth of 12 inches below the lowest adjacent grade. This bearing value may be increased by 20 percent for each additional foot of perimeter grade beam embedment to a maximum of 1,440 psf. This value may be also be increased by one-third for short-term loads including wind or seismic. Reinforcement of the mat-slab foundation should be designed in accordance with local codes and structural considerations, including the intended use.

The mat-slab foundation may be either ribbed or uniform thickness (UTF) with a perimeter beam measuring at least 12 inches wide. The perimeter beam should extend at least 12, 18, or 24 inches below the lowest adjacent grade into approved compacted fill overlying suitable, unweathered Friars Formation or metavolcanics for a slab subgrade that is low, medium, and high in expansion potential, respectively. The need and arrangement of internal grade beams will be in accordance with the project structural engineer's recommendations. The passive resistance and lateral sliding resistance values recommended in the preceding "Post-Tensioned Slab Foundation Systems" section of this report should also be considered in the design of mat-slab foundations.

All exterior columns not integrated into the mat-slab foundation should be supported by isolated spread footings that are a minimum of 24 inches square in dimension and extend at least 24 inches into approved compacted fill overlying dense, unweathered Friars Formation or metavolcanics. Exterior column footings should be tied to the perimeter of the mat-slab foundation with 12 square inch, reinforced grade beams in at least two directions.

The moduli of subgrade reaction (K_S) and effective plasticity indices (P.I.s) for consideration in mat-slab foundation design for various expansive soil conditions are presented in the following table:

LOW EXPANSION (E.I. = 21-50)	MEDIUM EXPANSION (E.I. = 51-90)	HIGH EXPANSION (E.I. = 91-130)
K_S =100 pci/inch PI <25	K_S =85 pci/inch, PI = 25	K_S =70 pci/inch, PI > 35

The modulus of subgrade reaction is a unit value for a 1-foot square footing and should be reduced in accordance with the following equation when used with the design of larger foundations.

$$K_R = K_S\left[\frac{B+1}{2B}\right]^2$$

where: K_S = unit subgrade modulus
K_R = reduced subgrade modulus
B = foundation width (in feet)

Slab Subgrade Pre-Soaking

Slab subgrade pre-soaking should conform to the recommendations previously provided in the "Post-Tensioned Slab Foundation Systems" section of this report.

FOUNDATION SETTLEMENT

Provided that the earthwork and foundation recommendations in this report are followed, shallow foundations bearing on approved compacted fill overlying dense, unweathered Friars Formation or metavolcanics should be designed to accommodate a maximum total settlement of 1½ inches and a differential settlement of ¾-inch over a 40-foot horizontal span (angular distortion = 1/640).

SOIL MOISTURE TRANSMISSION CONSIDERATIONS

GSI has evaluated the potential for moisture or water vapor transmission through the proposed concrete slab-on-grade floor, in light of typical floor coverings and improvements. Slab moisture emission rates range from about 2 to 27 lbs/24 hours/1,000 square feet from a typical slab (Kanare, 2005), while floor covering manufacturers generally recommend about 3 lbs/24 hours as an upper limit. The recommendations in this section are not intended to preclude the transmission of moisture or water vapor through the foundation or slab-on-grade floor. Foundation systems and concrete slab-on-grade floors shall not allow moisture or water vapor to enter into the structure so as to cause damage to another building component or to limit the installation of the type of flooring materials typically used for the particular application (State of California, 2023). These recommendations may be exceeded or supplemented by a "water proofing" consultant, the project architect, or the structural consultant. Thus, the client will need to evaluate the following in light of a cost vs. benefit analysis (owner expectations and repairs/replacement), along with disclosure to all interested/affected parties. It should also be noted that moisture or water vapor transmission will occur in new slab-on-grade floors as a result of chemical reactions taking place within the curing concrete. Moisture or water vapor transmission through concrete floor slabs as a result of concrete curing has the potential to adversely affect sensitive floor coverings depending on the thickness of the concrete floor slab and the duration of time between the placement of concrete, and the floor covering installation. It is possible that a slab moisture testing and a slab sealant may be needed prior to the placement of sensitive floor coverings if a thick slab-on-grade floor is used and the time frame between concrete and floor covering placement is relatively short.

Considering the E.I. test results presented herein, and known soil conditions in the region, the anticipated typical moisture or water vapor transmission rates, floor coverings, and improvements (to be chosen by the client and project architect) that can tolerate moisture or water vapor transmission rates without significant distress, the following alternatives are provided:

- Concrete floor slabs, including garage slabs-on-grade, should be thicker.

- Concrete floor slab underlayment should consist of a 15-mil vapor retarder, or equivalent, with all laps sealed per the 2022 CBC and the manufacturer's recommendation. The vapor retarder should comply with the ASTM E 1745 - Class A criteria (i.e., Stego Wrap or approved equivalent), and be installed in accordance with ACI 302.1R-05 and ASTM E 1643.

- The 15-mil vapor retarder (ASTM E 1745 - Class A) should be installed per the recommendations of the manufacturer, including <u>all</u> penetrations (i.e., pipe, ducting, rebar, etc.).

- Concrete floor slabs, including the garage areas, should be underlain by 2 inches of clean, washed sand (sand equivalent [SE] $\geq$ 30) above a 15-mil vapor retarder (ASTM E-1745 -Class A, per Engineering Bulletin 119 [Kanare, 2005]), installed per the recommendations of the manufacturer, including all penetrations (i.e., pipe, ducting, rebar, etc.). The manufacturer shall provide instructions for lap sealing, including minimum width of lap, method of sealing, and either supply or specify suitable products for lap sealing (ASTM E 1745), and per code.

  ACI 302.1R-15 (ACI, 2015) states, "Experience has shown, however, that the greatest level of protection for floor coverings, coatings, or building environments is provided when the vapor retarder/barrier is placed in direct contact with the slab.

Placing concrete in direct contact with the vapor retarder/barrier eliminates the potential for water from sources such as rain, saw-cutting, curing, cleaning, or compaction to become trapped within the fill course. Wet or saturated fill above the vapor retarder/barrier can significantly lengthen the time required for a slab to dry to a level acceptable to the manufacturers of floor coverings, adhesives, and coatings. A fill layer sandwiched between the vapor retarder/barrier and the concrete also serves as an avenue for moisture to enter and travel freely beneath the slab, which can lead to an increase in moisture within the slab once it is covered. Moisture can enter the fill layer through voids, tears, or punctures in the vapor retarder/barrier." Therefore, additional observation and testing will be necessary for the cushion or sand layer for moisture content, and relatively uniform thicknesses, prior to the placement of concrete.

Conversely, ACI 302.1R-15 indicates that placing concrete directly upon the vapor retarder requires additional design and construction considerations to avoid potential slab-related problems, such as excessive concrete settlement and significantly larger length change during casting and drying shrinkage, and when the concrete is subject to environmental changes. In addition, dominant joint behavior can be made worse when the slab is placed in direct contact with the vapor retarder. Further, settlement cracking over reinforcing steel is more likely because of increased settlement resulting from a longer bleeding period. There is also a potential for enhanced slab curl. Lastly, if rapid surface drying conditions are present, the surface of the concrete (i.e., top fraction of an inch [millimeter]) placed directly upon the vapor retarder would have a greater propensity to dry and crust over leaving the underlying concrete relatively less stiff or unhardened. This may impact surface flatness of the concrete slab and result in blistering or delamination. Design and construction measures should be implemented to offset or reduce these effects.

Given the above, GSI recommends that all responsible parties participate in a risk/benefit evaluation regarding the specified location of the vapor retarder during project design.

- Owing to the expansive characteristics of the onsite earth materials, the vapor retarder should be directly underlain by a capillary break consisting of at least 4 inches of clean crushed gravel with a maximum dimension of ¾ inch and less than 5 percent passing the No. 200 sieve that has been placed upon a properly compacted and moisture conditioned slab subgrade.

- Concrete used in the construction of the building foundations and slab-on-grade floors should have a maximum water-to-cement ratio (W/C) of 0.50. This does not supercede Table 19.3.2.1 of American Concrete Institute 318-14 ([ACI], 2014) for corrosion or other corrosive requirements. Additional concrete mix design recommendations should be provided by the structural consultant or waterproofing consultant. Concrete finishing and workability should be addressed by the structural consultant and a waterproofing consultant.

- Where slab water to cement ratios are as indicated herein, or admixtures used, the structural consultant should also make changes to the concrete in the grade beams and footings in kind, so that the concrete used in the foundation and floor slab is designed or treated for more uniform moisture protection.

- The owner(s) should be specifically advised which areas are suitable for tile flooring, vinyl flooring, or other types of moisture/vapor-sensitive flooring and which areas are not suitable for these types of flooring applications. In all planned floor areas, flooring shall be installed per the manufacturer's recommendations.

- Additional recommendations regarding moisture or water vapor transmission should be provided by the architect/structural engineer/slab or foundation designer and should be consistent with the specified floor coverings indicated by the architect.

Regardless of the mitigation, some limited moisture/ water vapor transmission through the building foundation and concrete slab-on-grade floor cannot be entirely precluded and should be anticipated. Construction crews may require special training for installation of certain product(s), as well as concrete finishing techniques. The use of specialized product(s) should be approved by the slab designer and waterproofing consultant. A technical representative of the flooring contractor should review the slab and moisture retarder plans and provide comment prior to the construction of the foundation or improvement. The vapor retarder contractor should have representatives onsite during the initial installation.

RETAINING WALL DESIGN PARAMETERS

General

In the following sections, GSI provides preliminary geotechnical recommendations for the design and construction of conventional concrete masonry unit (CMU) and cast-in-place concrete (CIPC) retaining walls, if incorporated into the proposed development. Recommendations for specialty walls (i.e., crib, earthstone, segmental, etc.) can be provided upon request, and would be based on the site-specific conditions.

Conventional Retaining Walls

The design parameters provided below assume that either very low expansive soils (typically Class 2 permeable filter material or Class 3 aggregate base) or native onsite earth materials with an E.I. up to 20 and a P.I. up to 14 are used to backfill any retaining wall. The type of backfill (i.e., select or native), should be specified by the wall designer, and clearly shown on the retaining wall plans. Based on our observations and laboratory testing, most of the onsite soils are not considered suitable for use as retaining wall backfill. Thus, compliance testing is recommended prior to placing the onsite soils for retaining wall backfill. In order to reduce lateral earth pressures acting upon retaining walls, select or native backfill with an E.I. up to 20 and a P.I. up to 14 should be placed above a 1:1 (h:v) plane projected up and toward the retained soils from the heel of the retaining wall foundation.

Waterproofing should be provided for proposed retaining walls in order to reduce the potential for efflorescence staining at the face. The use of a waterstop should also be considered for all concrete and masonry joints. Waterproofing should be in accordance with the recommendations provided by the project waterproofing consultant and wall designer.

Preliminary Retaining Wall Foundation Design

Preliminary foundation design for the proposed retaining walls should incorporate the following recommendations:

Minimum Footing Embedment - 24 inches below the lowest adjacent grade into tested and approved compacted fill overlying suitable, unweathered Friars Formation or metavolcanics or into suitable, unweathered Friars Formation or metavolcanics (embedment depth excludes any landscape layer [typically the upper 6 inches of soil]).

Minimum Footing Width - 24 inches.

Allowable Bearing Pressure - An allowable bearing pressure of 1,000 psf may be used in the preliminary design of the retaining wall foundation provided that the footing maintains a minimum width of 24 inches and extends at least 24 inches into tested and approved compacted fill overlying suitable, unweathered Friars Formation or metavolcanics or into suitable, unweathered Friars Formation or metavolcanics. This pressure may be increased by one-third for short-term wind and seismic loads.

Passive Earth Pressure - A passive earth pressure of 100 psf/ft with a maximum earth pressure of 1,000 psf may be used in the preliminary design of the proposed retaining wall foundation.

Lateral Sliding Resistance - Multiply the foundation contact area by a cohesion value of 130 psf, except where limited by Section 1806.3.2 of the 2022 CBC. When combining passive pressure and lateral sliding resistance, the passive pressure component should be reduced by one-third.

Backfill Soil Density - Soil densities ranging between 125 pcf and 130 pcf may be used in the preliminary design of the retaining walls. This assumes the use of a granular backfill with an average compaction of at least 90 percent of the laboratory standard (per ASTM D 1557).

Any retaining wall footings near the perimeter of the site will likely need to be deepened into suitable, unweathered Friars Formation or metavolcanics for adequate vertical and lateral bearing support. Retaining wall footing setbacks from slopes should comply with Figure 1808.7.1 of the 2019 CBC. GSI recommends a minimum horizontal setback distance of 7 feet, as measured from the bottom, outboard edge of the footing to the slope face.

Restrained Walls

Retaining walls that will be restrained prior to placing and compacting backfill material or that have re-entrant or male corners, should be designed for an at-rest equivalent fluid pressure (EFP) of 55 pcf and 65 pcf for select and very low expansive native backfill, respectively. The design should include any applicable surcharge loading. For areas of male or re-entrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall (2H) laterally from the corner.

Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls up to 10 feet high. Design parameters for walls less than 3 feet in height may be superceded by San Diego Regional Standard Design. However, the regional standard design requires that the wall backfill consist of clean sands or gravels, or mixtures of the aforementioned. Based on the onsite soil conditions, imported backfill will likely be necessary if regional standard design retaining walls are to be installed at the subject property.

Active earth pressure may be used for retaining wall design, provided the top of the wall is not restrained from minor deflections. An equivalent fluid pressure approach may be used to compute the horizontal pressure against the wall. Appropriate fluid unit weights are given below for specific slope gradients of the retained material. These do not include other superimposed loading conditions due to traffic, structures, seismic events or adverse geologic conditions. When wall configurations are finalized, the appropriate loading conditions for superimposed loads can be provided upon request.

For preliminary planning purposes, the structural consultant/wall designer should incorporate the surcharge of traffic on retaining walls if vehicular traffic could occur within a horizontal distance of “H” from the back of the retaining wall (where “H” equals the retained soil height). The traffic surcharge may be taken as 100 psf/ft in the upper 5 feet of the wall for light passenger truck and car traffic, excluding wall freeboard. For heavy axle loads (HS20), a 300 psf/ft traffic surcharge should be applied in the upper 5 feet of the wall, excluding wall freeboard. This does not include the surcharge of parked vehicles, which should be evaluated at a higher surcharge to account for the effects of seismic loading.

Equivalent fluid pressures for the design of cantilevered retaining walls are provided in the following table:

SURFACE SLOPE OF RETAINED MATERIAL (HORIZONTAL:VERTICAL)	EQUIVALENT FLUID WEIGHT P.C.F. (SELECT BACKFILL)[2]	EQUIVALENT FLUID WEIGHT P.C.F. (NATIVE BACKFILL)[3]
Level[1]	35	45
2 to 1	50	65
1.5 to 1	70	90

[1] Level backfill behind a retaining wall is defined as compacted earth materials, properly drained, without a slope for a distance of 2H behind the wall, where H is the height of the wall.
[2] SE $\geq$ 30, P.I. $\leq$ 14, E.I. $\leq$ 20, and $\leq$ 10% passing No. 200 sieve.
[3] E.I. = 0 to 20, SE $\geq$ 30, P.I. $\leq$ 14, E.I. $\leq$ 20, and $\leq$ 15% passing No. 200 sieve.

Seismic Surcharge

For retaining walls incorporated into a building, site retaining walls with more than 6 feet of retained materials (as measured vertically from the bottom of the wall footing at the heel to the ground surface), or retaining walls that could present ingress/egress constraints to emergency personnel and equipment, in the event of failure, GSI recommends that the walls be evaluated for seismic surcharge in general accordance with 2022 CBC requirements. The retaining walls in this category should maintain an overturning Factor-of-Safety (FOS) of approximately 1.1 when the seismic surcharge (seismic increment), is applied. For restrained walls, the seismic surcharge should be applied as a uniform surcharge load from the bottom of the footing (excluding shear keys) to the top of the backfill at the heel of the wall footing. For cantilevered walls, the seismic surcharge should be applied as an inverted triangular pressure distribution for the portion of the wall located above 0.6H up from the bottom of the footing to the top of the retained soils, where “H” equals the retained soil height. For the evaluation of the seismic surcharge, the bearing pressure may exceed the static value by one-third, considering the transient nature of this surcharge. This is for local wall stability only.

This seismic surcharge may be taken as 15H, where “H” equals the height of the retained soils measured vertically from the bottom of the footing to the ground surface. The 15H seismic increment is derived from the guidelines set forth in City of Los Angeles Department of Building and Safety (LADBS) Information Bulletin Document No.: P/BC 2020-83 (LADBS, 2020), which are based on Seed and Whitman (1970).

$$\gamma_{EFP\ (seismic)} = \tfrac{3}{4} k_h \gamma_{soil}$$

Where:

$\gamma_{EFP\ (seismic)}$	is the seismic increment expressed as equivalent fluid pressure (pounds per cubic foot [pcf]);
k_h	is the seismic lateral earth pressure coefficient equivalent to one-half of two-thirds of PGA_M ($0.462\ g \times \tfrac{2}{3} \times \tfrac{1}{2} = 0.155\ g$);
γ_{soil}	is the total unit weight of the retained soils (130 pcf).

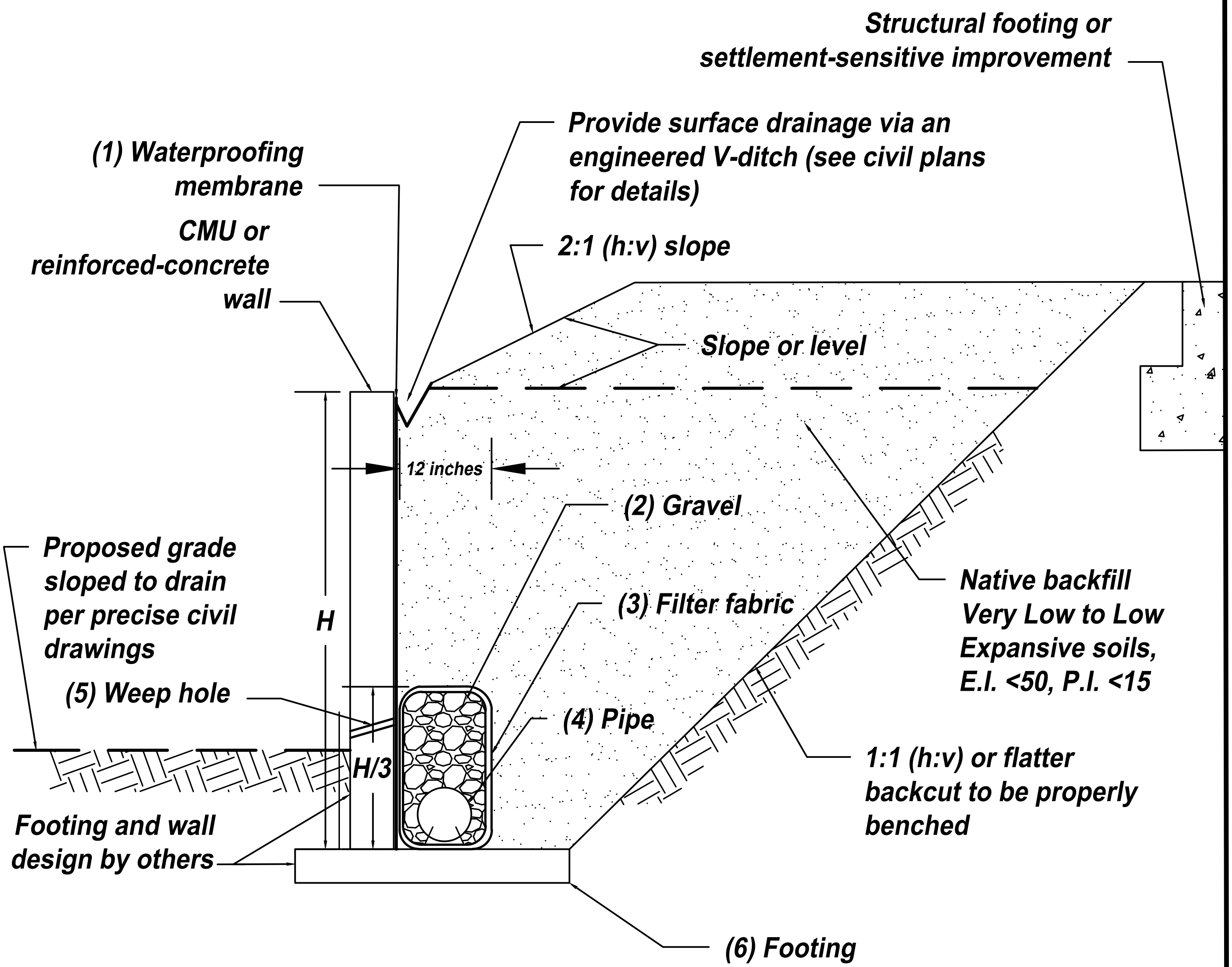
Thus, for the proposed retaining wall:

$$\gamma_{EFP\ (seismic)} = \tfrac{3}{4} \times \tfrac{1}{2} \times \tfrac{2}{3} \times 0.462 \times 130 \text{ pcf} = 15.1 \text{ pcf (use 15 pcf [15H])}$$

Retaining Wall Backfill and Drainage

Positive drainage should be provided behind all retaining walls in the form of gravel wrapped in geofabric and outlets. Otherwise, the retaining walls should be designed for hydrostatic pressure by adding the unit weight of water (62.4 pcf) to the equivalent fluid pressure. A backdrain system is considered necessary for retaining walls that are 2 feet or greater in height. Details 1, 2, and 3, present the backdrainage options discussed below. At a minimum, backdrains should consist of a 4-inch diameter perforated Schedule 40 or SDR 35 drain pipe, with perforations oriented down, encased in either Class 2 permeable filter material or ¾-inch to 1½-inch gravel wrapped in approved filter fabric (Mirafi 140N or equivalent). The backdrain should flow via gravity (minimum 1 percent slope) toward an approved drainage facility. For select backfill, the filter material should extend a minimum of 1 horizontal foot behind the base of the walls and upward at least 1 foot. For native backfill that has an E.I. up to 20 and P.I. up to 14, continuous Class 2 permeable drain materials should be used behind the wall. This material should be continuous (i.e., full height) behind the wall, and it should be constructed in accordance with the enclosed Detail 1 (Alternative "A"). For limited access and confined areas, (panel) drainage behind the wall should be constructed in accordance with Detail 2 (Alternative "B"). Materials with an expansion index (E.I.) greater than 20 and a P.I. greater than 14 should not be used as backfill for retaining walls. If the retained soils are expansive, backfill and drainage behind the retaining wall should conform with Detail 3 (Alternative "C"). Retaining wall backfill should be uniformly moisture conditioned to at least the soil's optimum moisture content, placed in relatively thin lifts, and compacted to a minimum relative density of 90 percent of the laboratory standard (ASTM D 1557).

Subdrain outlets should consist of minimum 4-inch diameter solid PVC or ABS drain pipes spaced no greater than about 100 feet apart, with a minimum of two outlets, one on each



(1) Waterproofing membrane.

(2) Gravel: Clean, crushed, ¾ to 1½ inch.

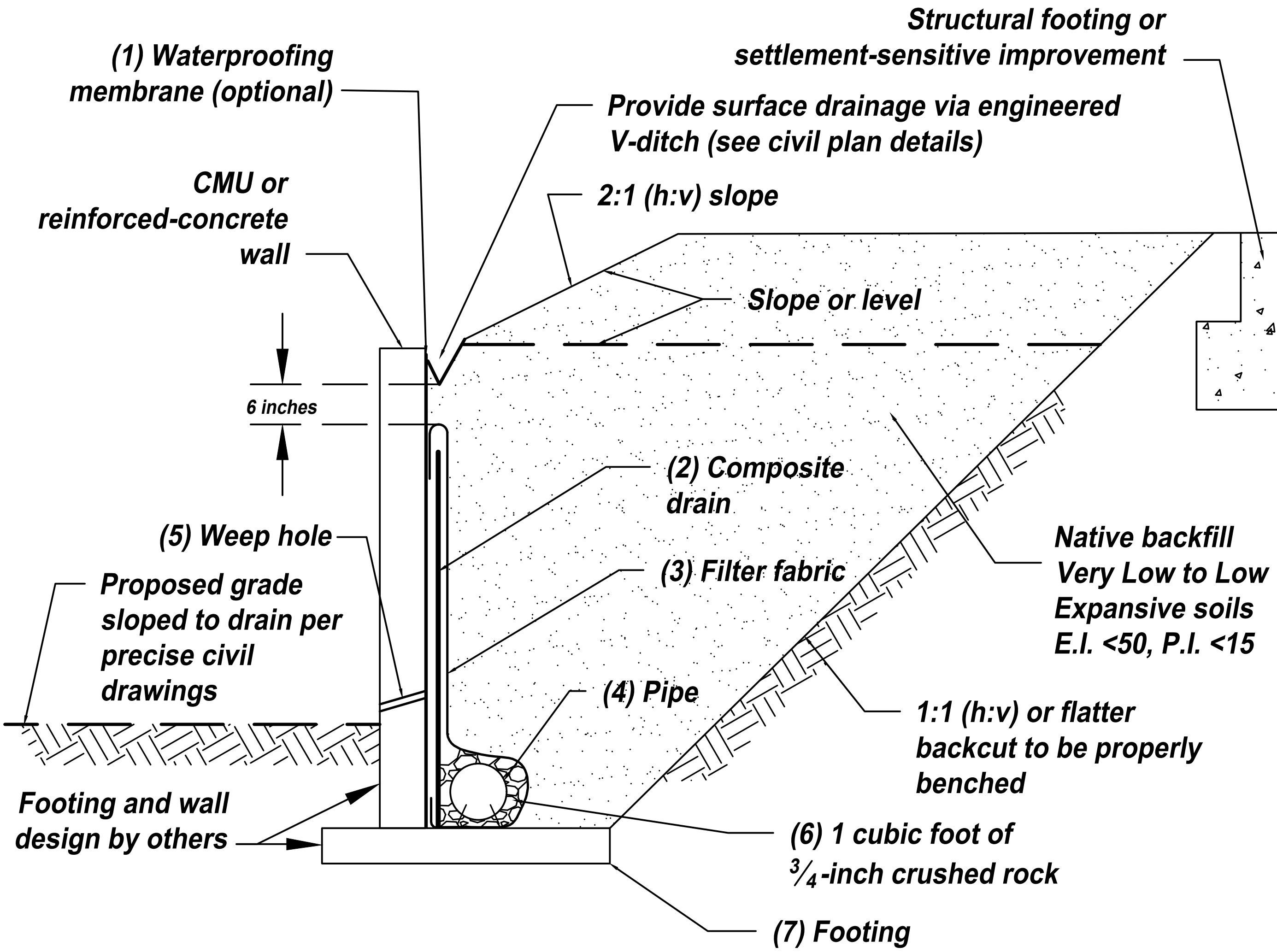
(3) Filter fabric: Mirafi 140N or approved equivalent.

(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient sloped to suitable, approved outlet point (perforations down).

(5) Weep holes: For CMU walls, Omit grout every other block, at or slightly above finished surface. For reinforced concrete walls, minimum 2-inch diameter weep holesspaced at 20 foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

(6) Footing: If bench is created behind the footing greater than the footing width using level fill or cut natural earth materials, an additional "heel" drain will likely be required by geotechnical consultant.

GeoSoils, Inc.	RETAINING WALL DETAIL — ALTERNATIVE A	Detail 1



(1) Waterproofing membrane (optional): Liquid boot or approved mastic equivalent.

(2) Drain: Miradrain 6000 or J-drain 200 or equivalent for non-waterproofed walls; Miradrain 6200 or J-drain 200 or equivalent for waterproofed walls (all perforations down).

(3) Filter fabric: Mirafi 140N or approved equivalent; place fabric flap behind core.

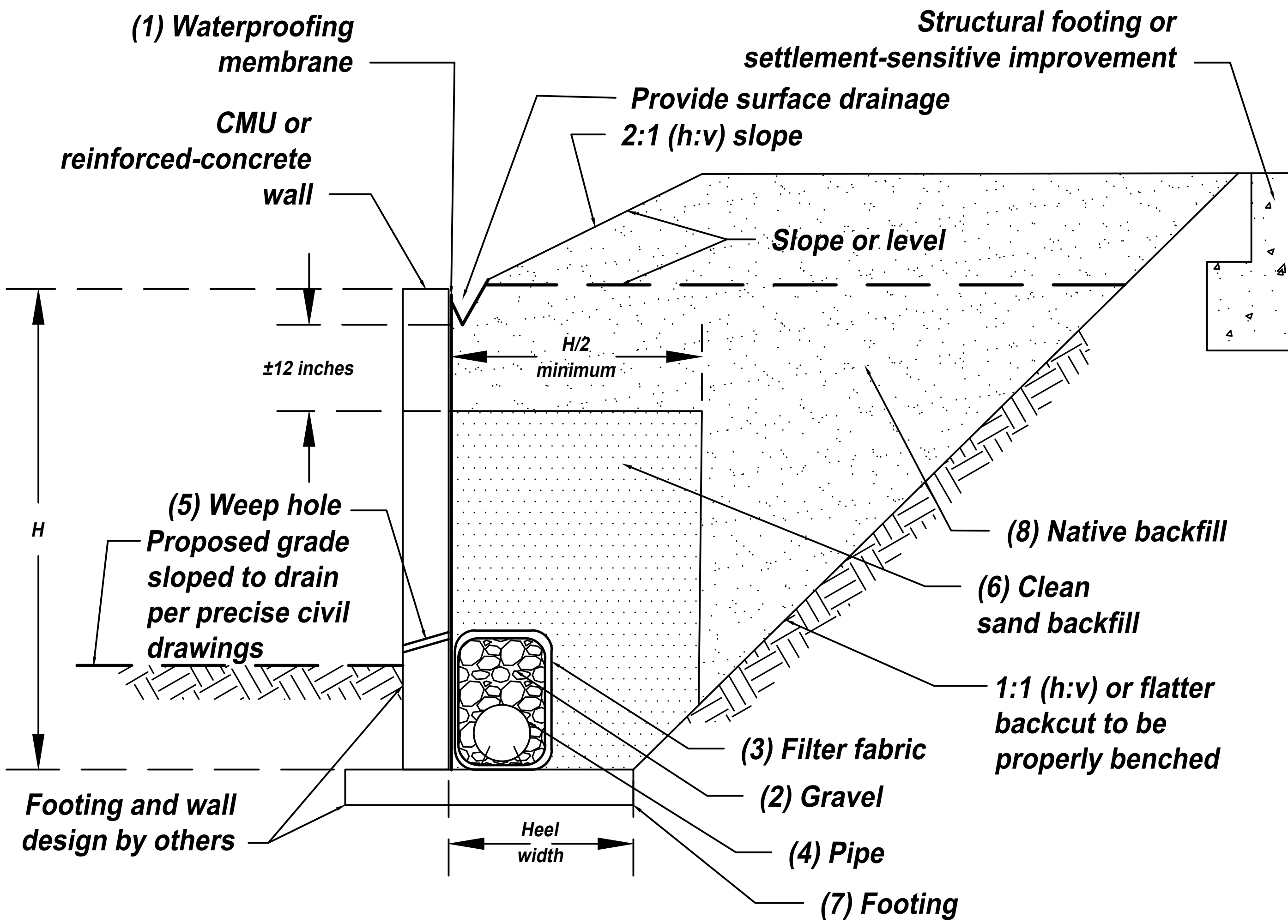
(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

(5) Weep holes: For CMU walls, Omit grout every other block, at or slightly above finished surface. For reinforced concrete walls, minimum 2-inch diameter weep holesspaced at 20 foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

(6) Gravel: Clean, crushed, ¾ to 1½ inch.

(7) Footing: If bench is created behind the footing greater than the footing width using level fill or cut natural earth materials, an additional "heel" drain will likely be required by geotechnical consultant.

GSI GeoSoils, Inc.	RETAINING WALL DETAIL — ALTERNATIVE B	Detail 2



(1) Waterproofing membrane: Liquid boot or approved masticequivalent.

(2) Gravel: Clean, crushed, $^3/_4$ to $1^1/_2$ inch.

(3) Filter fabric: Mirafi 140N or approved equivalent.

(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

(5) Weep hole: For CMU walls, Omit grout every other block, at or slightly above finished surface. For reinforced concrete walls, minimum 2-inch diameter weep holesspaced at 20 foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

(6) Clean sand backfill: Must have sand equivalent value (S.E.) of 35 or greater; can be densified by water jetting upon approval by geotechnical engineer.

(7) Footing: If bench is created behind the footing greater than the footing width using level fill or cut natural earth materials, an additional "heel" drain will likely be required by geotechnical consultant.

(8) Native backfill: If E.I. <21 and S.E. $\geq$35 then all sand requirements also may not be required and will be reviewed by the geotechnical consultant.

