PRELIMINARY GEOTECHNICAL EVALUATION
2850 EL FUERTE STREET (3-LOT SUBDIVISION),
APN 215-370-28-00, CARLSBAD,
SAN DIEGO COUNTY, CALIFORNIA

FOR

C2 FUERTE HOLDINGS, LLC
1056 QUARTZ CT
SAN MARCOS, CALIFORNIA 92078

W.O. 7795-A-SC APRIL 29, 2020

GeoSoils, Inc.

Geotechnical • Geologic • Coastal • Environmental

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April 29, 2020

W.O. 7795-A-SC

C2 Fuerte Holdings, LLC
1056 Quartz Court
San Marcos, California 92078

Attn: Mr. Chad Clifford

Subject: Preliminary Geotechnical Investigation and Infiltration Feasibility Testing, 2850 El Fuerte St. (3-Lot Subdivision), Carlsbad, San Diego County, California, APN 215-370-28

Dear Mr. Clifford:

In accordance with your request and authorization, GeoSoils, Inc. (GSI) is pleased to present the results of our preliminary geotechnical investigation of the subject site as it relates to the proposed multi-family residential development thereon. The purpose of our study was to evaluate the onsite geologic and geotechnical conditions in order to develop preliminary geotechnical recommendations for earthwork, and the design of foundations, retaining walls, pavements, and other improvements applicable to the project.

EXECUTIVE SUMMARY

Based on our review of the available data (see Appendix A), as well as field exploration (see Appendix B and C), seismicity analysis (see Appendix D), laboratory testing (see Appendix E), and geologic and engineering analysis, the proposed three single-family residential development of the property appears to be feasible from a geotechnical viewpoint, provided that mitigation measures presented in the text of this report are properly incorporated into design and construction of the project. Stormwater infiltration is not recommended. The most significant elements of this study are summarized below:

- Based on our review of V & ME (2004, 2009) and subsurface data acquired during our recent subsurface exploration, surficial site earth materials generally consist of localized areas of undocumented fill and Quaternary-age colluvium, underlain by bedrock consisting of Mesozoic-Age metasedimentary and metavolcanic rocks (undivided). Bedrock was observed outcropping on the northwesterly property margin. A discontinuous weathering profile was encountered in the upper approximately four to five feet of the bedrock.

- Soils considered unsuitable for the support of proposed improvements and new planned fills consist of undocumented fill, colluvium, and weathered bedrock. For remedial treatment, these earth materials should be removed to expose the underlying unweathered sedimentary bedrock and may be reused as engineered fill per the recommendations contained herein. Based on the available subsurface data, remedial grading excavations for compressible soils are anticipated to range between approximately 2½ and 15¼ feet below the existing grades.

- Groundwater was not encountered during our site-specific field work to the depths explored. In general and based upon the available data to date, regional groundwater is not expected to be a major factor in the currently proposed development. The potential for perched water conditions to manifest along zones of contrasting permeabilities/densities (i.e., along fill lifts, fill/bedrock contacts, etc.) and along geologic discontinuities in the bedrock, during and following site development, cannot be precluded and should be anticipated.

- GSI's review and field exploration indicates no known Holocene-active faults are within the site, and the site is not located within an Alquist-Priolo Earthquake Fault Zone (California Geological Survey, 2018 [see Appendix A]). However, owing to the site's location within a seismically active region, it retains the potential to experience moderate to strong ground shaking, should an earthquake occur along any of the nearby regional active faults.

- The proposed structures and foundations, as well as other supporting infrastructure should be designed to resist seismic forces and deformation in accordance with the criteria contained in the 2019 California Building Code ([2019 CBC], California Building Standards Commission [CBSC], 2019a). Based on our site-specific seismic hazard analysis, appropriate seismic design parameters, conforming to the 2019 California Building Codes are provided herein.

- The potential for the proposed development to be adversely affected by mass wasting is considered low, provided our recommendations are incorporated into project design, construction, and post-development surface drainage practices and ground management. However, the onsite soils are considered highly susceptible to erosion. Thus, properly designed and maintained surface drainage is recommended. Surface drainage should not be allowed to pond near the tops or toes of slopes, or flow uncontrollably down or along slopes.

- Based on our analysis, the potential for seismic-induced liquefaction/densification to adversely affect the proposed development is considered low. Regardless, some seismic-induced deformation should be anticipated due to offsite densification near the subdivision boundaries, and is discussed herein.

- Assuming the use of standard, heavy earth-moving equipment with ripper attachments that is are in good working condition, it is our opinion that the bedrock will not pose significant constraints to the planned excavations currently shown on Advanced Civil Engineering, PLLC plans ([ACE], 2018). Non-productive trenching into the bedrock for foundations and underground utilities may be realized, especially if relatively lightweight trenching equipment such as mini-excavators and/or rubber-tire backhoes are used. Thus, the owner/developer may consider overexcavating and replacing the bedrock with compacted fills during grading to help facilitate trenching for improvement construction.

- A representative sample of near-surface undocumented fill materials was tested for expansion potential. The Expansion Index (E.I.) test indicated an E.I. of 102. This corresponds to high expansion potential. Atterberg Limits testing performed on a representative sample yielded a plasticity index (P.I.) of 49. Based on our experience, the P.I. is suggestive of high expansion potential. Visual classification of much of the unweathered bedrock indicates that these earth materials have a high to very high expansion potential. Thus, our observations and laboratory testing indicate that soils with very high to high expansion potentials are present at the subject site, and that some of the onsite earth materials meet the criteria of expansive soils, as defined in Section 1803.5.3 of the 2019 CBC (CBSC, 2019a). As required in Section 1808.6 of the 2019 CBC, foundations for buildings and structures founded on expansive soils necessitate structural design to mitigate their adverse shrink/swell effects. Alternatively, Section 1808.6 of the 2019 CBC stipulates that expansive soils within the influence of the building foundations will need to be removed and replaced with very low expansive soils (E.I. $\leq$ 20 and P.I. $\leq$ 14), or stabilized in place. Preliminary geotechnical recommendations for the design of post-tensioned foundation and mat foundation systems are provided herein for expansive soil mitigation.

- A representative sample of site material was evaluated for corrosion, soluble sulfates, and soluble chlorides. Laboratory testing indicates that the tested sample is strongly acidic with respect to soil acidity/alkalinity; is severely corrosive to exposed, buried metals when saturated; presents negligible sulfate exposure to concrete ("Exposure Classes S0, C1, and W0" per Table 19.3.1.1 of American Concrete Institute (ACI) 318-14); and contains very low concentrations of soluble chlorides. It should be noted that 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 the Project Architect, Civil Engineer, and/or Structural Engineer. Minimally, Exposure Classes S0 and W1, per ACI 318-14, should be utilized.

- Based on our review of ACE (2018), significant graded slopes are not proposed. It is our opinion that proposed graded slopes with overall heights of $\leq$ 15 feet will be grossly and surficially stable provided the recommendations in this report are incorporated into their design and construction.

- It should be noted that the 2019 CBC (CBSC, 2019a) indicates that removals of unsuitable soils be performed across all areas to be graded under the purview of the grading permit, and not just within the influence of the proposed residential structures. 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 removals, if removals cannot be performed onsite or offsite. In general, any planned settlement-sensitive improvement located above a 1:1 (h:v) projection up from the bottom, outboard edge of the remedial grading excavation at the subdivision boundary would be affected by perimeter conditions. On a preliminary basis, any planned settlement-sensitive improvements located within Parcel 1 and Parcel 2 existing improvements that need to remain in service (e.g., an existing storm drain that crosses the property) would require deepened foundations into unweathered bedrock or additional reinforcement by means of ground improvement or specific structural design. Otherwise, these proposed improvements may be subject to distress and a reduced service life. This will also require proper disclosure to all interested/affected parties should this condition exist at the conclusion of grading. In order to further evaluate perimeter conditions, it would be beneficial for GSI to review the as-built plans for the existing storm drain that crosses the property and the utilities along the north easterly property boundary, prior to final grading and/or foundation plans.

- Our stormwater infiltration feasibility evaluation indicates that "no infiltration" is feasible in the vicinity of the property. Therefore, GSI recommends against stormwater infiltration into the onsite earth materials.

- Adverse geologic structures that would preclude project feasibility were not encountered.

- The recommendations presented in this report should be incorporated into the design and construction considerations of the project.

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

David W. Skelly
Civil Engineer, RCE 47857

MK/DWS/JPF/mn

Distribution: (3) Addressee (wet signed)

TABLE OF CONTENTS

FIGURES:

ATTACHMENTS:

PRELIMINARY GEOTECHNICAL EVALUATION
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SAN DIEGO COUNTY, CALIFORNIA

SCOPE OF SERVICES

The scope of our services has included the following:

1. Review of available soils and geologic data, and aerial photographs that pertain to the site and the immediate site vicinity.

2. Review of the current "Preliminary Grading Plan," prepared by ACE (2018).

3. Surficial geologic mapping of the site.

4. Subsurface exploration consisting of advancement of two (2) hollow-stem auger borings with a truck-mounted drill rig to evaluate the near-surface geologic conditions and sample the onsite earth materials (see Appendix B).

5. Percolation testing in four (4) of the hollow-stem auger borings to evaluate storm water infiltration feasibility. Calculations for the conversion of percolation rates to infiltration rates, and the completion of City of Carlsbad (2016) Worksheets C.4-1 and D.5-1 are provided herein (see Appendix C).

6. Site-specific seismic hazard evaluation and seismicity analysis (see Appendix D).

7. Laboratory testing on selected relatively undisturbed and representative bulk soil samples collected during subsurface exploration (see Appendix E).

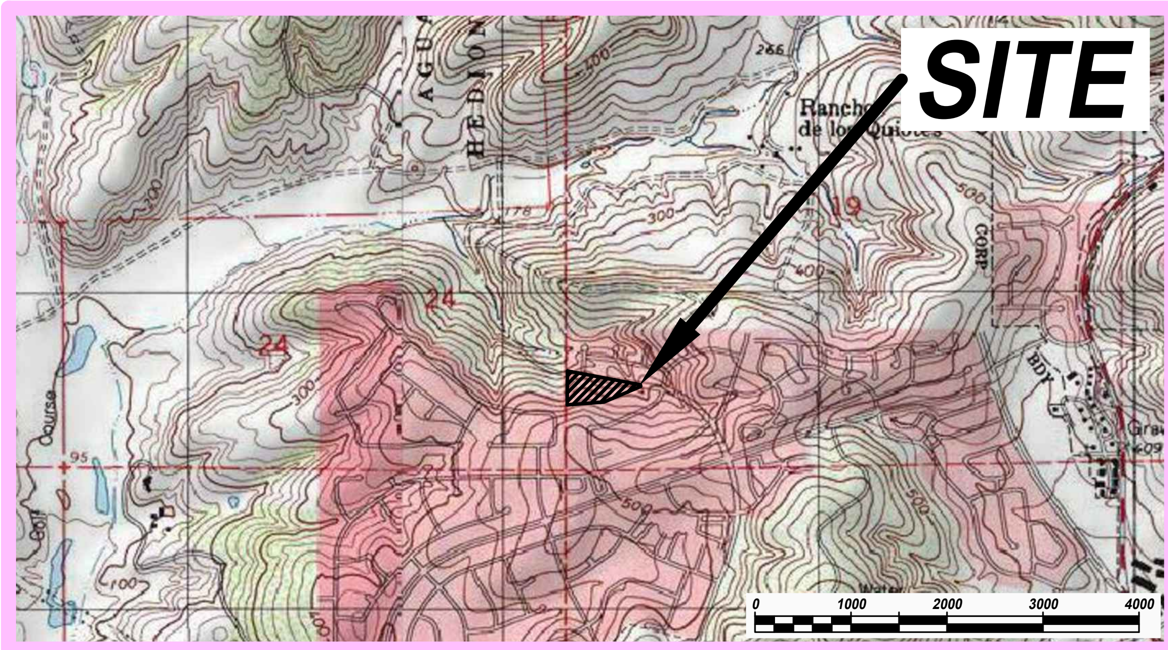
8. Analysis of data.

9. Preparation of this appropriately illustrated summary report of our preliminary geotechnical evaluation and accompaniments.

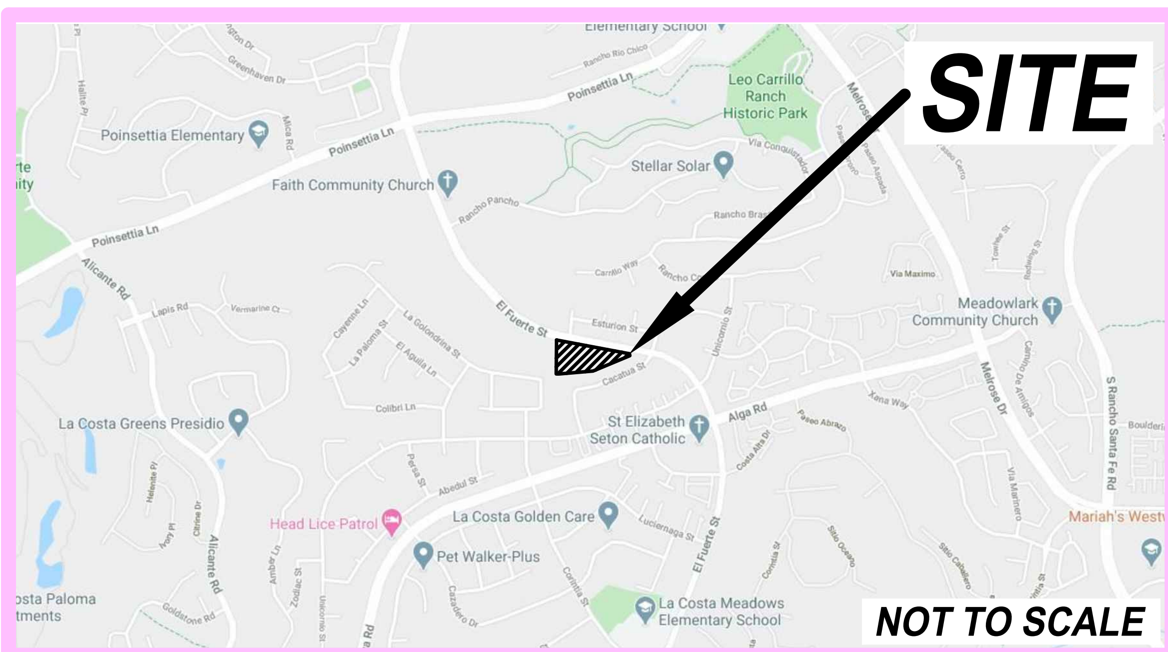
PROPERTY DESCRIPTION/PROPOSED SITE DEVELOPMENT

The subject site consists of approximately 3.9 acres of undeveloped land with north facing hillside terrain along the south side of El Fuerte Street in Carlsbad, San Diego County, California (see Figure 1, Site Location Map). The geographic coordinates of the approximate centroid of the site are 33.1115˚,-117.2431˚.