GeoSoils, Inc.	RETAINING WALL DETAIL — ALTERNATIVE C	Detail 3

end of the wall. The sole use of weep holes in walls higher than 2 feet, is not recommended. The surface of the backfill should be sealed by pavement or the top 18 inches of the backfill should consist of compacted native soil (E.I. ≤ 50). Proper surface drainage should also be provided.

Wall/Retaining Wall Footing Transitions

Site walls are anticipated to be supported by foundations designed in accordance with the recommendations in this report. Should wall footings transition from compacted fill to Friars Formation or metavolcanics, the wall designer may specify either:

a) The overexcavation and replacement of the Friars Formation or metavolcanics with compacted fill to at least 2 feet below the bearing surface elevation of the wall footing and for a minimum distance of 2H, from the point of transition, where "H" equals the height of the wall.

b) Increase of the amount of reinforcing steel and wall detailing (i.e., expansion joints or crack control joints) such that an angular distortion of 1/360 for a distance of 2H on either side of the transition may be accommodated. Expansion joints should be placed no greater than 20 feet on-center, in accordance with the structural engineer's/wall designer's recommendations, regardless of whether or not transition conditions exist. Expansion joints should be sealed with a flexible, non-shrink grout.

c) Embed the footings entirely into suitable, unweathered Friars Formation or metavolcanics (i.e., deepened footings).

If transitions from compacted fill to Friars Formation or metavolcanics transect the wall footing alignment at an angle of less than 45 degrees (plan view), then the wall designer should follow recommendation "a" (above) and until such transition is between 45 and 90 degrees to the wall alignment.

TOP-OF-SLOPE WALLS/FENCES/IMPROVEMENTS AND EXPANSIVE SOILS

Expansive Soils and Slope Creep

Most of the onsite soils are expansive (i.e., E.I. $\geq$ 21 and P.I. $\geq$ 15); and therefore, become desiccated when allowed to dry. Such soils are susceptible to surficial slope creep, especially with seasonal changes in moisture content. Typically in southern California, during the hot and dry summer period, these soils become desiccated and shrink, thereby developing surface cracks. The extent and depth of these desiccation cracks depend on many factors such as the nature and the expansion potential of the soils, temperature and humidity, and extraction of moisture from surface soils by plants and roots. When seasonal rains occur, water percolates into the cracks and fissures, causing slope surfaces to expand, with a corresponding loss in soil density and shear strength near the slope surface. With the passage of time and several moisture cycles, the outer 3 to 5 feet of slope materials experience a very slow, but progressive, outward and downward movement, known as slope creep. For slope heights greater than 10 feet, this creep-related soil movement will typically affect all flatwork and other secondary improvements that are located within about 15 feet from the top of slopes, such as exterior concrete slabs-on-grade, swimming pools, spas, etc., and in particular top-of-slope fences/walls. This influence is normally in the form of detrimental settlement, and tilting of the proposed improvements. The desiccation/swelling and creep discussed above continues over the life of the improvements, and generally becomes progressively worse. Accordingly, this information should be provided to all interested/affected parties.

Top-of-Slope Walls/Fences

Due to the potential for slope creep along slopes higher than about 10 feet, some settlement and tilting of the top-of-slope walls/fences with corresponding distress should be expected. To mitigate the tilting of top-of-slope walls/fences, we recommend that the walls/fences be constructed on a combination of grade beam and caisson foundations. At a minimum, the grade beam should be 12 inches by 12 inches in cross section, supported by drilled caissons, a minimum of 12 inches in diameter, placed at a minimum spacing of 3 diameters on-center, and with a minimum embedment length of 5 feet into tested and approved compacted fill or suitable, unweathered Friars Formation or metavolcanics. The design of the grade beam and caissons should be in accordance with the recommendations of the project structural engineer, and consider the following geotechnical parameters:

Creep Zone: 5-foot vertical zone below the slope face and projected upward parallel to the slope face.

Creep Load: The creep load projected on the area of the grade beam should be taken as an equivalent fluid approach, having a density of 60 pcf. For the caisson, it should be taken as a uniform pressure of 900 pounds per foot of the caisson's depth, located within the creep zone.

Point of Fixity: Located at a depth of 1.5 times the caisson's diameter, below the creep zone.

Passive Resistance: Passive earth pressure of 250 psf per foot of depth per foot of caisson diameter, to a maximum value of 2,500 psf may be used to determine caisson depth and spacing, provided that they meet or exceed the minimum requirements stated above. To determine the total lateral resistance, the contribution of the creep-prone zone above the point of fixity, to passive resistance, should be disregarded.

Allowable Axial Capacity:

Shaft capacity : 50 psf applied below the point of fixity (in approved compacted fill) and 250 psf applied below the point of fixity (in unweathered Friars Formation or metavolcanics) over the surface area of the shaft.

Tip capacity: 1,500 psf, 3,000 psf, and 6,000 psf for a caisson bearing on approved compacted fill, suitable, unweathered Friars Formation, and suitable, unweathered metavolcanics, respectively (tip capacity assumes that the bottom of the drilled excavation is clear of loose soil).

PRELIMINARY ASPHALTIC CONCRETE OVER AGGREGATE BASE (AC/AB) PAVEMENT DESIGN/CONSTRUCTION

In order to evaluate the preliminary design of the structural section for asphaltic concrete pavements, GSI has assumed a Traffic Index (T.I.) of 4.5 for the proposed driveway. Owing to the fine-grained nature of most of the onsite soils, it is our opinion that a subgrade R-value of 5 is appropriate for preliminary design purposes. Preliminary recommendations for the structural section of an asphaltic concrete driveway relative to the assumed T.I. and R-value are provided in the table below. The actual T.I. for the driveway should be evaluated and confirmed by the project civil engineer or traffic engineer. Final pavement design should be based on the R-value test results of the soils located near the pavement subgrade, following grading and underground utility trench backfill.

VEHICULAR TRAFFIC AREA	TRAFFIC INDEX (T.I.)[1]	STANDARD PAVEMENT DESIGN		
		R-VALUE[2]	AC INCHES	CLASS 2 AGGREGATE BASE[3] INCHES
Driveway	4.5	5	3.0	8.0
Driveway (Alternative Section)	4.5	5	4.0	6.0

1 - Assumed T.I. To be confirmed by the project civil engineer or traffic engineer.
2 - Assumed subgrade R-value to be re-evaluated at the conclusion of grading and underground utility backfill.
3 - Assumed R-value for Class 2 aggregate base R=78 - Caltrans standard Class 2 Aggregate Base.

PRELIMINARY VEHICULAR PORTLAND CEMENT CONCRETE PAVEMENT (PCCP) DESIGN

Preliminary recommendations for the design of vehicular PCCP are provided in the table below.

PORTLAND CONCRETE CEMENT PAVEMENTS (PCCP)					
TRAFFIC AREAS	**CONCRETE TYPE**	**PCCP THICKNESS (inches)**	**TRAFFIC AREAS**	**CONCRETE TYPE**	**PCCP THICKNESS (inches)**
Light Vehicles	520-C-2500	7.0	Heavy Truck Traffic	520-C-2500	9.0
	560-C-3250	6.0		560-C-3250	8.0
NOTE: All PCCP is designed as un-reinforced and bearing directly on compacted subgrade. However, a 6-inch thick layer of compacted Class 2 aggregate base may be considered for increased performance. All PCCP should be properly detailed (jointing, etc.) per the industry standard. Pavements may be additionally reinforced with #4 reinforcing bars, placed 12 inches on center, each way, for improved performance. Trash truck loading pads (aprons) shall adhere to the City of San Diego's minimum thickness and detailing.					

OTHER CONSIDERATIONS REGARDING VEHICULAR PAVEMENT DESIGN

The recommended pavement sections provided above are intended as minimum guidelines. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. If the ADT (average daily traffic) or ADTT (average daily truck traffic) increases beyond that intended, as reflected by the T.I. used for design, increased maintenance and repair could be required for the pavement section. Consideration should be given to the increased potential for distress from overuse of paved street areas by heavy equipment or construction-related heavy traffic (e.g., telehandlers, concrete trucks, loaded supply trucks, etc.). Best management construction practices should be followed at all times, especially during inclement weather.

VEHICULAR PAVEMENT SECTION CONSTRUCTION

General

The following recommendations should be incorporated into the construction of vehicular pavements:

Pavement Subgrade

The recommended remedial grading should occur within the vehicular pavement areas prior to subgrade preparation. The pavement subgrade should be free of any loose materials, scarified at least 6 to 8 inches, uniformly moisture conditioned to the soil's optimum moisture content, and then compacted to a minimum relative density of 95 percent of the laboratory standard (per ASTM D 1557). The pavement subgrade should be proof-rolled under the observation of the geotechnical consultant prior to placing the Class 2 aggregate base. Field density tests should be performed during the compaction of the pavement subgrade.

Class 2 Aggregate Base

The Class 2 aggregate base should be placed in lifts not exceeding 6 inches, uniformly moisture conditioned to at least optimum moisture content, and compacted to a minimum relative density of 95 percent of the laboratory standard (per ASTM D 1557). Field density tests should be performed during the compaction of the aggregate base layer. Base aggregate should be in accordance to the Caltrans or "Greenbook" specifications for Class 2 base rock (minimum R-value=78).

Asphaltic Concrete

Asphaltic concrete paving should conform to the standards in Section 302-5 of the 2021 "Greenbook" (BNI Publications, Inc., 2021). Geotechnical observations and field density testing should be conducted during asphaltic concrete paving. The asphaltic concrete should be compacted to a minimum of 95 percent of the density obtained on samples tested in accordance with California Test Methods 304 and 308, Method "A." Method "C" may be used if the absorption of the compacted specimen is less than 2 percent.

Prime coat may be omitted if all of the following conditions are met:

1. The asphaltic concrete pavement layer is placed within two weeks of completion of the aggregate base course.

2. Traffic is not routed over the completed aggregate base course before paving.

3. Construction is completed during the dry season of May through October.

4. The aggregate base is kept free of debris prior to placement of the asphaltic concrete.

If construction is performed during the wet season of November through April, prime coat may be omitted if no rain occurs between completion of the aggregate base course and paving, <u>and</u> the time between completion of the base and paving is reduced to three (3) days, provided the aggregate base is free of loose soil or debris. Where prime coat has been omitted and rain occurs, traffic is routed over the aggregate base course, or paving is delayed, measures shall be taken to restore the base course, and the subgrade to conditions that will meet specifications as directed by the County of San Diego or recommended by the geotechnical consultant.

PRELIMINARY OUTDOOR POOL/SPA DESIGN RECOMMENDATIONS

The following preliminary recommendations are provided for consideration in outdoor swimming pool/spa design and planning. Recommendations for a swimming pool, spa, and associated deck flatwork in areas with differential settlements exceeding ¼ inch over 40 feet horizontally, will be more conservative than the preliminary recommendations presented below. If segmental retaining walls will be included with the project, the proposed swimming pool/spa and associated deck flatwork should be located below a 1:1 (h:v) plane projected up and toward the retained soils from the heel of the wall facing and the heel of the geogrid-reinforced backfill, owing to strain incompatibilities between these improvements.

General

1. Owing to the expansive nature of most of the onsite earth materials, it is recommended that the entire bottoms and sides of the pool/spa shells and their foundations be surrounded by at least 5 feet of tested and approved compacted fill that is very low in expansion potential (E.I. of 20 or less and P.I. of 14 or less) overlying suitable unweathered Friars Formation or metavolcanics. Pool deck slabs-on-grade should be underlain by a similar compacted fill mat that extends at least 3 feet below the slab subgrade and at least 3 feet horizontally outside the outboard edges of the deck slab. Should a transition from compacted fill to unweathered Friars Formation or metavolcanics occur beneath the pool/spa shells, the Friars Formation or metavolcanics should be overexcavated to a depth of at least 5 feet below the shell and be replaced with compacted fill that is very low in expansion potential. If the maximum depth of compacted fill beneath the pool/spa shell exceeds 15 feet, additional bedrock overexcavation should be performed such that the maximum to minimum fill thickness below the shell does not exceed a ratio of 3:1 (maximum:minimum). The bottom of the overexcavation should be sloped away from the pool/spa area.

2. An allowable bearing value of 1,500 psf may be assumed for continuous footings, a minimum of 12 inches wide and embedded at least 12 inches below the lowest adjacent grade into tested and approved compacted fill overlying suitable unweathered Friars Formation or metavolcanics. Footing embedment excludes soft soils, landscape zones, slab and underlayment sections, etc. The compacted fill material should be very low in expansion potential (E.I. of 20 or less and P.I. of 14 or less).

3. The equivalent fluid pressure to be used for the swimming pool/spa design should be 60 pcf for pool/spa walls with level backfill, and 75 pcf for 2:1 (h:v) sloping backfill conditions. In addition, backdrains should be provided behind pool/spa walls subjacent to slopes. Alternatively, the pool/spa walls may be designed for full hydrostatic pressure by adding 62.4 pcf to the equivalent fluid pressures recommended above for drained conditions.

4. If laterally supported by compacted fill that is very low in expansion potential, the passive earth pressure used in the design of the pool/spa foundations may be

computed as an equivalent fluid having a density of 250 pcf, with a maximum lateral earth pressure of 2,500 pounds per square foot (psf).

5. An allowable coefficient of friction between very low expansive soil and concrete of 0.35 may be used with the dead load forces.

6. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

7. Where pools/spas are planned near structures, appropriate surcharge loads need to be incorporated into design and construction by the pool/spa designer. This includes, but is not limited to landscape berms, decorative walls, footings, built-in barbeques, utility poles, etc.

8. All pool/spa walls should be designed as "free standing" and be capable of supporting the water in the pool/spa without soil support. The shape of the pool/spa in cross section and plan view may affect the performance of the pool/spa, from a geotechnical standpoint. The pool and spa should also be designed in accordance with the latest adopted Code. The bottom of the pool/spa, should maintain a distance of H/3, where H is the height of the slope (in feet), from the slope face. This distance should not be less than 7 feet, nor need not be greater than 40 feet.

9. Hydrostatic pressure relief valves should be incorporated into the pool and spa designs.

10. All fittings and pipe joints, particularly fittings in the side of the pool/spa, should be properly sealed to prevent water from leaking into the adjacent soils materials, and be fitted with slip or expandible joints between connections transecting varying soil conditions.

11. An elastic expansion joint (flexible waterproof sealant) should be installed to reduce the potential for water to infiltrate into the soil at all deck joints.

12. A reinforced grade beam should be placed around skimmer inlets to provide support and mitigate cracking around the skimmer face.

13. In order to reduce unsightly cracking, deck slabs should be a minimum of 4 inches thick, and be reinforced with No. 3 steel reinforcing bars at 18 inches on-center, in two perpendicular directions. All slab reinforcement should be supported by chairs to ensure proper mid-slab positioning during the placement of concrete. "Hooking" of the steel reinforcement is not recommended. Wire mesh reinforcing should not be used. Deck slabs should not be tied to the pool/spa structures. The deck subgrade should be lightly moisturized immediately prior to concrete placement to

promote uniform curing and to reduce the potential for the loss of concrete moisture following placement. For increased performance, the deck slab underlayment should consist of a 1- to 2-inch thick leveling course of sand (S.E. > 30) and a minimum of 4 to 6 inches of Class 2 aggregate base compacted to a minimum relative density of 90 percent of the laboratory standard. Deck slabs within the H/3 zone, where H is the height of the slope (in feet), will have an increased potential for distress relative to other areas outside of the H/3 zone. If distress is undesirable, improvements, deck slabs or flatwork should not be constructed closer than H/3 or 7 feet (whichever is greater) from the slope face, in order to reduce, but not eliminate, this potential.

14. In order to reduce unsightly cracking, the outer edges of the pool/spa deck slab that is bordered by landscaping, and the edges immediately adjacent to the pool/spa, should incorporate an 8-inch wide concrete cutoff barrier (thickened edge) extending to a depth of at least 12 inches below the bottoms of the deck slab to mitigate excessive lateral migration of water under the pool/spa deck slab. These thickened edges should be reinforced with two No. 4 bars, with one bar near the top and one bar near the bottom of the thickened edges.

15. Surface and shrinkage cracking of the finished pool/spa shells and deck slab may be reduced if the shotcrete/concrete has a low slump and water-to-cement ratio that are maintained during placement. Concrete used should have a minimum compressive strength of 4,000 psi. Excessive water added to concrete prior to placement is likely to cause shrinkage cracking, and should be avoided. Some shrinkage cracking, however, is unavoidable.

16. Joint and sawcut locations for the pool/spa deck slab should be determined by the design engineer and contractor. However, spacings should not exceed 6 feet on center.

17. Considering the nature of the onsite earth materials, caving or sloughing could be a factor in subsurface excavations and trenching. Shoring or excavating the trench walls/backcuts at the angle of repose (typically 25 to 45 degrees), should be anticipated. <u>All</u> excavations should be observed by a licensed engineering geologist or engineer, prior to workers entering the excavation or trench, and <u>minimally</u> conform to the recommendations for temporary slopes previously provided in this report, as well as CAL/OSHA and local safety codes. Should adverse conditions exist, appropriate recommendations should be offered at that time by the geotechnical consultant.

18. It is imperative that adequate provisions for surface drainage are incorporated by the homeowner(s) into their overall improvement scheme. Positive surface drainage should be maintained over the life of the proposed development. Ponding water, ground saturation and flow over slope faces, are all situations which must be

avoided to enhance long-term performance of the pool/spa and associated improvements, and reduce the likelihood of distress.

19. If the pool/spa ever require emptying, it should be done in accordance with the recommendations of the pool/spa designer.

20. The temperature of the water lines for spas and pools may affect the corrosion properties of the onsite soils. Thus, a corrosion specialist should be retained to review all spa and pool plans, and provide mitigative recommendations, as warranted. Concrete mix design should be reviewed by a qualified corrosion consultant and materials engineer.

21. All pool/spa underground utility trenches should be compacted to at least 90 percent of the laboratory standard, under the full-time observation and field density testing of the geotechnical consultant. Underground utility trench bottoms should be sloped away from the primary structures on the property (typically the residential structure).

22. Pool and spa underground utility lines should not cross the primary structure's underground utility lines (i.e., not stacked, or sharing of trenches, etc.).

23. The pool/spa or associated underground utilities should not intercept, interrupt, or otherwise adversely impact any area drain, roof drain, or other drainage conveyances. If it is necessary to modify, move, or disrupt existing area drains, subdrains, or tightlines, then the design civil engineer should be consulted, and mitigative measures provided. Such measures should be further reviewed and approved by the geotechnical consultant, prior to proceeding with any further construction.

24. The geotechnical consultant should review and approve all aspects of the pool/spa and flatwork designs prior to construction. A design civil engineer should review all aspects of such design, including drainage and setback conditions. Prior to acceptance of the pool/spa construction, the project builder, geotechnical consultant and civil designer should evaluate the performance of the area drains and other site drainage pipes, following pool/spa construction.

25. All aspects of construction should be reviewed and approved by the geotechnical consultant, including during excavation, prior to the placement of any additional fill, and prior to the placement of any steel reinforcement and concrete.

26. Any changes in the design or location of the pool/spa should be reviewed and approved by the geotechnical and design civil engineer prior to construction. Field adjustments should not be allowed until written approval of the proposed field changes are obtained from the geotechnical and design civil engineer.

27. Disclosure should be made to all builders, contractors, and any interested/affected parties, that pools/spas built within about 15 feet of the top of a slope, and H/3, where “H” is the height of the slope will experience some movement or tilting. While the pool/spa shell or coping may not necessarily crack, the levelness of the pool/spa will likely tilt toward the slope, and may not be aesthetically pleasing. The same is true with decking, flatwork and other improvements in this zone.

28. Failure to adhere to the above recommendations will significantly increase the potential for distress to the pool/spa, flatwork, etc.

29. Local seismicity or the design earthquake will cause some distress to the pool/spa and decking or flatwork, possibly including total functional and economic loss.

30. The information and recommendations discussed above should be provided to any contractors and subcontractors, or homeowner(s), interested/affected parties, etc., that may perform or may be affected by such work.

FLATWORK AND OTHER IMPROVEMENTS

Most of the near-surface onsite earth materials exhibit expansive characteristics. The effects of expansive soils are cumulative, and typically occur over the lifetime of any improvement. On relatively level areas, when the soils are allowed to dry, the desiccation and swelling process tends to cause heaving and distress to flatwork and other improvements. The resulting potential for distress to improvements may be reduced, but not totally eliminated. To that end, it is recommended that the long-term potential for distress be communicated to any interested/affected parties. To reduce the likelihood of distress, the following recommendations are presented for all exterior concrete flatwork:

1. Please refer to the preceding “Preliminary Outdoor Pool/Spa Design Recommendations” section for the design and construction of pool/spa deck slabs-on-grade.

2. Exterior concrete slabs-on-grade should be cast entirely on properly compacted fill materials that have been tested and approved by the geotechnical consultant or on suitable, unweathered Friars Formation, or suitable, unweathered metavolcanics. The subgrade area for the concrete slabs should be compacted to achieve a minimum 90 percent relative compaction (per ASTM D 1557), and then be presoaked to 2 to 3 percent above the soils’ optimum moisture content to a depth of 18 inches below the subgrade. This moisture content should be maintained in the subgrade soils during concrete placement to promote uniform curing of the concrete and to reduce the development of unsightly shrinkage cracks. Although unlikely without selective grading, if very low expansive soils (E.I. $\leq$ 20 and P.I. < 14) are present in the upper 3 feet of the subgrade, only optimum moisture

content, or greater is required and specific presoaking is not warranted. The moisture content of the subgrade should be tested within 72 hours prior to concrete placement.

3. The exterior concrete slabs-on-grade should be cast over a non-yielding surface consisting of a 4-inch layer of crushed rock, gravel, or clean sand that should be compacted and level prior to placing concrete. Although unlikely, if very low expansive soils are present, the rock, gravel, or sand may be omitted. This layer or the subgrade should be moisturized completely prior to placing concrete, to reduce the loss of concrete moisture to the surrounding earth materials.

4. Exterior concrete slabs-on-grade that will receive pedestrian traffic should be a minimum of 4 inches thick. Driveway approach slabs or other concrete slabs, adjacent to landscape areas, that will receive vehicular traffic should include a thickened edge extending at least 12 inches below the subgrade to help impede the lateral migration of landscape water under the slab.

5. The use of transverse and longitudinal control joints are recommended to help control slab cracking due to concrete shrinkage or expansion. Two ways to mitigate such cracking are: a) add a sufficient amount of reinforcing steel, increasing tensile strength of the slab; and b) provide an adequate amount of control or expansion joints to accommodate the anticipated concrete shrinkage and expansion.

   In order to reduce the potential for unsightly cracks, exterior concrete slabs-on-grade should be reinforced at mid-height with a minimum of No. 3 bars placed at 18 inches on center, in each direction. If subgrade soils within the top 7 feet from the subgrade are very low in expansion potential (i.e., E.I. $\leq$ 20, P.I. $\leq$ 14), then 6x6-W1.4xW1.4 welded-wire mesh may be substituted for the rebar, provided the reinforcement is placed on chairs, at slab mid-height. The exterior slabs should be scored or saw cut, ½ to ⅜ inches deep, often enough so that no section is greater than 10 feet by 10 feet. For sidewalks or narrow slabs, control joints should be provided at intervals of every 5 feet. The slabs should be separated from the foundations and sidewalks with expansion joint filler material.

6. No traffic should be allowed upon the new concrete slabs until they have been properly cured to within 75 percent of design strength. Concrete compression strength for concrete slabs that will only receive pedestrian traffic should be a minimum of 2,500 psi.

7. Driveways, sidewalks, and patio slabs adjacent to the proposed building should be separated from the structure with thick expansion joint filler material. In areas directly adjacent to a continuous source of moisture (i.e., irrigation, planters, etc.), all joints should be additionally sealed with flexible mastic.

8. Planters and walls should not be structurally tied to the proposed building.

9. Overhang structures should be supported on the slabs, or structurally designed with continuous footings tied to the perimeter building foundation(s) in at least two directions.

10. Any masonry landscape or site retaining walls that are to be constructed throughout the property should be grouted and articulated in segments no more than 20 feet long. These segments should be keyed or doweled together.

11. Positive site drainage should be maintained at all times. Finish grade on the property should provide a minimum of 1 to 2 percent fall to approved drainage facilities identified by the project civil engineer, as indicated herein or conform to Section 1804.3 of the 2022 CBC (whichever is more conservative). It should be kept in mind that drainage reversals could occur, including post-construction settlement, if relatively flat yard drainage gradients are not periodically maintained by the property owner. This should be disclosed to all interested/affected parties.

12. Air conditioning (A/C) units should be supported by concrete slabs that are incorporated into the building foundation or constructed on a rigid slab with flexible couplings for plumbing and electrical lines. A/C waste water lines should be drained to a suitable non-erodible outlet.

13. Shrinkage cracks could become excessive if proper finishing and curing practices are not followed. Finishing and curing practices should be performed per the Portland Cement Association Guidelines. Mix design should incorporate rate of curing for climate and time of year, sulfate content of soils, corrosion potential of soils, and fertilizers used on site.

STRUCTURAL CONCRETE MIX DESIGN

The project architect, structural engineer, and civil engineer should review the results of the corrosion tests provided in the “Laboratory Testing” section of this report and specify the appropriate mix design for structural concrete on their respective plans.

PERMANENT POST-CONSTRUCTION STORM WATER BEST MANAGEMENT PRACTICES

As previously indicated herein, infiltration of storm water into the onsite soils for permanent post-construction storm water BMPs is not recommended from a geotechnical perspective. Since storm water infiltration into the onsite soils is not advised, any proposed permanent post-construction storm water BMP should consist of a fully contained system or storm water filtration, or detention basins should receive an impermeable liner and an under-drain system.

Impermeable liners used in conjunction with storm water basins should consist of a 30-mil polyvinyl chloride (PVC) membrane that is covered by a minimum of 12 inches of clean soil, free from rocks and debris. The impermeable liner should extend a few inches above the 100-year flood elevation (Q_{100} elevation). In addition, the design and construction of storm water basins should consider the following:

1. The 30-mil impermeable liner should have the following minimum engineering properties:

   Specific Gravity (ASTM D792): 1.2 (g/cc, min.); Tensile (ASTM D882): 73 (lb/in-width, min); Elongation at Break (ASTM D882): 380 (%, min); Modulus (ASTM D882): 30 (lb/in-width, min.); and Tear Strength (ASTM D1004): 8 (lb/in, min); Seam Shear Strength (ASTM D882) 58.4 (lb/in, min); Seam Peel Strength (ASTM D882) 15 (lb/in, min).

2. Subdrains for the under-drain system should consist of a minimum 4-inch diameter Schedule 40 or SDR 35 perforated drain pipe with the perforations oriented down. The drain pipe should be sleeved with filter sock or wrapped in filter fabric (Mirafi 140N or approved equivalent).

3. Areas adjacent to, or within, the storm water basins that are subject to inundation should be properly protected against scouring, undermining, and erosion, in accordance with the recommendations of the design engineer.

4. Long-term stability of the basin slopes will require them to be constructed at gradients no steeper than 3:1 (h:v). Alternatively, the sides of the basin may be supported by retaining structures/walls designed for the appropriate earth and hydrostatic pressures. Footings for the retaining walls should extend at least 2 feet below the bottom of the basin into earth materials deemed suitable for bearing by the project geotechnical consultant. Refer to the “Retaining Wall Design Parameters” section of this report for other geotechnical recommendations for the design and construction of retaining walls.

5. Due to the potential for piping and adverse seepage conditions, a burrowing rodent control program should also be implemented onsite.

6. Any trenches for inlet/outlet piping or other subsurface utilities, located within or near the proposed basins may become saturated and induce backfill settlement. This is due to the potential for piping, water migration, or seepage along the trench line backfill. Underground utility trenches, excluding underdrains, adjacent to and within basins, should be backfilled with a 1-sack sand-cement slurry.

7. Separation geotextiles or slurry backfill should be used to reduce the potential for the piping of fine soil particles into open-graded gravel backfill layers in the trenches.

8. The use of storm water basins above or near existing or planned underground utilities that might degrade/corrode with the introduction of water/seepage should be avoided. Alternatively, a corrosion consultant may provide recommendations for corrosion protection.

9. Basins should not occur below a 1:1 (h:v) plane projected down and away from foundations or within 10 horizontal feet from the proposed building or site retaining walls.

10. Basins should not occur within 50 feet of the tops and toes of slopes.

DEVELOPMENT CRITERIA

Slope Deformation

Compacted fill slopes designed using customary factors of safety for gross or surficial stability and constructed in general accordance with the design specifications should be expected to undergo some differential vertical heave or settlement in combination with differential lateral movement in the out-of-slope direction, after grading. This post-construction movement occurs in two forms: slope creep, and lateral fill extension (LFE). Slope creep is caused by the alternating wetting and drying of the fill soils which results in slow downslope movement. This type of movement is expected to occur throughout the life of the slope, and is anticipated to potentially affect improvements or structures (e.g., separations or cracking), placed near the tops of slopes, up to a maximum distance of approximately 15 feet from the tops of slopes, depending on the slope height. This movement generally results in rotation and differential settlement of improvements located within the creep zone. LFE occurs due to deep wetting from irrigation and rainfall on slopes composed of expansive materials. Although some movement should be expected, long-term movement from this source may be reduced, but not eliminated, by placing the fill throughout the slope region, wet of the fill's optimum moisture content.

It is generally not practical to attempt to eliminate the effects of either slope creep or LFE. Suitable mitigative measures to reduce the potential for lateral deformation typically include: setback of improvements from the slope faces (per the 2022 CBC), positive structural separations (i.e., joints) between improvements, and stiffening and deepening of foundations. Expansion joints in walls should be placed no greater than 20 feet on-center, and in accordance with the structural engineer's recommendations. All of these measures are recommended for the design of structures and improvements. The ramifications of the above conditions, and recommendations for mitigation, should be provided to all interested/affected parties.

Slope Maintenance and Planting

Water has been shown to weaken the inherent strength of all earth materials. Slope stability is significantly reduced by overly wet conditions. Positive surface drainage away from slopes should be maintained and only the amount of irrigation necessary to sustain plant life should be provided for slopes. Over-watering should be avoided as it adversely affects site improvements, and causes perched groundwater conditions. Graded slopes constructed of the onsite materials would be erodible. Eroded debris may be reduced and surficial slope stability enhanced by establishing and maintaining a suitable vegetation cover soon after construction. Compaction to the face of fill slopes would tend to reduce short-term erosion until vegetation is established. Plants selected for landscaping should be lightweight, deep rooted types that require little water and are capable of surviving the prevailing climate. Jute-type matting or other fibrous covers may aid in the establishment of a sparse plant cover. Using plants other than those recommended above will increase the potential for perched water, staining, mold, etc. to develop. A rodent control program to prevent burrowing should be implemented. Irrigation of natural (ungraded) slope areas is generally not recommended. These recommendations regarding plant type, irrigation practices, and rodent control should be provided to all interested/affected parties. Over-steepening of slopes should be avoided during building construction activities and landscaping practices.

Drainage

Adequate surface drainage within the site is a very important factor in reducing the likelihood of adverse performance of improvements. Surface drainage should be sufficient to prevent ponding of water anywhere on the building pad, and especially near the residential structure. Pad surface drainage should be carefully taken into consideration during fine grading, landscaping, and building construction. Therefore, care should be taken that future landscaping or construction activities do not create adverse drainage conditions. Positive site drainage should be provided and maintained at all times. Water should be directed away from foundations and tops of slopes. Consideration should be given to avoiding the construction of planters adjacent to the residential structure and pavements. Building pad drainage should be directed toward approved drainage facilities identified by the project civil engineer. Roof gutter downspouts, or drainage devices should outlet a minimum of 5 feet from the residential structure or into a subsurface drainage system. Areas of seepage may develop due to irrigation or heavy rainfall, and should be anticipated. Minimizing irrigation will lessen this potential. If areas of seepage develop, recommendations for reducing this effect could be provided upon request.

Erosion Control

Onsite earth materials are erodible. Consideration should be given to providing hay bales and silt fences for the temporary control of surface water, from a geotechnical viewpoint.

Landscape Maintenance and Open-Bottom Planter Location

We recommend that any proposed open-bottom planters adjacent to the proposed residential structure be eliminated for a minimum distance of 10 feet. As an alternative, closed-bottom type planters could be used. An outlet placed in the bottom of the planter, could be installed to direct drainage away from the structure or any exterior concrete flatwork.

If planters are constructed adjacent to the residential structure, the sides and bottom of the planters should be provided with a moisture barrier to prevent infiltration of irrigation water into the subgrade. Provisions should be made to drain the excess irrigation water from the planters without saturating the subgrade below or adjacent to the planters. Consideration should be given to the type of vegetation chosen and their potential effect upon surface improvements (i.e., some trees will have an effect on concrete flatwork with their extensive root systems). From a geotechnical standpoint leaching is not recommended for establishing landscaping.