Topographically, the site is characterized by nearly level graded areas adjacent to El Fuerte Street that give way to slopes that ascend onto adjacent residential development to the south. Most of the site supports a light cover of native grasses and large trees near El Fuerte Street situated along the easterly side of an anthropogenically modified drainage



Base Map: TOPO!® ©2003 National Geographic, U.S.G.S. Oceanside Quadrangle, California -- San Diego Co., 7.5 Minute, dated 1996, current, 2000.



Base Map: Google Maps, Copyright 2020 Google, Map Data Copyright 2020 Google

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SITE LOCATION MAP

Figure 1

course and the westerly flank of a knoll. The site is generally flat-lying to locally moderately sloping in a predominant northwesterly direction. According to ACE (2018), site elevations vary between approximately 384 and 430 feet (unknown survey datum), for an overall relief of about 46 feet. Existing slope gradients are on the order of 3:1 (horizontal:vertical [h:v]) or flatter. Site drainage is accommodated by sheet flow runoff directed northward toward El Fuerte Street. A north-northwesterly trending sanitary sewer main crosses the property near its center. Site vegetation predominately consists of weeds, grasses, and brush with sparse trees. A small cluster of boulders occurs toward the northeasterly property margin. It is currently unknown if the boulders were placed there by anthropogenic processes or are natural in origin, as the immediately surrounding subsurface was not evaluated.

A review of ACE (2018) indicates that proposed development consists of preparing the site to receive three (3) single-family residences with associated underground utility and street improvements, and a stormwater retention basin. ACE (2018) shows maximum planned cut and fill thicknesses on the order of 10 feet and 4 feet, respectively. In general, ACE (2018) shows that grade transitions will be utilized, and GSI anticipates that the grade transitions will be accommodated by the construction of retaining walls with a maximum height of roughly 10 feet to 14 feet. Although the retaining wall designs are not clearly indicated on ACE(2018), GSI anticipates the walls will consist of concrete masonry unit (CMU) and mechanical stabilized earth (MSE) construction. GSI anticipates that the proposed residences will be two- to three-stories (including basements) in height. We further anticipate that the proposed residences will consist of wood frame and CMU construction with slab-on-grade floors, and be supported by shallow foundations. Building loads are assumed to be typical for this type of relatively light residential construction. Sanitary sewage disposal for is understood to be accommodated by tying into the municipal sewage system. The need for import soils is currently unknown.

FIELD STUDIES

GSI conducted field studies at the subject site in late February 2020. Field work consisted of surficial mapping and the advancement of six (6) hollow-stem auger borings with a truck-mounted drill rig. The purpose of the field work was to evaluate the near-surface geologic conditions and to sample the onsite earth materials. Percolation testing was conducted in four (4) of the hollow-stem auger borings to evaluate stormwater infiltration feasibility in the vicinity of the anticipated stormwater basin. The approximate locations of borings are included on the Geotechnical Map (see Plate 1), which uses ACE (2018) as a base. Logs of the borings are provided in Appendix B. Our field percolation test data and associated infiltration conversion calculations are provided in Appendix C.

REGIONAL GEOLOGIC SETTINGS

The subject property lies within the coastal plain physiographic region of the Peninsular Ranges Geomorphic Province of southern California. This region consists of dissected, mesa-like terraces that transition inland to rolling hills. The encompassing Peninsular Ranges Geomorphic Province is characterized by elongated mountain ranges and valleys that trend northwesterly. This geomorphic province extends from the base of the east-west aligned Santa Monica - San Gabriel Mountains, and continues south into Baja California. 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.

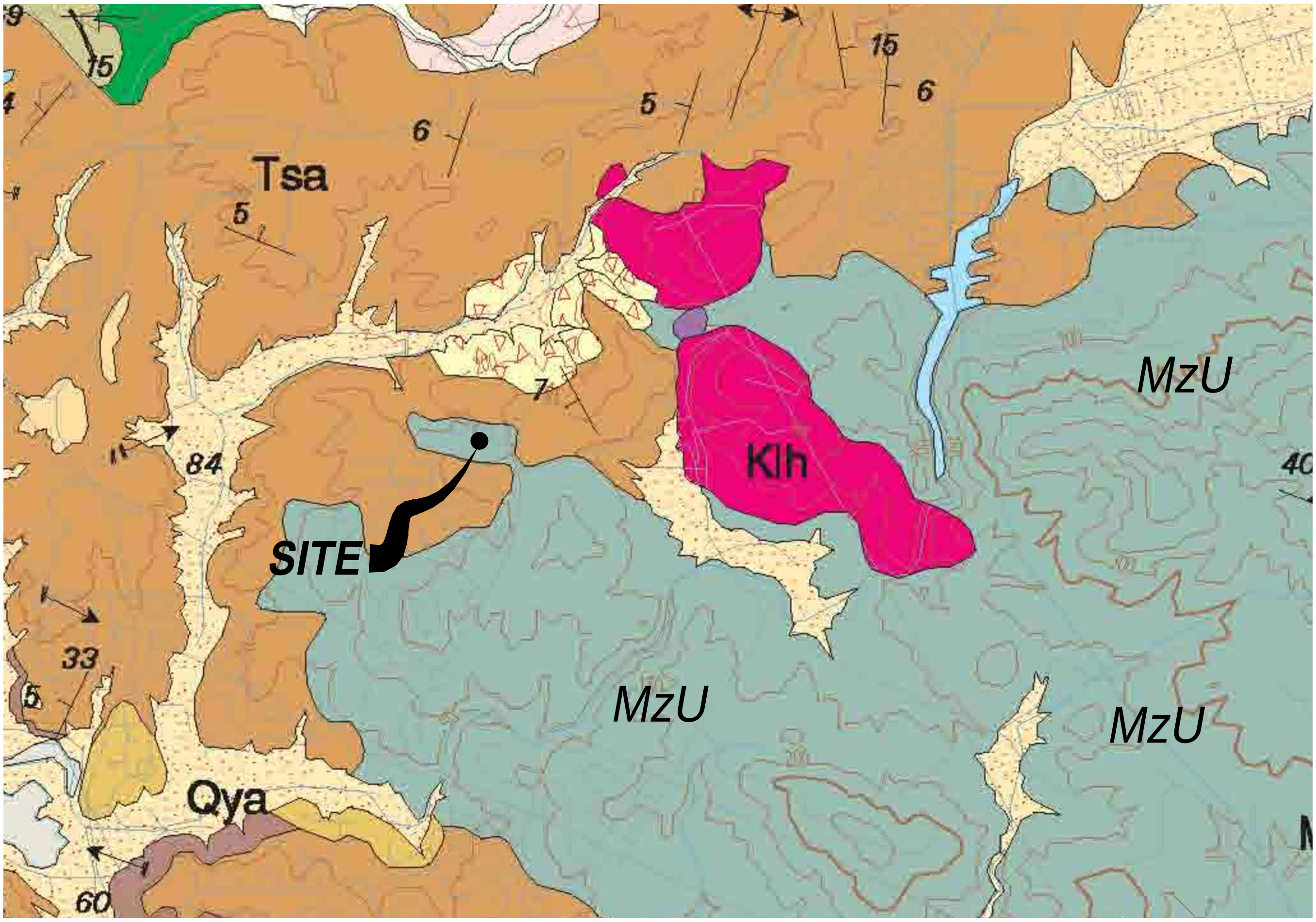
In the southern California region, deposition occurred during the Cretaceous Period and Cenozoic Era in the continental margin of a forearc basin. Sediments, derived from Cretaceous-age plutonic rocks and Jurassic-age volcanic rocks, were deposited during the Tertiary Period (Eocene-age) into the narrow, steep, coastal plain and continental margin of the basin. These rocks have been uplifted, eroded, and deeply incised. During the early Pleistocene Epoch, a broad coastal plain was developed from the deposition of marine terrace deposits. During the mid- to late-Pleistocene Epoch, this plain was uplifted, eroded and incised. Alluvial deposits have since filled the lower valleys, and young marine sediments are currently being deposited/eroded within coastal and beach areas (see Figure 2). Regional geologic mapping by Kennedy and Tan (2007) indicates the site is underlain by Mesozoic-age bedrock, consisting of undivided metasedimentary and metavolcanic rocks. For the purposes of this evaluation, the more recent nomenclature of Kennedy and Tan (2007) will be adhered to.

SITE GEOLOGIC UNITS

The site geologic units encountered during our mapping and subsurface exploration included undocumented artificial fill, Quaternary-age colluvium (Map Symbol - Qcol), and undivided Mesozoic-age metasedimentary and metavolcanic rocks (Map Symbol - MzU) The earth materials are generally described below from the youngest to the oldest. The distribution of the mappable units across the site is shown on Plate 1 (Geotechnical Map).

Artificial Fill - Undocumented (Map Symbol - Afu)

Undocumented artificial fill was encountered at the surface in he hollow-stem auger borings B-1 & B-2. As observed therein, the undocumented fill consisted of brown, fine- to coarse-grained sand, and brown, olive brown, and dark brown clayey sand. The fill was generally dry to moist and loose to dense/hard. Where field measurements were taken within the subsurface explorations, the thickness of the undocumented fill was on the order of 5 to 10 feet, or so. Owing to a lack of documentation regarding the suitability of the existing fill to support the proposed engineered improvements and its placement atop potentially compressible colluvium, as well as non-uniformity, the undocumented fill is considered unsuitable for supporting proposed improvements or new fill in its existing state, and will require removal and recompaction.



GEOLOGIC MAP OF THE OCEANSIDE 30' X 60' QUADRANGLE, CALIFORNIA

Compiled by

Michael P. Kennedy and Siang S. Tan

2005

DESCRIPTION OF MAP UNITS

Qya	Young alluvial flood-plain deposits (Holocene and late Pleistocene)
Tsa	Santiago Formation (middle Eocene)
Klh	Leucogranodiorite of Lake Hodges (mid-Cretaceous)
Mzu	Metasedimentary and metavolcanic rocks, undivided (Mesozoic)

ONSHORE MAP SYMBOLS

Contact - Contact between geologic units; generally approximately located; dotted where concealed.

Strike and dip of beds

- 70 Inclined
- 70 Overturned
- Vertical
- Horizontal

Strike and dip of igneous foliation

- 45 Inclined
- Vertical

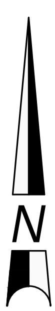
Strike and dip of igneous joints

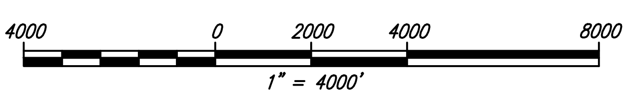
- 60 Inclined
- Vertical

Strike and dip of metamorphic foliation

- 55 Inclined

Strike and dip of sedimentary joints





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REGIONAL GEOLOGIC MAP

Figure 2

W.O. 7795-A-SC	DATE: 04/20	SCALE: 1" = 5000'

Quaternary-Age Colluvium (Map Symbol - Qcol)

Quaternary Colluvium was encountered at five to ten feet from the surface in the hollow-stem auger borings B-1 & B-2. Where encountered, these soils generally consist of brown to dark brown, fine to coarse grained sand. Colluvium was typically moist and stiff to very stiff. Where field measurements were taken within the subsurface explorations, the thickness of the Colluvium was on the order of 3 to 4 feet. Existing deposits of colluvium are considered potentially compressible in their existing state. As such, colluvium should not be used for the support of settlement-sensitive improvements and/or any planned fill, unless adequately remediated.

Mesozoic-Age Metasedimentary and Metavolcanic Rocks, Undivided (Map Symbol - Mzu)

Bedrock belonging to the Mesozoic-age metasedimentary and metavolcanic rocks, undivided was encountered beneath the colluvium in our Borings B-1 and B-2. Based on the available subsurface data, the bedrock generally occurs near the surface on the northwesterly property margin, and up to 13½ feet deep on easterly property margin below existing grade, where exploration was performed. However, as observed by GSI in exploratory borings B-1 and B-2, the upper approximately 1 foot to 4 feet of the bedrock was weathered. Weathered metasedimentary rock and metavolcanic rock generally consisted of reddish brown to pale grey silty clay; and brownish metavolcanic rocks with trace to locally abundant cobble. Weathered metasedimentary rock and metavolcanic rock was generally moist, and hard to very stiff. Unweathered metavolcanic bedrock was encountered by GSI at depths ranging between approximately 1 foot on the northwest side of the property, to 17 feet deep on south side of the property, below existing grade. Unweathered bedrock was generally moist and very hard to very stiff. Unweathered bedrock is considered suitable bearing materials.

GROUNDWATER

Groundwater was not observed at the explored depths (see Appendix B). Seasonal and cyclical variations in groundwater conditions from those encountered during field work, performed in preparation of this report, cannot be entirely precluded and should be anticipated. Due to its depth below the site, the regional groundwater table is not considered a significant geotechnical concern. However, perched groundwater may occur locally (as the result of heavy/frequent precipitation, up-gradient irrigation practices, or damaged “wet” underground utilities). Perched groundwater has the potential to occur along zones of contrasting permeabilities/densities (fill/Sedimentary bedrock contacts, sandy/clayey fill lifts, etc.) or along geologic discontinuities in the bedrock. This potential should be anticipated and disclosed to all interested/affected parties.

Due to the potential for post-development perched water to manifest near the surface, owing to as-graded permeability/density contrasts, more onerous slab design is necessary for any new slab-on-grade floor (State of California, 2019). Recommendations for reducing the amount of water and/or water vapor through slab-on-grade floors are provided in the "Soil Moisture Considerations" sections of this report.