Subsurface and Surface Water

Subsurface and surface water are not anticipated to affect site development, provided that the recommendations contained in this report are incorporated into final design and construction and that prudent surface and subsurface drainage practices are incorporated into the construction plans and implemented over the life of the project. Perched groundwater conditions along zones of contrasting permeabilities may not be precluded from occurring in the future due to meteorological or climatic factors, site irrigation, poor drainage conditions, or damaged underground utilities, and should be anticipated. Should perched groundwater conditions develop, this office could assess the affected area(s) and provide the appropriate recommendations for mitigation. Groundwater conditions may change with the introduction of irrigation, rainfall, or other factors.

Site Improvements

If any additional improvements (e.g., outbuildings, walls, hardscape, etc.) are planned for the site, recommendations concerning the geological or geotechnical aspects of the design and construction of said improvements could be provided upon request. This office should be notified in advance of any fill placement, grading of the site, or trench backfilling after rough grading has been completed. This includes any grading, foundation construction, the placement of underground utility trench and retaining wall backfills, flatwork construction, etc. This information should be provided to all interested/affected parties.

Foundation Excavations

All foundation excavations should be observed by a representative of this firm after excavation and prior to concrete form and steel reinforcement placement. The purpose of the observations is to evaluate that the excavations have been made into the recommended bearing material and to the minimum widths and depths recommended for construction. If loose or compressible earth materials are exposed within the foundation excavation, a deeper excavation or removal and recompaction of the subgrade materials would be recommended at that time.

Trenching/Temporary Construction Backcuts

Considering the nature of the onsite earth materials, caving or sloughing could be a factor in subsurface excavations and trenching into onsite granular soils. Shoring or excavating the trench walls/backcuts at a maximum angle of 45 degrees (except as specifically superceded within the text of this report), should be anticipated. All excavations should meet a minimum FOS for temporary slope, backcut, shoring conditions of at least 1.25, and be observed by a licensed engineering geologist or engineer prior to workers entering the excavation or trench, and conform to CAL/OSHA, state, and local safety codes. Should adverse conditions (such as groundwater or running sands) exist, appropriate recommendations would be offered at that time. The above recommendations should be provided to any contractors and subcontractors, etc., that may perform such work. If groundwater or running sands are present or exposed during the trench excavation, trench shields, shoring and dewatering should be used to complete excavations.

Underground Utility Trench Backfill

1. All underground utility trench backfill consisting of the onsite soils should be uniformly moisture conditioned to at least 2 to 3 percent above the soil's optimum moisture content and then be compacted to obtain a minimum relative density of 90 percent of the laboratory standard (ASTM D 1557). However, the moisture content of trench backfill located within 2 feet of vehicular pavement subgrade should be close to optimum. Imported trench backfill that is very low in expansion potential should be uniformly moisture conditioned to at least the soil's optimum moisture content and then be compacted to a minimum relative density of 90 percent of the laboratory standard (ASTM D 1557). Geotechnical observation, selective probing, and field density testing should be provided to evaluate the desired results.

2. Sand backfill should not be used in exterior trenches adjacent to, and within areas extending below a 1:1 (h:v) plane projected down from the outside, bottom edge of the footings for the proposed residential structure, and all trenches beneath hardscape features, unless excavated from the trench. Compaction testing and observations, along with selective probing, should be performed to evaluate the desired results.

3. Underground utilities crossing grade beams, perimeter beams, or footings should either pass below the footing or grade beam using a hardened collar or foam

spacer, or pass through the footing or grade beam in accordance with the recommendations of the structural engineer.

4. Prior to underground utility installation in areas located outside the limits of remedial grading, the bottom of underground utility trenches, exposing unsuitable soils should be overexcavated to expose suitable, unweathered Friars Formation or metavolcanics, and then be backfilled to the planned cut elevation with the excavated soils in accordance with the "Compacted Fill Placement" section of this report.

SUMMARY OF RECOMMENDATIONS REGARDING GEOTECHNICAL OBSERVATION AND TESTING

We recommend that observation and testing be performed by GSI at each of the following construction stages:

- During grading/pad recertification.
- During excavation deeper than 4 feet, including any planned cut slope and temporary slope construction.
- During and following the excavation for a swimming pool or spa, prior to installing concrete forms and reinforcing steel.
- Subsequent to the excavation for foundations and prior to the placement of concrete forms and steel reinforcement.
- During placement of subdrains or other subdrainage devices, prior to placing fill or backfill.
- During drilling for the installation of caissons used to support top-of-slope walls/fences.
- After excavation of retaining wall footings/foundations, and free standing walls footings, prior to the placement of reinforcing steel or concrete.
- Prior to pouring any slabs or flatwork, after pre-moisturizing/presoaking of the building pad and other flatwork subgrade, before the placement of concrete, reinforcing steel, capillary break (i.e., sand, pea-gravel, etc.), or vapor retarders.
- During retaining wall subdrain installation, prior to backfill placement.

- During placement of backfill for area drains, under-slab and underground utility line trenches, and retaining walls.

- During the compaction of the driveway pavement subgrade, aggregate base course, and asphaltic concrete.

- When any unusual soil conditions are encountered during any construction operations, subsequent to the issuance of this report.

- When any additional site improvements are proposed, prior to construction. GSI should review and approve such plans prior to construction.

- A report of geotechnical observation and testing should be provided at the conclusion of each of the above stages, in order to provide concise and clear documentation of site work, and to comply with code requirements.

OTHER DESIGN PROFESSIONALS/CONSULTANTS

The design civil engineer, structural engineer, architect, landscape architect, wall designer, etc., should review the recommendations provided herein, incorporate those recommendations into all their respective plans, and by explicit reference, make this report part of their project plans. This report presents minimum design criteria for the design of slabs, foundations and other elements possibly applicable to the project. These criteria should not be considered as substitutes for actual designs by the structural engineer/designer. The recommendations contained herein are not intended to preclude the transmission of moisture or water vapor through the building slab-on-grade floor or foundation. The structural engineer/foundation or slab designer should provide recommendations to not allow moisture or water vapor to enter into the structure so as to cause damage to another building component, or so as to limit the installation of the type of flooring materials typically used for the particular application.

The structural engineer/designer should analyze actual soil-structure interaction and consider, as needed, bearing, expansive soil influence, and strength, stiffness and deflections in the various slab, foundation, and other elements in order to develop appropriate, design-specific details. As conditions dictate, it is possible that other influences will also have to be considered. The structural engineer/designer should consider all applicable codes and authoritative sources where needed. If analyses by the structural engineer/designer result in less critical details than are provided herein as minimums, the minimums presented herein should be adopted. It is considered likely that some, more restrictive details will be required.

If the structural engineer/designer has any questions or requires further assistance, they should not hesitate to call or otherwise transmit their requests to GSI. In order to mitigate potential distress, the foundation or improvement's designer should confirm to GSI and the governing agency, in writing, that the proposed foundations or improvements can tolerate the amount of differential settlement and expansion characteristics, and other design criteria specified herein.

PLAN REVIEW

Final project plans (grading, precise grading, foundation, retaining wall, landscaping, etc.), should be reviewed by this office prior to construction, so that construction is in accordance with the conclusions and recommendations of this report. Based on our review, supplemental recommendations or further geotechnical studies may be warranted.

LIMITATIONS

The materials encountered on the project site and used for our analysis are believed representative of the area; however, soil and bedrock materials vary in character between excavations and natural outcrops or conditions exposed during mass grading. Site conditions may vary due to seasonal changes or other factors.

Inasmuch as our study is based upon our review and engineering analyses and laboratory data, the conclusions and recommendations are professional opinions. These opinions have been derived in accordance with current standards of practice, and no warranty, either express or implied, is given. Standards of practice are subject to change with time. GSI assumes no responsibility or liability for work or testing performed by others, or their inaction; or work performed when GSI is not requested to be onsite, to evaluate if our recommendations have been properly implemented. Use of this report constitutes an agreement and consent by the user to all the limitations outlined above, notwithstanding any other agreements that may be in place. In addition, this report may be subject to review by the controlling authorities. Thus, this report brings to completion our scope of services for this portion of the project. All samples will be disposed of after 30 days, unless specifically requested by the client, in writing.

APPENDIX A

REFERENCES AND COAST GEOTECHNICAL (2018)

APPENDIX A

REFERENCES

American Concrete Institute (ACI) Committee 302, 2015, Guide for concrete floor and slab construction, ACI 302.1R-15, dated June.

_____, 2014, Building code requirements for structural concrete (ACI 318-14), and commentary (ACI 318R-14): reported by ACI Committee 318, dated September.

American Society for Testing and Materials (ASTM), 1998, Standard practice for installation of water vapor retarder used in contact with earth or granular fill under concrete slabs, Designation: E 1643-98 (Reapproved 2005).

_____, 1997, Standard specification for plastic water vapor retarders used in contact with soil or granular fill under concrete slabs, Designation: E 1745-97 (Reapproved 2004).

American Society of Civil Engineers, 2017, Minimum design loads and associated criteria for buildings and other structures, provisions, ASCE Standard ASCE/SEI 7-16.

_____, American Society of Civil Engineers, 2010, Structural design of interlocking concrete pavement for municipal streets and roadways, ASCE Standard 58-10.

Blake, Thomas F., 2000a, EQFAULT, A computer program for the estimation of peak horizontal acceleration from 3-D fault sources; Windows 95/98 version.

_____, 2000b, EQSEARCH, A computer program for the estimation of peak horizontal acceleration from California historical earthquake catalogs; Updated to May 8, 2021, Windows 95/98 version.

Bozorgnia, Y., Campbell K.W., and Niazi, M., 1999, Vertical ground motion: Characteristics, relationship with horizontal component, and building-code implications; Proceedings of the SMIP99 seminar on utilization of strong-motion data, September 15, Oakland, pp. 23-49.

California Building Standards Commission, 2022, California Building Code, California Code of Regulations, Title 24, Part 2, Volumes 1 and 2, based on the 2021 International Building Code effective January 1, 2023.

California Department of Conservation, California Geological Survey (CGS), 2018, Earthquake fault zones, a guide for government agencies, property owners/developers, and geoscience practitioners for assessing fault rupture hazards in California: California Geological Survey Special Publication 42 (revised 2018), 93 p.

Cao, T., Bryant, W.A., Rowshandel, B., Branum, D., and Wills, C.J., 2003, The revised 2002 California probabilistic seismic hazard maps, dated June, http://www.conservation.ca.gov/cgs/rghm/psha/fault_parameters/pdf/Documents/2002_CA_Hazard_Maps.pdf.

City of Los Angeles Department of Building and Safety, 2020, Information bulletin / public - building code, reference no.: LABC 1610.1, 1807.2, document no.: P/BC 2020-083, dated January 1.

Coast Geotechnical, 2018, Geotechnical evaluation of existing fill slope, APN 264-110-30-00, Via De Las Flores, Rancho Santa Fe, California, W.O. 6811217, dated January 11.

County of San Diego, 2020, County of San Diego BMP design manual for permanent site design, storm water treatment and hydromodification management, storm water requirements for development applications, dated September 15 (second update to February 2016 manual).

Jennings, C.W., and Bryant, W.A., 2010, Fault activity map of California, scale 1:750,000, California Geological Survey, Geologic Data Map No. 6.

Kanare, H.M., 2005, Concrete floors and moisture, Engineering Bulletin 119, Portland Cement Association.

Kennedy, M.P., and Tan, SS., 2007, Geologic map of the Oceanside 30' by 60' quadrangle, California, Map no. 2, scale 1:100,000,California Geologic Survey and U.S. Geological Survey.

Norris, R.M. and Webb, R.W., 1990, Geology of California, second edition, John Wiley & Sons, Inc.

Park Aerial Surveys, Inc., 1953, Stereoscopic aerial photographs, flight AXN-1953, frame nos. 4M-19 and 4M-20, 1:20,000-scale, dated March 31.

Romanoff, M., 1957, Underground corrosion, originally issued April 1.

Seed, 2005, Evaluation and mitigation of soil liquefaction hazard “evaluation of field data and procedures for evaluating the risk of triggering (or inception) of liquefaction”, *in* Geotechnical earthquake engineering; short course, San Diego, California, April 8-9.

Seed, H. B. and Whitman, R. V., 1970, Design of earth retaining structures for dynamic loads, *ASCE Specialty Conference, Lateral Stresses in the Ground and Design of Earth Retaining Structures*, pp. 103-147.

Sowers and Sowers, 1979, Unified soil classification system (After U. S. Waterways Experiment Station and ASTM 02487-667) in Introductory Soil Mechanics, New York.

Snipes-Dye Associates, undated, Grading plan for Honarvar residence and equestrian pads (grading violation), grading permit no.: PDS2019-LDGRMJ-30214, 13 sheets, various scales.

State of California, 2023, Civil Code, Sections 896 et seq.

Tan, S.S., and Giffen, D.G., 1995, Landslide hazards in the northern part of the San Diego Metropolitan area, San Diego County, California, Landslide hazard identification map no. 35, Plate 35E, Department of Conservation, Division of Mines and Geology, DMG Open File Report 95-04.

United States Geological Survey, 1996, Topographic survey map of the Rancho Santa Fe quadrangle, San Diego County, California, 7.5 minute series, 1:24,000-scale.

______, 1968, Topographic survey map of the Rancho Santa Fe quadrangle, San Diego County, California, 7.5 minute series, 1:24,000-scale (photorevised 1983).

COAST GEOTECHNICAL
CONSULTING ENGINEERS AND GEOLOGISTS

January 11, 2018

Susan Drawdy
16059 Avenida Calma
Rancho Santa Fe, CA 92091

RE: GEOTECHNICAL EVALUATION OF EXISTING FILL SLOPE
APN-264-110-30-00
Via De Las Flores
Rancho Santa Fe, California

Dear Ms. Drawdy:

In response to your request and in accordance with our Agreement dated December 11, 2017, we have performed a geotechnical evaluation for the previous undocumented grading on the site. The findings of the evaluation, laboratory test results, and conclusions are presented in this report.

No geologic hazards which could preclude development of the site are present. However, significant amounts of undocumented fill deposits are present which will require mitigation for development.

If you have any questions, please do not hesitate to contact us at (858) 755-8622. This opportunity to be of service is appreciated.

Respectfully submitted,
COAST GEOTECHNICAL

Wyatt Bartholomew
Project Geologist

Mark Burwell, C.E.G.
Engineering Geologist

GEOTECHNICAL EVALUATION OF EXISTING FILL SLOPE

APN-264-110-30-00
Via De Las Flores
Rancho Santa Fe, California

Prepared for:
Susan Drawdy
16059 Avenida Calma
Rancho Santa Fe, CA 92091

Prepared by:
COAST GEOTECHNICAL
P.O. Box 230163
Encinitas, California 92023

January 11, 2018
W.O. 6811217

TABLE OF CONTENTS

1. INTRODUCTION

This report presents the results of our background review, subsurface investigation, laboratory testing, geotechnical analyses, and conclusions regarding the conditions at APN 264-110-30-00 Via De Las Flores. The purpose of this study is to evaluate the nature and characteristics of the earth materials on the existing graded lot.

2. SCOPE OF SERVICES

The scope of services provided included a review of background data, reconnaissance of the site geology, and evaluation of the existing graded lot. The performed tasks specifically included the following:

- Reviewing geologic and hazard (seismic, landslide, and tsunami) maps, recently published regarding the seismic potential of nearby faults, and a site plan for the project. All background data is listed in the References portion of this report.

- Performing a site reconnaissance, including the observation of geologic conditions and other hazards of the existing graded lot.

- Excavation of exploratory trenches consisting of excavating, logging, and sampling of earth materials to evaluate the subsurface conditions.

- Performing geotechnical laboratory testing of recovered soil samples.

- Analyzing data obtained from our research, subsurface exploration, and laboratory testing.

- Preparing this geotechnical evaluation report.

3. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

3.1 Site Description

The subject site is a vacant lot on Via De Las Flores, north of Aliso Canyon Road, in the city of Rancho Santa Fe (Figure 1). The property is a rectangular-shaped area that descends to the west. The elevation of the lot ranges from approximately 400 to 450 feet.

The subject site includes two existing graded pads. Both pads have fill slopes on the western perimeter sloping to the west at an approximate gradient of 2:1(horizontal to vertical). To the east of the pads, there is natural terrain that is sloping to the west at an approximate gradient of 5:1 (horizontal to vertical). From the relative pad area, the property ascends at a gradient of approximately 6:1 (horizontal to vertical) for approximately 315 feet horizontally. The site is bounded to the north and south by developed residential lots. The site is bounded to the east by a dirt road and vacant sloping lot. The site is bounded to the west by a paved road, Via De Las Flores.

Vegetation on the site consists of a sparse growth of trees and bushes. Drainage is generally by sheet flows to the west.

3.2 Proposed Development

Preliminary grading plans for the development of the site were prepared by ERB & Associates, LLC. The project is anticipated to include remedial grading and the construction of a single family residence (Figure 2). Grading is anticipated to include maximum cuts of 10 feet at a 2:1 (horizontal to vertical) gradient and fills up to 4 feet.

4. SITE INVESTIGATION AND LABORATORY TESTING

4.1 Site Investigation

Site exploration was performed on January 11, 2018. It included a visual reconnaissance of the site and the excavation of three (3) exploratory trenches (Figures 5-7). The trenches were excavated using a tractor-mounted backhoe. All the trenches extended into the underlying artificial fill, weathered metavolcanics, and Friars formation to a maximum depth of 16 feet in Trench 3. Earth materials encountered were visually classified and logged by our field project geologist, and sampled for future laboratory testing. Undisturbed, representative chunk and bulk samples of earth materials were obtained at selected intervals by driving a thin-walled steel sampler and excavating into the desired strata. Samples were retained in waterproof containers and transported to Coast Geotechnical Soils Laboratory for testing and analysis.

4.2 Laboratory Testing and Analysis

The laboratory tests were performed in accordance with the generally accepted American Society for Testing and Materials (ASTM) test methods or suggested procedures. All lab descriptions and results can be found in the Test Results section of Appendix B of this report.

The following tests were preformed:

1. Classification of Soils
2. Moisture/Density
3. Maximum Dry Density and Optimum Moisture Content

5. GEOLOGIC CONDITIONS

The geologic conditions at the site are based on our field exploration and review of available geologic and geotechnical literature.

5.1 Regional Geologic Settings

The subject property is located in the Coastal Plains subdivision of the Peninsular Ranges geomorphic province of San Diego. The coastal plain area is characterized by Pleistocene marine terrace landforms. These surfaces are relatively flat erosional platforms that were shaped by wave action along the former coastlines. The step-like elevation of the marine terraces was caused by changes in sea level throughout the Pleistocene and by seismic activity along the Rose Canyon Fault Zone located 8.7 miles west of the coastline. The Rose Canyon Fault Zone is one of many northwest trending, sub-parallel faults and fault zones that traverse the nearby vicinity. Several of these faults, including the Rose Canyon Fault Zone, are considered active faults. Further discussion of faulting in regards to the site is discussed in the Geologic Hazards section of this report.

5.2 Site Geology

Previously published geologic maps conducted by Kennedy and Tan (2008) indicate that the subject property is underlain at depth by Mesozoic-aged undivided metasedimentary and metavolcanic rocks (Mzu). The Mesozoic metasedimentary and metavolcanic rocks are covered by artificial fill deposits (Qaf) with a thin layer of top soil in between the metavolcanic rocks and the artificial fill as observed in the test excavation trenches The general geologic conditions are depicted on cross-section A-A' enclosed on Figure 4. A brief description of the earth materials encountered on the site are as follows:

Artificial Fill (Qaf)

All three trenches encountered artificial fill. In Excavation Trenches No. 1, 9 feet of artificial fill was encountered, No. 2 had 7 feet of artificial fill, and No. 3 had 10 feet of artificial fill. The artificial fill is tan, fine to medium grained sand with pebbles. The upper most 6 inches were moist. In the lower sections the fill is dark brown, silty fine sandy clay to clayey sand with rocks and pebbles with metavolcanic clasts, some greater than 12 inches in diameter. The fill contains concrete chunks, plastics, bricks, ropes, pipes, pipelines and wire mesh.

Topsoil(Qs)

Underlying the surficial materials is a relatively thin layer of soil material. In Trench No. 1, the soil was approximately 10 inches thick, in Trench No. 2 approximately 12-14 inches thick, and in Trench No. 3 approximately 3 feet thick. The soil is described as a brown to black silty fine grained clayey sand and grades into a lower layer of reddish brown clay.

Friars Formation (Tf)

The only trench to encounter the Friars formation was Trench No. 1.The top of the formation was encountered at a depth of 10 feet and continued to the bottom depth of the trench at 14 feet. Through a gradational contact from the overlying soil horizon, the Friars formation is a gray to brown clay with pale tan green silty clay, and is damp and in a soft to stiff condition.

Metasedimentary/ Metavolcanic rocks (Mzu)

Underlying the soil horizon in Trench Nos. 2 and 3 is the Mesozoic-aged metasedimentary and metavolcanic rock unit. The contact grades from the overlying soil. The unit is described as a tan brown to gray, fine to silty sand, slightly clayey, slightly moist, and moderately dense. The weathered bedrock in both trenches extended to depths of 13 and 16 feet in Trenches 2 and 3, respectively.

5.3 Expansive Soil

Based on our experience in the area and previous laboratory testing of selected samples, the deposits reflect an expansion potential in the medium to high range.

6. GEOLOGIC HAZARDS

6.1 Faulting and Seismicity

The subject site is located within the seismically active Southern California region, which is generally characterized by northwest trending, right-lateral strike-slip faults and fault zones. Several of these fault segments and zones are classified as active by the California Geologic Survey (Alquist-Priolo Earthquake Fault Zoning Act.) As a result, ground shaking is a potential hazard throughout the region.

Based on a review of published geologic maps, no known active faults traverse the site (Figure 3). Thus, ground surface rupture is not likely to occur as a result of an earthquake or seismic event.

The nearest active fault to the site is the Rose Canyon Fault Zone (offshore), located approximately 8.7 miles west of the site. It should be noted that the Rose Canyon Fault is one of four main fault strands that make up the Newport-Inglewood/Rose Canyon (NIRC) fault system (Treiman, 1984). The four strands form a series of right-stepping en echelon faults situated along the Southern California coastline. A recent study by Sahakian et al. (2017) concluded that the geometry of the NIRC fault system may enable rupture along the entire length of the fault zone. The study also modeled several rupture scenarios in light of the newly defined geometry which suggest earthquake ruptures up to magnitudes (M) of 7.4 are possible along the NIRC system. While the models are intriguing, the paper recommends further research and modeling on the NIRC fault geometry to improve our understanding of potential hazards and ground shaking along the Southern California coast. Therefore, the modeled rupture magnitude of M = 7.4 on the Rose Canyon Fault was not used for the recommendations for this investigation.

Other nearby faults that may affect the site include the Newport-Inglewood, Coronado Bank, Elsinore-Julian, and Elsinore-Temecula. The proximity of major faults to the site, and their estimated maximum earthquake magnitudes and peak site accelerations are enclosed on Table 1 and were determined by EQFAULT version 3.00 software (Blake, 2000).

Table 1: Principal Active Faults

Fault Name	Approximate Distance from site (mi)	Maximum EQ Magnitude (Mmax)	Peak Site Accel. (g)
ROSE CANYON	8.7	6.9	0.175
NEWPORT-INGLEWOOD	16.2	6.9	0.173
CORONADO BANK	23.2	7.4	0.171
ELSINORE-JULIAN	23.5	7.1	0.145
ELSINORE-TEMECULA	24.3	6.8	0.121
EARTHQUAKE VALLEY	35.7	6.5	0.077
ELSINORE-GLEN IVY	41.9	6.8	0.080
SAN JACINTO-ANZA	46.2	7.2	0.091
PALOS VERDES	46.8	7.1	0.085

The Rose Canyon Fault is capable of generating a magnitude earthquake which would cause strong ground motions at the subject site. Further analysis on seismicity and the site specific seismic parameters are discussed in the Recommendations chapter of this report.

Although the likelihood of ground rupture on the site is remote, the property will be exposed to moderate to high levels of ground motion resulting from the release of energy should an earthquake occur along the numerous known and unknown faults in the region. The Rose Canyon (offshore) Fault Zone located approximately 8.7 miles west of the property is the nearest known active fault, and is considered the design fault for the site. In addition to the Rose Canyon fault, several other active faults may affect the subject site.

Seismic design parameters were determined as part of this investigation in accordance with Chapter 16, Section 1613 of the 2016 California Building Code (CBC) and ASCE 7-10 Standard using the web-based United States Geological Survey (USGS) Seismic Design Tool. The generated results for the parameters are presented on Table 2

Table 2: Seismic Design Parameters

Factors	**Values**
Site Class	D
Seismic Design Category	I/ II/ III
Site Coefficient, F_a	1.108
Site Coefficient, F_v	1.636
Mapped Short Period Spectral Acceleration, S_S	0.979
Mapped One-Period Spectral Acceleration, S_1	0.382
Short Period Spectral Acceleration Adjusted for Site Class, S_{MS}	1.085
One-Second Period Spectral Acceleration Adjusted for Site, S_{M1}	0.625
Design Short Period Spectral Acceleration, S_{DS}	0.724
Design One-Second Period Spectral Acceleration, S_{D1}	0.417

6.2 Landslide Potential

A landslide is the displacement of a mass of rock, debris, or earth down a slope caused by topographic, geological, geotechnical and/or subsurface water conditions. Potential landslide hazards for the site were assessed using the review of published geologic and topographic maps for the area.

According to the Landslide Hazards map, Rancho Santa Fe Quadrangle (Tan and Giffen, 1995), the site is located within Susceptibility Area 3-1 where slopes are generally susceptible. Most slopes in this area do not contain landslide deposits, but they can be subject to failure if they are adversely modified."

Owing to the gentle topography at the site, the potential for deep-seated landsliding at the subject site is considered low.

6.3 Liquefaction Potential

Liquefaction is a process by which a sand mass loses its shearing strength completely and flows. The temporary transformation of the material into a fluid mass is often associated with ground motion resulting from an earthquake, and high groundwater conditions.

Owing to the moderately dense nature of the Metasedimentary/ Metavolcanic Rocks and the anticipated depth to groundwater, the potential for seismically-induced liquefaction and soil instability is considered low.

6.4 Tsunami Potential

Tsunamis are large sea waves generated by earthquakes, volcanic eruptions, or landslides that potentially cause the displacement of substantial volumes of water.

The Tsunami Inundation Map for Emergency Planning: Rancho Santa Fe Quadrangle (California Emergency Management Agency, 2009) suggests that the site is not susceptible to flooding from tsunamis.

7. CONCLUSIONS

1) Fill deposits were encountered in all three (3) exploratory trenches. The depth of fill ranged from 8 feet to 10 feet. The fill is generally composed of clayey sand to sandy clay and is expansive.

2) The fill contains large rock fragments, concrete, concrete with rebar and welded wire mesh, rope, plastic containers and other assorted construction debris.

3) The fill was placed over a natural soil zone and no evidence of benching was observed in the exploratory trenches.

4) Based on our site exploration and limited laboratory testing, the fill does not meet minimum standards in the California Building Code or County of San Diego grading requirements.

5) Underlying the fill and soil, two (2) distinctively different geologic formations were encountered. In Trench No. 1, the fill and soil is underlain by claystone which is generally correlated with the Friars formation on published geologic maps. The claystone is generally soft to stiff, expansive and has been associated with slope instability.

   In Trench Nos. 2 and 3, the fill and soil is underlain by weathered meta-volcanic rock. The bedrock is well exposed along the rear cut slope, as well as cut slopes in the immediate area.

6) Based on the results of our geotechnical evaluation, the fill deposits were placed inadequately and without geotechnical observation and testing. In order to develop the site, the fill deposits would have to be removed and replaced as properly compacted fill. An adequate keyway along the base of the slope and benching into competent bedrock will also be necessary. The existing depth of fill is significant. Although the existing fill may be reused, after removal of debris and proper mixing, the final fill mixture will be expansive. Expansive soils will require a special foundation and special consideration in site development.

7) Although no geologic hazards which would preclude the development of the site for residential purposes were encountered, the existing fill deposits could pose an economic concern. Onsite discussion with Lane Fukuda of Pacific Coast Grading suggests remedial grading could cost up to $175,000.

8. LIMITATIONS

This report is presented with the provision that it is the responsibility of the owner or the owner's representative to bring the information and recommendations given herein to the attention of the project's architects and/or engineers so that they may be incorporated into the plans.

If conditions encountered during construction appear to differ from those described in this report, our office should be notified so that we may consider whether modifications are needed. No responsibility for construction compliance with design concepts, specifications, or recommendations given in this report is assumed unless on-site review is performed during the course of construction.

The subsurface conditions, excavation characteristics, and geologic structure described herein are based on individual exploratory excavations made on the subject property. The subsurface conditions, excavation characteristics, and geologic structures discussed should in no way be construed to reflect any variations which may occur among the exploratory excavations.

Please note that fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time measurements were made and reported herein. Coast Geotechnical assumes no responsibility for variations which may occur across the sire.

The conclusions and recommendations of this report apply as of the current date. In time, however, changes can occur on a property whether caused by acts of man or nature on this or adjoining properties. Additionally, changes in professional standards may be brought about by legislation or the expansion of knowledge. Consequently, the conclusions and recommendations of this report may be rendered wholly or partially invalid by event beyond our control. This report is therefore subject to review and should not be relied upon after the passage of two years.

The professional judgements presented herein are founded partly on our assessment of the technical data gathered, partly on our understanding of the proposed construction, and partly on our general experience in the geotechnical field. However, in no respect do we guarantee the outcome of the project.

This study has been provided solely for the benefit of the client, and is in no way intended to benefit or extend any right or interest to any third party. This report is not to be used on other projects or extensions to this project except by agreement in writing with Coast Geotechnical.

REFERENCES

Blake, T. F. (2000). EQFAULT: A Computer Program for the Deterministic Estimation of Peak Acceleration using Three-Dimensional California Faults as Earthquake Sources, Version 3.0, Thomas F. Blake Computer Services and Software, Thousand Oaks, CA.

California Geologic Survey, (1994), Fault Activity Map of California, Map Scale 1"=750,00'.

Kennedy, M. P., and Tan, S. S. (2008). California Geological Survey, Regional Geologic Map No. 2, 1:100,000 scale.

ERB & Associates, LLC (2017). Remedial Grading Plans:APN 264-110-30-00, California, Scale 1" = 20'.

Sahakian, V., et al. (2017). Seismic Constraints on the Architecture of the Newport-Inglewood/Rose Canyon Fault: Implications for the Length and Magnitude of Future Earthquake Ruptures. American Geophysical Union. DOI: 10.1002/2016JB013467

Seed, H.B., and Whitman, R.V. (1970). Design of earth retaining structures for dynamic loads. In Proceedings of the ASCE Special Conference on Lateral Stresses, Ground Displacement and Earth Retaining Structure, Ithaca, N.Y., pp. 103–147.

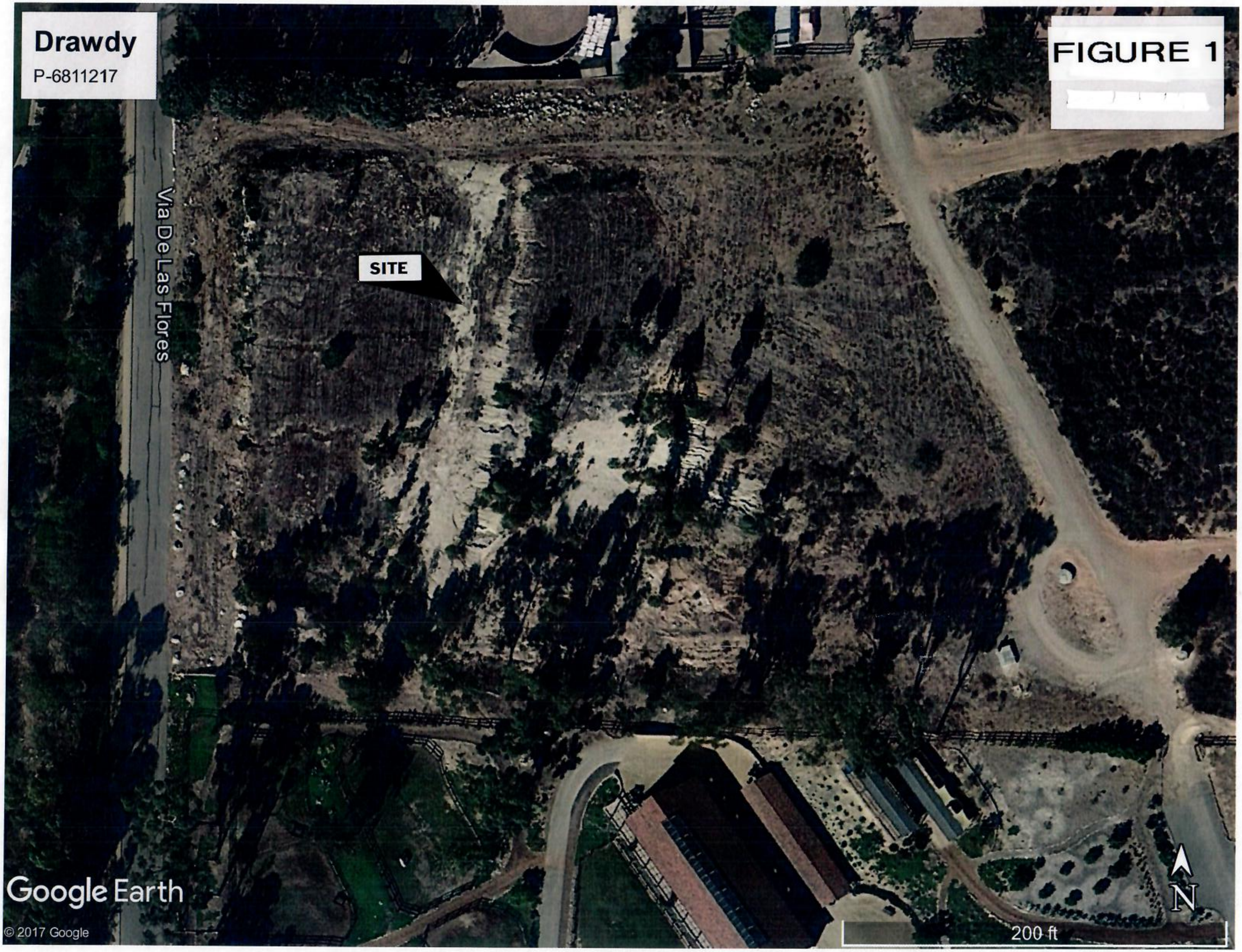
USGS, U. S. Seismic Design Maps, Scale = Variable.

https://earthquake.usgs.gov/designmaps/us/application.php

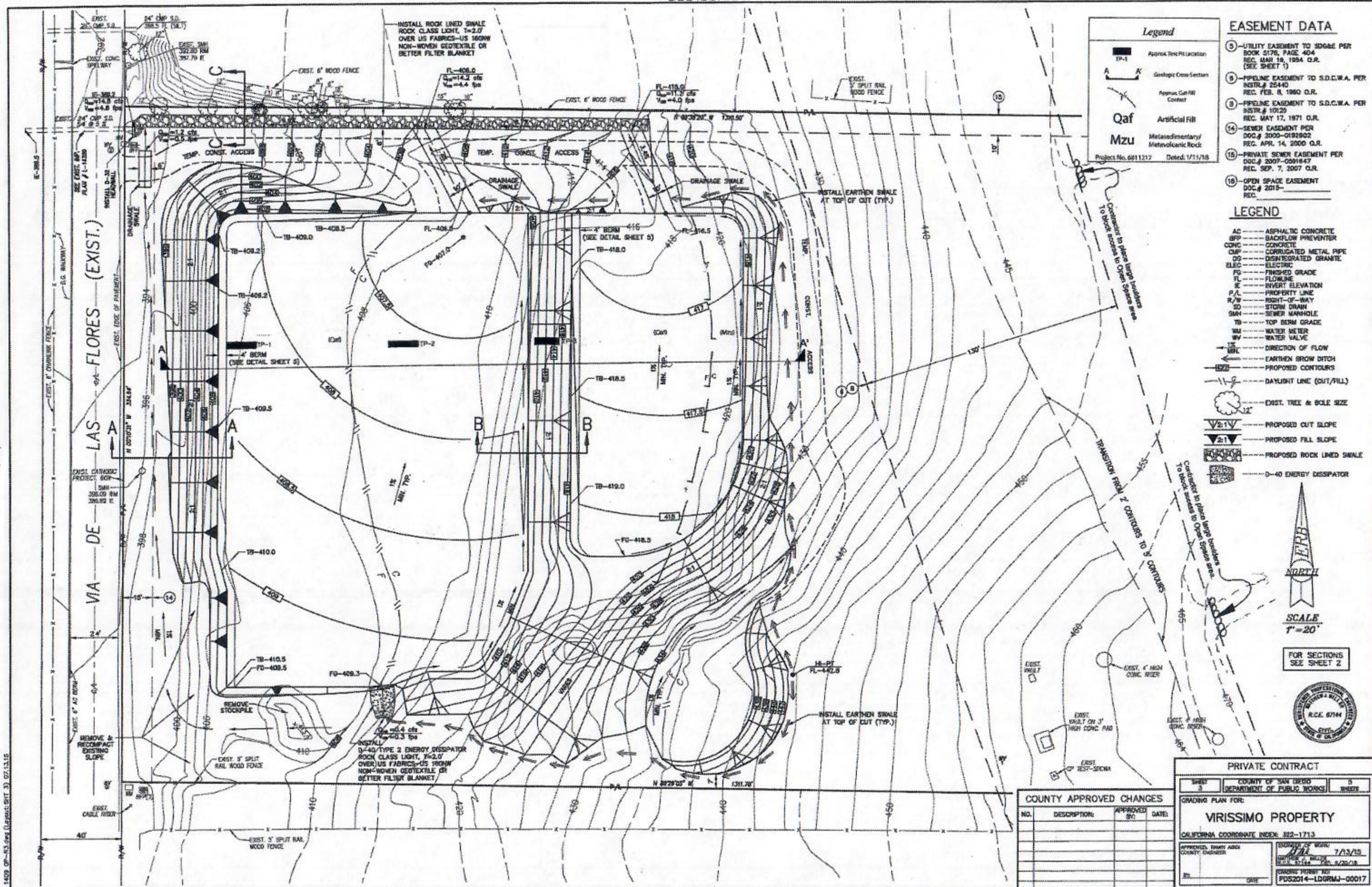
Tan, S. S., and Giffen, D. G. (1995). Landslide Hazards in the Northern Part of the San Diego Metropolitan Area, San Diego County, California, OFR 95-04,

Treiman, J. A. (1984). The Rose Canyon Fault Zone: A Review and Analysis, California Division of Mines and Geology, Fault Evaluation Report 216.

APPENDIX A


Drawdy
P-6811217
FIGURE 1
SITE
Via De Las Flores
Google Earth
© 2017 Google
200 ft
N

GEOTECHNICAL MAP



CALIFORNIA FAULT MAP

Drawdy

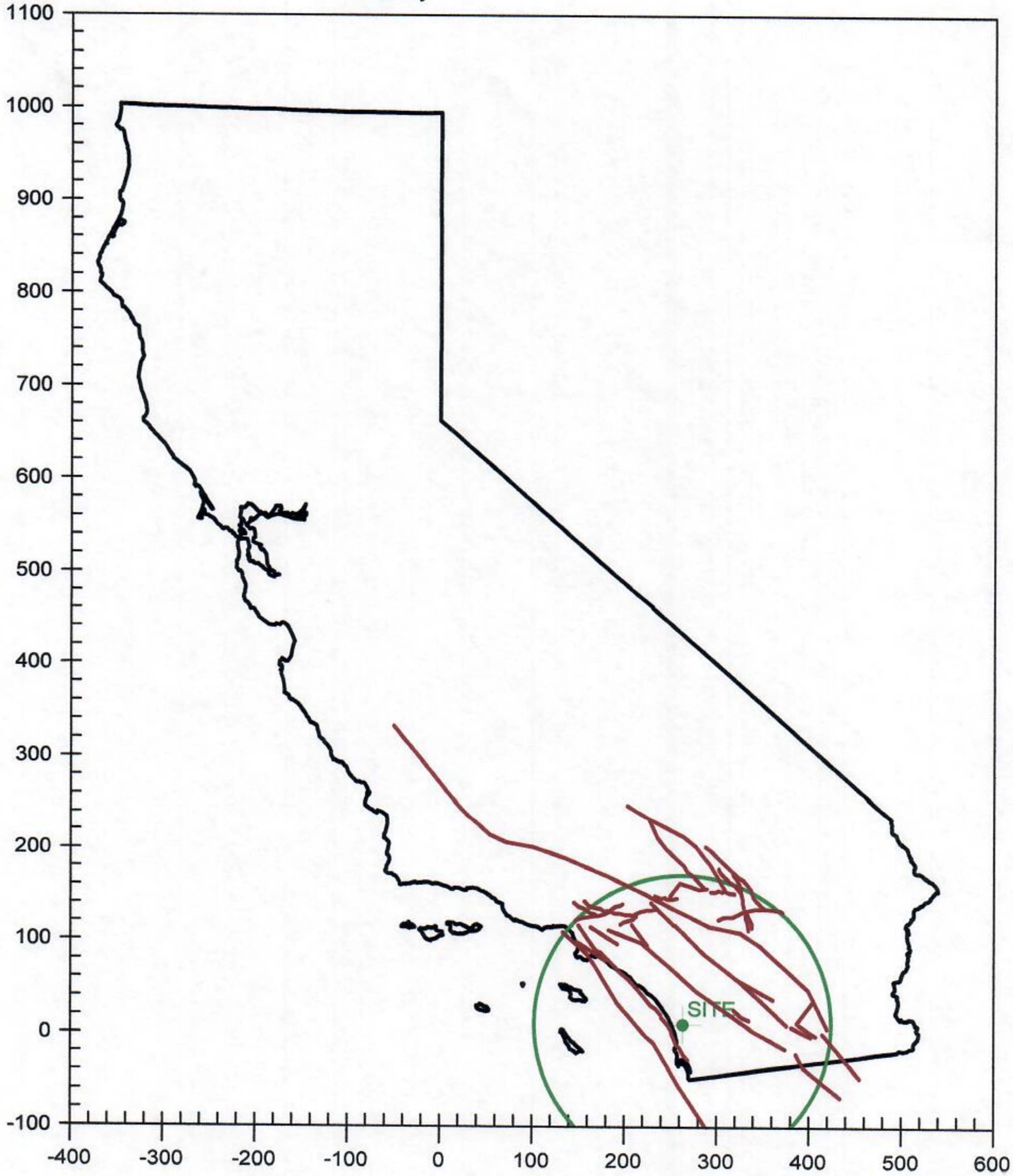
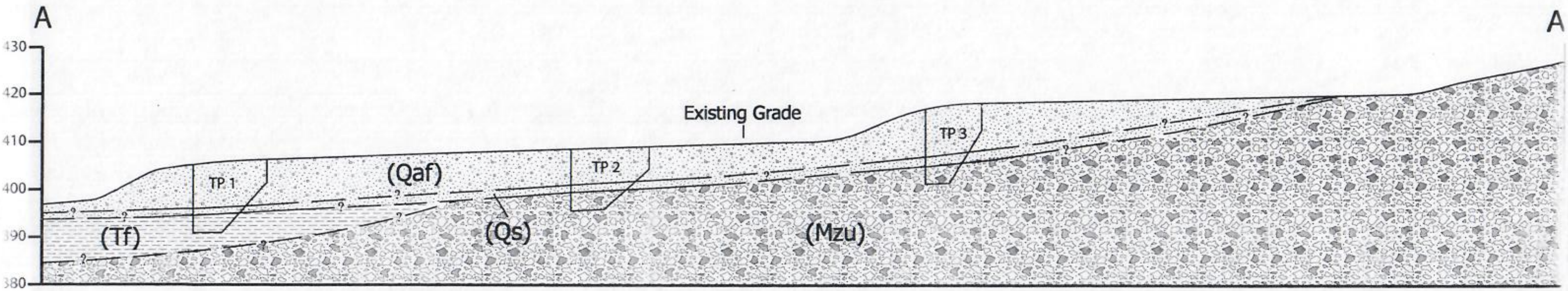


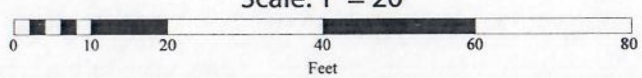
FIGURE 3

GEOLOGIC CROSS SECTION A-A'

APN-264-110-30-00
Via De Los Flores
Rancho Santa Fe, California



Scale: 1″ = 20′



Legend		*Geologic Units*	
-?—?-	Approximate Contact	Qaf	Artificial Fill
——	Geologic Contact	Qs	Top Soil
TP-1	Test Pit	Tf	Friars Formation
		Mzu	Metavolcanic Unit

COAST GEOTECHNICAL
5931 Sea Lion Place, Suite 109
Carlsbad, CA 92010

Figure 4
W.O. P-6811217

Project: Drawdy Project Location: Via De Los Flores Project Number: 6811217	Log of Trench 1	**Coast Geotechnical** 5931 Sea Lion Place, Suite 109 Carlsbad, CA 92010 (858) 755-8622

Date(s) Drilled		Logged By	WB	Checked By	
Drilling Method		Drill Bit Size/Type		Total Depth of Trench	
Drill Rig Type		Drilling Contractor		Approximate Surface Elevation	
Groundwater Level and Date Measured		Sampling Method(s)		Hammer Data	
Borehole Backfill		Location			

Elevation (feet)	Depth (feet)	Sample Type	Sample Number		Material Type	Graphic Log	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
	0						(Qaf) Artificial Fill -Tan, fine to medium grained sand with pebbles; moist upper 6." -Dark brown, silty fine sandy clay to clayey sand with rocks and pebbles; metavolcanic boulder-sized rocks (some >12"). -Red to gray stained weathering.	
	1							
	2	■						
	3							
	4	□						
	5						(Qs) Top Soil -Black coarse to fine to silty grained clayey sand (thickness is approximately 10"), gradational contact into a lower layer of reddish brown clay.	
	8	■						
	9							
	10							
	12	□					(Tf) Friars Formation - Gradational contact from the overlying Qs into a gray to brown clay with pale tan green silty clay, moist, stiff, and damp.	
	14	□					Bottom of TP1 @ 14 feet -No GW -No Caving	

Figure #5

Project: Drawdy Project Location: Via De Los Flores Project Number: 6811217	Log of Trench 2	**Coast Geotechnical** 5931 Sea Lion Place, Suite 109 Carlsbad, CA 92010 (858) 755-8622

Date(s) Drilled		Logged By	WB	Checked By	
Drilling Method		Drill Bit Size/Type		Total Depth of Trench	
Drill Rig Type		Drilling Contractor		Approximate Surface Elevation	
Groundwater Level and Date Measured		Sampling Method(s)		Hammer Data	
Borehole Backfill		Location			

Elevation (feet)	Depth (feet)	Sample Type	Sample Number		Material Type	Graphic Log	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
	0						**(Qaf) Artificial Fill** - Tan, fine to medium grained sand with pebbles; moist upper 6." - Dark brown, silty fine sandy clay to clayey sand with rocks and pebbles; metavolcanic boulder-sized rocks (some >12"). - Red to grey stained weathering - Rock fragments, concrete, pipeline 4' long, brick, wood, asphalt, plastic.	
	1							
	2							
	3							
	4							
	5						**(Qs) Top Soil** -Black coarse to fine to silty grained clayey sand (thickness is approximately 12"-14"), gradational contact into a lower layer of reddish brown clay.	
	7							
	8	□					**(Mzu) Weathered Bedrock** -Gradational contact from the overlying Qs to a tan brown to gray, fine to silty sand, slightly clayey, slightly moist, dense.	
	10							
	12							
	11	□						
	13						Bottom of TP2 @ 13 feet -No GW -No Caving	

Figure #6

Project: Drawdy Project Location: Via De Los Flores Project Number: 6811217	Log of Trench 3	**Coast Geotechnical** 5931 Sea Lion Place, Suite 109 Carlsbad, CA 92010 (858) 755-8622

Date(s) Drilled		Logged By	WB	Checked By	
Drilling Method		Drill Bit Size/Type		Total Depth of Trench	
Drill Rig Type		Drilling Contractor		Approximate Surface Elevation	
Groundwater Level and Date Measured		Sampling Method(s)		Hammer Data	
Borehole Backfill		Location			

Elevation (feet)	Depth (feet)	Sample Type	Sample Number		Material Type	Graphic Log	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
	0						(Qaf) Artificial Fill - Tan, fine to medium grained sand with pebbles; moist upper 6." - Dark brown, silty fine sandy clay to clayey sand with rocks and pebbles; metavolcanic boulder-sized rocks (some >12"). - Red to grey stained weathering - Rock fragments, concrete, pipeline 4' long, brick, wood, asphalt, plastic.	Dirty fill: concrete, asphalt, plastic, ropes, pipes, and wire mesh.
	1							
	2							
	3							
	4							
	5							
	6	□					(Qs) Top Soil -Black coarse to fine to silty grained clayey sand, gradational contact into a lower layer of reddish brown clay.	
	10							
	13	□					(Mzu) Weathered Bedrock -Gradational contact from the overlying Qs to a tan brown to gray, fine to silty sand, slightly clayey, slightly moist, dense.	
	16						Bottom of TP3 @ 16 feet -No GW -No Caving	

Figure # 7

USGS Design Maps Summary Report

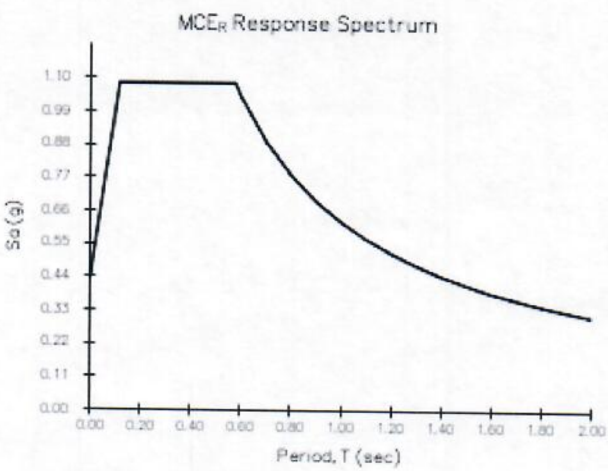
User–Specified Input

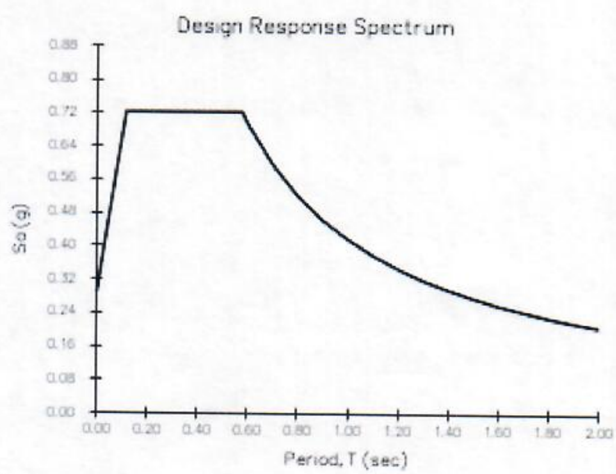
Report Title	P-6811217 Tue January 9, 2018 16:29:05 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	33.054°N, 117.178°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

USGS–Provided Output

S_S =	0.979 g	S_{MS} =	1.085 g	S_{DS} =	0.724 g
S_1 =	0.382 g	S_{M1} =	0.625 g	S_{D1} =	0.417 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

FIGURE 8

USGS Design Maps Detailed Report

ASCE 7-10 Standard (33.054°N, 117.178°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 0.979$ g

From Figure 22-2 [2] $S_1 = 0.382$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 0.979 g, F_a = 1.108

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.382 g, F_v = 1.636

Equation (11.4–1): $S_{MS} = F_aS_S = 1.108 \times 0.979 = 1.085$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.636 \times 0.382 = 0.625$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

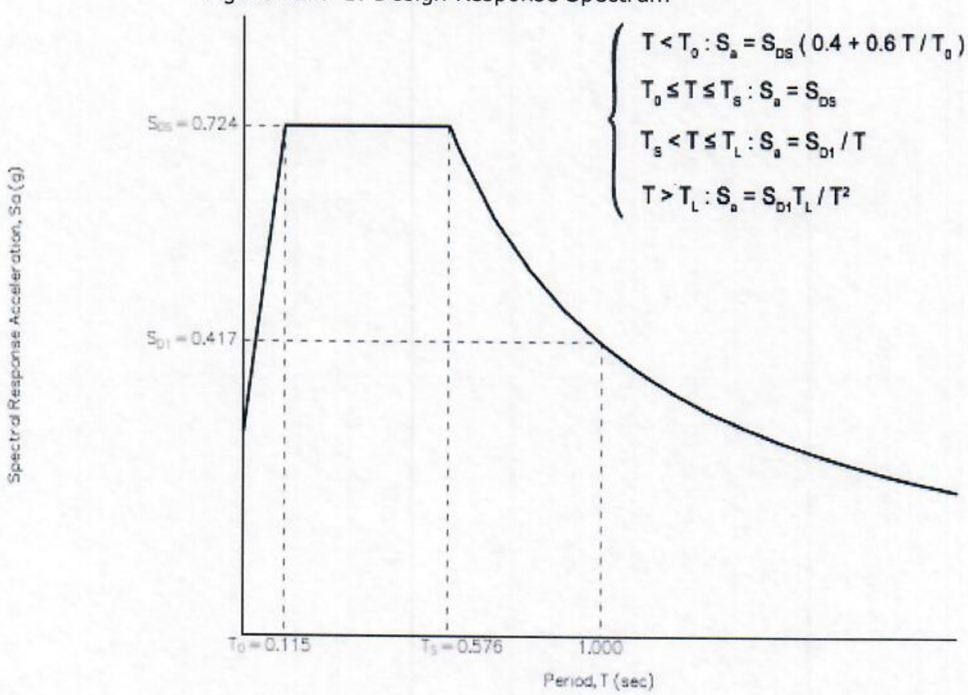
Equation (11.4–3): $S_{DS} = ⅔\ S_{MS} = ⅔ \times 1.085 = 0.724$ g

Equation (11.4–4): $S_{D1} = ⅔\ S_{M1} = ⅔ \times 0.625 = 0.417$ g

Section 11.4.5 — Design Response Spectrum

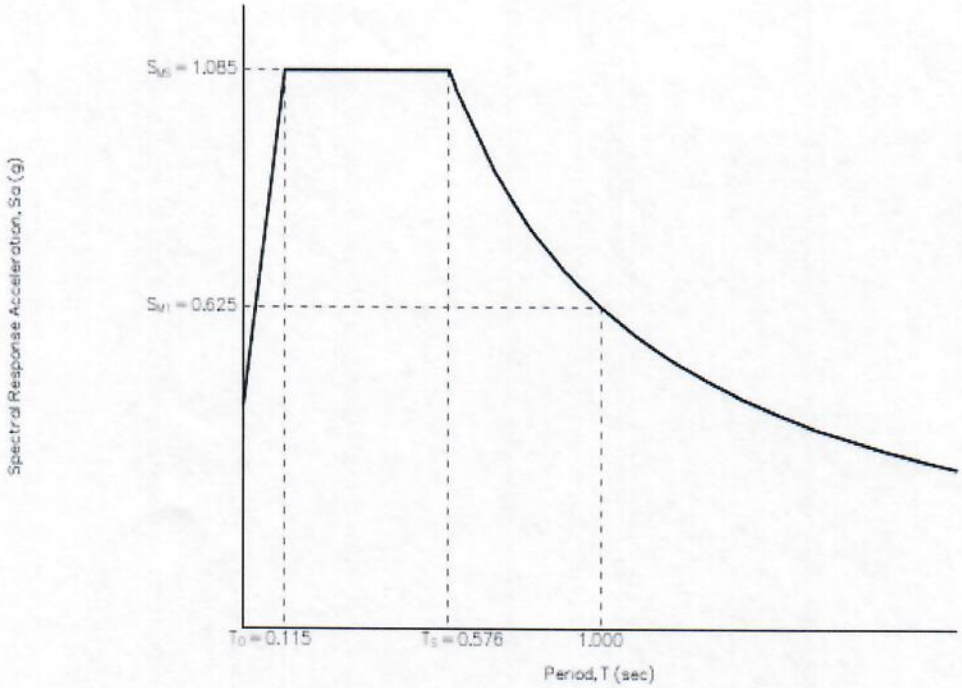
From Figure 22-12 [3] $T_L = 8$ seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.371

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.129 \times 0.371 = 0.419$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.371 g, F_{PGA} = 1.129

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 1.004

From Figure 22-18 [6] C_{R1} = 1.060

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.724 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.417 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

APPENDIX B

LABORATORY TESTING AND RESULTS

Earth materials encountered in the exploratory test pits were closely examined and sampled for laboratory testing. The laboratory tests were performed in accordance with the generally accepted American Society for Testing and Materials (ASTM) test methods or suggested procedures.

Classification: The field classification was verified through laboratory examination, in accordance with the Unified Soil Classification System. The final classification is shown on the enclosed Exploratory Logs provided in Appendix A.

Maximum Dry Density and Optimum Moisture Content: The maximum dry density and optimum moisture content were determined for selected samples of earth materials taken from the site. The laboratory standard tests were in accordance with ASTM D-1557-12. The test result is presented on Table 6.

TABLE 6

Location	Soil Type	Maximum Dry Density (γm-pcf)	Optimum Moisture Content (ωopt-%)
TP-1 @ 8'	1	117.0	13.2

Moisture/Density: The field moisture content and dry unit weight were determined for each of the undisturbed soil samples. Test procedures were conducted in accordance with ASTM D2216-10 and ASTM D7263-09 (Method A), respectively. This information is useful in providing a gross picture of the soil consistency or variation among exploratory excavation. The field moisture content was determined as a percentage of the dry unit weight. The dry unit weight was determined in pounds per cubic foot (pcf). The test results are presented on Table 7 and 8.

TABLE 7: Moisture Content ASTM D 2216 - 10

Sample	Wet Wt (gr)	Dry Wt (gr)	Moisture Content
TP-1 @ 4'	100.0	83.2	20.2
TP-3 @ 6'	100.0	83.3	20.0

TABLE 8: Density ASTM D 7263 - 09 (Method A)

Sample Location	Soil Type	Field Moisture Content (%)	Field Dry Density (γd-pcf)	Max. Dry Density (γm-pcf)	In-place Relative Compaction (%)	Degree of Saturation (%)
TP-1 @ 4'	1	20.2	85.6	117.0	73.1	57.0
TP-3 @ 6'	1	20.0	82.8	117.0	70.7	53.0

APPENDIX B

TEST PIT LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM					
Major Divisions			**Group Symbols**	**Typical Names**	
Coarse-Grained Soils More than 50% retained on No. 200 sieve	Gravels 50% or more of coarse fraction retained on No. 4 sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines	
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines	
		Gravel with	GM	Silty gravels gravel-sand-silt mixtures	
			GC	Clayey gravels, gravel-sand-clay mixtures	
	Sands more than 50% of coarse fraction passes No. 4 sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines	
			SP	Poorly graded sands and gravelly sands, little or no fines	
		Sands with Fines	SM	Silty sands, sand-silt mixtures	
			SC	Clayey sands, sand-clay mixtures	
Fine-Grained Soils 50% or more passes No. 200 sieve	Silts and Clays Liquid limit 50% or less		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands	
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
			OL	Organic silts and organic silty clays of low plasticity	
	Silts and Clays Liquid limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts	
			CH	Inorganic clays of high plasticity, fat clays	
			OH	Organic clays of medium to high plasticity	
Highly Organic Soils			PT	Peat, mucic, and other highly organic soils	

CONSISTENCY OR RELATIVE DENSITY

CRITERIA

Standard Penetration Test

Penetration Resistance N (blows/ft)	Relative Density
0 - 4	Very loose
4 - 10	Loose
10 - 30	Medium
30 - 50	Dense
> 50	Very dense

Standard Penetration Test

Penetration Resistance N (blows/ft)	Consistency	Unconfined Compressive Strength (tons/ft^2)
<2	Very Soft	<0.25
2 - 4	Soft	0.25 - .050
4 - 8	Medium	0.50 - 1.00
8 - 15	Stiff	1.00 - 2.00
15 - 30	Very Stiff	2.00 - 4.00
>30	Hard	>4.00

	3"		3/4"	#4	#10	#40	#200 U.S. Standard Sieve
Unified Soil Classification	Cobbles	Gravel		Sand			Silt or Clay
		coarse	fine	coarse	medium	fine	

MOISTURE CONDITIONS

Dry	Absence of moisture: dusty, dry to the touch
Slightly Moist	Below optimum moisture content for compaction
Moist	Near optimum moisture content
Very Moist	Above optimum moisture content
Wet	Visible free water; below water table

MATERIAL QUANTITY

trace	0 - 5 %
few	5 - 10 %
little	10 - 25 %
some	25 - 45 %

OTHER SYMBOLS

C Core Sample
S SPT Sample
B Bulk Sample
– Groundwater
Qp Pocket Penetrometer

BASIC LOG FORMAT:
Group name, Group symbol, (grain size), color, moisture, consistency or relative density. Additional comments: odor, presence of roots, mica, gypsum, coarse grained particles, etc.

EXAMPLE:
Sand (SP), fine to medium grained, brown, moist, loose, trace silt, little fine gravel, few cobbles up to 4" in size, some hair roots and rootlets.

GeoSoils, Inc.

W.O. 8543-A-SC
Honarvar
APN 264-110-30-00, Rancho Santa Fe
Logged By: RBB
February 27, 2023

LOG OF EXPLORATORY TEST PIT

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-1	~506	0-¾	SM	BULK @ ¾-2½ UND @ 2	13.9 16.8	 105.6	**ARTIFICIAL FILL - UNDOCUMENTED:** SILTY SAND, variegated light gray and dark brownish gray, wet, loose; trace CLAY; trace angular and subangular gravels and cobbles, abundant organic matter.
		¾-2½	CL				SANDY CLAY, brown and light gray, wet, soft; trace angular and subangular gravels and cobbles.
		2½-3¾	SC				**QUATERNARY COLLUVIUM**: CLAYEY SAND, dark grayish brown, saturated, loose; trace subangular gravels and cobbles, perched groundwater seepage between approximately 2½ and 3¾.
		3¾-5¼	CH	SM BAG @ 3¾-4½	29.8		**WEATHERED MESOZOIC METAVOLCANIC BEDROCK**: SANDY CLAY, reddish brown and olive brown, wet, stiff; trace subangular and angular gravels.
		5¼-8½	CL	BULK @ 7½-8½	12.5		**MESOZOIC METAVOLCANIC BEDROCK**: METAVOLCANICS, light tan and light gray, moist, very stiff; disintegrates to SANDY CLAY with angular and subangular gravels upon excavation.
	UND - Relatively Undisturbed Sample BULK - Representative Bulk Soil Sample SM BAG - Small Bag Sample						Total Depth = 8½' Perched groundwater seepage encountered between approximately 2½' and 3¾' Caving encountered between approximately 0 and 2' Backfilled 2-27-23

GeoSoils, Inc.

W.O. 8543-A-SC
Honarvar
APN 264-110-30-00, Rancho Santa Fe
Logged By: RBB
February 27, 2023

LOG OF EXPLORATORY TEST PIT

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-2	~530	0-1	SC				**QUATERNARY COLLUVIUM**: CLAYEY SAND, dark grayish brown, wet to locally saturated, loose; trace angular and subangular gravels.
		1-4	SC	UND @ 1¼	17.9	101.2	**WEATHERED MESOZOIC METAVOLCANIC BEDROCK**: CLAYEY, SAND, light reddish brown, moist, medium dense; trace subangular and angular gravels.
		4-5½					**MESOZOIC METAVOLCANIC BEDROCK**: METAVOLCANICS, dark gray and olive brown, dry, dense; disintegrates to angular gravels and cobbles with trace CLAYEY SAND upon excavation. Joints: N70°W, 50°NE N2°W, 49°NE N75°W, 64°SW N70°E, 60°NW
	UND= Relatively Undisturbed Sample						Practical refusal at approximately 5½' Perched groundwater seepage encountered at approximately 2¾' No caving encountered Backfilled 2-27-23

GeoSoils, Inc.