PRELIMINARY STORMWATER INFILTRATION FEASIBILITY AND DISCUSSION

Subsurface Exploration

During our site-specific field studies, four (4) infiltration test borings (IB-1, IB-2, IB-3 and IB-4) were advanced with a hollow-stem auger drill rig to depths ranging between approximately 5½ and 7½ feet below the existing grades, respectively. Percolation testing was performed in these borings to develop preliminary soil infiltration rates with respect to permanent stormwater best management practices (BMPs)/low impact development (LID). The test borings (IB-1, IB-2, IB-3 and IB-4) were logged by a GSI representative. The logs of these borings are provided in Appendix B. Their approximate locations are presented on the Geotechnical Map (Plate 1).

United States Department of Agriculture (USDA)/Natural Resources Conservation Service (NRCS) Soil Survey

A review of the United States Department of Agriculture/Natural Resources Conservation Service database ([USDA/NRCS]; 1973, 2019) indicates that the surficial soil units mapped at the location of the currently proposed permanent stormwater BMP/LID primarily consist of the Altamont Clay (AtD), 9 to 15 percent slopes with lesser Huerhuero loam (HrE2), 15 to 30 percent slopes (eroded) near its westerly margin. The USDA/NRCS indicates the capacity of the most limiting layers in soil unit AtD to transmit water (K_{sat}) are moderately low to moderately high (0.06 to 0.2 inches per hour [in/h]) and soil unit HrE2 to transmit water (K_{sat}) are very low to moderately low (0.00 to 0.06 inches per hour [in/h]). The Hydrologic Soil Group (HSG) designations for AtD soil unit is "C." and HrE2 soil unit is "D." According to City of Carlsbad (2016), the occurrence of HSG "D" soils provides a reasonable basis to conclude that infiltration is not feasible for a given drainage management area (DMA), if confirmed by available data.

Infiltration Screening Feasibility

Percolation testing was performed in Infiltration Borings IB-1, IB-2, IB-3 and IB-4 in general accordance with Riverside County Flood Control and Water Conservation District ([RCFC & WCD], 2011) guidelines. As indicated previously, the logs of Infiltration Borings IB-1 and IB-2 are presented in Appendix B.

Prior to testing, the soils within Infiltration Borings IB-1, IB-2, IB-3 and IB-4 were pre-soaked by filling them with more than 5 gallons of water. Since water was still observed in the test borings after a 4-hour waiting period, the borings were allowed to pre-soak overnight. At the onset of testing the following day, GSI evaluated the soil conditions in the test borings to determine whether"sandy soil criteria," as outlined in RCFC & WCD (2011) was met.

This was accomplished by adding water to the test borings and allowing the water level to fall over two (2) 25-minute test intervals to see if a greater than 6-inch change in water column height occurred within each test period. This test indicated that the soil conditions in all four test borings did not meet the "sandy soil criteria." Thus, in general accordance with RCFC & WCD (2011) protocol, percolation testing in these borings continued over a 6-hour period, with readings taken every 30 minutes. At the beginning of each test interval, the boring was refilled with water and the water level was allowed to drop. Both initial and final readings were rounded to the nearest ¼ inch.

The field percolation test data sheets are provided in Appendix C. The change in water height recorded during the last test interval was then used to calculate the infiltration rate using the Porchet Method, per RCFC & WCD (2011) guidelines. The calculation sheets showing the conversion of the field percolation test data to infiltration rates are also provided in Appendix C.

The following table presents the change in water column height in each infiltration test boring during the last test interval:

INFILTRATION BORING NO.	CHANGE IN WATER HEIGHT DURING FINAL TESTING PERIOD (INCHES)
IB-1	0.0
IB-2	0.0
IB-3	0.25
IB-4	0.25

The following table summarizes the calculated infiltration rate within each test boring using Porchet Method:

INFILTRATION TEST BORING	INFILTRATION RATE (INCHES PER HOUR)
IB-1	0.0
IB-2	0.0
IB-3	0.019
IB-4	0.019

Infiltration Conclusions

The results of the infiltration testing demonstrate soil infiltration rates that are generally slightly faster than the K_{sat} values published by the USDA/NRCS. This is likely due to the occurrence of undocumented fill and colluvium in the Infiltration Borings IB-1, IB-2 and undocumented fill and in the near-surface of Infiltration Borings IB-3 and IB-4.

Owing to the variable infiltration rates acquired from our testing, the lower rate should be used for design (0.0 in/hr). When the minimum site assessment safety factor is applied, the estimated reliable infiltration rate is 0.01 in/hr. This rate supports no infiltration. The completed Worksheets C.4-1 and D.5-1 of City of Carlsbad (2016) are included in Appendix C. The infiltration into the onsite earth materials at the proposed stormwater BMP/LID is not recommended due to the potential adverse effects, it could pose to existing offsite improvements and any proposed retaining walls.

GEOLOGIC HAZARDS EVALUATION

General

GSI has evaluated the site relative to geologic and seismic hazards that could have an effect on the proposed development. A summary of this assessment is provided 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 and/or surficial failures).

According to regional landslide susceptibility mapping by Tan and Giffen (1995), the site is located within landslide susceptibility Subarea 3-1, which is characterized as being "generally susceptible" to landsliding. Tan and Giffen (1995) indicate that slopes within Subarea 3-1 are at, or close to, their stability limits due to an amalgamation of weak earth materials and steep topography. Although Subarea 3-1 does not presently contain landslide deposits, slopes in this subarea can experience local failures, especially when adversely modified. Geomorphic expressions indicative of past mass wasting events (i.e., scarps and hummocky terrain) were not, however, observed on the property during our field studies nor our review of regional geologic mapping. Further, no adverse geologic structures were encountered during our subsurface exploration, and regional geologic maps (Tan & Kennedy; 1977, 2008) do not indicate the presence of landslides on the property. Given the absence of adverse geologic structure, the dense nature of the underlying bedrock, and the lack of evidence with respect to existing slope instability, the potential for deep seated landslides to affect the proposed site development is considered low.

The onsite soils are, however, considered erosive. Therefore, any slopes comprised of these materials may be subject to gullying, sloughing, and surficial slope failures depending on rainfall frequency and severity, and surface drainage practices. Such risks can be minimized through properly designed, and regularly and periodically maintained surface drainage and landscape practices.

FAULTING AND REGIONAL SEISMICITY

Regional Faults

Our review indicates that there are no known Holocene-active faults crossing the property and the site is not within an Alquist-Priolo Earthquake Fault Zone (California Geological Survey, 2018; Jennings and Bryant, 2010). However, the site is situated in a region subject to periodic earthquakes along active faults. The Rose Canyon fault (part of the Newport-Inglewood - Rose Canyon fault zone) is the closest known Holocene-active fault to the site (located at a distance of approximately 7.7 miles [12.4 kilometers]) to the west, and 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 D. 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

No faults were observed to specifically transect the site during the field investigation nor during our review of available regional geologic maps (Kennedy and Tan, 2007; Weber, 1982).

Surface Rupture

Owing to the absence of known Holocene-active faults within the site, the potential for the proposed development to be adversely affected by surface rupture from fault propagation to the surface is considered very 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 from an upper-bound event on the Rose Canyon fault may be on the order of 0.49g. The computer printouts of pertinent portions of the EQFAULT program are included within Appendix D.

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 August 15, 2018). This program performs a search of the historical earthquake records for magnitude 5.0 to 9.0 seismic events within a 100-kilometer radius, from 1800 through August 15, 2018. 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 August 15, 2018 was about 0.44g. A historic earthquake epicenter map and a seismic recurrence curve are also estimated/generated from the historical data. Computer printouts of the EQSEARCH program are presented in Appendix D.

Seismic Shaking Parameters

Based on the site conditions, the following table summarizes the site-specific design criteria obtained from the 2019 CBC (CBSC 2019), Chapter 16 Structural Design, Section 1613, Earthquake Loads. The computer program "OSHPD Seismic Design Maps" (https://seismicmaps.org/), provided by the Structural Engineers Association of California (SEAOC) and the State of California's Office of Statewide Health Planning and Development (OSHPD) was utilized for design. The short spectral response utilizes a period of 0.2 seconds.

CBC SEISMIC DESIGN PARAMETERS		
PARAMETER	**VALUE**	**2019 CBC or REFERENCE**
Site class	C	Table 1604.5
Spectral Response - (0.2 sec), S_s	0.939 g	Section 1613.2.2/ Chap. 20 ASCE 7-16 (p. 203-204)
Spectral Response - (1 sec), S_1	0.344 g	Section 1613.2.1 Figure 1613.2.1(1)
Site Coefficient, F_a	1.2	Section 1613.2.1 Figure 1613.2.1(2)
Site Coefficient, F_v	1.5	Table 1613.2.3(1)

CBC SEISMIC DESIGN PARAMETERS		
PARAMETER	**VALUE**	**2019 CBC or REFERENCE**
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec), S_{MS}	1.126 g	Table 1613.2.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (1 sec), S_{M1}	0.516 g	Section 1613.2.3 (Eqn 16-36)
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	0.751 g	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.344 g	Section 1613.2.4 (Eqn 16-38)
PGA_M	0.49 g	Section 1613.2.4 (Eqn 16-39)
Seismic Design Category	D	ASCE 7-16 (Eqn 11.8.1)

GENERAL SEISMIC PARAMETERS	
PARAMETER	**VALUE**
Distance to Seismic Source (Rose Canyon fault)	7.7 mi (12.4 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 2019 CBC (CBSC 2019a) and regular maintenance and repair following locally significant seismic events (i.e., M_w4.5) will likely be necessary, as is the case in all of southern California.

SECONDARY SEISMIC HAZARDS

Liquefaction

Seismically-induced liquefaction is 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 sand boils, lateral movement/sliding, volumetric consolidation and settlement of loose sediments, and other damaging deformations as pore pressures dissipate. 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. Thus, one of the primary factors controlling liquefaction potential is the depth to groundwater.

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

Liquefaction susceptibility is related to numerous factors and the following conditions must generally exist, or have the potential to exist, for liquefaction to occur: 1) sediments must be relatively young in age and not have developed a large amount of cementation; 2) sediments must consist mainly of medium to fine 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 have a potential for a design seismic event of a sufficient duration and magnitude to induce straining of soil particles. Only one to perhaps two of these five necessary conditions have the potential to affect the site simultaneously.

Seismic Settlement/Seismic Densification

Seismic densification is a phenomenon that typically occurs in low relative density granular soils (i.e., USCS classified as SP, SW, SC, and SM) that are above the groundwater table and are significantly drier than the optimum moisture content. During seismically-induced ground shaking, these natural sediments deform under loading and volumetric strain, resulting in ground surface settlements. Some contribution to seismic settlement has been incorporated in our evaluations herein. Some densification of the adjoining un-mitigated onsite and offsite areas may influence improvements at the perimeter of the project area. These unsaturated granular soils are susceptible if left in their original density (unmitigated), and are significantly drier than the optimum water content (as defined by the ASTM D 1557).

Summary

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 bedrock that underlies the site in the near-surface, the depth to the regional groundwater table, and the herein provided recommendations for remedial earth and foundation design and construction. Our recommendations for remedial earthwork, fill placement, and foundation design, and construction would further reduce any significant densification potential. Some seismic densification of the adjoining un-mitigated site(s) may adversely influence planned improvements at the perimeter of the site. However, given the remedial earthwork and foundation recommendations provided herein, the potential for the site to be affected by significant seismic densification or liquefaction of adjoining offsite soils may be considered low.

Other Seismic Hazards

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

- Surface Fault Rupture
- Dynamic Settlement
- Ground Lurching or Shallow Ground Rupture
- Tsunami
- Seiche

It is important to keep in perspective that in the event of an upper bound earthquake occurring on any of the nearby major faults, strong ground shaking would occur in the subject site's general area. Potential damage to any structure(s) would likely be greatest from the vibrations and impelling force caused by the inertia of a structure's mass than from those induced by the hazards considered above. Following implementation of remedial earthwork and design of foundations described herein, this potential would be no greater than that for other existing structures and improvements in the immediate vicinity that comply with current and adopted building standards.

LABORATORY TESTING

General

Laboratory tests were performed on relatively undisturbed and representative bulk samples of the onsite earth materials collected during our exploratory subsurface work, in order to evaluate their physical characteristics and engineering properties. The test procedures used and subsequent results are presented below.

Classification

Soils were classified with respect to the Unified Soil classification System (U.S.C.S.) in general accordance with ASTM D 2487 and D 2488. The soil classifications are presented on the Exploration Logs (test pit and boring logs) in Appendix B.

Field Moisture and Density

The field moisture contents and dry unit weights were determined for relatively "undisturbed" samples of earth materials obtained from the test pits. The dry unit weight was evaluated in general accordance with ASTM D 2937 (reported in pounds per cubic foot [pcf]), and the field moisture content was evaluated as a percentage of the dry weight in general accordance with ASTM D 2216. Results of these tests are summarized on the Test Pit Logs (see Appendix B).

Laboratory Standard

The maximum density and optimum moisture content was evaluated for a composite sample of the undocumented artificial fill encountered in the test pits, in general accordance with the laboratory standard, ASTM D 1557. The moisture-density relationships obtained for these soils are shown on the following table:

SAMPLE LOCATION AND DEPTH (FT)	MAXIMUM DENSITY (PCF)	OPTIMUM MOISTURE CONTENT (%)
B-2 @ 5-10(Composite)	120.3	13.7
T-1 @ 2 (V&ME, 2004)	115.5	15.6
T-2 @ 7 (V&ME, 2004)	113.1	15.8
T-7 @ 4 (V&ME, 2004)	116.6	15.3

Expansion Index

A representative sample of near-surface soil was tested for expansivity. The Expansion Index (E.I.) tests were performed in general accordance with ASTM Standard D 4829. The laboratory test results are presented in the following table.

SAMPLE LOCATION AND DEPTH	EXPANSION INDEX	EXPANSION POTENTIAL
B-2 @ 5-10	102	High
T-1 @ 2 (V&ME, 2004)	132	Very High
T-2 @ 7 (V&ME, 2004)	25	Low
T-7 @ 4 (V&ME, 2004)	114	High
E.I. = 0-20 - Very Low Expansion Potential E.I. = 21 to 50 - Low Expansion Potential E.I. = 51 to 90 - Medium Expansion Potential E.I. = 91 to 130 - High Expansion Potential E.I. $\geq$ 131 - Very High Expansion Potential		

Atterberg Limits

Atterberg Limits testing was performed on a representative fine-grained sample of weathered bedrock to evaluate its liquid limit, plastic limit, and plasticity index (P.I.) in general accordance with ASTM D 4318. Test results are presented in the following table and in Appendix E. Testing indicates that the sample is subject to plastic deformation.

SAMPLE LOCATION AND DEPTH (FT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-2 @ 5-10	65	16	49
T-1 @ 2 (V&ME, 2004)	50	22	28
T-2 @ 7 (V&ME, 2004)	31	28	3
T-7 @ 4 (V&ME, 2004)	43	33	10

Direct Shear

Strain-controlled direct shear tests were performed on a remolded representative soil sample in general conformance with the ASTM D 3080 test method. Prior to testing, the sample was remolded to 90 percent of its maximum density (per ASTM D 1557) and its optimum moisture content. The test results are presented in the following table and in Appendix E.

SAMPLE LOCATION AND DEPTH (FT)	COHESION (PSF)	FRICTION ANGLE (DEGREES)
T-1@ 2 (V&ME, 2004)	343	22
T-2@ 7 (V&ME, 2004)	191	31
T-7@ 4 (V&ME, 2004)	459	17

Saturated Resistivity, pH, and Soluble Sulfates, and Chlorides

Testing was performed on a representative sample of the onsite earth materials for general soil corrosivity and soluble sulfates, and chlorides testing. The testing included evaluation of soil pH, soluble sulfates, soluble chlorides, and saturated resistivity. Test results are presented in Appendix E and the following table:

SAMPLE LOCATION AND DEPTH (FT)	pH	SATURATED RESISTIVITY (ohm-cm)	SOLUBLE SULFATES (% by weight)	SOLUBLE CHLORIDES (ppm)
B-2 @ 5-10	5.3	1,400	0.059	530

Corrosion Summary

Laboratory testing indicates that the tested sample of the onsite soils is strongly acidic with respect to soil acidity/alkalinity; is severely corrosive to exposed, buried metals when saturated; presents negligible sulfate exposure to concrete (i.e., Exposure class S0, C1, and W0, per Table 19.3.1.1 of American Concrete Institute [ACI] 318-14); and contains low concentrations of soluble chlorides. It should be noted that 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 the Project Architect, Civil Engineer, and/or Structural Engineer.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

General

Based on our field exploration, laboratory testing, and geotechnical engineering analysis, it is our opinion that the site appears suitable for the proposed development from a geotechnical engineering and geologic viewpoint, provided that the recommendations presented in the following sections are incorporated into the design and construction phases of site development. The primary geotechnical concerns with respect to the proposed development are:

- The potentially compressible nature of the undocumented artificial fill, colluvium, and weathered bedrock.
- The depth to competent bearing material below existing grades.
- On-going expansion and corrosion potentials of the onsite soils.
- Constraints to remedial earthwork due to perimeter conditions.
- Excavations into the bedrock.
- Settlement potential (static and seismic).
- The potential for perched groundwater conditions to be encountered both during and following site development.
- Stormwater infiltration feasibility and the effects of infiltration on nearby improvements.
- Regional seismic activity.

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 have been completed using the information provided and obtained during our field work. In the event that any significant changes are made to proposed site development, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the recommendations of this report are evaluated 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.

- All undocumented artificial fill, Quaternary-age colluvium, and weathered portions of the bedrock are considered potentially compressible in their existing state; and therefore, should not be used to support the proposed settlement-sensitive improvements and new planned fills without mitigation.

- Remedial grading should consist of the removal and recompaction of all existing undocumented fill, colluvium, and the near-surface weathered Formation, where these earth materials are located within the influence of the proposed development. Based on the available data, the depth of remedial grading excavations are generally anticipated to range between approximately 2 feet and 17 feet below the existing grades. However, locally deeper remedial grading excavations cannot entirely be precluded and should be anticipated.

- Assuming the use of standard, heavy earth-moving equipment with ripper attachments in good working condition, it is our opinion that the sedimentary bedrock will not pose significant constraints to the planned excavations currently shown in Advanced Civil Engineering ([ACE] 2018). However, the localized use of rock breaking equipment cannot entirely be precluded. Non-productive trenching into the unweathered sedimentary bedrock could be realized, especially if relatively lightweight trenching equipment such as mini-excavators and/or rubber-tire backhoes are used. If more information regarding rock hardness/excavation feasibility is necessary, this office can perform seismic refraction (rock hardness) surveys upon request. These surveys would be best performed once improvement plans showing the planned cuts for underground utility installation have been prepared.

- Expansion index testing on a representative samples conducted by GSI and V&ME(2004) indicates an expansion index (E.I.) of 25 to 132. This correlates from low to very high expansion potential. Atterberg limits testing performed on a representative samples conducted by GSI and V&ME(2004) indicates a plasticity index (P.I.) of 3 to 49. In summary, our observations and laboratory testing indicate that soils with low to very high expansion potentials occur at the subject site, and that some of the onsite earth materials meet the criteria of expansive soils, as defined in Section 1803.5.3 of the 2019 CBC (CBSC, 2019]). As required in Section 1808.6 of the 2019 CBC, foundations for buildings and structures founded on expansive soils necessitate structural design to mitigate their adverse shrink/swell effects. Alternatively, Section 1808.6 of the 2019 CBC stipulates that expansive soils within the influence of the building foundations will need to be removed and replaced with very low expansive soils (E.I. $\leq$ 20 and P.I. $\leq$ 14), or stabilized in place. Preliminary geotechnical recommendations for the design of post-tensioned foundation and mat foundation systems are provided herein for expansive soil mitigation.

- Laboratory testing indicates that the tested sample of the onsite soils is strongly acidic with respect to soil acidity/alkalinity; is severely corrosive to exposed, buried metals when saturated; presents negligible sulfate exposure to concrete (i.e., Exposure class S0, C1 and W0, per Table 19.3.1.1 of American Concrete Institute [ACI] 318-14); and contains low concentrations of soluble chlorides. It should be noted that 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 the Project Architect, Civil Engineer, and/or Structural Engineer.

- It should be noted that the 2019 CBC (CBSC, 2019a) indicates that for fill placed under the purview of the grading permit, removals of unsuitable soils shall be performed across all areas to be graded, not just within the influence of the proposed residential structures. 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 removals, if removals cannot be performed onsite or offsite to mitigate site perimeter conditions or where limited by existing underground utilities. Thus, any settlement-sensitive improvements (walls, curbs, flatwork, etc.), constructed within this zone, may require deepened foundations, reinforcement, etc., or will retain some potential for settlement and associated distress. Based on the available subsurface data, the approximate width of this zone ranges between approximately 1 and 17 feet, depending on the particular location within the site, but may be wider locally if the depth of remedial grading excavations in the vicinity of property lines and existing improvements that are to remain in service exceeds the anticipated removal depths stated herein. In order to further evaluate perimeter conditions, it would be beneficial for GSI to review the as-built plans for the existing sewer main that crosses the property.

- In general and based upon the available data to date, the regional groundwater table is not expected to be a major factor in site development. The potential for perched water conditions to manifest along zones of contrasting permeabilities/densities (i.e., along fill lifts, fill/sedimentary bedrock contacts, etc.) and along geologic discontinuities during and following site development cannot be precluded and should be anticipated.

- On a preliminary basis, temporary slopes should be constructed in accordance with CAL-OSHA guidelines for Type "B" soils. All temporary slopes should be evaluated by the geotechnical consultant, prior to worker entry. Should adverse conditions be identified, the slope may need to be laid back to a flatter gradient or require the use of shoring.

- Our testing regarding stormwater infiltration feasibility indicates that "No infiltration" of stormwater into soils within the vicinity of the proposed permanent stormwater BMP is recommended.

- The seismicity-acceleration values provided herein should be considered during the design and construction of the proposed development.

- General Earthwork and Grading Guidelines are provided at the end of this report as Appendix F. Specific recommendations are provided in the following section.

EARTHWORK CONSTRUCTION RECOMMENDATIONS

General

Remedial earthwork will likely be necessary for the support of the proposed settlement-sensitive improvements. Remedial grading should conform to the guidelines presented in Appendix J of the 2019 CBC (CBSC, 2019a), the requirements of the City of Carlsbad, and the Grading Guidelines presented in Appendix F, except where specifically superceded in the text of this report. In case of conflict, the more onerous code or recommendations should govern. Prior to grading, a GSI representative should be present at the pre-construction meeting to provide additional grading guidelines, if needed, and review the earthwork schedule.

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 office and, if warranted, modified and/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 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.

Demolition/Grubbing

1. Vegetation should be removed from the areas of proposed grading/earthwork.

2. Cavities or loose soils remaining after clearing and grubbing should be cleaned out and observed by the geotechnical consultant. The cavities should be backfilled with fill materials that have been moisture conditioned to at least optimum moisture content and compacted to at least 90 percent of the laboratory standard (ASTM D 1557).

3. Although not anticipated, any septic systems encountered during remedial grading should be observed by the geotechnical consultant. Recommendations for remedial treatment should be provided by the geotechnical consultant based on the type of system observed.

4. Although not anticipated, the abandonment of any existing water wells encountered during remedial earthwork should be performed in accordance with County of San Diego Department of Environmental Health and State of California guidelines.

5. Abandoned existing underground utilities should be removed as part of the remedial earthwork recommended herein.

Removal and Recompaction of Potentially Compressible Earth Materials

Potentially compressible undocumented artificial fill, colluvium (topsoil), and any weathered sedimentary bedrock should be removed to exposed unweathered Sedimentary bedrock. Following removal, these soils should be cleaned of any vegetation and deleterious debris, moisture conditioned to at least 1.1 to 1.3 times the soil's optimum moisture content, and then be recompacted to at least 90 percent of the laboratory standard (per ASTM D 1557). Based on the available subsurface data, remedial grading excavations for the removal of unsuitable soils are anticipated to range between approximately 2½ feet and 17 feet below the existing grades. The potential to encounter localized thicker sections of unsuitable soils that require deeper remedial grading excavations, than that stated above, cannot be precluded and should be anticipated. Potentially compressible soils should be removed below a 1:1 (h:v) plane projected down from the bottom, outboard edge of any settlement-sensitive improvement or limits of planned fill, provided that the remedial excavation does not conflict with property lines and/or existing improvements that need to remain in service. The use of shoring and/or alternating slot excavations may be necessary to protect adjacent property and existing improvements during remedial earthwork. Remedial grading excavations should be observed by the geotechnical consultant prior to scarification and fill placement. Once observed and approved, the bottom of the remedial grading excavation should be scarified at least 6 to 8 inches, moisture conditioned to at least the soil's optimum moisture content, and then recompacted to a minimum 90 percent of the laboratory standard (ASTM D 1557). Remedial grading excavations adjacent to existing underground utilities that need to remain in service should not extend below a 1:1 (h:v) plane projected down from the ground surface at a point minimally located 5 horizontal feet from the center of the top of the underground utility pipe without the use of shoring and/or alternating slot excavations. Underground utility companies/agencies may have stricter requirements than those recommended above. This condition (limited removals) should be shown on the plans.

Alternating Slot Excavations

Alternating (A, B, and C) slot excavations may be performed as an alternative to shoring when conducting remedial earthwork adjacent to property lines or existing improvements that need to remain in service. Slot excavations should be a maximum of 6 feet in width. Multiple slots may be simultaneously excavated provided that open slots are separated by engineered fill or undisturbed soils that are at least 12 feet wide.

Overexcavation

In order to provide uniform support of shallow foundations, any suitable, unweathered sedimentary bedrock exposed within 24 inches below the lowest building foundation element, should be overexcavated and replaced with compacted fill. The maximum to minimum fill thickness beneath the proposed residential structures should not exceed a ratio of 3:1 (maximum:minimum). Overexcavation should be completed to at least 5 feet outside the perimeter foundation elements or below a 1:1 (h:v) plane projected down from the bottom, outboard edge of the perimeter foundation of the proposed residential structures, whichever is greater. The bottoms of the overexcavations should be sloped to the street/driveway. Once completed the overexcavation bottom should be observed by the geotechnical consultant, and then be scarified at least 6 to 8 inches, moisture conditioned to at least 1.1 to 1.3 times the soil's optimum moisture content, and recompacted to a minimum 90 percent of the laboratory standard (ASTM D 1557).

In order to help facilitate trenching for underground utilities and retaining wall foundations, the owner/developer may consider overexcavating the sedimentary bedrock to at least 1 foot below the lowest foundation element for retaining walls (including shear keys) and below the lowest underground utility invert elevation during grading, and then backfilling the resultant excavation with fill materials that have been moisture conditioned to at least 1.1 to 1.3 times the soil's optimum moisture content, and recompacted to a minimum 90 percent of the laboratory standard (ASTM D 1557). If performed, overexcavation should be completed for a minimum horizontal distance of 5 feet outside the perimeter of the retaining wall footings (in all directions) and to at least 2 feet outside retaining wall foundations (in all directions). It is not a geotechnical requirement for the owner/developer to perform overexcavation to assist with trenching operations.

Fill Placement

Following scarification of the bottom of the remedial grading excavation or overexcavation, the reused onsite soils and import (if necessary) should be placed in ±6- to ±8-inch lifts, cleaned of vegetation and debris, moisture conditioned to at least 1.1 to 1.3 times the soil's optimum moisture content, and compacted to achieve a minimum relative compaction of 90 percent of the laboratory standard (ASTM D 1557). Keying and benching should be performed when placing fills on prepared surfaces with gradients steeper than 5:1 (h:v).

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 five business days of lead time should be allowed by builders or contractors for proposed import submittals. This lead time will allow for environmental document review, partie size analysis, laboratory standard, expansion testing, and blended import/native characteristics as deemed necessary. At a minimum, import soils should have an E.I. of $<$ 20 and a P.I. $\leq$ 14. The use of subdrains at the bottom of the fill cap may be necessary, and may be subsequently recommended based on compatibility with onsite soils.

Earthwork Mitigation of Expansive Soils

Earthwork can be performed to mitigate the shrink/swell effects of the onsite expansive soils. This would include the removal and replacement of expansive soils (i.e., soils with an E.I. $\geq$ 21 and P.I. $\geq$ 15) with compacted fill comprised of onsite and/or import soils with an E.I. $\leq$ 20 and P.I. $\leq$ 14), such that the weighted P.I. of the upper 15 feet of soil below pad grade is $\leq$ 14. Alternatively, the expansive onsite soils could be treated with lime or Portland cement.