W.O. 8543-A-SC
Honarvar
APN 264-110-30-00, Rancho Santa Fe
Logged By: RBB
February 27, 2023

LOG OF EXPLORATORY TEST PIT

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-3	~486	0-½	SC				**MULCH**.
		½-1	SC				**QUATERNARY COLLUVIUM**: CLAYEY SAND, dark grayish brown, saturated, loose; trace roots.
		1-3	CH	SM BAG @ 1½-2	23.4		**WEATHERED MESOZOIC METAVOLCANIC BEDROCK**: SANDY CLAY, brown, wet, stiff.
		3-4½					**MESOZOIC METAVOLCANIC BEDROCK**: METAVOLCANICS, dark gray and olive brown, dry, dense to very dense; disintegrates to angular and subangular gravels, cobbles, and boulders. Joints: N37°E, 69°SE N58°W, 40°NE N45°E, Vertical N64°W, Vertical
	SM BAG - Small Bag Sample						Practical refusal at approximately 4½' Perched groundwater seepage encountered between approximately ½' and 1' No caving encountered Backfilled 2-27-23

W.O. 8543-A-SC
Honarvar
APN 264-110-30-00, Rancho Santa Fe
Logged By: RBB
February 27, 2023

LOG OF EXPLORATORY TEST PIT

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-4	~444	0-½	SC				**ARTIFICIAL FILL - UNDOCUMENTED:** SANDY CLAY, brown, moist, soft; trace angular to subangular gravels and cobbles.
		½-2½	SM	UND @ 2½	5.0	127.6	**MESOZOIC METAVOLCANIC BEDROCK**: METAVOLCANICS, gray, dry, very dense; disintegrates to angular and subangular gravels, cobbles, and boulders (boulders up to approximately 3½' in longest dimension), becomes massive at approximately 2½'. Joints: N75°E, 45°NW N77°W, Vertical
							Practical refusal at approximately 2½' No groundwater or caving encountered Backfilled 2-27-23

APPENDIX C

SEISMICITY DATA

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          *    E Q F A U L T    *
          *                     *
          *    Version 3.00     *
          *                     *
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DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 8543-A-SC

DATE: 03-01-2023

JOB NAME: HONARVAR

CALCULATION NAME: 8543

FAULT-DATA-FILE NAME: C:\Users\Ryan\Documents\EQFAULT1\CGSFLTE.DAT

SITE COORDINATES:
   SITE LATITUDE: 33.0542
   SITE LONGITUDE: 117.1769

SEARCH RADIUS: 62.4 mi

ATTENUATION RELATION: 13) Bozorgnia Campbell Niazi (1999) Hor.-Hard Rock-Cor.
   UNCERTAINTY (M=Median, S=Sigma): S   Number of Sigmas: 1.0
   DISTANCE MEASURE: cdist
   SCOND: 0
   Basement Depth: 5.00 km   Campbell SSR: 0   Campbell SHR: 1
   COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: C:\Users\Ryan\Documents\EQFAULT1\CGSFLTE.DAT

MINIMUM DEPTH VALUE (km): 3.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
ROSE CANYON	9.1(14.7)	7.2	0.413	X
NEWPORT-INGLEWOOD (Offshore)	16.3(26.3)	7.1	0.220	IX
CORONADO BANK	23.4(37.6)	7.6	0.217	VIII
ELSINORE (JULIAN)	23.6(38.0)	7.1	0.151	VIII
ELSINORE (TEMECULA)	24.3(39.1)	6.8	0.119	VII
EARTHQUAKE VALLEY	35.7(57.4)	6.5	0.065	VI
ELSINORE (GLEN IVY)	41.9(67.5)	6.8	0.067	VI
SAN JACINTO-ANZA	46.2(74.3)	7.2	0.080	VII
SAN JOAQUIN HILLS	46.9(75.5)	6.6	0.074	VII
PALOS VERDES	47.0(75.6)	7.3	0.085	VII
ELSINORE (COYOTE MOUNTAIN)	47.7(76.7)	6.8	0.059	VI
SAN JACINTO-COYOTE CREEK	47.7(76.8)	6.6	0.051	VI
SAN JACINTO-SAN JACINTO VALLEY	49.8(80.1)	6.9	0.060	VI
NEWPORT-INGLEWOOD (L.A.Basin)	57.7(92.9)	7.1	0.059	VI
CHINO-CENTRAL AVE. (Elsinore)	57.8(93.0)	6.7	0.063	VI
SAN JACINTO - BORREGO	57.8(93.1)	6.6	0.042	VI
WHITTIER	61.7(99.3)	6.8	0.045	VI

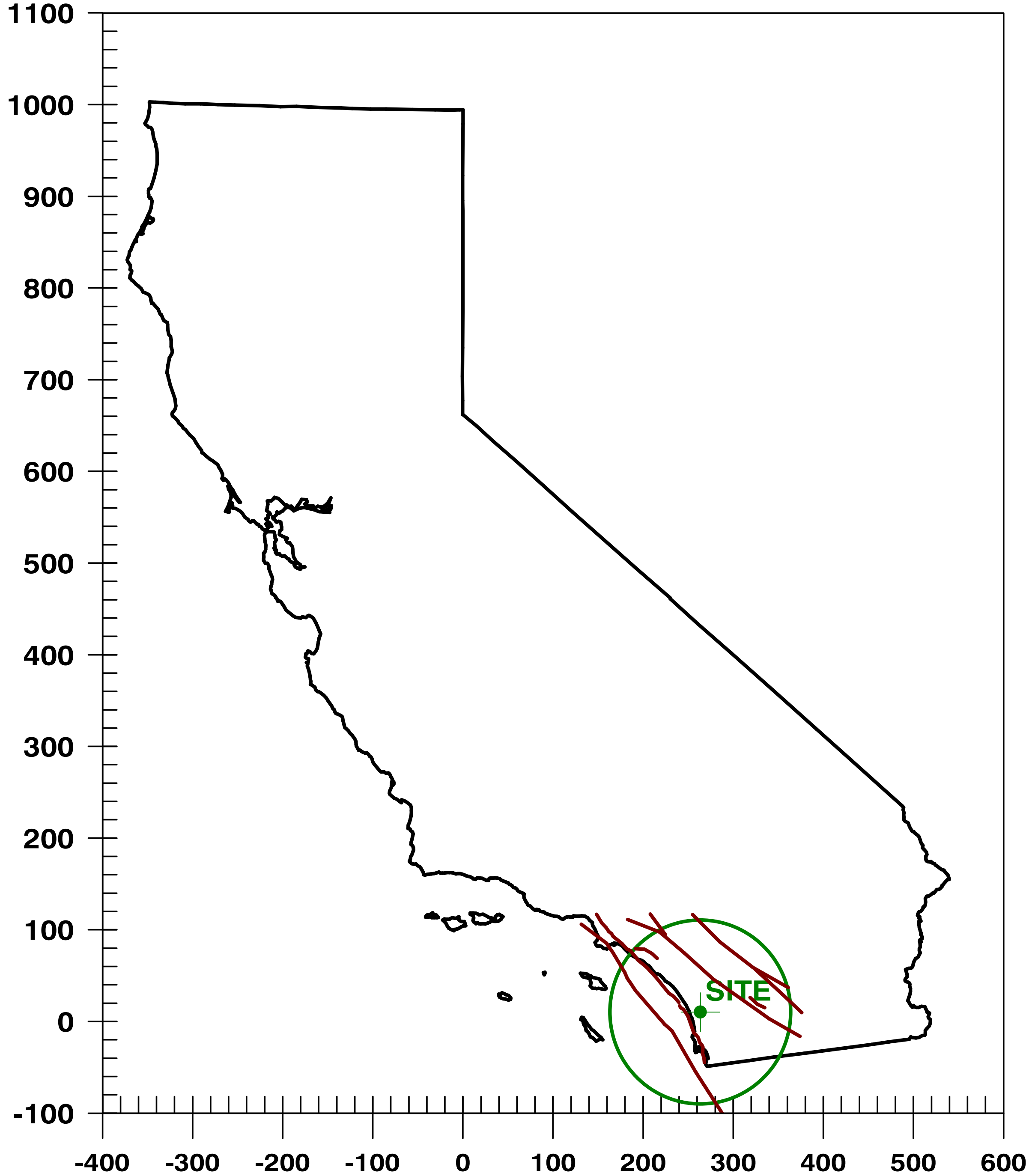
-END OF SEARCH- 17 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ROSE CANYON FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 9.1 MILES (14.7 km) AWAY.

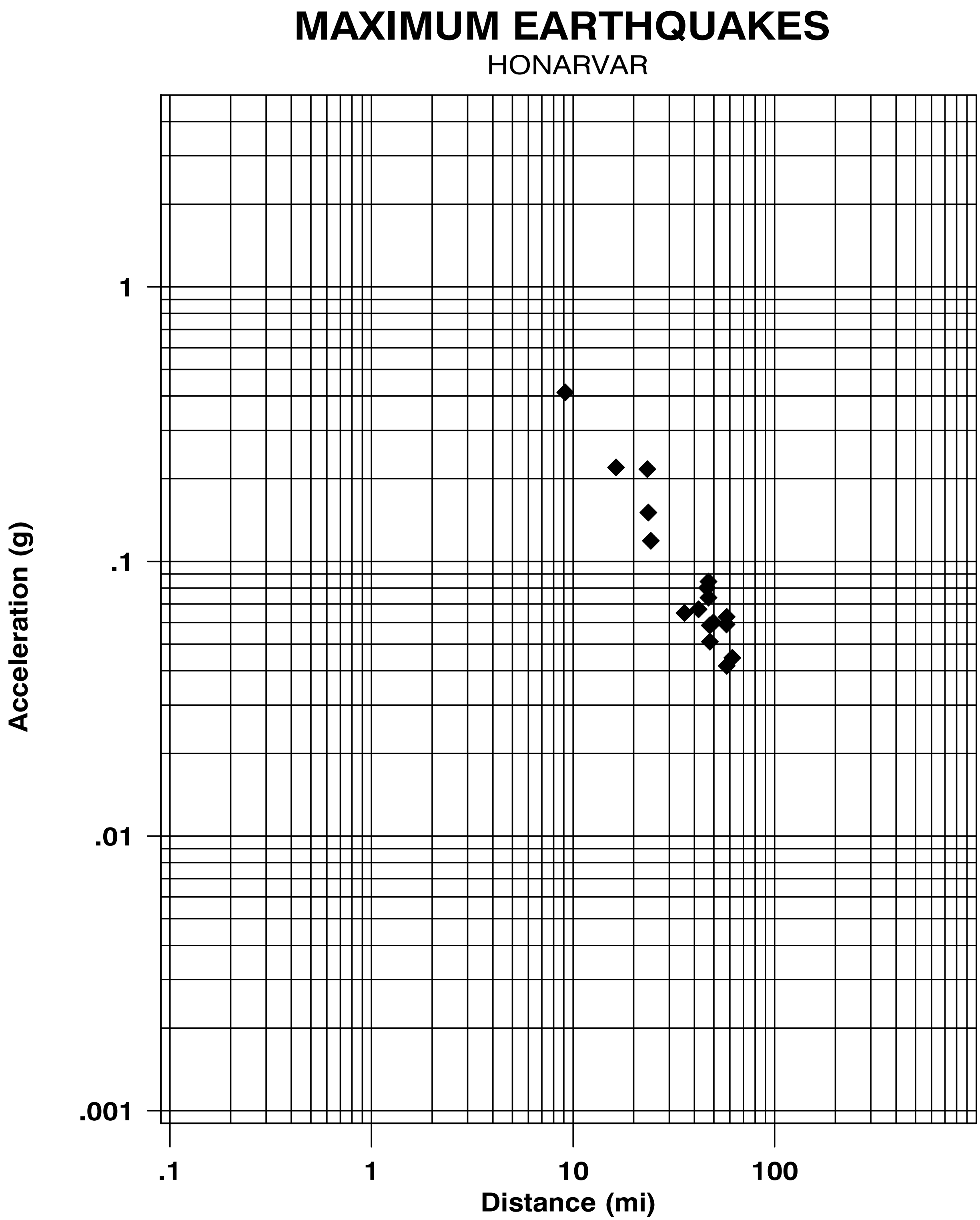
LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.4128 g

CALIFORNIA FAULT MAP

HONARVAR



W.O. 8543-A-SC
Plate C-3



W.O. 8543-A-SC
Plate C-4

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*                       *
*     Version 3.00      *
*                       *
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ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 8543-A-SC

DATE: 03-01-2023

JOB NAME: HONARVAR

EARTHQUAKE-CATALOG-FILE NAME: C:\Users\Ryan\Documents\EQSEARCH\ALLQUAKE-2021.DAT

MAGNITUDE RANGE:
MINIMUM MAGNITUDE: 5.00
MAXIMUM MAGNITUDE: 9.00

SITE COORDINATES:
SITE LATITUDE: 33.0542
SITE LONGITUDE: 117.1769

SEARCH DATES:
START DATE: 1800
END DATE: 2023

SEARCH RADIUS:
62.4 mi
100.4 km

ATTENUATION RELATION: 13) Bozorgnia Campbell Niazi (1999) Hor.-Hard Rock-Cor.
UNCERTAINTY (M=Median, S=Sigma): S Number of Sigmas: 1.0
ASSUMED SOURCE TYPE: SS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
SCOND: 0 Depth Source: A
Basement Depth: 5.00 km Campbell SSR: 0 Campbell SHR: 1
COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km): 3.0

EARTHQUAKE SEARCH RESULTS

Page 1

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	33.0000	117.3000	11/22/1800	2130 0.0	0.0	6.50	0.295	IX	8.0(12.9)
MGI	33.0000	117.0000	09/21/1856	730 0.0	0.0	5.00	0.083	VII	10.9(17.5)
MGI	32.8000	117.1000	05/25/1803	0 0 0.0	0.0	5.00	0.051	VI	18.1(29.1)
DMG	32.7000	117.2000	05/27/1862	20 0 0.0	0.0	5.90	0.065	VI	24.5(39.4)
T-A	32.6700	117.1700	12/00/1856	0 0 0.0	0.0	5.00	0.035	V	26.5(42.7)
T-A	32.6700	117.1700	10/21/1862	0 0 0.0	0.0	5.00	0.035	V	26.5(42.7)
T-A	32.6700	117.1700	05/24/1865	0 0 0.0	0.0	5.00	0.035	V	26.5(42.7)
DMG	32.8000	116.8000	10/23/1894	23 3 0.0	0.0	5.70	0.050	VI	28.0(45.1)
DMG	33.2000	116.7000	01/01/1920	235 0.0	0.0	5.00	0.031	V	29.4(47.2)
MGI	33.2000	116.6000	10/12/1920	1748 0.0	0.0	5.30	0.031	V	34.8(56.1)
PAS	32.9710	117.8700	07/13/1986	1347 8.2	6.0	5.30	0.027	V	40.5(65.2)
DMG	33.0000	116.4330	06/04/1940	1035 8.3	0.0	5.10	0.022	IV	43.2(69.6)
DMG	33.7000	117.4000	05/13/1910	620 0.0	0.0	5.00	0.019	IV	46.4(74.7)
DMG	33.7000	117.4000	04/11/1910	757 0.0	0.0	5.00	0.019	IV	46.4(74.7)
DMG	33.7000	117.4000	05/15/1910	1547 0.0	0.0	6.00	0.036	V	46.4(74.7)
GSG	33.4200	116.4890	07/07/2010	235333.5	14.0	5.50	0.026	V	47.1(75.7)
DMG	33.7100	116.9250	09/23/1963	144152.6	16.5	5.00	0.019	IV	47.5(76.5)
GSP	33.5290	116.5720	06/12/2005	154146.5	14.0	5.20	0.021	IV	47.9(77.1)
DMG	33.6990	117.5110	05/31/1938	83455.4	10.0	5.50	0.025	V	48.5(78.1)
DMG	33.7500	117.0000	04/21/1918	223225.0	0.0	6.80	0.057	VI	49.1(79.0)
DMG	33.7500	117.0000	06/06/1918	2232 0.0	0.0	5.00	0.018	IV	49.1(79.0)
PAS	33.5010	116.5130	02/25/1980	104738.5	13.6	5.50	0.025	V	49.2(79.2)
GSP	33.5080	116.5140	10/31/2001	075616.6	15.0	5.10	0.019	IV	49.5(79.6)
DMG	33.5000	116.5000	09/30/1916	211 0.0	0.0	5.00	0.018	IV	49.7(80.0)
GSP	33.4315	116.4427	06/10/2016	080438.7	12.3	5.19	0.020	IV	49.8(80.1)
DMG	33.3430	116.3460	04/28/1969	232042.9	20.0	5.80	0.028	V	52.0(83.6)
DMG	33.8000	117.0000	12/25/1899	1225 0.0	0.0	6.40	0.040	V	52.5(84.5)
DMG	33.4000	116.3000	02/09/1890	12 6 0.0	0.0	6.30	0.035	V	56.0(90.1)
DMG	32.7000	116.3000	02/24/1892	720 0.0	0.0	6.70	0.046	VI	56.4(90.8)
MGI	33.8000	117.6000	04/22/1918	2115 0.0	0.0	5.00	0.016	IV	57.0(91.7)
DMG	33.2000	116.2000	05/28/1892	1115 0.0	0.0	6.30	0.034	V	57.4(92.3)
DMG	33.4080	116.2610	03/25/1937	1649 1.8	10.0	6.00	0.028	V	58.3(93.8)
DMG	33.9000	117.2000	12/19/1880	0 0 0.0	0.0	6.00	0.028	V	58.4(94.0)
T-A	32.2500	117.5000	01/13/1877	20 0 0.0	0.0	5.00	0.015	IV	58.6(94.3)
DMG	33.5750	117.9830	03/11/1933	518 4.0	0.0	5.20	0.017	IV	58.8(94.6)
DMG	33.2830	116.1830	03/23/1954	41450.0	0.0	5.10	0.016	IV	59.6(95.9)
DMG	33.2830	116.1830	03/19/1954	95429.0	0.0	6.20	0.031	V	59.6(95.9)
DMG	33.2830	116.1830	03/19/1954	95556.0	0.0	5.00	0.015	IV	59.6(95.9)
DMG	33.2830	116.1830	03/19/1954	102117.0	0.0	5.50	0.020	IV	59.6(95.9)

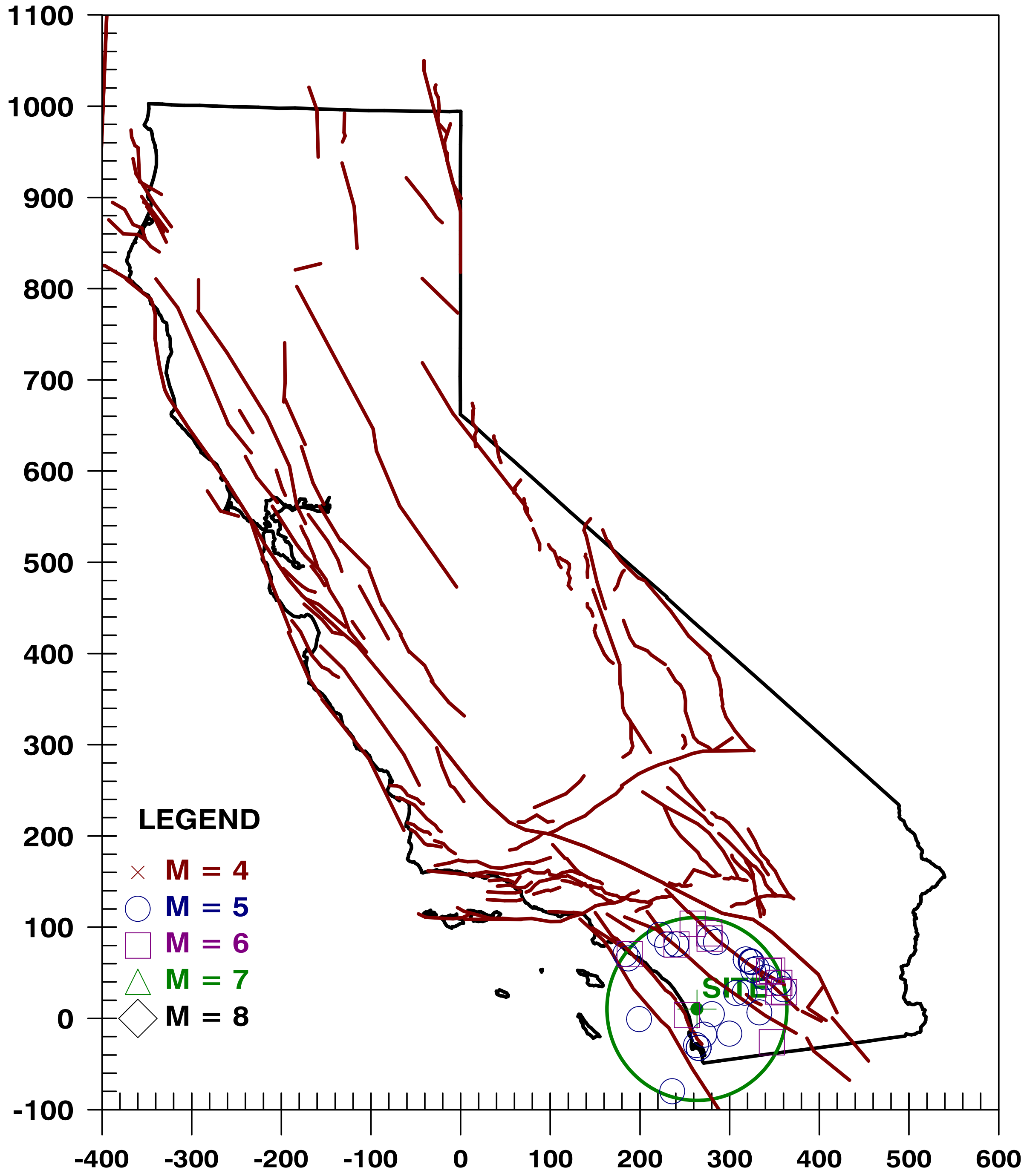
```
DMG |33.6170|117.9670|03/11/1933| 154 7.8|  0.0| 6.30| 0.033 |  V | 59.9( 96.4)
DMG |33.1900|116.1290|04/09/1968| 22859.1| 11.1| 6.40| 0.034 |  V | 61.3( 98.7)
DMG |33.2170|116.1330|08/15/1945|175624.0|  0.0| 5.70| 0.022 | IV | 61.4( 98.8)
DMG |33.6170|118.0170|03/14/1933|19 150.0|  0.0| 5.10| 0.015 | IV | 62.1(100.0)

*******************************************************************************
```

```
-END OF SEARCH-   43 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH:   1800  TO  2023

LENGTH OF SEARCH TIME:   224  years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 8.0 MILES (12.9 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 6.8

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.295 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
  a-value=  0.547
  b-value=  0.288
  beta-value=  0.664


------------------------------------
TABLE OF MAGNITUDES AND EXCEEDANCES:
------------------------------------
```

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	43	0.19196
4.5	43	0.19196
5.0	43	0.19196
5.5	20	0.08929
6.0	12	0.05357
6.5	3	0.01339

EARTHQUAKE EPICENTER MAP

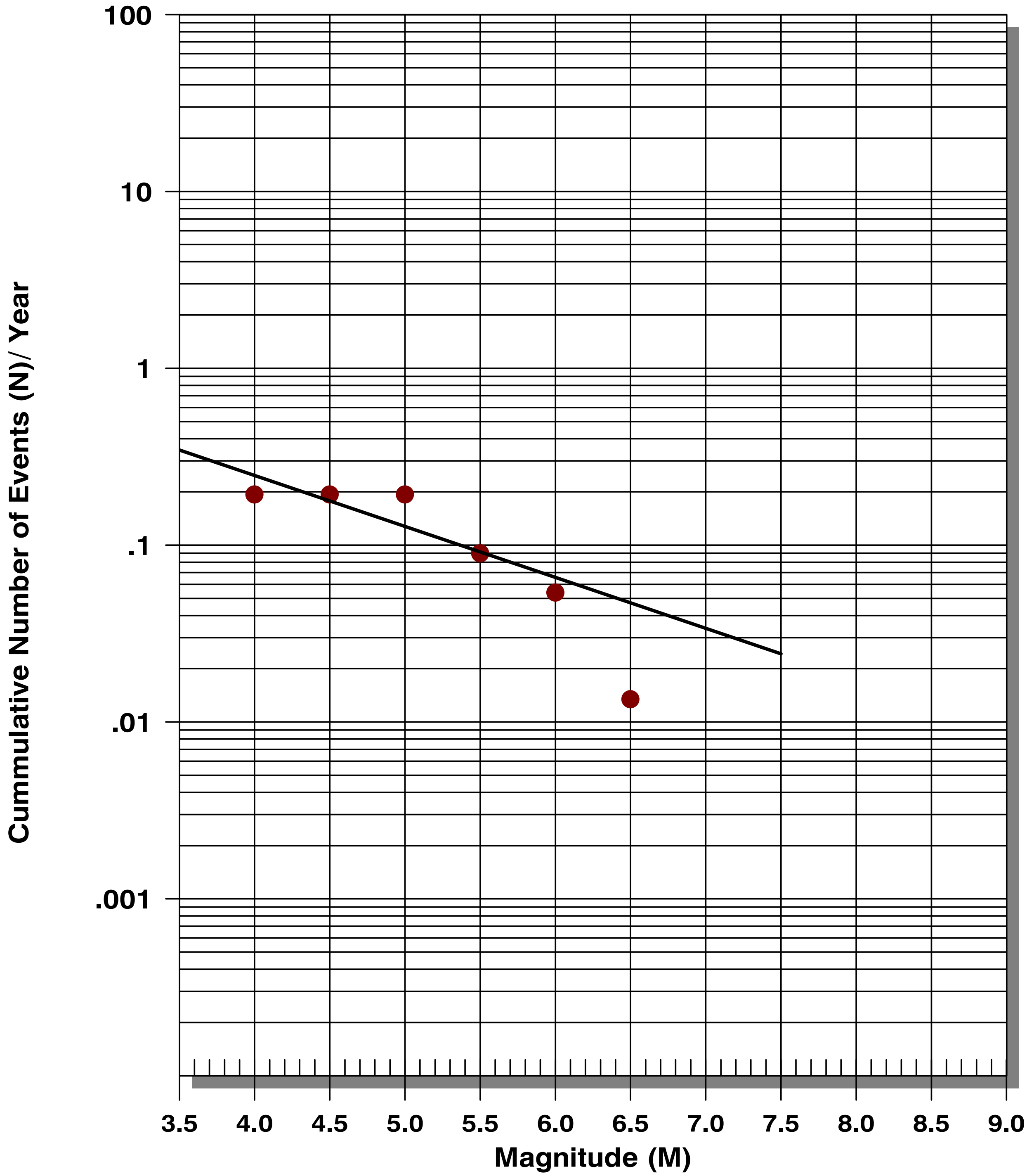
HONARVAR



W.O. 8543-A-SC
Plate C-8

EARTHQUAKE RECURRENCE CURVE

HONARVAR

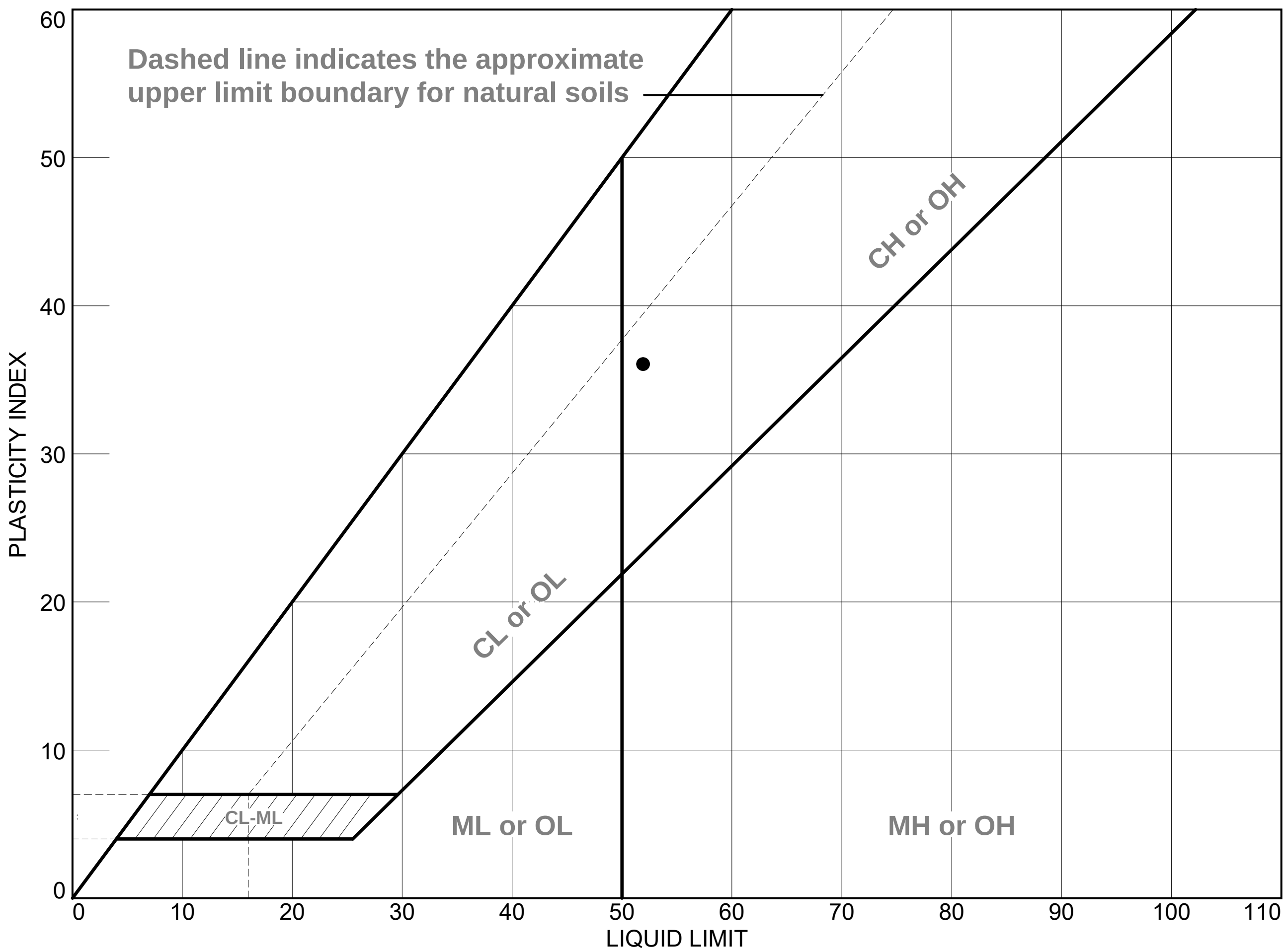


W.O. 8543-A-SC
Plate C-9

APPENDIX D

LABORATORY DATA

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

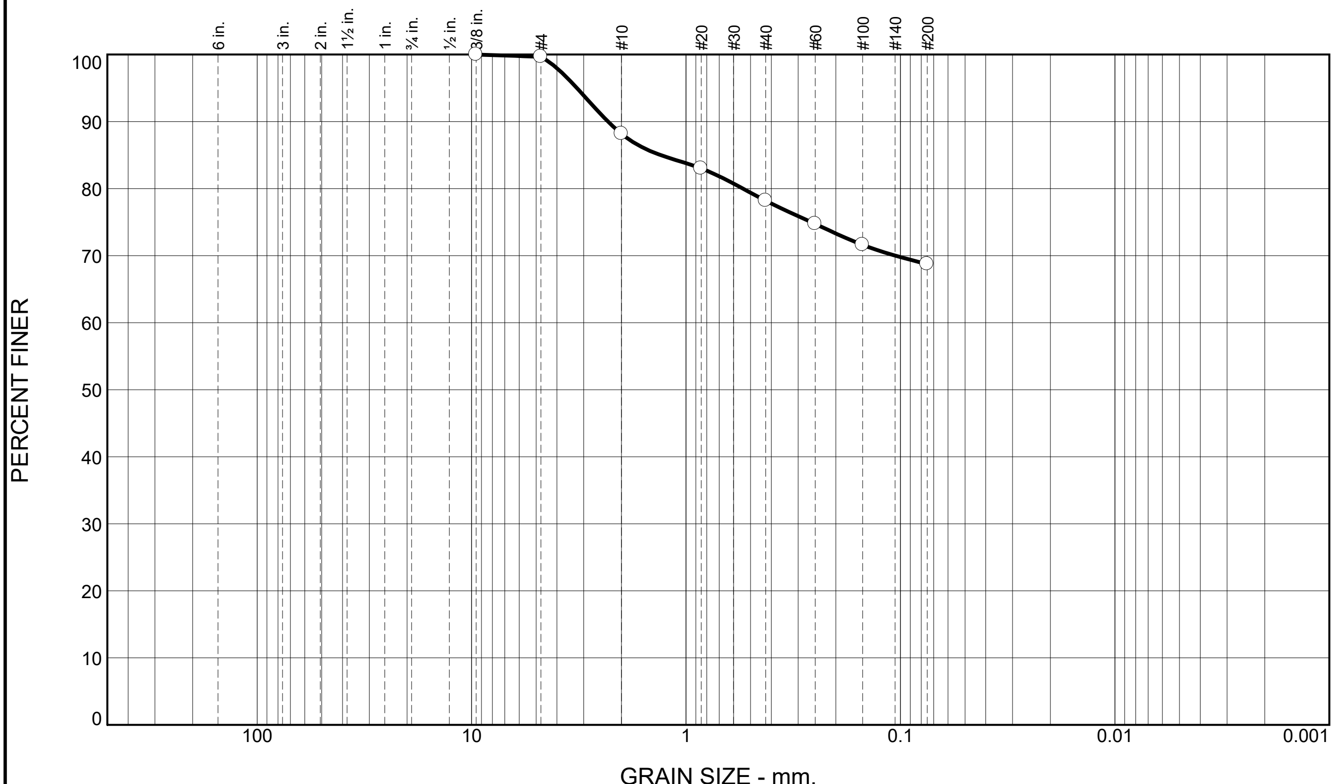
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	TP-101/T-103	TP-101/TP-103	1.5-4.5	-	16	52	36	CH

GeoSoils, Inc.

Client: Honarvar
Project: Rancho Santa Fe
Project No.: 8543-A-SC **Plate**

Tested By: TR **Checked By:** TR

Particle Size Distribution Report - ASTM C136



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.3	11.5	10.0	9.5	68.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375	100.0		
#4	99.7		
#10	88.2		
#20	83.0		
#40	78.2		
#60	74.7		
#100	71.6		
#200	68.7		

* (no specification provided)

Soil Description

Dark Reddish Brown Sandy Clay

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 2.3110 D_{85}= 1.3133 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CH AASHTO=

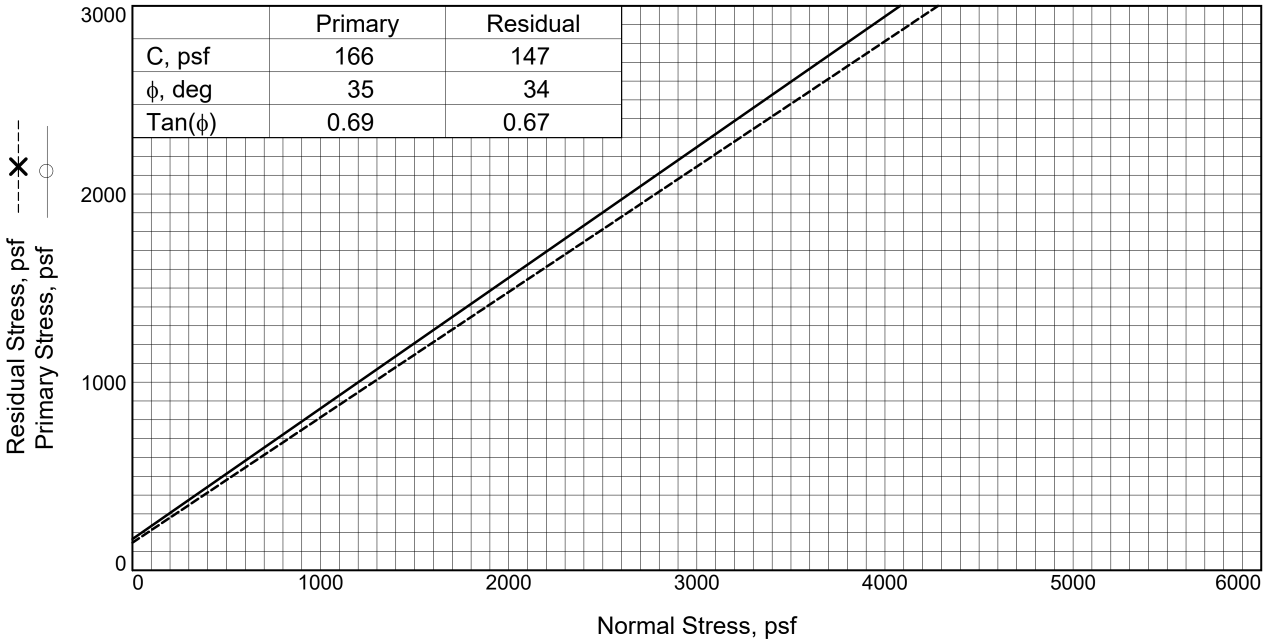
Remarks

Source of Sample: TP-101 **Depth:** 3.75-4.25
Sample Number: TP-101 **Date:** 3-17-23

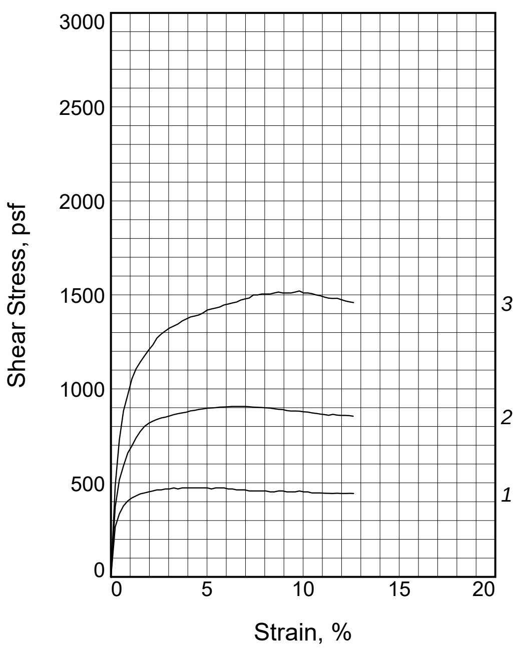
GeoSoils, Inc.

Client: Honarvar
Project: Rancho Santa Fe
Project No: 8543-A-SC **Plate**

Tested By: TR **Checked By:** TR



	Primary	Residual
C, psf	166	147
ϕ, deg	35	34
Tan(ϕ)	0.69	0.67



	Sample No.	1	2	3
Initial	Water Content, %	10.7	10.7	10.7
	Dry Density, pcf	109.7	109.7	109.5
	Saturation, %	53.8	53.8	53.6
	Void Ratio	0.5367	0.5367	0.5389
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	16.4	16.7	17.1
	Dry Density, pcf	109.8	110.2	111.3
	Saturation, %	82.8	85.2	89.8
	Void Ratio	0.5352	0.5290	0.5143
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	0.98
	Normal Stress, psf	500	1000	2000
	Primary Stress, psf	473	906	1521
	Strain, %	3.3	6.3	9.8
	Residual Stress, psf	447	863	1463
	Strain, %	10.5	11.1	12.4
	Strain rate, in./min.	0.002	0.002	0.002

Sample Type: Remolded
Description: Brown Clayey Sand

Specific Gravity= 2.70
Remarks: Remolded at 90% maximum density

Plate ________

Client: Honarvar

Project: Rancho Santa Fe

Source of Sample: TP-101 **Depth:** 0.75-2.5
Sample Number: TP-101
Proj. No.: 8543-A-SC **Date Sampled:**

GeoSoils, Inc.

Tested By: TR **Checked By:** TR

5741 Palmer Way, Carlsbad CA 92010
Phone (760) 438-3155

CORROSION REPORT SUMMARY

Project No: **8543-A-SC**
Project Name: **Honarvar**
Report Date: **March 21, 2023**

SAMPLE ID	pH (H+)	Minimum Resistivity (ohm/cm)	Sulfate Content (wt%)	Chloride Content (mg/kg)
TP-101, 0.75-2.5ft	6.5	1000	0.004	53

Sample testing in accordance with: pH - CTM 643, Resistivity - CTM 643
Sulfate - CTM 417, Chloride - CTM 422

Remarks: ______________________
____

APPENDIX E

TABLE B.2-1 OF THE COUNTY OF SAN DIEGO'S "BMP DESIGN MANUAL"

Table B.2-1: Infiltration Restrictions

Restriction Element		Is Element Applicable? (Yes/No)
Mandatory Considerations	BMP is within 100' of Contaminated Soils	
	BMP is within 100' of Industrial Activities Lacking Source Control	
	BMP is within 100' of Well/Groundwater Basin	
	BMP is within 50' of Septic Tanks/Leach Fields	
	BMP is within 10' of Structures/Tanks/Walls	
	BMP is within 10' of Sewer Utilities	
	BMP is within 10' of Groundwater Table	
	BMP is within Hydric Soils	
	BMP is within Highly Liquefiable Soils and has Connectivity to Structures	
	BMP is within 1.5 Times the Height of Adjacent Steep Slopes (≥25%)	
	County Staff has Assigned "Restricted" Infiltration Category	
Optional Considerations	BMP is within Predominantly Type D Soil	YES
	BMP is within 10' of Property Line	
	BMP is within Fill Depths of ≥5' (Existing or Proposed)	YES
	BMP is within 10' of Underground Utilities	
	BMP is within 250' of Ephemeral Stream	
	Other (Provide detailed geotechnical support)	YES *
Result	**Unrestricted**. None of the restriction elements above are applicable.	
	Restricted. One or more of the restriction elements above are applicable.	YES

* See "Onsite Soils and Storm Water Infiltration Feasibility" Section of Report Text.

APPENDIX F

GENERAL EARTHWORK AND GRADING GUIDELINES

GENERAL EARTHWORK AND GRADING GUIDELINES

General

These guidelines present general procedures and requirements for earthwork and grading as shown on the approved grading plans, including preparation of areas to be filled, placement of fill, installation of subdrains, excavations, and appurtenant structures or flatwork. The recommendations contained in the geotechnical report are part of these earthwork and grading guidelines and would supercede the provisions contained hereafter in the case of conflict. Evaluations performed by the consultant during the course of grading may result in new or revised recommendations which could supercede these guidelines or the recommendations contained in the geotechnical report. Generalized details follow this text.

The contractor is responsible for the satisfactory completion of all earthwork in accordance with provisions of the project plans and specifications and latest adopted Code. In the case of conflict, the most onerous provisions shall prevail. The project geotechnical engineer and engineering geologist (geotechnical consultant), and/or their representatives, should provide observation and testing services, and geotechnical consultation during the duration of the project.

EARTHWORK OBSERVATIONS AND TESTING

Geotechnical Consultant

Prior to the commencement of grading, a qualified geotechnical consultant (soil engineer and engineering geologist) should be employed for the purpose of observing earthwork procedures and testing the fills for general conformance with the recommendations of the geotechnical report(s), the approved grading plans, and applicable grading codes and ordinances.

The geotechnical consultant should provide testing and observation so that an evaluation may be made that the work is being accomplished as specified. It is the responsibility of the contractor to assist the consultants and keep them apprised of anticipated work schedules and changes, so that they may schedule their personnel accordingly.

All remedial removals, clean-outs, prepared ground to receive fill, key excavations, and subdrain installation should be observed and documented by the geotechnical consultant prior to placing any fill. It is the contractor's responsibility to notify the geotechnical consultant when such areas are ready for observation.

Laboratory and Field Tests

Maximum dry density tests to determine the degree of compaction should be performed in accordance with American Standard Testing Materials test method ASTM designation D 1557. Random or representative field compaction tests should be performed in accordance with test methods ASTM designation D 1556, D 2937 or D 2922, and D 3017, at intervals of approximately ± 2 feet of fill height or approximately every 1,000 cubic yards placed. These criteria would vary depending on the soil conditions and the size of the project. The location and frequency of testing would be at the discretion of the geotechnical consultant.

Contractor's Responsibility

All clearing, site preparation, and earthwork performed on the project should be conducted by the contractor, with observation by a geotechnical consultant, and staged approval by the governing agencies, as applicable. It is the contractor's responsibility to prepare the ground surface to receive the fill, to the satisfaction of the geotechnical consultant, and to place, spread, moisture condition, mix, and compact the fill in accordance with the recommendations of the geotechnical consultant. The contractor should also remove all non-earth material considered unsatisfactory by the geotechnical consultant.

Notwithstanding the services provided by the geotechnical consultant, it is the sole responsibility of the contractor to provide adequate equipment and methods to accomplish the earthwork in strict accordance with applicable grading guidelines, latest adopted Codes or agency ordinances, geotechnical report(s), and approved grading plans. Sufficient watering apparatus and compaction equipment should be provided by the contractor with due consideration for the fill material, rate of placement, and climatic conditions. If, in the opinion of the geotechnical consultant, unsatisfactory conditions such as questionable weather, excessive oversized rock or deleterious material, insufficient support equipment, etc., are resulting in a quality of work that is not acceptable, the consultant will inform the contractor, and the contractor is expected to rectify the conditions, and if necessary, stop work until conditions are satisfactory.

During construction, the contractor shall properly grade all surfaces to maintain good drainage and prevent ponding of water. The contractor shall take remedial measures to control surface water and to prevent erosion of graded areas until such time as permanent drainage and erosion control measures have been installed.

SITE PREPARATION

All major vegetation, including brush, trees, thick grasses, organic debris, and other deleterious material, should be removed and disposed of off-site. These removals must be concluded prior to placing fill. In-place existing fill, soil, alluvium, colluvium, or rock materials, as evaluated by the geotechnical consultant as being unsuitable, should be removed prior to any fill placement. Depending upon the soil conditions, these materials may be reused as compacted fills. Any materials incorporated as part of the compacted fills should be approved by the geotechnical consultant.

Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipelines, or other structures not located prior to grading, are to be removed or treated in a manner recommended by the geotechnical consultant. Soft, dry, spongy, highly fractured, or otherwise unsuitable ground, extending to such a depth that surface processing cannot adequately improve the condition, should be overexcavated down to firm ground and approved by the geotechnical consultant before compaction and filling operations continue. Overexcavated and processed soils, which have been properly mixed and moisture conditioned, should be re-compacted to the minimum relative compaction as specified in these guidelines.

Existing ground, which is determined to be satisfactory for support of the fills, should be scarified (ripped) to a minimum depth of 6 to 8 inches, or as directed by the geotechnical consultant. After the scarified ground is brought to optimum moisture content, or greater and mixed, the materials should be compacted as specified herein. If the scarified zone is greater than 6 to 8 inches in depth, it may be necessary to remove the excess and place the material in lifts restricted to about 6 to 8 inches in compacted thickness.

Existing ground which is not satisfactory to support compacted fill should be overexcavated as required in the geotechnical report, or by the on-site geotechnical consultant. Scarification, disc harrowing, or other acceptable forms of mixing should continue until the soils are broken down and free of large lumps or clods, until the working surface is reasonably uniform and free from ruts, hollows, hummocks, mounds, or other uneven features, which would inhibit compaction as described previously.

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical [h:v]), the ground should be stepped or benched. The lowest bench, which will act as a key, should be a minimum of 15 feet wide and should be at least 2 feet deep into firm material, and approved by the geotechnical consultant. In fill-over-cut slope conditions, the recommended minimum width of the lowest bench or key is also 15 feet, with the key founded on firm material, as designated by the geotechnical consultant. As a general rule, unless specifically recommended otherwise by the geotechnical consultant, the minimum width of fill keys should be equal to ½ the height of the slope.

Standard benching is generally 4 feet (minimum) vertically, exposing firm, acceptable material. Benching may be used to remove unsuitable materials, although it is understood that the vertical height of the bench may exceed 4 feet. Pre-stripping may be considered for unsuitable materials in excess of 4 feet in thickness.

All areas to receive fill, including processed areas, removal areas, and the toes of fill benches, should be observed and approved by the geotechnical consultant prior to placement of fill. Fills may then be properly placed and compacted until design grades (elevations) are attained.

COMPACTED FILLS

Any earth materials imported or excavated on the property may be used in the fill provided that each material has been evaluated to be suitable by the geotechnical consultant. These materials should be free of roots, tree branches, other organic matter, or other deleterious materials. All unsuitable materials should be removed from the fill as directed by the geotechnical consultant. Soils of poor gradation, undesirable expansion potential, or substandard strength characteristics may be designated by the consultant as unsuitable and may require blending with other soils to serve as a satisfactory fill material.

Fill materials derived from benching operations should be dispersed throughout the fill area and blended with other approved material. Benching operations should not result in the benched material being placed only within a single equipment width away from the fill/bedrock contact.

Oversized materials defined as rock, or other irreducible materials, with a maximum dimension greater than 12 inches, should not be buried or placed in fills unless the location of materials and disposal methods are specifically approved by the geotechnical consultant. Oversized material should be taken offsite, or placed in accordance with recommendations of the geotechnical consultant in areas designated as suitable for rock disposal. GSI anticipates that soils to be utilized as fill material for the subject project may contain some rock. Appropriately, the need for rock disposal may be necessary during grading operations on the site. From a geotechnical standpoint, the depth of any rocks, rock fills, or rock blankets, should be a sufficient distance from finish grade. This depth is generally the same as any overexcavation due to cut-fill transitions in hard rock areas, and generally facilitates the excavation of structural footings and substructures. Should deeper excavations be proposed (i.e., deepened footings, utility trenching, swimming pools, spas, etc.), the developer may consider increasing the hold-down depth of any rocky fills to be placed, as appropriate. In addition, some agencies/jurisdictions mandate a specific hold-down depth for oversize materials placed in fills. The hold-down depth, and potential to encounter oversize rock, both within fills, and occurring in cut or natural areas, would need to be disclosed to all interested/affected parties. Once approved by the governing agency, the hold-down depth for oversized rock (i.e., greater than 12 inches) in fills on this project is provided as 10 feet, unless specified differently in the text of this report. The governing agency may require that these materials need to be deeper, crushed, or reduced to less than 12 inches in maximum dimension, at their discretion.

To facilitate future trenching, rock (or oversized material), should not be placed within the hold-down depth feet from finish grade, the range of foundation excavations, future utilities, or underground construction unless specifically approved by the governing agency, the geotechnical consultant, and/or the developer's representative.

If import material is required for grading, representative samples of the materials to be utilized as compacted fill should be analyzed in the laboratory by the geotechnical consultant to evaluate it's physical properties and suitability for use onsite. Such testing should be performed three (3) days prior to importation. If any material other than that previously tested is encountered during grading, an appropriate analysis of this material should be conducted by the geotechnical consultant as soon as possible.

Approved fill material should be placed in areas prepared to receive fill in near horizontal layers, that when compacted, should not exceed about 6 to 8 inches in thickness. The geotechnical consultant may approve thick lifts if testing indicates the grading procedures are such that adequate compaction is being achieved with lifts of greater thickness. Each layer should be spread evenly and blended to attain uniformity of material and moisture suitable for compaction.

Fill layers at a moisture content less than optimum should be watered and mixed, and wet fill layers should be aerated by scarification, or should be blended with drier material. Moisture conditioning, blending, and mixing of the fill layer should continue until the fill materials have a uniform moisture content at, or above, optimum moisture.

After each layer has been evenly spread, moisture conditioned, and mixed, it should be uniformly compacted to a minimum of 90 percent of the maximum density as evaluated by ASTM test designation D 1557, or as otherwise recommended by the geotechnical consultant. Compaction equipment should be adequately sized and should be specifically designed for soil compaction, or of proven reliability to efficiently achieve the specified degree of compaction.

Where tests indicate that the density of any layer of fill, or portion thereof, is below the required relative compaction, or improper moisture is in evidence, the particular layer or portion shall be re-worked until the required density and/or moisture content has been attained. No additional fill shall be placed in an area until the last placed lift of fill has been tested and found to meet the density and moisture requirements, and is approved by the geotechnical consultant.

In general, per the latest adopted Code, fill slopes should be designed and constructed at a gradient of 2:1 (h:v), or flatter. Compaction of slopes should be accomplished by over-building a minimum of 3 feet horizontally, and subsequently trimming back to the design slope configuration. Testing shall be performed as the fill is elevated to evaluate compaction as the fill core is being developed. Special efforts may be necessary to attain the specified compaction in the fill slope zone. Final slope shaping should be performed by trimming and removing loose materials with appropriate equipment. A final evaluation of fill slope compaction should be based on observation and/or testing of the finished slope face. Where compacted fill slopes are designed steeper than 2:1 (h:v), prior approval from the governing agency, specific material types, a higher minimum relative compaction, special reinforcement, and special grading procedures will be recommended.

If an alternative to over-building and cutting back the compacted fill slopes is selected, then special effort should be made to achieve the required compaction in the outer 10 feet of each lift of fill by undertaking the following:

1. An extra piece of equipment consisting of a heavy, short-shanked sheepsfoot should be used to roll (horizontal) parallel to the slopes continuously as fill is placed. The sheepsfoot roller should also be used to roll perpendicular to the slopes, and extend out over the slope to provide adequate compaction to the face of the slope.

2. Loose fill should not be spilled out over the face of the slope as each lift is compacted. Any loose fill spilled over a previously completed slope face should be trimmed off or be subject to re-rolling.

3. Field compaction tests will be made in the outer (horizontal) ± 2 to ± 8 feet of the slope at appropriate vertical intervals, subsequent to compaction operations.

4. After completion of the slope, the slope face should be shaped with a small tractor and then re-rolled with a sheepsfoot to achieve compaction to near the slope face. Subsequent to testing to evaluate compaction, the slopes should be grid-rolled to achieve compaction to the slope face. Final testing should be used to evaluate compaction after grid rolling.

5. Where testing indicates less than adequate compaction, the contractor will be responsible to rip, water, mix, and recompact the slope material as necessary to achieve compaction. Additional testing should be performed to evaluate compaction.

SUBDRAIN INSTALLATION

Subdrains should be installed in approved ground in accordance with the approximate alignment and details indicated by the geotechnical consultant. Subdrain locations or materials should not be changed or modified without approval of the geotechnical consultant. The geotechnical consultant may recommend and direct changes in subdrain line, grade, and drain material in the field, pending exposed conditions. The location of constructed subdrains, especially the outlets, should be recorded/surveyed by the project civil engineer. Drainage at the subdrain outlets should be provided by the project civil engineer.

EXCAVATIONS

Excavations and cut slopes should be examined during grading by the geotechnical consultant. If directed by the geotechnical consultant, further excavations or overexcavation and refilling of cut areas should be performed, and/or remedial grading of cut slopes should be performed. When fill-over-cut slopes are to be graded, unless otherwise approved, the cut portion of the slope should be observed by the geotechnical consultant prior to placement of materials for construction of the fill portion of the slope. The geotechnical consultant should observe all cut slopes, and should be notified by the contractor when excavation of cut slopes commence.

If, during the course of grading, unforeseen adverse or potentially adverse geologic conditions are encountered, the geotechnical consultant should investigate, evaluate, and make appropriate recommendations for mitigation of these conditions. The need for cut slope buttressing or stabilizing should be based on in-grading evaluation by the geotechnical consultant, whether anticipated or not.

Unless otherwise specified in geotechnical and geological report(s), no cut slopes should be excavated higher or steeper than that allowed by the ordinances of controlling governmental agencies. Additionally, short-term stability of temporary cut slopes is the contractor’s responsibility.

Erosion control and drainage devices should be designed by the project civil engineer and should be constructed in compliance with the ordinances of the controlling governmental agencies, and/or in accordance with the recommendations of the geotechnical consultant.

COMPLETION

Observation, testing, and consultation by the geotechnical consultant should be conducted during the grading operations in order to state an opinion that all cut and fill areas are graded in accordance with the approved project specifications. After completion of grading, and after the geotechnical consultant has finished observations of the work, final reports should be submitted, and may be subject to review by the controlling governmental agencies. No further excavation or filling should be undertaken without prior notification of the geotechnical consultant or approved plans.

All finished cut and fill slopes should be protected from erosion and/or be planted in accordance with the project specifications and/or as recommended by a landscape architect. Such protection and/or planning should be undertaken as soon as practical after completion of grading.

JOB SAFETY

General

At GSI, getting the job done safely is of primary concern. The following is the company's safety considerations for use by all employees on multi-employer construction sites. On-ground personnel are at highest risk of injury, and possible fatality, on grading and construction projects. GSI recognizes that construction activities will vary on each site, and that site safety is the prime responsibility of the contractor; however, everyone must be safety conscious and responsible at all times. To achieve our goal of avoiding accidents, cooperation between the client, the contractor, and GSI personnel must be maintained.

In an effort to minimize risks associated with geotechnical testing and observation, the following precautions are to be implemented for the safety of field personnel on grading and construction projects:

Safety Meetings: GSI field personnel are directed to attend contractor’s regularly scheduled and documented safety meetings.

Safety Vests: Safety vests are provided for, and are to be worn by GSI personnel, at all times, when they are working in the field.

Safety Flags: Two safety flags are provided to GSI field technicians; one is to be affixed to the vehicle when on site, the other is to be placed atop the spoil pile on all test pits.

Flashing Lights: All vehicles stationary in the grading area shall use rotating or flashing amber beacons, or strobe lights, on the vehicle during all field testing. While operating a vehicle in the grading area, the emergency flasher on the vehicle shall be activated.

In the event that the contractor's representative observes any of our personnel not following the above, we request that it be brought to the attention of our office.

<u>Test Pits Location, Orientation, and Clearance</u>

The technician is responsible for selecting test pit locations. A primary concern should be the technician's safety. Efforts will be made to coordinate locations with the grading contractor's authorized representative, and to select locations following or behind the established traffic pattern, preferably outside of current traffic. The contractor's authorized representative (supervisor, grade checker, dump man, operator, etc.) should direct excavation of the pit and safety during the test period. Of paramount concern should be the soil technician's safety, and obtaining enough tests to represent the fill.

Test pits should be excavated so that the spoil pile is placed away from oncoming traffic, whenever possible. The technician's vehicle is to be placed next to the test pit, opposite the spoil pile. This necessitates the fill be maintained in a driveable condition. Alternatively, the contractor may wish to park a piece of equipment in front of the test holes, particularly in small fill areas or those with limited access.

A zone of non-encroachment should be established for all test pits. No grading equipment should enter this zone during the testing procedure. The zone should extend approximately 50 feet outward from the center of the test pit. This zone is established for safety and to avoid excessive ground vibration, which typically decreases test results.

When taking slope tests, the technician should park the vehicle directly above or below the test location. If this is not possible, a prominent flag should be placed at the top of the slope. The contractor's representative should effectively keep all equipment at a safe operational distance (e.g., 50 feet) away from the slope during this testing.

The technician is directed to withdraw from the active portion of the fill as soon as possible following testing. The technician's vehicle should be parked at the perimeter of the fill in a highly visible location, well away from the equipment traffic pattern. The contractor should inform our personnel of all changes to haul roads, cut and fill areas or other factors that may affect site access and site safety.

In the event that the technician's safety is jeopardized or compromised as a result of the contractor's failure to comply with any of the above, the technician is required, by company policy, to immediately withdraw and notify his/her supervisor. The grading contractor's representative will be contacted in an effort to affect a solution. However, in the interim, no further testing will be performed until the situation is rectified. Any fill placed can be considered unacceptable and subject to reprocessing, recompaction, or removal.

In the event that the soil technician does not comply with the above or other established safety guidelines, we request that the contractor bring this to the technician's attention and notify this office. Effective communication and coordination between the contractor's representative and the soil technician is strongly encouraged in order to implement the above safety plan.

<u>Trench and Vertical Excavation</u>

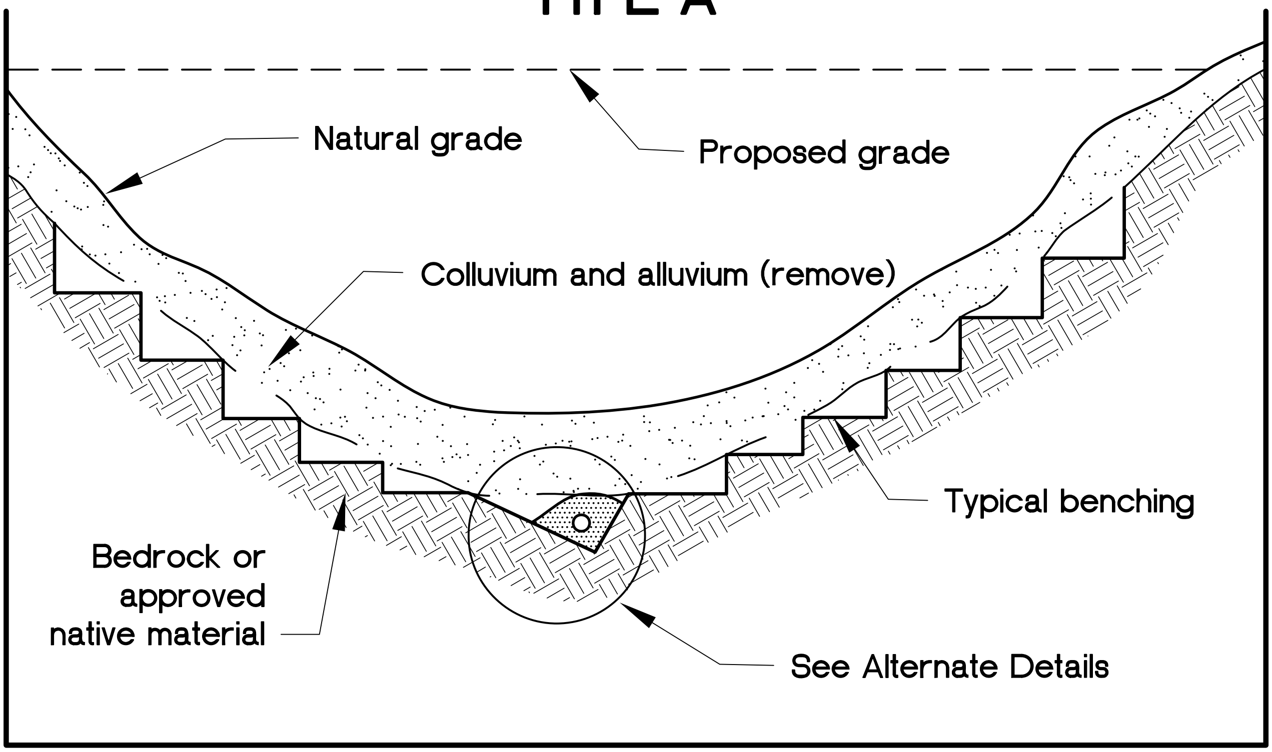
It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Our personnel are directed not to enter any excavation or vertical cut which: 1) is 5 feet or deeper unless shored or laid back; 2) displays any evidence of instability, has any loose rock or other debris which could fall into the trench; or 3) displays any other evidence of any unsafe conditions regardless of depth.

All trench excavations or vertical cuts in excess of 5 feet deep, which any person enters, should be shored or laid back. Trench access should be provided in accordance with Cal/OSHA and/or state and local standards. Our personnel are directed not to enter any trench by being lowered or "riding down" on the equipment.

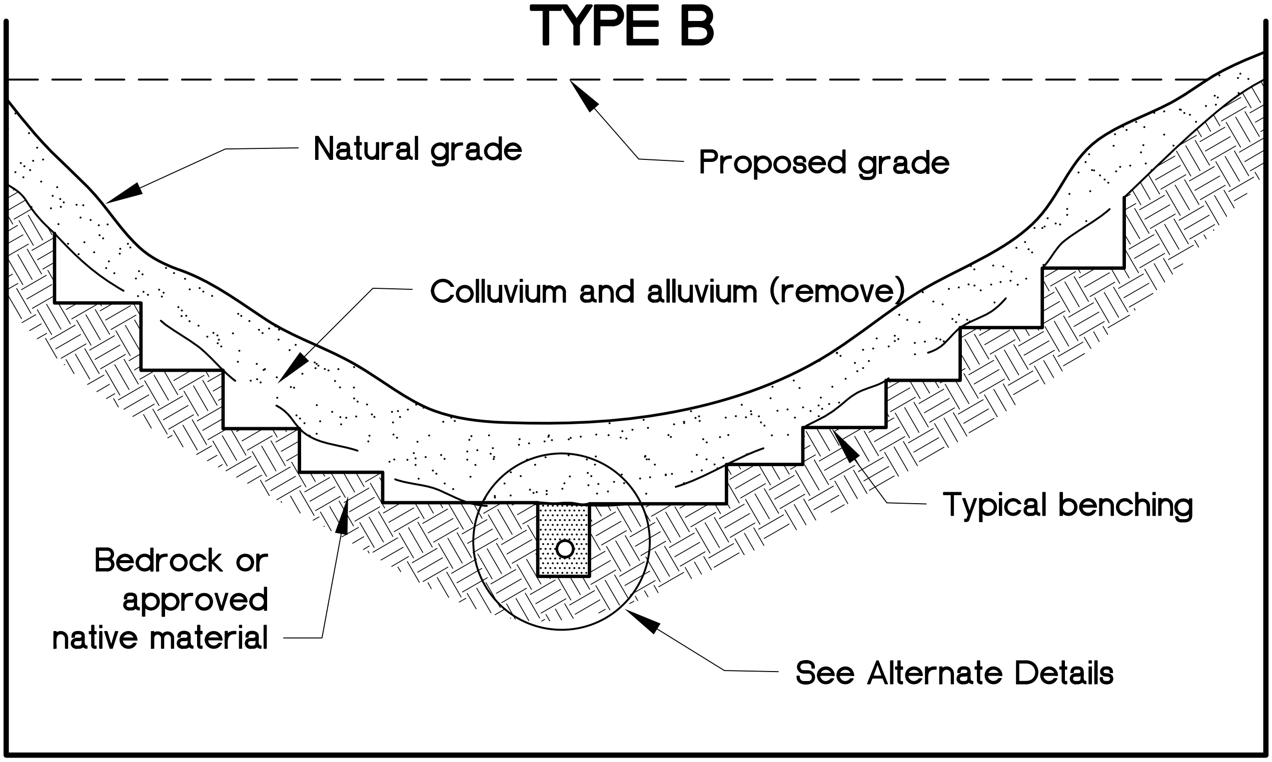
If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraw and notify his/her supervisor. The contractor's representative will be contacted in an effort to affect a solution. All backfill not tested due to safety concerns or other reasons could be subject to reprocessing and/or removal.