Graded Slope Construction

General

Based on our review of ACE (2018), graded slopes are not proposed. However, the following recommendations for graded slope construction have been provided in case they become a part of the civil site design, as the project evolves toward final engineering.

Cut Slopes

Graded 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 undocumented fill, colluvium, or highly weathered/fractured sedimentary bedrock) and/or adverse structures (i.e., daylighted joint and/or fracture planes). Should adverse conditions be exposed, 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 detail showing recommended stabilization fills is provided in Appendix F. Although not anticipated at this time, any planned graded cut slope greater than 15 feet in gross overall height should be further evaluated prior to construction. Cut slopes should not be constructed at gradients steeper than 2:1 (h:v) without further evaluation, and prior approval from the City of Carlsbad and the geotechnical consultant.

Fill Slopes

Graded fill slopes should be properly keyed and benched, and be compacted to at least 90 percent relative compaction 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. Keyway excavations should be a minimum of 15 feet wide and should extend at least 2 feet into suitable, unweathered sedimentary bedrock at the toe. The bottom of keyway excavations should be sloped at a minimum of 2 percent from the toe to the heel. Benching should be performed on all surfaces 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 City of Carlsbad. Geogrid reinforcements are typically required for the stability of fill slopes with gradients steeper than 2:1 (h:v).

Fill-Over-Cut Slopes

Prior to the construction of fill-over-cut slopes, unsuitable earth materials should be removed to expose unweathered sedimentary bedrock. A keyway should then be constructed in accordance with the recommendations contained in the preceding section. However, the 2-foot embedment into bedrock at the toe of the keyway is not required. Benching should be performed on all surfaces steeper than 5:1 (h:v), prior to fill placement. The slope should then be constructed at gradients no steeper than 2:1 (h:v). There is a potential for perched water to accumulate along the contact between the fill portion and cut portion of fill-over-cut slopes. This may be mitigated by the installation of subdrains at the heel of the keyway or by reconstructing the entire slope as a stability fill (see Appendix F).

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 City-approved erosion control devices.

- 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 sedimentary bedrock may be problematic. Soil amendments may be necessary.

- Surface drainage should be directed away from the tops of graded slopes. Conveyance of surface runoff along the toe of slopes should be avoided or transported in 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. 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 greater than 4 feet, but less than 20 feet in overall height should conform to CAL-OSHA and/or OSHA requirements for Type "B" soils, provided groundwater or seepage and/or running sands are <u>not</u> present. Temporary slopes, up to a maximum height of ±20 feet, may be excavated at a 1:1 (h:v) gradient, or flatter, provided groundwater, seepage, and/or running sands are <u>not</u> exposed. Heavy equipment storage or traffic nor construction materials/soil stockpiles should not occur within 'H' of any temporary slope, where 'H' equals the height of the temporary slope. All temporary slopes should be observed by a licensed engineering geologist and/or geotechnical engineer prior to worker entry into the excavation. Based on the exposed field conditions, inclining temporary slopes to flatter gradients or the use of shoring may be necessary if adverse conditions are observed. If adverse conditions are exposed or if temporary slopes conflict with property boundaries, or existing improvements that need to remain in service, shoring or alternating slot excavations may be necessary. The need for shoring or alternating slot excavations could be further evaluated during the grading plan review stage and during site earthwork.

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 construction 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 taken which could possibly prevent serious damage to existing improvements. Second, the responsibility for damage to the existing improvement can be determined 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 type of the 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 Engineer-of-Record, prior to excavation.

Reference points on existing walls, buildings, and other settlement-sensitive improvements. These points should be placed as low as possible on the wall and building 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 final 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 reading should be plotted by the Surveyor and then reviewed by the Geotechnical Engineer. In addition to the monitoring system, it would be prudent for the Geotechnical Engineer and the Contractor to make a complete inspection of the existing structures both before and after construction. The inspection should be directed toward detecting any signs of damage, particularly those caused by settlement. Pre-construction notes should be made and photographs should be taken where necessary.

Observation

It is recommended that all excavations be observed by the Geologist and/or Geotechnical Engineer. Any fill which is placed should be approved, tested, and verified if used for engineered purposes. Should the observation reveal any unforseen hazard, the Geologist or Geotechnical 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 may be approximated by using the following parameters:

Undocumented Fill and Colluvium . . . . 5% to 15% shrinkage
Weathered MetaSedimentary and MetaVolcanic Rock
- 75% Soil/25% Rock . . . . 5% to 11% shrinkage
- 50% Soil/50% Rock . . . . 2% to 8% shrinkage
- 75% Rock/25% Soil . . . . 7% to 17% bulking

It should be noted that the above factors are estimates only, based on preliminary data. The undocumented fill, colluvium, and weathered and unweathered sedimentary bedrock 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 92 percent of the laboratory standard (ASTM D 1557). In addition, extensive rodent burrowing may result in higher shrinkage values. 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 - FOUNDATIONS

General

Preliminary recommendations for foundation design and construction are provided in the following sections. These preliminary recommendations have been developed from our understanding of the currently proposed site development, 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. In the event that the information concerning the proposed development plan is not correct, or any changes in the design, location or loading conditions of the proposed residential structures 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.

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 highly expansive soils (E.I. $\geq$ 91). In the following sections, GSI provides preliminary design and construction recommendations for foundations and slab-on-grade floor systems underlain by this type of soil condition. Footings for the proposed residential structures should be founded into approved engineered fill, observed and tested by this office, which is directly underlain by suitable, unweathered sedimentary bedrock.

Preliminary Foundation Design - Conventional Shallow Spread Footings and Slab-on-Grade Floor Systems

The following recommendations may be used for the preliminary design of conventional shallow spread footings and slab-on-grade floor systems if the weighted P.I. of the upper 15 feet of soils below pad grade is less than or equal to 14.

1. The foundation systems should be designed and constructed in accordance with guidelines presented in the 2019 CBC.

2. An allowable bearing value of 2,000 pounds per square foot (psf) may be used for the design of continuous spread footings that maintain a minimum width of 12 inches and a minimum depth of 12 inches (below the lowest adjacent grade), into approved engineered fill overlying dense, unweathered sedimentary bedrock. A similar bearing value may be used in the design of isolated spread footings that have a minimum dimension of at least 24 inches square and a minimum embedment of 24 inches below the lowest adjacent grade, into approved engineered fill overlying dense, unweathered sedimentary bedrock. Foundation embedment depth excludes concrete slabs-on-grade, and/or slab underlayment. The bearing value may be increased by 20 percent for each additional 12 inches in footing depth to a maximum value of 2,500 psf for footings founded into approved engineered fill overlying dense, unweathered sedimentary bedrock. The bearing value may be increased by one-third when considering short duration seismic or wind loads.

3. For foundations deriving passive resistance from approved very low expansive engineered fill, a passive earth pressure may be computed as an equivalent fluid having a density of 250 pcf, with a maximum earth pressure of 2,500 psf.

4. The upper 6 inches of passive pressure should be neglected if not confined by slabs or pavement.

5. For lateral sliding resistance, a 0.35 coefficient of friction may be utilized for a concrete to soil contact when multiplied by the dead load.

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

7. All 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 face of descending slopes.

8. Footings for structures adjacent to retaining walls should be deepened so as to extend below a 1:1 projection up from the heel of the wall footing. This includes structures adjacent to the existing, offsite retaining walls.

PRELIMINARY CONVENTIONAL FOUNDATION CONSTRUCTION RECOMMENDATIONS

Conventional Foundation and Slab-On-Grade Floor Systems

The following recommendations are intended to support foundations and slab-on-grade floor systems underlain by approved engineered fill overlying dense, unweathered sedimentary bedrock, where the weighted P.I. in the upper 15 feet of pad grade is 14 or less.

1. Exterior and interior continuous footings should be founded into approved very low expansive engineered fill overlying dense, unweathered sedimentary bedrock at a minimum depth of 12 or 18 inches below the lowest adjacent grade for one- or two-story floor loads, respectively. For one- or two-story floor loads, continuous footing widths should be 12 or 15 inches, respectively. Isolated, column, panel pad, or retaining wall footings, should be at least 24 inches square, and be founded at a minimum depth of 24 inches below the lowest adjacent grade into approved very low expansive engineered fill overlying dense, unweathered sedimentary bedrock. All footings should be minimally reinforced with four No. 4 reinforcing bars, two placed near the top and two placed near the bottom of the footing.

2. All interior and exterior isolated column footings should be tied to the perimeter foundation via a reinforced grade beam in at least two directions. The grade beam should be at least 12 inches square in cross section, and should be provided with a minimum of one No. 4 reinforcing bar at the top, and one No. 4 reinforcing bar at the bottom of the grade beam. The base of the reinforced grade beam should be

at the same elevation as the adjoining footings. This may require the use of a stepped grade beam if there are differences in the bearing elevations.

3. A grade beam, reinforced as previously recommended and at least 12 inches square, should be provided across large (garage) entrances. The base of the reinforced grade beam should be at the same elevation as the adjoining footings.

4. A minimum concrete slab-on-grade thickness of 4½ inches is recommended. This includes garage slabs-on-grade.

5. Concrete slabs should be reinforced with a minimum of No. 3 reinforcement bars placed at 18 inches on center, in two horizontally perpendicular directions (i.e., long axis and short axis).

6. All slab reinforcement should be supported to ensure proper mid-slab height positioning during placement of the concrete. "Hooking" of reinforcement is not an acceptable method of positioning.

7. Slab subgrade pre-soaking is not required for very low expansive soil conditions. However, the client/Developer should consider pre-wetting the slab subgrade materials to at least the soil's optimum moisture content to a minimum depth of 12 inches, prior to the placement of the underlayment sand and vapor retarder.

8. Soils generated from footing excavations to be used onsite should be compacted to a minimum relative compaction of 90 percent of the laboratory standard (ASTM D 1557), whether the soils are to be placed inside the foundation perimeter or in the yard/right-of-way areas. This material must not alter positive drainage patterns that direct drainage away from the structural areas and toward the street.

9. Reinforced concrete mix design should be provided by the Project Structural Engineer, and should consider the corrosion potential of the onsite soils and the maximum water to cement ratio recommended herein to reduce moisture/vapor transmission through the foundations and floor slabs.

POST-TENSION FOUNDATION SYSTEMS

Post-tension foundation systems may be used to mitigate the damaging shrink/swell effects of the onsite expansive soil conditions on the proposed building foundations and slab-on-grade floors. However, the use of post-tension foundation systems may be limited due to the size of the proposed building footprints and/or the incorporation of retaining walls for some building walls.

The post-tension foundation designer may elect to exceed the minimal recommendations, provided herein, to increase slab stiffness performance. Post-tension (PT) design may be either ribbed or mat-type. The latter is also referred to as uniform thickness foundation (UTF). The use of a UTF is an alternative to the traditional ribbed-type. The UTF offers a reduction in grade beams. That is to say a UTF typically uses a single perimeter grade beam and “shovel” footings, but has a thicker slab than the ribbed-type. UTF perimeter footings may utilize an allowable vertical bearing value of 1,500 psf if founded into approved engineered fill overlying dense, unweathered bedrock.

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 design. Post-tension foundations should be designed using sound engineering practice and be in accordance with local and 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 related to post-tension foundation design.

From a soil expansion/shrinkage standpoint, a common contributing factor to distress of structures using post-tensioned slabs 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 leads to what is commonly referred to as the center lift condition. A well-designed, post-tensioned slab 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 slabs 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-tension slab construction. This effectively reduces soil moisture migration from the area located outside the building toward the soils underlying the post-tension slab. Perimeter cut-off walls are thickened edges of the concrete slab that impedes 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 equal to or 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 for low, medium, or highly expansive soil conditions, respectively).

Pre-moistening and/or pre-soaking should be evaluated by the soils engineer 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 soil moisture 2 percent over optimum (or 1.2 times)	Wetting and/or reprocessing	Periodically wet or cover with plastic after trenching. Evaluation 72 hours prior to placement of concrete.
Medium (E.I. = 51-90)	Upper 18 inches of pad soil moisture 2 percent over optimum (or 1.2 times)	Berm and flood or wetting and reprocessing	Periodically wet or cover with plastic after trenching. Evaluation 72 hours prior to placement of concrete.
High to Very High (E.I. = 91-130+)	Upper 24 inches of pad soil moisture 3 percent over optimum (or 1.3 times)	Berm and flood or wetting and reprocessing	Periodically wet or cover with plastic after trenching. Evaluation 72 hours prior to placement of concrete.

Perimeter Cut-Off Walls

Perimeter cut-off walls should be at least 12, 18, or 24 inches deep for low, medium, or highly expansive soil conditions, respectively. The cut-off walls may be integrated into the slab design or independent of the slab. 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 reinforcement per the structural engineer.

Post-Tension Foundation Design

The following recommendations for design of post-tensioned slabs have been prepared in general compliance 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-Tension 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.)*	<15-50

* - The weighted plasticity index should be evaluated for the upper 13 feet of foundation soils either during or following grading.
** - The onsite PI values obtained from lab testing is 49.

Based on the above, the recommended post-tension soil support parameters are tabulated below:

TABLE 1 - POST-TENSION FOUNDATION DESIGN

DESIGN PARAMETER[4]	EXPANSION POTENTIAL			
	CATEGORY I VERY LOW TO LOW[5] (E.I. 0-50)	CATEGORY II MEDIUM[5] (E.I. 51-90)	CATEGORY III HIGH[5] (E.I. 91-130)	CATEGORY IV VERY HIGH (E.I. >130)
e_m center lift	9.0 feet	8.7 feet	8.5	7.5 feet
e_m edge lift	5.0 feet	4.5 feet	4.0	3.25 feet
y_m center lift	0.4 inches	0.50 inches	0.66 inches	1.1 inches
y_m edge lift	0.7 inch	1.3 inch	1.7 inch	2.5 inches
Bearing Value [1]	1,500 psf [1]	1,000 psf [2]	1,000 psf	1,000 psf
Lateral Pressure	250 psf	250 psf	250 psf	250 psf
Subgrade Modulus (k)	100 pci/inch	85 pci/inch	70 pci/inch	60 pci/inch
Minimum Perimeter Footing Embedment [3]	12 inches	18 inches	24 inches	30 inches

[1] Internal bearing values within the perimeter of the post-tension slab for very low to low expansive soil conditions may be increased to 2,000 psf for a minimum embedment of 12 inches, then by 20 percent for each additional foot of embedment to a maximum of 2,500 psf.
[2] For medium to very high expansive soil conditions, internal bearing values within the perimeter of the post-tension slab may be increased to 1,500 psf for a minimum embedment of 12 inches, then by 20 percent for each additional foot of embedment to a maximum of 2,000 psf.
[3] As measured below the lowest adjacent compacted subgrade surface (not including slab underlayment layer thickness).
[4] Post-tension slab design should also be evaluated with respect to the potential differential settlements provided in this report.
[5] Category Criteria:
Category I Expansion Index < 50 (very low to low), and/or Max fill less than 25 feet thick, or fill differential less than 10 feet.
Category II Expansion Index 51-90 (Medium), and/or max fill less 25 feet thick, or fill differential less than 20 feet.
Category III Expansion Index 90-130 (High), and/or fill greater than 25 feet thick, or differential fill thickness greater than 20 feet.
Category IV Expansion Index 130+ (Very High), fill greater than 50 feet thick.