If GSI personnel become aware of anyone working beneath an unsafe trench wall or vertical excavation, we have a legal obligation to put the contractor and owner/developer on notice to immediately correct the situation. If corrective steps are not taken, GSI then has an obligation to notify Cal/OSHA and/or the proper controlling authorities.

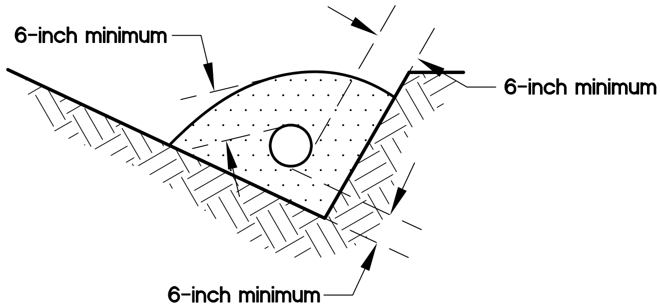
TYPE A



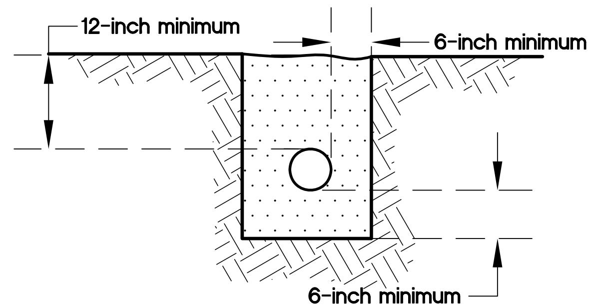
TYPE B



Selection of alternate subdrain details, location, and extent of subdrains should be evaluated by the geotechnical consultant during grading.



A-1



B-1

Filter material: Minimum volume of 9 cubic feet per lineal foot of pipe.

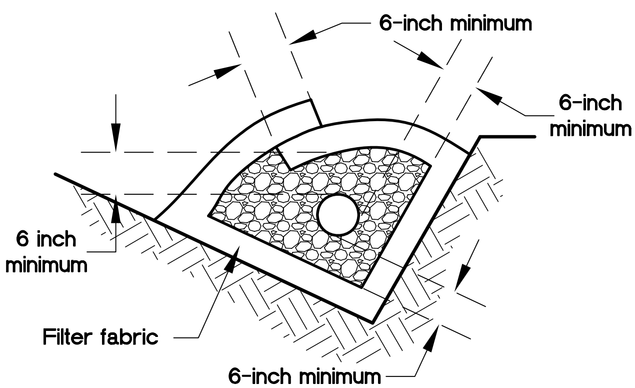
Perforated pipe: 6-inch-diameter ABS or PVC pipe or approved substitute with minimum 8 perforations ($^1/_4$-inch diameter) per lineal foot in bottom half of pipe (ASTM D-2751, SDR-35, or ASTM D-1527, Schd. 40).

For continuous run in excess of 500 feet, use 8-inch-diameter pipe (ASTM D-3034, SDR-35, or ASTM D-1785, Schd. 40).

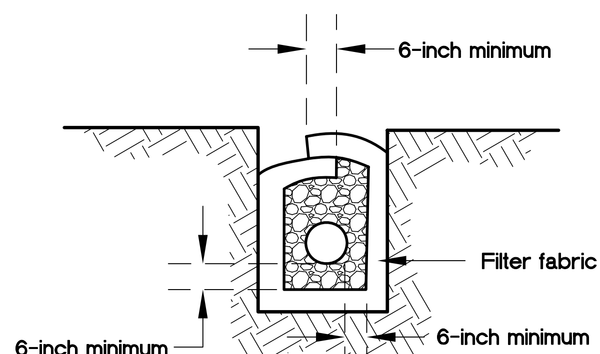
FILTER MATERIAL

Sieve Size	Percent Passing
1 inch	100
$^3/_4$ inch	90-100
$^3/_8$ inch	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

ALTERNATE 1: PERFORATED PIPE AND FILTER MATERIAL



A-2

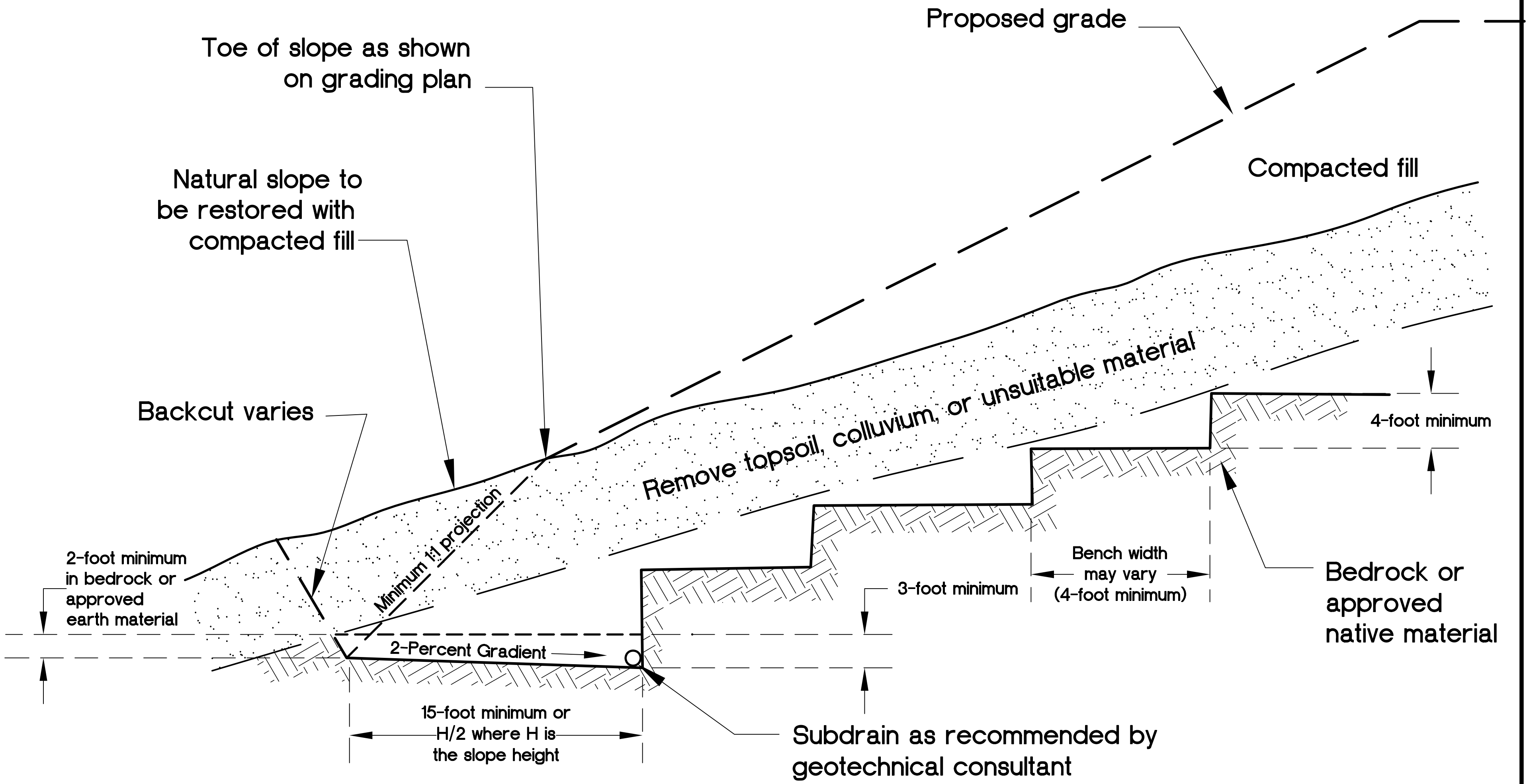


B-2

Gravel Material: 9 cubic feet per lineal foot.
Perforated Pipe: See Alternate 1
Gravel: Clean $^3/_4$-inch rock or approved substitute.
Filter Fabric: Mirafi 140 or approved substitute.

ALTERNATE 2: PERFORATED PIPE, GRAVEL, AND FILTER FABRIC

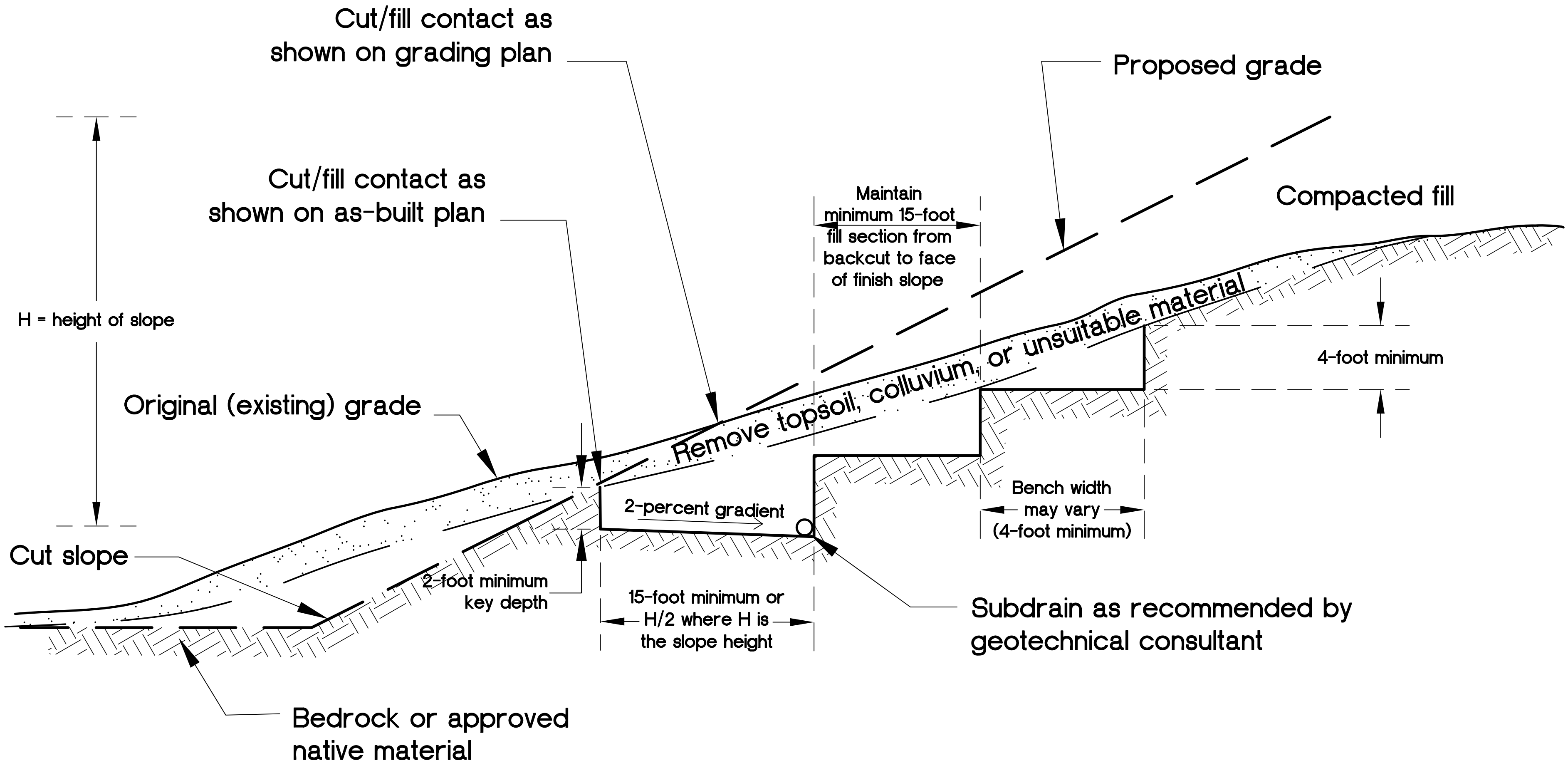
GeoSoils, Inc.	CANYON SUBDRAIN ALTERNATE DETAILS	Plate F-2



NOTES:
1. Where the natural slope approaches or exceeds the design slope ratio, special recommendations would be provided by the geotechnical consultant.

2. The need for and disposition of drains should be evaluated by the geotechnical consultant, based upon exposed conditions.

GeoSoils, Inc.	FILL OVER NATURAL (SIDEHILL FILL) DETAIL	Plate F-7

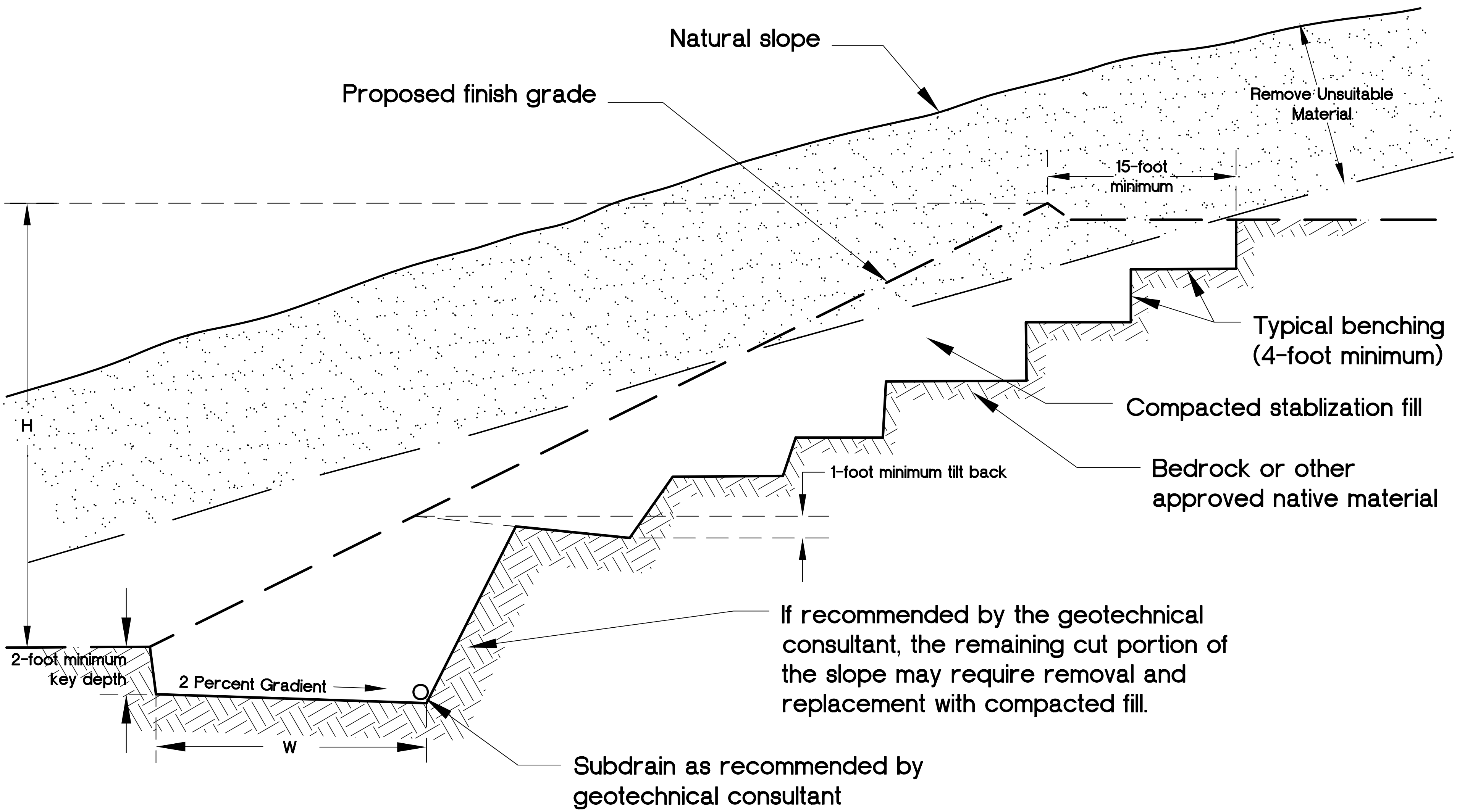


NOTE: The cut portion of the slope should be excavated and evaluated by the geotechnical consultant prior to construction of the fill portion.

GeoSoils, Inc.

FILL OVER CUT DETAIL

Plate F-8

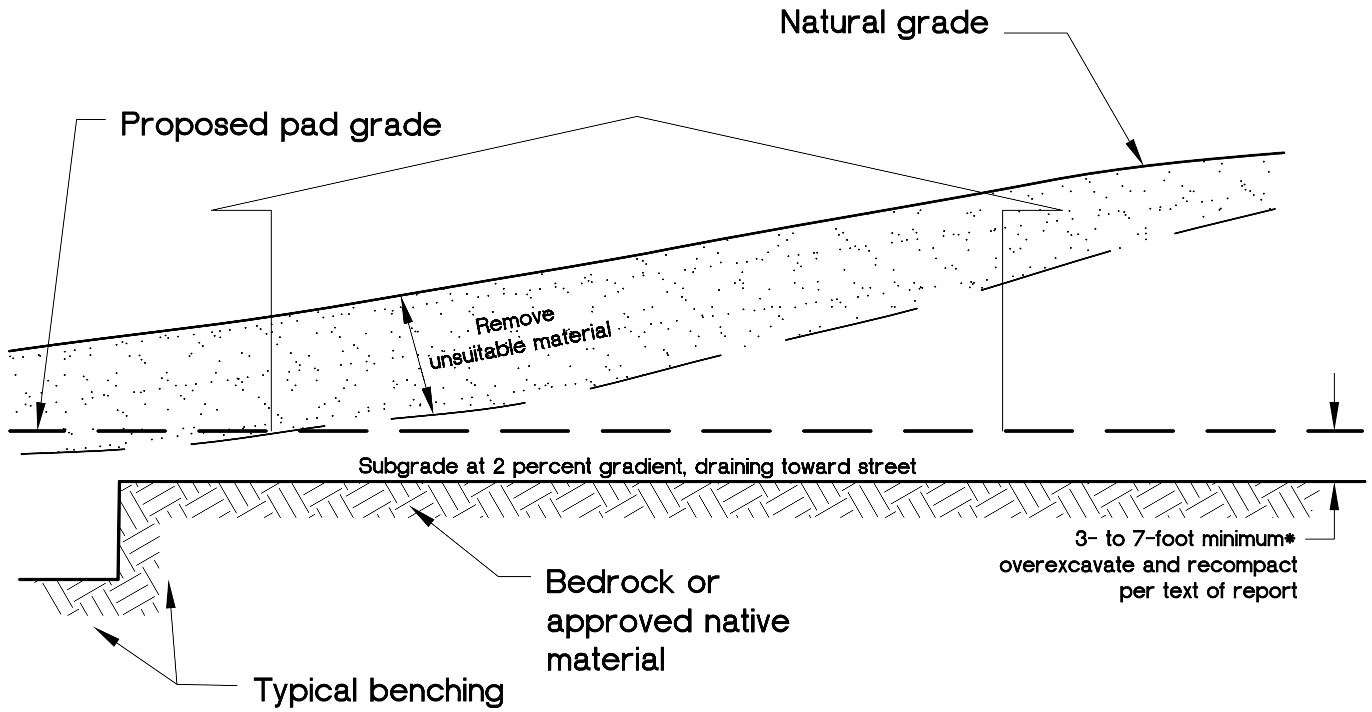


NOTES:

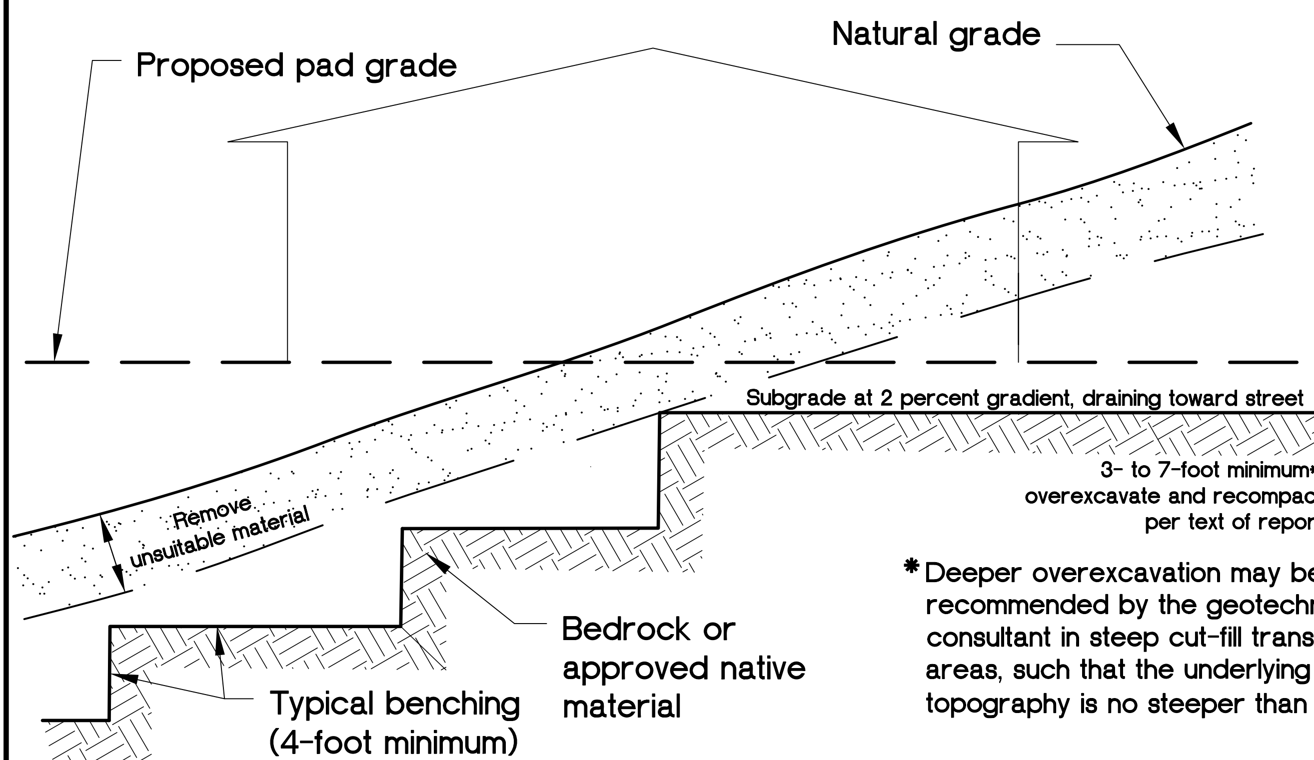
1. Subdrains may be required as specified by the geotechnical consultant.

2 W shall be equipment width (15 feet) for slope heights less than 25 feet. For slopes greater than 25 feet, W shall be evaluated by the geotechnical consultant. At no time, shall W be less than H/2, where H is the height of the slope.

GeoSoils, Inc. | STABLIZATION FILL FOR UNSTABLE MATERIAL EXPOSED IN CUT SLOPE DETAIL | Plate F-9



CUT LOT OR MATERIAL-TYPE TRANSITION

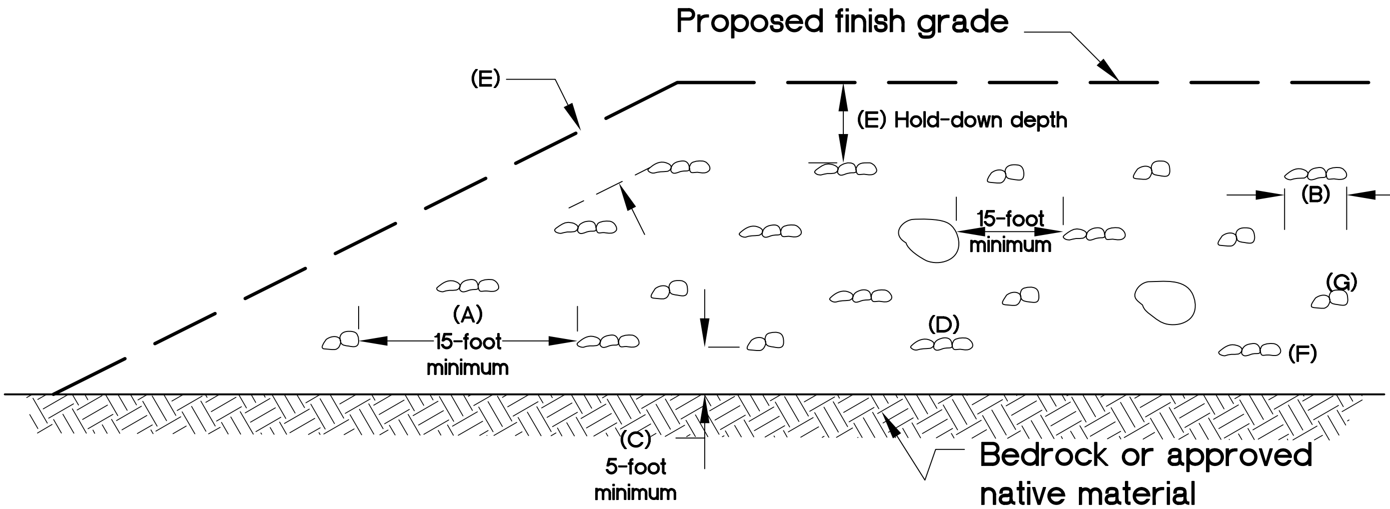


* Deeper overexcavation may be recommended by the geotechnical consultant in steep cut-fill transition areas, such that the underlying topography is no steeper than 3:1 (H:V)

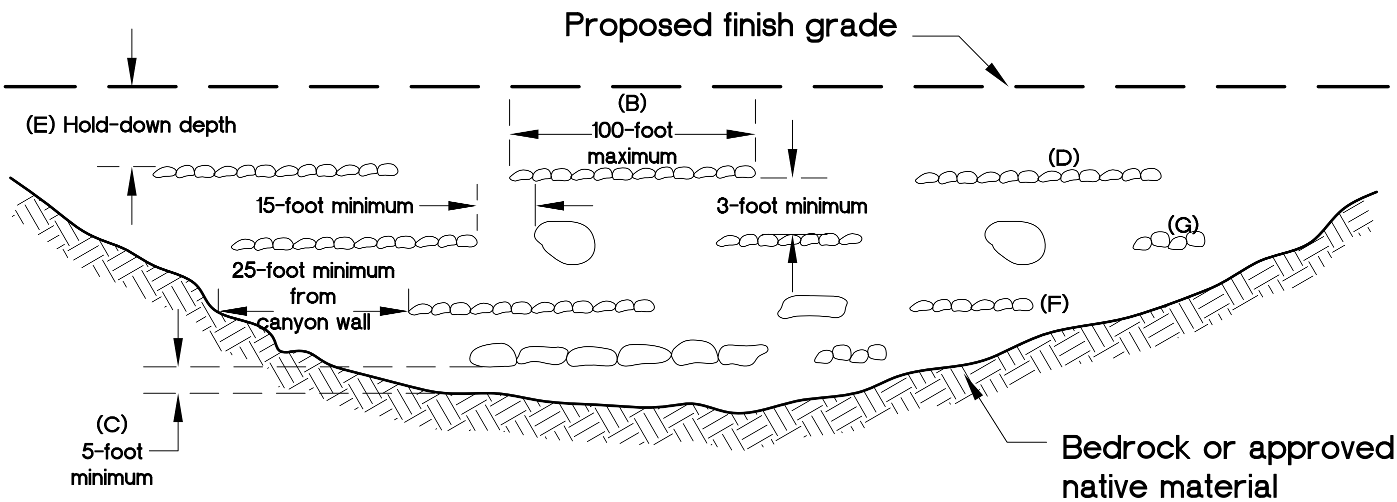
CUT-FILL LOT (DAYLIGHT TRANSITION)

GeoSoils, Inc. | TRANSITION LOT DETAILS | Plate F-12

VIEW NORMAL TO SLOPE FACE



VIEW PARALLEL TO SLOPE FACE



NOTES:

A. One equipment width or a minimum of 15 feet between rows (or windrows).

B. Height and width may vary depending on rock size and type of equipment. Length of windrow shall be no greater than 100 feet.

C. If approved by the geotechnical consultant, windrows may be placed direclty on competent material or bedrock, provided adequate space is available for compaction.

D. Orientation of windrows may vary but should be as recommended by the geotechnical engineer and/or engineering geologist. Staggering of windrows is not necessary unless recommended.

E. Clear area for utility trenches, foundations, and swimming pools; Hold-down depth as specified in text of report, subject to governing agency approval.

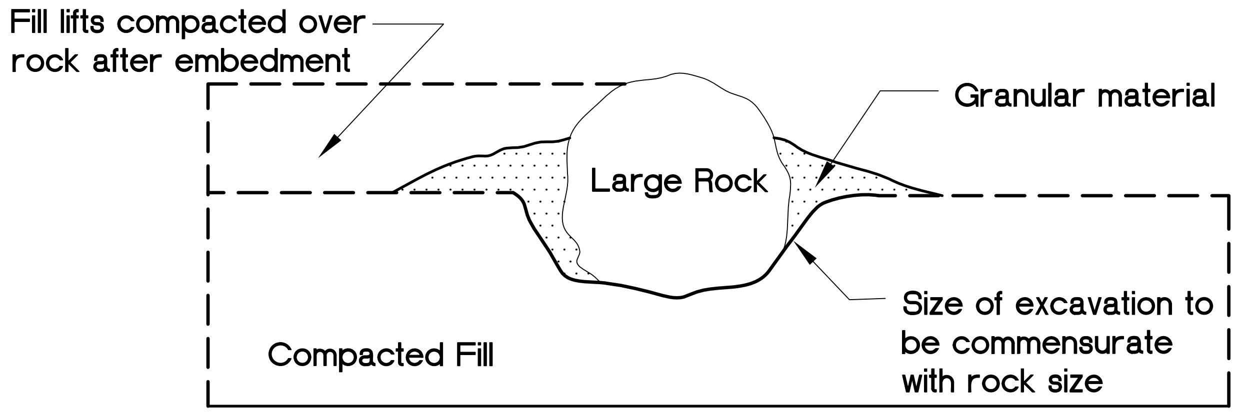
F. All fill over and around rock windrow shall be compacted to at least 90 percent relative compaction or as recommended.

G. After fill between windrows is placed and compacted, with the lift of fill covering windrow, windrow should be proof rolled with a D-9 dozer or equivalent.