The parameters are considered minimums and may not be adequate to represent all expansive soils and site conditions such as adverse drainage and/or improper landscaping and maintenance. The above parameters are applicable provided the grades around the structure provide positive drainage that is maintained away from the building foundation. In addition, no trees with significant root systems are to be planted within 15 feet of the perimeter of foundations. Therefore, it is important that information regarding drainage, site maintenance, trees, settlements, and effects of expansive soils be passed on to future 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 supported by the post-tensioned foundation should be supported by 24 square inch isolated footings extending at least 24 inches into approved engineered fill. Exterior column footings should be tied to the post-tensioned foundation with 12 square inch, reinforced grade beams in at least two directions.

MAT FOUNDATIONS

In lieu of using a post-tension foundation to tolerate expansive soil effects, the client may consider a mat foundation which uses steel bar reinforcement instead of post-tensioned cables. The structural engineer may supercede the following recommendations based on the planned building loads and use. WRI (Wire Reinforcement Institute) methodologies for design may be used. Mat foundations may be incorporate exterior and interior stiffening beams or a uniform thickness slab (UTF). Minimum mat embedment should be 12 inches below the lowest adjacent grade into approved engineered fill overlying dense, unweathered sedimentary bedrock, if a uniform thickness slab is utilized.

Mat Foundation Design

The design of mat foundations should incorporate the vertical modulus of subgrade reaction. This value 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. This assumes that the bearing soils will consist of at least 2 feet of newly compacted fills with an average relative compaction of 90 percent of the laboratory (ASTM D 1557).

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

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

The modulus of subgrade reaction (K_S) and effective plasticity index (P.I.) to be used in mat 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 weighted plasticity index for the upper 15 feet of the foundation soils should be performed during or following grading. Lateral pressures and sliding coefficients for mat foundation design should conform to those previously provided in the "Preliminary Foundation Design - Conventional Shallow Spread Footings and Slab-on-Grade Floor Systems" section of this report.

Slab Subgrade Pre-Soaking

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

Perimeter Cut-Off Walls

Perimeter cut-off walls should be at least 12, 18, or 24 inches deep for low, medium, or highly expansive soil conditions, respectively. The cut-off walls may be integrated into the slab design or independent of the slab. 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 steel reinforcement per the structural engineer.

FOUNDATION SETTLEMENT

Provided that the earthwork and foundation recommendations in this report are adhered, foundations bearing on approved engineered fill overlying dense, unweathered sedimentary bedrock should be minimally designed to accommodate a total settlement of 1 inch and a differential settlement of 1 inch over a 40-foot horizontal span (angular distortion = 1/480).

SOIL MOISTURE CONSIDERATIONS

GSI has evaluated the potential for vapor or water transmission through the concrete floor slab, in light of typical floor coverings and improvements. Please note that 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 water or vapor through the foundation or slabs. Foundation systems and slabs shall not allow water 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, 2020). These recommendations may be exceeded or supplemented by a water "proofing" specialist, project architect, or 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 vapor transmission will occur in new slab-on-grade floors as a result of chemical reactions taking place within the curing concrete. 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. It is possible that a slab moisture 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 water vapor transmission rates, floor coverings, and improvements (to be chosen by the client and/or project architect) that can tolerate vapor transmission rates without significant distress, the following alternatives are provided:

1. Concrete slab-on-grade floor thicknesses, including garage slabs, should be increased.

2. Concrete slab underlayment should consist of a 15-mil vapor retarder, or equivalent, with all laps sealed per the 2019 CBC and the manufacturer's recommendation. The vapor retarder should comply with the ASTM E 1745 - class A criteria, and be installed in accordance with ACI 302.1R-04 and ASTM E 1643.

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

4. Concrete slab-on-grade floors, including garage slabs, should be directly underlain by 2 inches of ean, washed sand (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-04 (2004) states "If a cushion or sand layer is desired between the vapor retarder and the slab, care must be taken to protect the sand layer from taking on additional water from a source such as rain, curing, cutting, or cleaning. Wet cushion or sand layer has been directly linked in the past to significant lengthening of time required for a slab to reach an acceptable level of dryness for floor covering applications." Therefore, additional observation and/or testing will be necessary for the cushion or sand layer for moisture content, and relatively uniform thicknesses, prior to the placement of concrete.

5. For building pads with very low expansive soil conditions E.I. $\leq$ 20 and P.I. $\leq$ 14, the vapor retarder should be underlain by 2 inches of sand (SE $\geq$ 30) placed directly on the prepared, moisture conditioned, subgrade and should be sealed to provide a continuous retarder under the entire slab, as discussed above. This layer of import sand may be eliminated below the vapor retarder, if laboratory testing indicates that the slab subgrade soil has a sand equivalent (SE) of 30 or greater. For building pads with an E.I. $\geq$ 21 and P.I. $\geq$ 15, the vapor retarder shall be underlain by a capillary break consisting of at least 4 inches of clean crushed gravel with a maximum dimension of ¾ inch (less than 5 percent passing the No. 200 sieve).

6. Concrete with a maximum water to cement ratio of 0.50 should be used in foundation and slab-on-grade floor construction.

7. Where slab water/cement ratios are as indicated herein, and/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 slabs are designed and/or treated for more uniform moisture protection.

8. The developer and homeowners should be specifically advised which areas are suitable for tile flooring, vinyl flooring, or other types of water/vapor-sensitive flooring and which are not suitable. In all planned floor areas, flooring shall be installed per the manufactures recommendations.

9. Additional recommendations regarding water or 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/moisture vapor transmission through the slab 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 water-proofing 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 foundations or improvements. The vapor retarder contractor should have representatives onsite during the initial installation.

Corrosion and Concrete Mix

Upon completion of grading, laboratory testing should be performed of earth materials near finish grade for corrosion to concrete and reinforcing steel.

PRELIMINARY RETAINING WALL DESIGN PARAMETERS

Conventional Retaining Walls

The design parameters provided below assume that either select materials (typically class 2 permeable filter material or class 3 aggregate base) or native onsite materials with an E.I. $\leq$ 20 and P.I. $\leq$ 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 early shown on the wall plans. Compliance testing would be recommended if the onsite soils are intended for use as retaining wall backfill. Retaining walls incorporated into the proposed buildings should be water-proofed. Waterproofing should also be considered for any site retaining wall in order to reduce the potential for efflorescence staining at the face. Recommendations for mechanically stabilized earth (MSE) retaining walls (i.e., crib, earthstone, geogrid, etc.) are also provided herein.

Preliminary Retaining Wall Foundation Design

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

Minimum Footing Embedment - 24 inches below the lowest adjacent grade (exuding any topsoil/colluvium, or landscape layer [upper 6 inches]), into approved, engineered fill overlying dense, unweathered sedimentary bedrock or into dense, unweathered sedimentary bedrock.

Minimum Footing Width - 24 inches.

Allowable Bearing Pressure - An allowable bearing pressure of 2,500 psf may be used in the preliminary design of retaining wall foundations provided that the footing maintains a minimum width of 24 inches and extends at least 24 inches into approved, engineered fill overlying dense, unweathered sedimentary bedrock. For a retaining wall footing of similar width extending at least 24 inches into dense, unweathered sedimentary bedrock, an allowable bearing pressure of 3,000 psf may be used. The allowable bearing pressures may be increased by one-third for short-term wind and/or seismic loads.

Passive Earth Pressure - A passive earth pressure of 250 pcf with a maximum earth pressure of 2,500 psf may be used in the preliminary design of retaining wall foundations provided the foundation is embedded into properly compacted very low expansive fill (E.I. $\leq$ 20 and P.I. $\leq$ 14). For retaining wall footings embedded into dense, unweathered, very low expansive sedimentary bedrock (E.I. $\leq$ 20 and P.I. $\leq$ 14), a passive earth pressure of 300 pcf with a maximum earth pressure of 3,000 psf may be used in the preliminary design of retaining wall foundations. The passive earth pressure should be reduced to 150 pcf if the retaining wall footing is founded into expansive engineered fills or expansive sedimentary bedrock (E.I. $\geq$ 21 and P.I. $\geq$ 15).

Lateral Sliding Resistance - A 0.35 coefficient of friction may be utilized for a concrete to soil contact when multiplied by the dead load. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

Backfill Soil Density - Soil densities ranging between 135 pcf and 140 pcf may be used in the design of retaining wall foundations. This assumes an average engineered fill compaction of at least 90 percent of the laboratory standard (ASTM D 1557).

Any proposed retaining wall footings near the perimeter of the site will likely need to be deepened into unweathered sedimentary bedrock for adequate vertical and lateral bearing support. All 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. Proposed retaining walls should not be located above a 1:1 (h:v) plane projected up from the bottom, inboard edge of the foundation for the offsite retaining walls.

Restrained Walls

Any 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 (level backfill). 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 site retaining walls less than 3 feet in height may be superceded by San Diego Regional Standard Design (SDRSD), provided that select backfill will be used. 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 the back of retaining walls where vehicular traffic could occur within horizontal distance “H” from the back of the retaining wall (where “H” equals the wall 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. For heavy (HS-20) axle loads, a 300 psf/ft traffic surcharge should be applied in the upper 5 feet of the wall. Traffic surcharge does not include the surcharge of parked vehies, 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. (SELECT NATIVE BACKFILL)[3]
Level[1]	38	50
2 to 1	55	65

[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. < 15, E.I. < 21, and $\leq$ 10% passing No. 200 sieve.
[3] E.I. = 0 to 30, SE $\geq$ 20, P.I. < 20, and $\leq$ 20% passing No. 200 sieve; confirmation testing required.

Earthquake Loads (Seismic Surcharge)

For engineered retaining walls with more than 6 feet of retained materials, as measured vertically from the bottom of the wall footing at the heel to daylight, for retaining walls incorporated into buildings, and for retaining walls that could impede ingress/egress for emergency equipment and personnel in the event of failure, GSI recommends that the walls be evaluated for a seismic surcharge (in general accordance with 2019 CBC requirements). The site walls in this category should maintain an overturning Factor-of-Safety (FOS) of approximately 1.25 when the seismic surcharge (increment), is applied. For restrained walls, the seismic surcharge should be applied as a uniform surcharge load from the bottom of the footing (exuding shear keys) to the top of the backfill at the heel of the wall footing. This seismic surcharge pressure (seismic increment) may be taken as 15H where "H" for retained walls is the dimension previously noted as the height of the backfill to the bottom of the footing. The resultant force should be applied at a distance 0.6 H up from the bottom of the footing. 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. For cantilevered walls, the pressure should be applied as an inverted triangular distribution using 15H. For restrained walls, the pressure should be applied as a rectangular distribution. Please note this is for local wall stability only.

The 15H is derived from a Mononobe-Okabe solution for both restrained cantilever walls. This accounts for the increased lateral pressure due to shakedown or movement of the sand fill soil in the zone of influence from the wall or roughly a $45^\circ - \phi/2$ plane away from the back of the wall. The 15H seismic surcharge is derived from the formula:

$P_h = \frac{3}{8} \bullet a_h \bullet \gamma_t H$

Where:	P_h	=	Seismic increment
	a_h	=	Probabilistic horizontal site acceleration with a percentage of "g"
	γ_t	=	total unit weight (135 to 140 pcf for site soils @ 90% relative compaction).
	H	=	Height of the wall from the bottom of the footing or point of pile fixity.