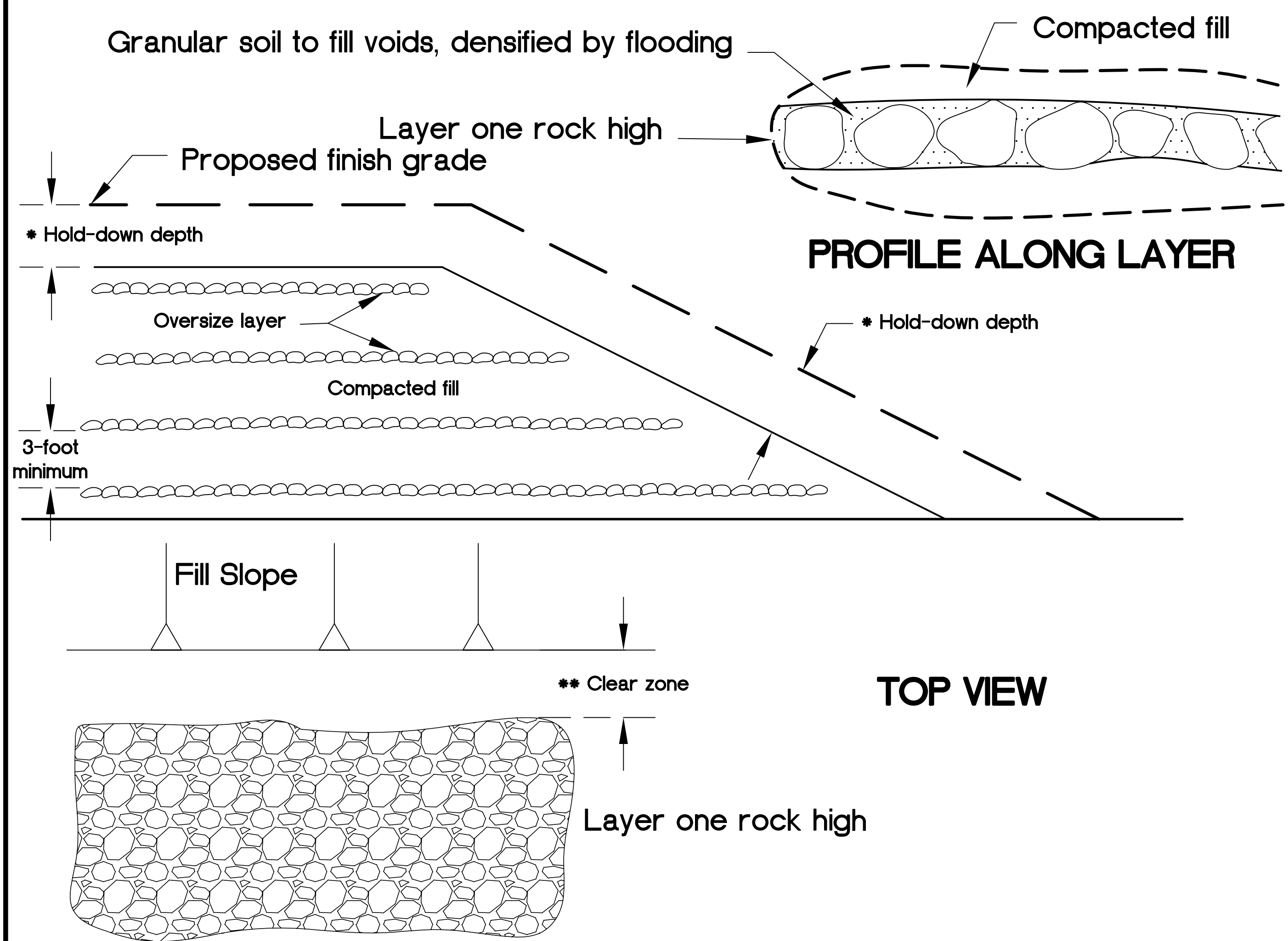
VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE
ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED

GeoSoils, Inc.	OVERSIZE ROCK DISPOSAL DETAIL	Plate F-13

ROCK DISPOSAL PITS



ROCK DISPOSAL LAYERS



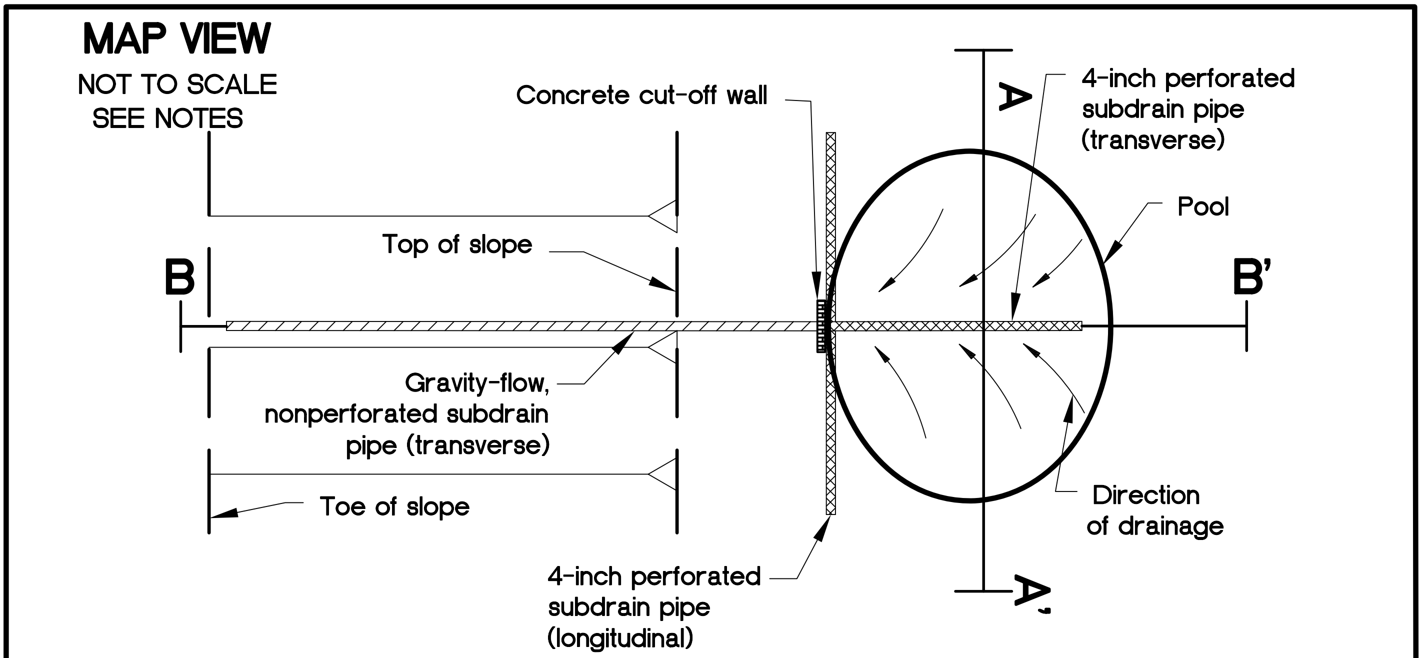
* Hold-down depth or below lowest utility as specified in text of report, subject to governing agency approval.

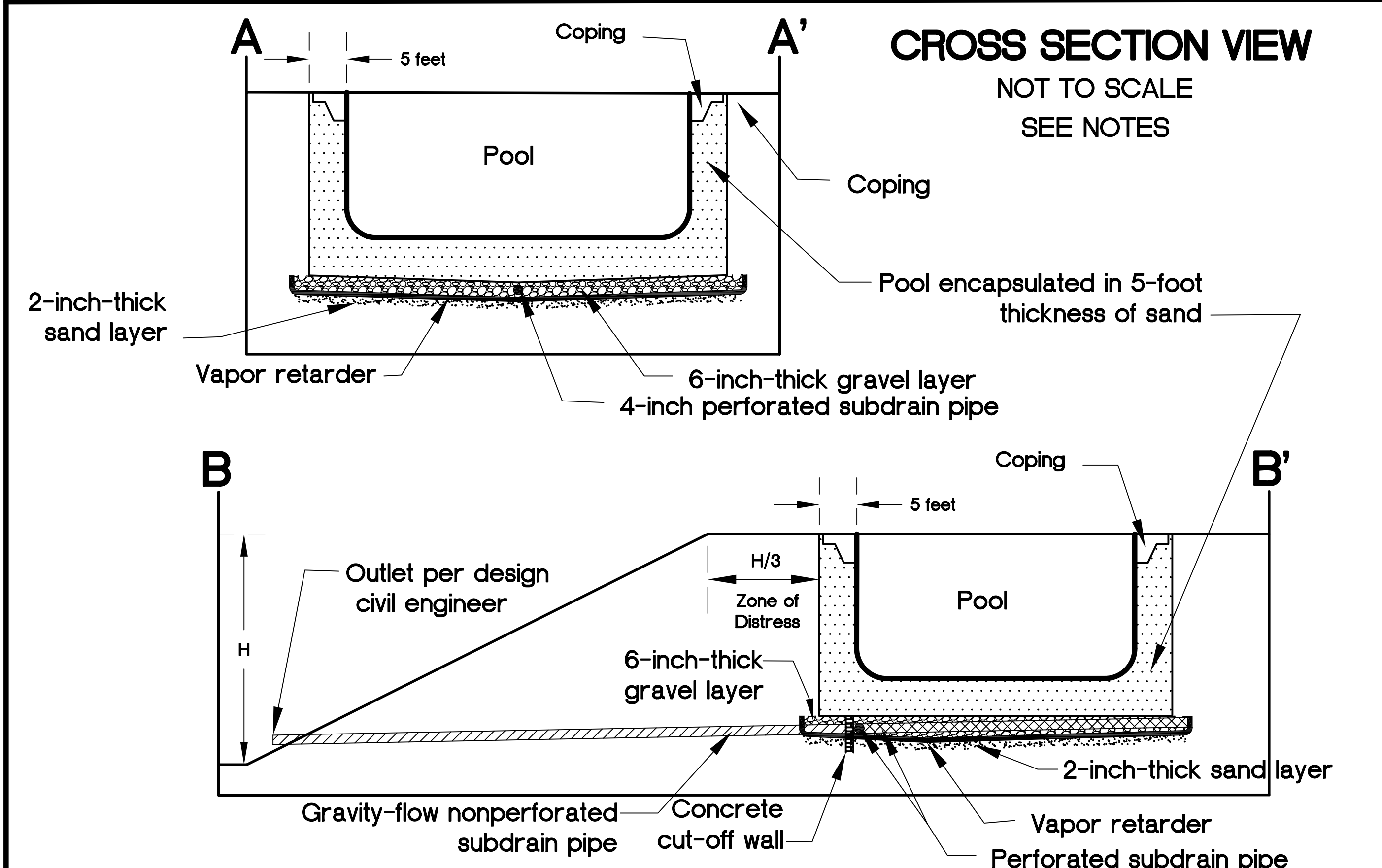
** Clear zone for utility trenches, foundations, and swimming pools, as specified in text of report.

VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE

ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED IN

GeoSoils, Inc.	ROCK DISPOSAL DETAIL	Plate F-14





NOTES:

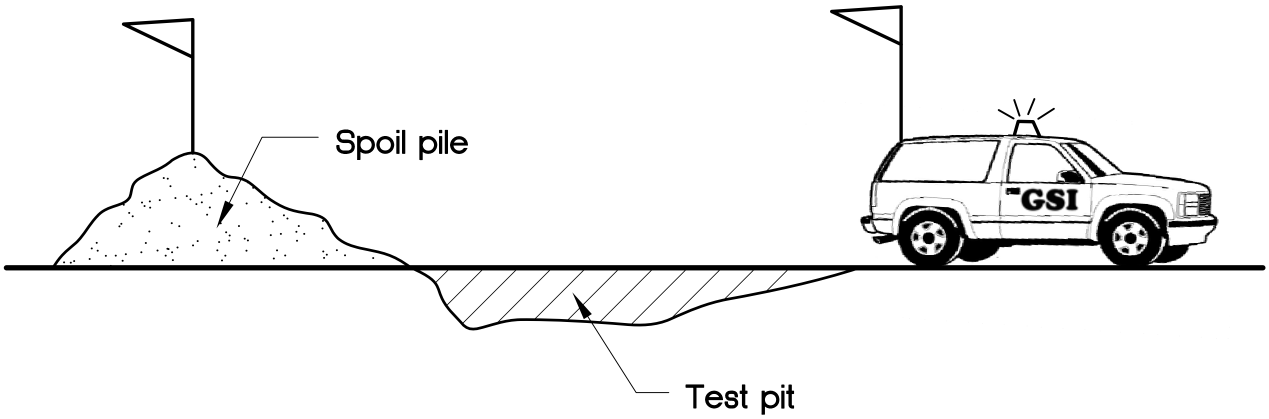
1. 6-inch-thick, clean gravel (¾ to 1½ inch) sub-base encapsulated in Mirafi 140N or equivalent, underlain by a 15-mil vapor retarder, with 4-inch-diameter perforated pipe longitudinal connected to 4-inch-diameter perforated pipe transverse. Connect transverse pipe to 4-inch-diameter nonperforated pipe at low point and outlet or to sump pump area.
2. Pools on fills thicker than 20 feet should be constructed on deep foundations; otherwise, distress (tilting, cracking, etc.) should be expected.
3. Design does not apply to infinity-edge pools/spas.

GeoSoils, Inc.

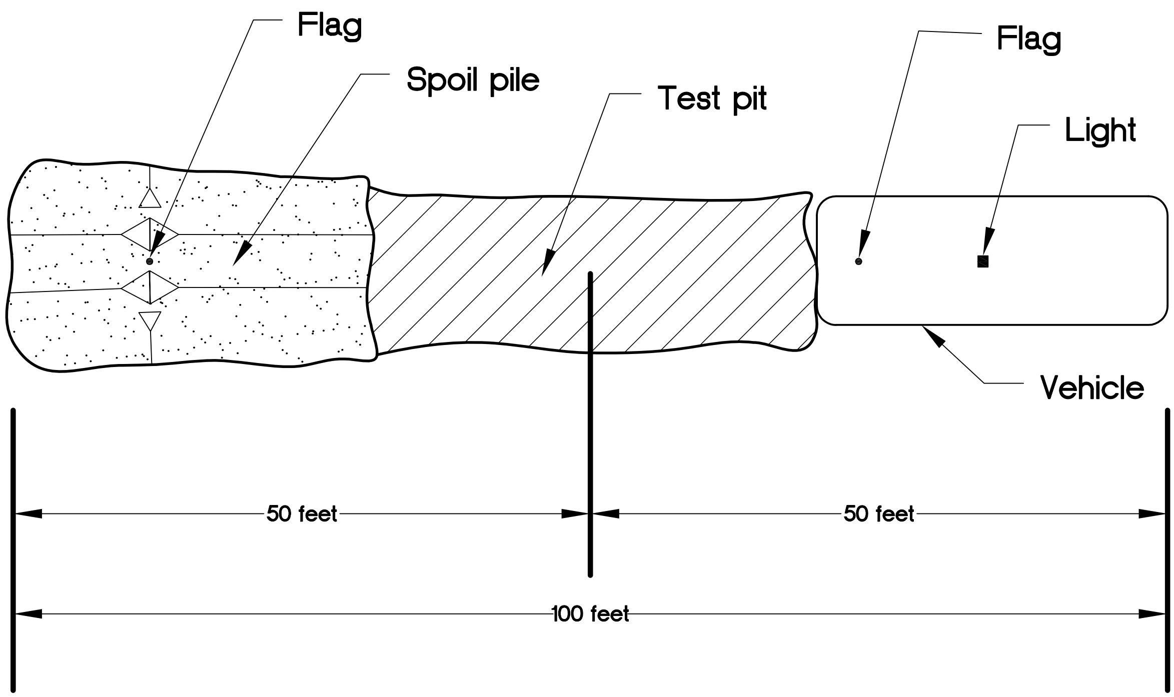
TYPICAL POOL/SPA DETAIL

Plate F-17

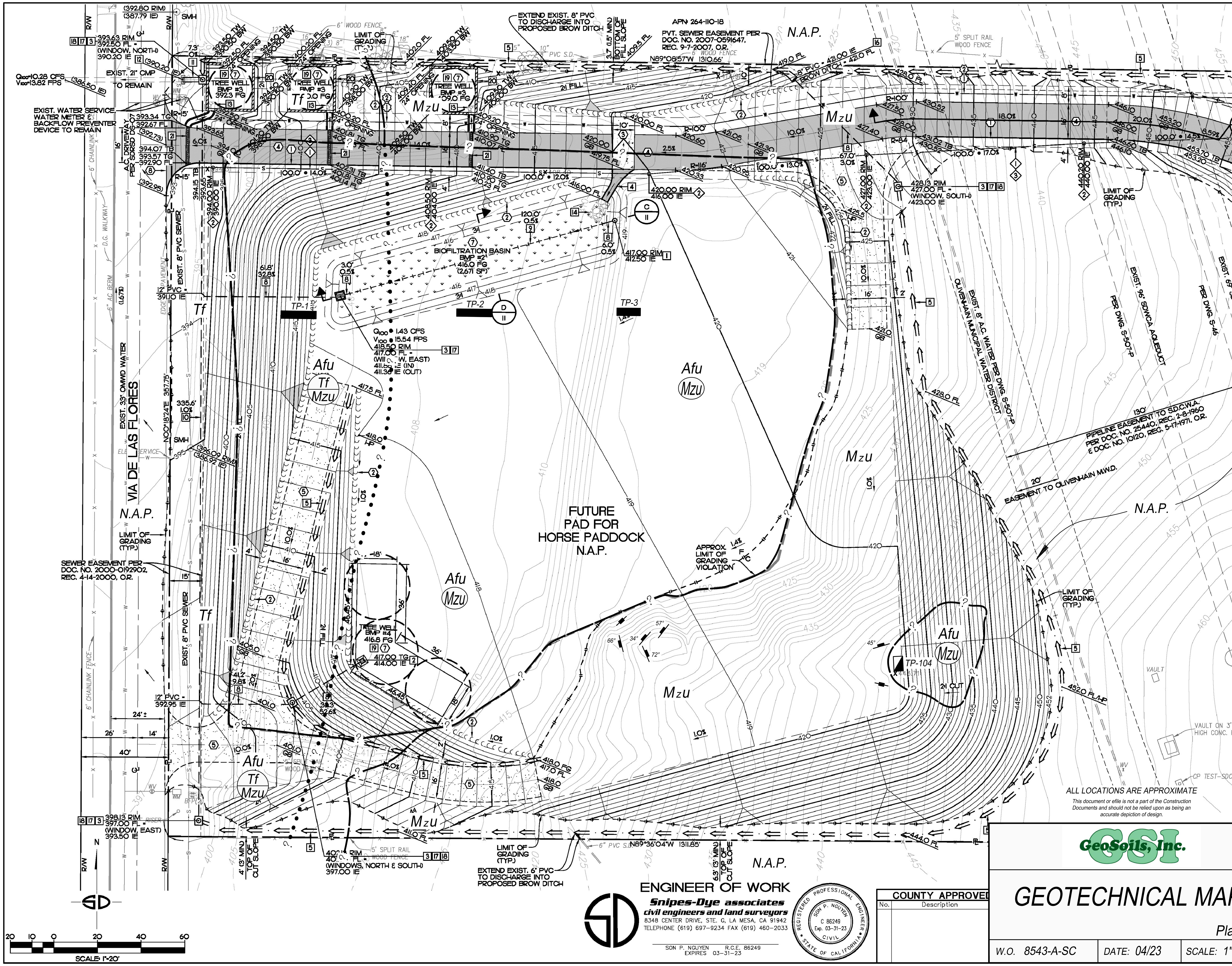
SIDE VIEW



TOP VIEW



GeoSoils, Inc. | TEST PIT SAFETY DIAGRAM | Plate F-20



KEY NOTES

PVT. IMPROVEMENTS

1. PVT. 6" AC BERM TYPE A PER SDRSD G-05 (TYP.).
2. EARTHEN BERM PER DETAIL 1, SHT. 3 (TYP.)
3. PVT. CONC. CROSS GUTTER PER SDRSD G-13.
4. MIN. 4" A.C. PAVEMENT OVER 6" CLASS II AGGREGATE BASE (TYP.).
5. 4" DECOMPOSED GRANITE PER DETAIL 7, SHT. 3 (TYP.).
7. ALL PROPOSED VEGETATION TO BE CONSTRUCTED/INSTALLED WITHIN BIOFILTRATION BASIN AND TREE WELLS SHALL BE CONFORMANCE WITH APPROVED LANDSCAPE AND IRRIGATION PLAN NO. PDS2022-LP-________, AND SHALL BE INCLUDED IN PROJECT SWQMP.
8. MATCH EXIST. A.C. PAVEMENT PER DETAIL 2, SHT. 3.

PVT. STORM DRAIN

1. PVT. STORM DRAIN CLEANOUT PER DETAIL 3, SHT. 3.
2. PVT. 24" CATCH BASIN W/ CONC. APRON PER DETAIL 4, SHT. 3.
3. PVT. CATCH BASIN TYPE F PER SDRSD D-07.
4. PVT. AC SPILL WAY PER SDRSD D-22.
5. PVT. DRAINAGE DITCH TYPE B PER SDRSD D-75 (TYP.).
6. PVT. DRAINAGE DITCH TYPE D PER SDRSD D-75 (TYP.).
8. PVT. 8" PVC SDR-35 PER SDRSD D-60.
9. PVT. 8" PVC SDR-35 PERFORATED PIPE.
10. PVT. 12" PVC SDR-35 PER SDRSD D-60.
11. PVT. 24" PVC SDR-35 PER SDRSD D-60.
12. PIPE COLLAR PER SDRSD D-62.
13. PVT. 3"-6" ROCKS (3'X3'X0.7' THICK).
14. PVT. 3"-6" ROCKS (6'X10'X0.7' THICK).
15. PVT. NO. 2 BACKING (6'X10'X1.1') ROCK RIP-RAP PER SDRSD D-40, TYPE I.
16. PVT. STRAIGHT HEADWALL PER SDRSD D-32.
17. MARK ALL INLETS WITH THE WORDS "NO DUMPING-DRAINS TO WATERWAYS" OR SIMILAR. SEE STENCIL TEMPLATE ON SHT. 10.
18. TRANSITION BROW DITCH TO DISCHARGE INTO TYPE F CATCH BASIN.
19. TREE WELL PER DETAILS ON SHT. 11.
20. GRAVITY RETAINING WALL TYPE C PER SDRSD C-09.
21. PVT. 8" TRENCH DRAIN PER DETAIL 8, SHT. 3.

GSI LEGEND

Afu	ARTIFICIAL FILL – UNDOCUMENTED
Tf	TERTIARY FRIARS FORMATION, CIRCLED WHERE BURIED
Mzu	MESOZOIC METAVOLCANIC BEDROCK, CIRCLED WHERE BURIED
	APPROXIMATE LOCATION OF GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN, DOTTED WHERE BURIED
72°	JOINT ATTITUDE WITH DIP IN DEGREES
	ATTITUDE OF VERTICAL JOINT
TP-104	APPROXIMATE LOCATION OF EXPLORATORY TEST PIT (THIS STUDY)
TP-3	APPROXIMATE LOCATION OF EXPLORATORY TEST PIT (CG, 2018)
A A'	APPROXIMATE LOCATION OF GEOLOGIC CROSS SECTION
N.A.P.	NOT A PART OF THIS STUDY

ENGINEER OF WORK
Snipes-Dye associates
civil engineers and land surveyors
8348 CENTER DRIVE, STE. C, LA MESA, CA 91942
TELEPHONE (619) 697-9234 FAX (619) 460-2033
SON P. NGUYEN R.C.E. 86249
EXPIRES 03-31-23

REGISTERED PROFESSIONAL ENGINEER SON P. NGUYEN C 86249 Exp. 03-31-23 CIVIL STATE OF CALIFORNIA

COUNTY APPROVED CHANGES	
No.	Description

GeoSoils, Inc.

GEOTECHNICAL MAP

Plate 1

W.O. 8543-A-SC | DATE: 04/23 | SCALE: 1" = 20'

PRIVATE CONTRACT

SHEET 5 | COUNTY OF SAN DIEGO DEPARTMENT OF PUBLIC WORKS | 13 SHEETS

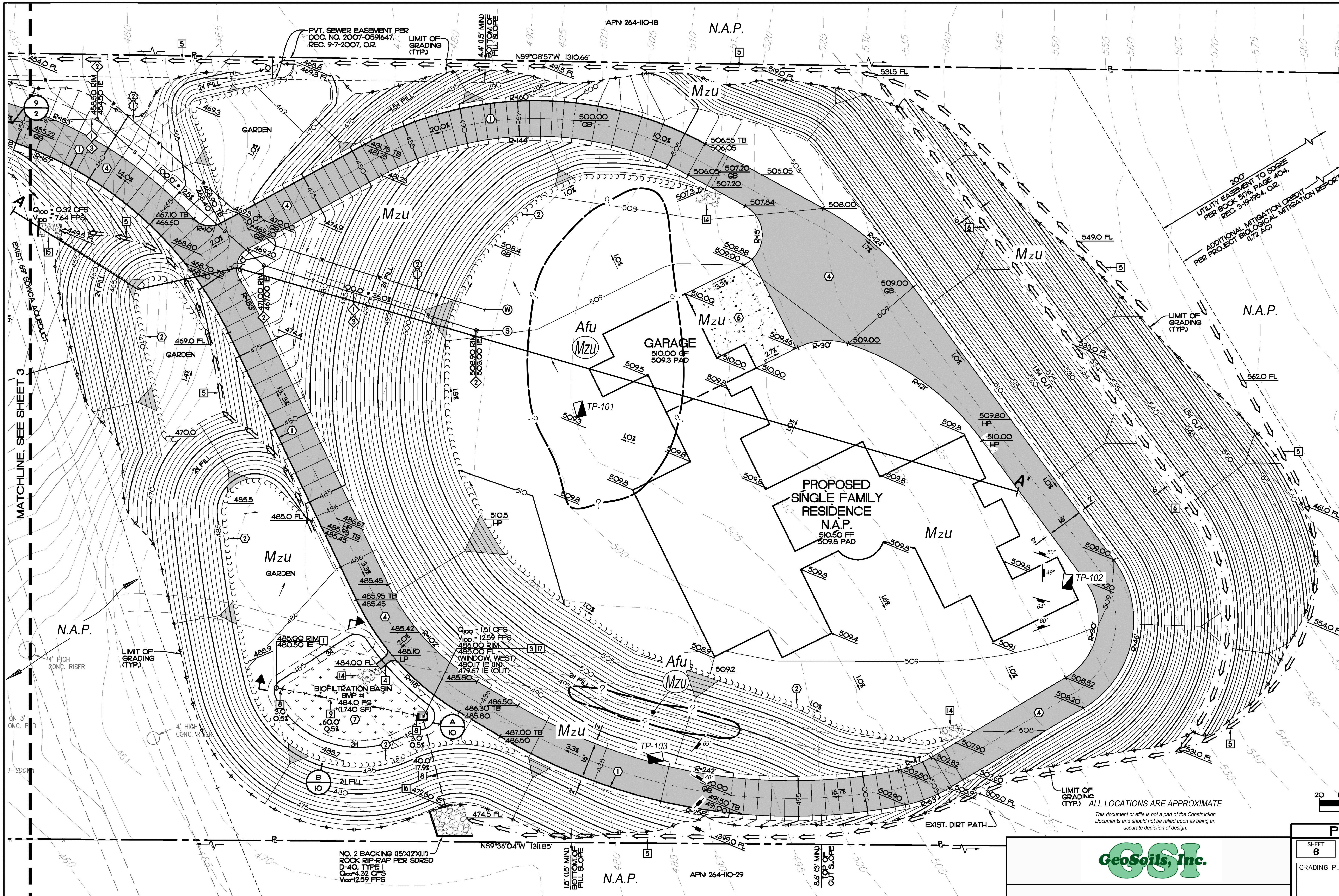
GRADING PLAN FOR:
HONARVAR RESIDENCE AND EQUESTRIAN PAD
CALIFORNIA COORDINATE INDEX 322-1713

APPROVED: FOR WILLIAM P. MORGAN COUNTY ENGINEER
BY:
DATE

ENGINEER OF WORK
SON P. NGUYEN R.C.E. 86249

PDS2019-LDGRMJ-30214

ENGINEER'S NAME - SNIPES-DYE ASSOCIATES



KEY NOTES

PVT. IMPROVEMENTS

(1) PVT. 6" AC BERM TYPE A PER SDRSD G-05 (TYP.).

(2) EARTHEN BERM PER DETAIL 1, SHT. 3 (TYP.)

(4) MIN. 4" A.C. PAVEMENT OVER 6" CLASS II AGGREGATE BASE (TYP.).

(7) ALL PROPOSED VEGETATION TO BE CONSTRUCTED/INSTALLED WITHIN BIOFILTRATION BASIN AND TREE WELLS SHALL BE CONFORMANCE WITH APPROVED LANDSCAPE AND IRRIGATION PLAN NO. PDS2022-LP-__________, AND SHALL BE INCLUDED IN PROJECT SWQMP.

PVT. STORM DRAIN

[1] PVT. STORM DRAIN CLEANOUT PER DETAIL 3, SHT. 3.

[3] PVT. CATCH BASIN TYPE F PER SDRSD D-07.

[5] PVT. DRAINAGE DITCH TYPE B PER SDRSD D-75 (TYP.).

[6] PVT. TERRACE & DITCH PER DS-10 & TYPE D PER SDRSD D-75 (TYP.).

[8] PVT. 8" PVC SDR-35 PER SDRSD D-60.

[9] PVT. 8" PVC SDR-35 PERFORATED PIPE.

[14] PVT. 3"-6" ROCKS (6'X10'X0.7' THICK).

[15] PVT. NO. 2 BACKING (6'X10'X1.1') ROCK RIP-RAP PER SDRSD D-40, TYPE I.

[16] PVT. STRAIGHT HEADWALL PER SDRSD D-32.

[17] MARK ALL INLETS WITH THE WORDS "NO DUMPING-DRAINS TO WATERWAYS" OR SIMILAR. SEE STENCIL TEMPLATE ON SHT. 10.

PVT. WATER

(1) PVT. 1" PVC SCH. 80 (TYP.).

(2) PVT. TRENCH AND BEDDING PER DETAIL 5, SHT. 3 (TYP.).

PVT. SEWER

<1> PVT. 4" PVC SDR-35 (TYP.).

<2> PVT. SEWER CLEANOUT PER DETAIL 3, SHT. 3.

<3> PVT. TRENCH AND BEDDING PER DETAIL 6, SHT. 3 (TYP.).

SEE PLATE 1 FOR LEGEND

ENGINEER OF WORK
Snipes-Dye associates
civil engineers and land surveyors
8348 CENTER DRIVE, STE. C, LA MESA, CA 91942
TELEPHONE (619) 697-9234 FAX (619) 460-2033
SON P. NGUYEN R.C.E. 86249
EXPIRES 03-31-23

REGISTERED PROFESSIONAL ENGINEER SON P. NGUYEN C 86249 Exp. 03-31-23 CIVIL STATE OF CALIFORNIA

COUNTY APPROVED CHANGES	
No.	Description

GeoSoils, Inc.

GEOTECHNICAL MAP

Plate 2

W.O. 8543-A-SC | DATE: 04/23 | SCALE: 1" = 20'

PRIVATE CONTRACT

SHEET 6 | COUNTY OF SAN DIEGO DEPARTMENT OF PUBLIC WORKS | 13 SHEETS

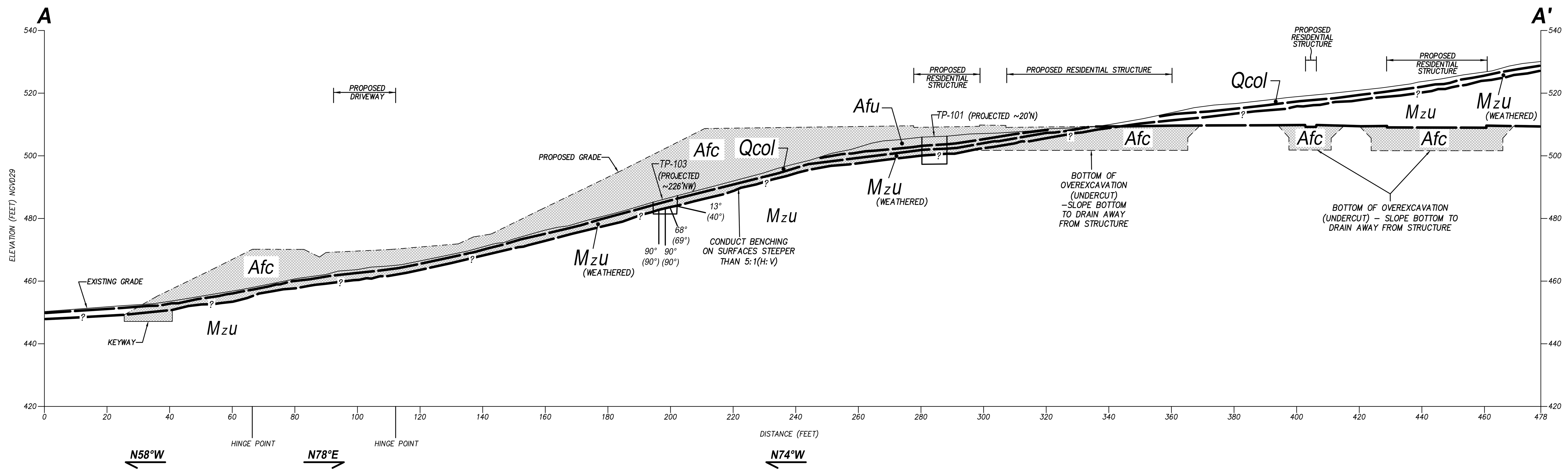
GRADING PLAN FOR:
HONARVAR RESIDENCE AND EQUESTRIAN PAD
CALIFORNIA COORDINATE INDEX 322-1713

APPROVED: FOR WILLIAM P. MORGAN COUNTY ENGINEER
BY:
DATE

ENGINEER OF WORK
SON P. NGUYEN R.C.E. 86249

PDS2019-LDGRMJ-30214

ENGINEER'S NAME - SNIPES-DYE ASSOCIATES



GRAPHIC SCALE

20 0 10 20 40 80

1" = 20'

GSI LEGEND

- Afc — ARTIFICIAL FILL – COMPACTED
- Afu — ARTIFICIAL FILL – UNDOCUMENTED
- Qcol — QUATERNARY COLLUVIUM
- Mzu (WEATHERED) — WEATHERED MESOZOIC METAVOLCANIC BEDROCK
- Mzu — MESOZOIC METAVOLCANIC BEDROCK
- —?— — APPROXIMATE LOCATION OF GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN
- 90° (90°) — APPARENT DIP OF JOINT (IN DEGREES), WITH TRUE DIP IN PARENTHESES
- — APPROXIMATE LIMITS OF PLANNED AND REMEDIAL COMPACTED FILLS

ALL LOCATIONS ARE APPROXIMATE

This document or efile is not a part of the Construction Documents and should not be relied upon as being an accurate depiction of design.

GeoSoils, Inc.

GEOLOGIC CROSS SECTION A-A'

Plate 3

W.O. 8543-A-SC	DATE: 04/23	SCALE: 1" = 20'