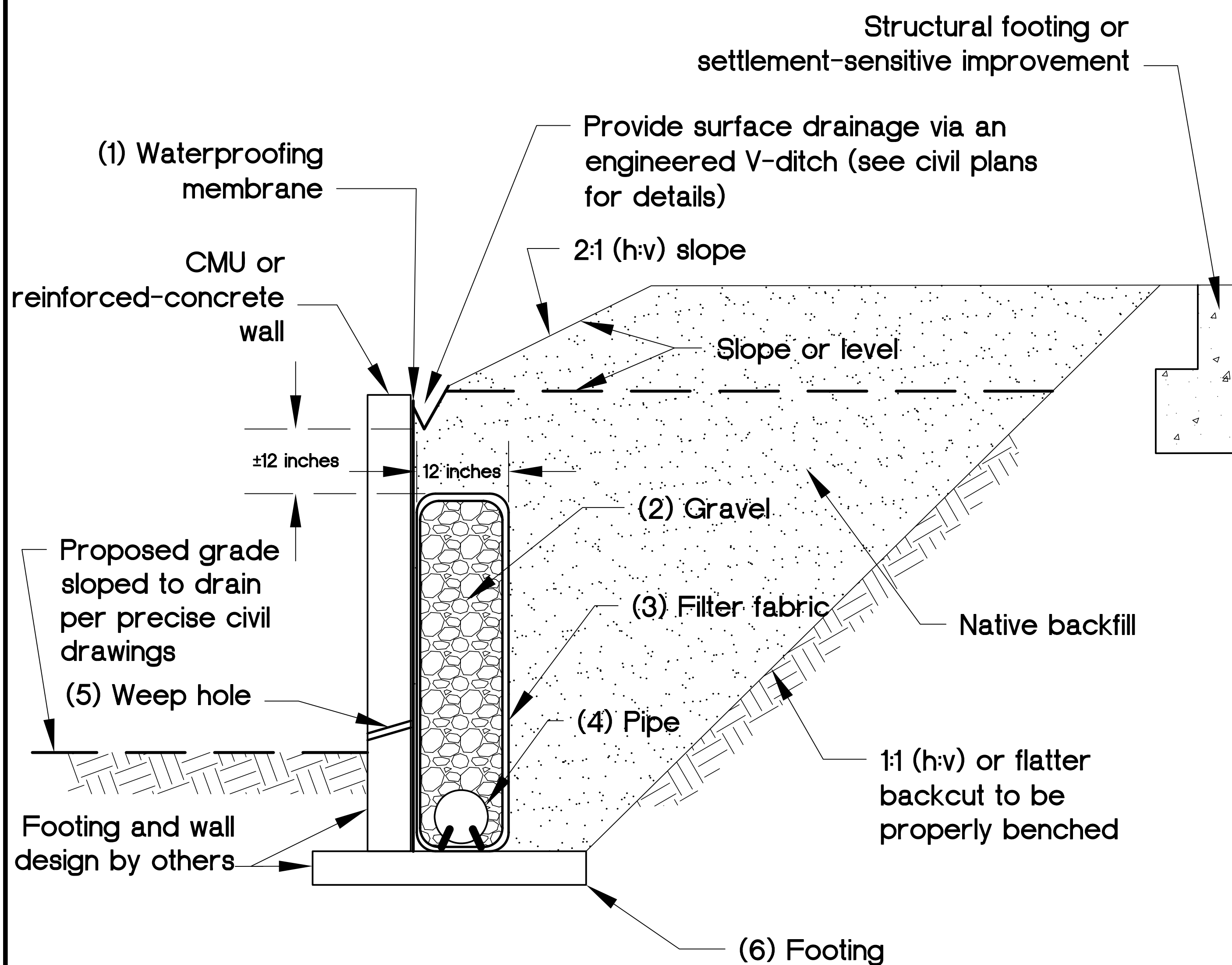
Retaining Wall Backfill and Drainage

Positive drainage must be provided behind all retaining walls in the form of gravel wrapped in geofabric and outlets. 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. Backdrains should consist of a 4-inch diameter perforated PVC or ABS pipe encased in either class 2 permeable filter material or ¾-inch to 1½-inch gravel wrapped in approved filter fabric (Mirafi 140 or equivalent). For low expansive backfill, the filter material should extend a minimum of 1 horizontal foot behind the base of the walls and upward at least 1 foot. This material should be continuous (i.e., full height) behind the wall, and it should be constructed in accordance with the enclosed Detail 1 (Typical Retaining Wall Backfill and Drainage Detail). For limited access and confined areas, (panel) drainage behind the wall may be constructed in accordance with Detail 2 (Retaining Wall Backfill and Subdrain Detail Geotextile Drain). Any materials (if encountered) with an Expansion Index (E.I.) potential of greater than 20 should not be used as backfill for retaining walls. For more onerous expansive situations, backfill and drainage behind the retaining wall should conform with Detail 3 (Retaining Wall And Subdrain Detail Clean Sand Backfill).

Outlets should consist of a 4-inch diameter solid PVC or ABS pipe spaced no greater than ±100 feet apart, with a minimum of two outlets, one on each end. The use of weep holes, only, in walls 2 feet or greater in height, is not recommended. The surface of the backfill should be sealed by pavement or the top 18 inches compacted with native soil (E.I. $\leq$ 50). Proper surface drainage should also be provided. For additional mitigation, consideration should be given to applying a water-proof membrane to the back of all retaining structures. The use of a waterstop should be considered for all concrete and masonry joints.

Wall/Retaining Wall Footing Transitions

Site walls are anticipated to be founded on footings designed in accordance with the recommendations in this report. Should wall footings transition from bedrock to fill, the civil designer may specify either:



(1) Waterproofing membrane.

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

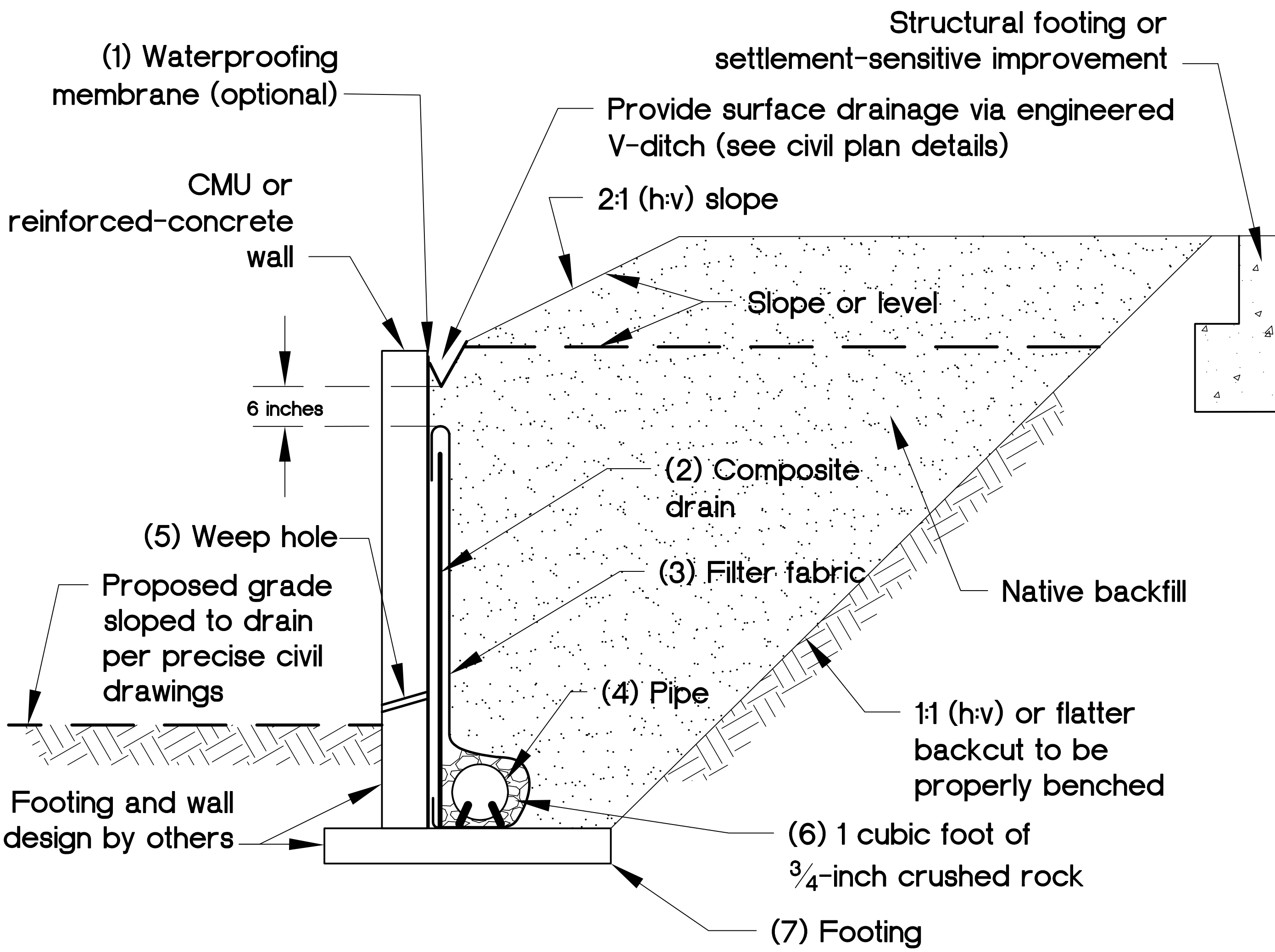
(3) Filter fabric: Mirafi 140N or approved equivalent.

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

(5) Weep hole: Minimum 2-inch diameter placed 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, use 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.

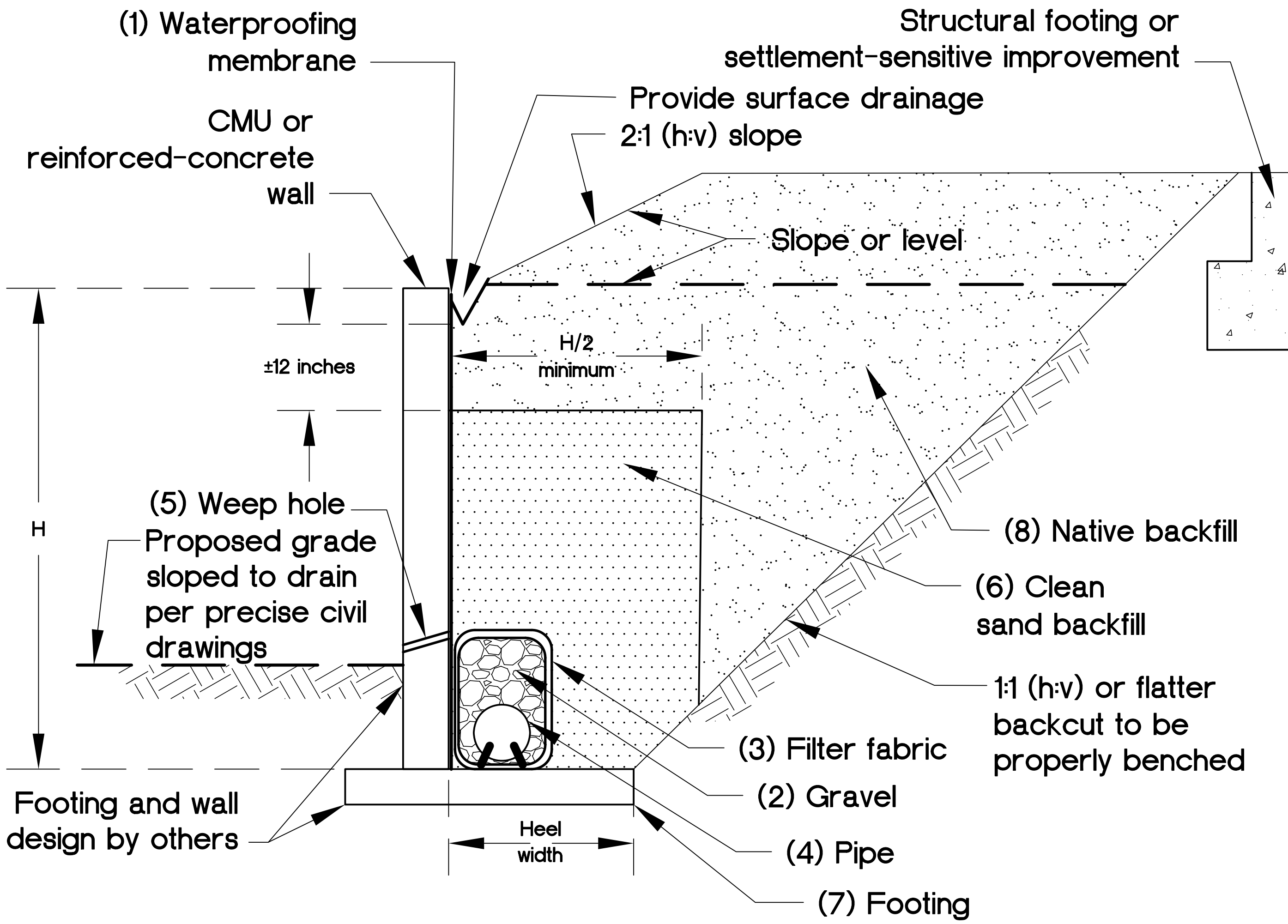
(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

(5) Weep hole: Minimum 2-inch diameter placed 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, $\frac{3}{4}$ to $1\frac{1}{2}$ inch.

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

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



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

(2) Gravel: Clean, crushed, $\frac{3}{4}$ to $1\frac{1}{2}$ 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: Minimum 2-inch diameter placed 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, use 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

a) A minimum of a 2-foot overexcavation and recompaction of cut materials for a distance of 2H, from the point of transition.

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 unweathered sedimentary bedrock material (i.e., deepened footings).

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

MECHANICAL STABILIZED EARTH (MSE) RETAINING WALLS

General

It is our understanding that proposed development will include the construction of an MSE retaining wall to support the driveway "hammerhead" turnaround area, near the northerly property margin. This type of wall improvement is also commonly referred to as a segmental, or geogrid retaining wall in the construction industry. Preliminary geotechnical recommendations for the design and construction of MSE retaining wall systems are provided below.

The recommended design parameters assume that either non-expansive soils (typically class 2 permeable filter material or class 3 aggregate base) or select soil import materials (with an E.I. $\leq$ 20, and a P.I. $\leq$10) are used to backfill any MSE retaining wall. The type of backfill, should be specified by the wall designer, and early shown on the MSE retaining wall plans. Additional evaluations are recommended prior to the release of construction bid documents to evaluate the suitability of the onsite earth materials for use in MSE retaining wall construction. Based on the available subsurface data, most of the onsite earth materials would not be appropriate for MSE retaining wall backfill. If the onsite soils are deemed unsuitable for use in this application, import/export operations would be necessary. For preliminary bidding purposes, GSI recommends that the client request that prospective MSE retaining wall installers provide construction estimates for MSE retaining walls with and without the use of the onsite earth materials in the geogrid-reinforced backfill zone. During the course of remedial and planned grading, should testing demonstrate that exhumed onsite earth are suitable for use in MSE retaining wall construction, they may be stockpiled and later used during the installation of these walls. However, please note that given the size of the site and the footprint of the proposed development, careful planning and construction sequencing will be necessary if the onsite earth materials are to be stockpiled for later use in MSE retaining wall construction.

Guidelines for MSE Retaining Wall Construction

MSE retaining walls are, by nature, a flexible system and, as such, not suited for every slope support condition, which will need to be considered and ultimately determined by the Project Civil Engineer, Project Structural Engineer, and the owner/developer. Slope and structural setbacks from the heel of MSE retaining walls and the heel of the geogrid reinforcements will be necessary, owing to potential deformation/movement. The necessary setbacks should be defined by the various project consultants and approved by the governing agency prior to final design. At a minimum, the building setback should be the area located above a 1:1 (h:v) plane projected up from the heel of the MSE retaining wall as well as the area located above a 1:1 (h:v) plane projected up from heel of the lowermost layer of geogrid reinforcement. The setback zones should be shown on the precise grading plans prepared by the Project Civil Engineer. Building setback mitigation may be accomplished by deepening any foundations within the setback zones such that they extend below these 1:1 (h:v) planes, provided disturbance of any geogrid reinforcement layer is deemed acceptable by the wall designer. GSI does not recommend placing geogrid reinforcement layers beneath foundations or above underground utilities. In addition, building, traffic, and sloping surcharge within the zone of influence for the MSE retaining wall should be included in the walls' internal stability evaluations.

In addition to the previous recommendations, the following are specific recommendations for MSE retaining wall design and construction. These recommendations have been provided in an effort to achieve the most desirable and efficient means of construction. Some of these do not deal specifically with geotechnical aspects, but do have significant effects on the quality of the end product. As the project geotechnical consultant, we feel that strong consideration should be given to these recommendations. If more onerous project specifications are required by the manufacturer or governing agency, then those guidelines should be followed.

Compared to conventional retaining walls, MSE retaining walls require significantly more geotechnical observation and testing. The costs for these services depend on wall size, conscientiousness of the contractor, the number of import sources for wall construction, and other factors. GSI should evaluate the geotechnical aspects of the wall layouts (offsets, cross-section, alignments) prior to construction. This approval by the geotechnical consultant should be sought (in writing) prior to 100 percent submittal by the wall designer.

Foundation

1. Prior to excavation for the wall base, the alignment and grade for the wall should be established in the field by the Project Civil Engineer or Project Surveyor.

2. The contractor should have a qualified grade checker onsite to continually verify the gradient (or batter) and alignment of the base excavation and wall during construction.

3. Defective segments or wall units should not be utilized in MSE retaining wall construction.

4. The Project Surveyor should spot-check wall gradient (face of wall slope) and alignment and using this data, the Civil Engineer/Wall Designer should evaluate if the wall installation is per plan.

5. When locating the base of the wall, structural setbacks established by the City of Carlsbad and/or governing agency, and/or geotechnical engineer should be followed.

6. Prior to MSE retaining wall construction, remedial grading should be performed to treat potentially compressible earth materials within the influence of the wall.

7. Although unlikely, if expansive soils are exposed in the MSE retaining wall foundation, overexcavation and replacement fills will be necessary so the wall is founded on a minimum 2-foot thick layer of very low expansive (E.I,. $\leq$ 20 and P.I. $\leq$ 14 or less) compacted fill overlying the unweathered sedimentary bedrock.

8. The recommended equivalent fluid pressure for the design of MSE retaining walls should be 45 pcf for level backfill and 65 pcf for 2:1 (h:v) backfill, assuming the use of granular backfill material (E.I. $\leq$20, P.I. $\leq$10, $\phi \geq$29 degrees, c $\leq$ 100 psf). These equivalent fluid pressures are based solely on static soil conditions and do not include seismic loading, expansive soil pressures, earthwork surcharge, or traffic loading which will need to be included, as necessary. Additional evaluation would be necessary if the gradient of sloping backfill exceeds 2:1 (h:v).

9. Utilize a seismic increment of 15H when evaluating internal stability of MSE retaining walls. For global stability of segmental retaining walls, a seismic factor (pseudo-static) of 0.15 *i*, should be used. The seismic increment should be applied in an inverted triangular distribution.

10. Wall foundation embedment should conform to NCMA guidelines, but not be less than 1 foot. Where MSE retaining walls are located adjacent to descending slopes, the horizontal setback from the bottom, outboard edge of the basal wall course to daylight should conform to 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 basal wall course to the slope face. Greater setbacks would be recommended if the overall height of the adjacent, descending slope exceeds 21 feet.

11. A bearing value of 1,500 psf may be utilized for a 1 foot deep MSE wall foundation. A friction coefficient of 0.35 may be used for a concrete to very low expansive (E.I. $\leq$ 20 and P.I. $\leq$ 14 or less) soil contact. A minimum friction angle of 28 degrees and a soil total unit weight of 135 to 140 pcf may be utilized for very low expansive (E.I. of 20 or less and P.I. of 14 or less) compacted fill, overlying dense, unweathered sedimentary bedrock, as evaluated by observation and/or testing. In addition, a cohesion value of 0.00 psf or less for reinforced fill, 100 psf for retained fill, and 100 psf for foundation fill may be utilized. For passive resistance of an MSE retaining wall foundation embedded 1 foot into very low expansive (E.I. $\leq$ 20 and P.I. $\leq$ 14) compacted fill, overlying dense, unweathered sedimentary bedrock, the wall designer may use 250 pcf with a maximum earth pressure of 2,500 psf. For an MSE retaining wall founded 1 foot into expansive soils (E.I. $\geq$ 21 and P.I. $\geq$ 15), the wall designer should use a passive resistance of 150 pcf.

12. Prior to placement of the MSE retaining wall units, the excavation for the wall base should be observed by representatives of this firm, and should be a minimum of 12 inches below the lowest adjacent grade.

13. A crushed stone leveling pad may be used to provide a uniform surface for the wall base.

14. If it is necessary to locally deepen the wall base to obtain suitable bearing materials, the contractor should consult the Project Civil Engineer and Wall Designer to determine if the wall location or design of the wall is affected.

15. MSE retaining wall height at the terminal ends of the wall should <u>not</u> exceed 4 feet, unless lateral support is provided.

<u>Backfill</u>

1. Any fill placed within the geogrid reinforcement zone, behind and in front of the MSE retaining walls, should be compacted to a minimum of 90 percent relative compaction (ASTM D 1557) unless otherwise specified by the manufacturer. Any backfill other than the "unit core fill (¾-inch crushed rock or stone)" should be placed in controlled lifts not to exceed 6 inches in thickness, moisture-conditioned as necessary to achieve at least optimum moisture content, and then compacted by mechanical means. Backfill within and immediately behind the walls should also be as indicated on the (precise and rough) grading plans. The Wall Designer should specify setbacks for heavy construction equipment from the back of the MSE retaining wall facing, as necessary.

2. Backfill materials should be free draining, and free from organic materials, with a maximum of 10 percent fines passing the No. 200 sieve. Lifts should be placed horizontally and compaction equipment should not be allowed to damage the

geogrid reinforcements, where utilized. No partie sizes greater than 2 inches should be used in the geogrid-reinforced backfill zone for all MSE retaining walls, unless approved by the Wall Designer.

3. If gravel or other select granular material is used as backfill within or behind the MSE retaining wall, it should be capped with a minimum 18 inches compacted fill with an E.I. of 50 or less or composed of relatively impervious material (i.e., pavements). A layer of filter fabric (Mirafi 140N or approved equivalent) should separate any gravel from the soil cap.

4. During construction, the unfilled section of wall should not be stacked more than 2 feet above the fill behind the wall. If gravel is used to backfill the wall, the wall may be stacked 3 feet above adjacent grades. The maximum gravel size should not exceed ¾-inch. If this option is selected, additional review with respect to drainage and potential for backfill scouring and/or piping at the face of the wall should be performed. Gravel (if used) should be separated from any adjacent soil with filter fabric (Mirafi 140N or approved equivalent).

5. Adequate space should be provided both behind and in front of the wall so that sufficient compaction can be obtained for all backfill. The slope of the MSE retaining wall(s) and benching (in cross-section and alignment) should be in accordance with the manufacturer's recommendations and as approved by the geotechnical consultant.

Wall Back Drains

A drainage system should be installed for all MSE retaining walls in excess of 3 feet in overall height. The design of the system will depend on specific conditions. For most cases, a Schedule 40 (or approved equivalent) perforated drain pipe, encased in clean crushed ½- to ¾-inch gravel, and wrapped in Mirafi 140N filter fabric (or approved equivalent) is recommended. The drain should be placed at the heel of the wall with a full height gravel drain, separated from the backfill materials by Mirafi 140, or equivalent. In areas where bedrock and/or perched groundwater are exposed in the backcut of the geogrid reinforced backfill zone, a secondary backdrain system, of similar construction, should be placed at the toe of the backcut and along zones of perched water seepage. If necessary, outlets may pass below the base of the wall at a minimum 2 percent gradient. Outlets should be tight-lined via a solid drain pipe (Schedule 40 or approved equivalent) that drain toward an approved outlet area in accordance with the design civil engineer recommendations. A concrete cut-off wall should be constructed at the connection between solid and perforated drain pipes to force seepage water into the solid pipe. The cut-off wall should surround the pipe connection and extend at least 12 inches beyond the outer diameter of the pipe (360 degrees). The trenches for the solid drain pipe should be backfilled with either compacted fill material or gravel. If gravel is used, it should be separated from the surrounding soils with Mirafi 140N filter fabric and should be capped with at least 12 inches of compacted fill material. Seepage should be anticipated below all MSE retaining walls, and this should be disclosed to all interested/affected parties.

Materials and Wall Construction

Only sound MSE retaining wall units/members and components that meet all required specifications should be used for construction of the walls. Wall units/members should be free of honeycombing, cracks, broken lugs, or slumped bearing surfaces. All geogrid reinforcement should comply with the required technical specifications for the project. Geogrid reinforcement should be placed horizontally to the required length/width behind the wall. The strong axis of the geogrid reinforcement should be placed perpendicular to the wall alignment if uniaxial geogrid is used.

Structural Setbacks from Proposed MSE Retaining Walls

It is recommended that settlement-sensitive structures be constructed behind a 1:1 (h:v) plane projected up from the heel of the lowermost geogrid reinforcement layer. In addition, all footings should be setback behind a 1:1 (h:v) plane projected up from the heel of the MSE retaining wall. If structures are located between the two 1:1 (h:v) projections, the MSE wall should be designed to accommodate the additional surcharge loading from the structures, and deepened building footings may be required depending on the height of the MSE wall. Alternately, footings may be constructed such that bearing surfaces are below both 1:1 (h:v) planes. All appurtenant structures (i.e., A/C pads, screen walls, light standards, pools, spas, etc.) should be placed behind a 1:1 (h:v) plane projected up from the heel of the wall. In addition, GSI recommends that geogrid reinforcement not be placed beneath spas, pools, or other significant appurtenant structures. Appurtenant structures, including pools, utilities, and landscaping, should not disrupt the geogrid reinforcement behind the walls. This should be disclosed to all interested affected parties. All structures proposed within the setback zone will be subject to both horizontal and vertical deflections and potential distress and/or replacement. All construction proposed within the setback area should be reviewed by the Project Civil Engineer and the geotechnical consultant. This review should be provided in writing to the owner prior to installation in the field.

Review of Segmental Retaining Wall Plans and Structural Calculations

A qualified geotechnical consultant should review the proposed MSE retaining walls for global stability. MSE retaining walls must meet City, local code, and slope stability factors-of-safety of 1.5 and 1.1 for static and seismic conditions, respectively. Criteria for limitations of land use within geogrid reinforced backfill areas should be provided by the wall designer and reviewed by both the owner and the geotechnical consultant. These limitations should be disclosed to all interested/affected parties.

Soil Expansion

The available data indicates that highly expansive soils are present onsite. If necessary, lateral pressures due to expansive soil conditions can be provided on request, <u>or</u> the effect of expansive soil may be minimized by using select, very low expansive backfill within the active zone, behind the wall system. The active zone is generally defined as the area above a 1:1 (h:v) plane projected up and away from the heel of the lowermost layer of geogrid reinforcement for the MSE wall. The actual slope of the projection is based on the friction angle of the soil and the slope of the backfill behind the wall.

Other Considerations

- Surcharge loads (slopes, traffic, etc.) should be applied by the design engineer as necessary. For preliminary planning purposes, the wall designer should incorporate the surcharge of traffic on the back of retaining walls where vehicular traffic could occur within horizontal distance “H” from the back of the retaining wall (where “H” equals the wall height). The traffic surcharge may be taken as 100 psf/ft in the upper 5 feet of backfill for light passenger vehies. Traffic surcharge for heavy axle (HS20) vehie loads should be minimally applied as 300 psf per lineal foot in the upper 5 feet of wall(s).

- A slope stability analysis should demonstrate a factor of safety of at least 1.5 (Global, static) and 1.1 (Global, seismic), for any slope being retained by an MSE retaining wall. Internal stability (local stability) of the MSE retaining walls should meet, or exceed, City and/or local standards.

- The alternative use of paver stone flatwork should be considered where Portland Cement Concrete (PCC) and asphaltic concrete (AC) pavements are planned above a 1:1 (h:v) projection up from the heel of MSE retaining walls because paver stone flatwork can better tolerate the ground deformations related to MSE retaining walls without displaying extensive distress features.

- Wall drainage should be reviewed by this office as plans become available. A filter fabric, pipe and gravel drainage system should be provided behind each wall greater than 3 feet in height. The gravel pad provided for the support of the base course should be adequately drained. Wall drainage should be reviewed by this office as plans become available.

- The geogrid reinforced backfill zone and wall should be supported upon a suitable bearing material, such as compacted fill or dense, unweathered bedrock. Walls may require deeper excavation in order to penetrate any over-lying unsuitable material and encounter these “suitable” materials.

- As with any settlement-sensitive structure, setbacks from adjacent descending slopes should be included in the wall design. A setback (lateral distance) equivalent to H/3 (where H is the height of the slope) should be provided for top of slope improvements. The minimum setback should be 7 feet and need not be greater than 40 feet. A setback (lateral distance) equivalent to H/2 (where H is the height of the slope) should be provided from the outside bottom edge of the wall to adjacent toe of slope improvements. The setback need not be more than 15 feet for these conditions.

- Periodic testing of earth materials will be recommended in order to evaluate that soils with the minimum strength parameters are provided during construction of the walls.

Additional Testing

The parameters provided are preliminary, as exact wall locations, design, and the nature of earth materials used in wall construction are determined, additional testing during earthwork is recommended in order to evaluate and/or modify the preliminary design values used.

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

Expansive Soils and Slope Creep

Some of the onsite soils may be 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 shrinkage cracks depend on many factors such as the nature and expansivity 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 cyes, 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 impact all rear yard flatwork and other secondary improvements that are located within about 15 feet from the top of slopes, such as concrete flatwork, 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, the developer should provide this information to all interested/affected parties.

Top of Slope Walls/Fences

Due to the potential presence of loose/soft bearing soils along property lines, some settlement and tilting of the walls/fence with the corresponding distresses, should be expected. Furthermore, due to the potential for slope creep for slopes higher than about 10 feet, some settlement and tilting of the walls/fence 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. The grade beam should be at a minimum of 12 inches by 12 inches in cross section, supported by drilled caissons, 12 inches minimum in diameter, placed at a maximum spacing of 6 feet on center, and with a minimum embedment length of 7 feet below the bottom of the grade beam. The strength of the concrete and grout should be evaluated by the structural engineer of record. The proper ASTM tests for the concrete and mortar should be provided along with the slump quantities. The concrete used should be appropriate to mitigate severe sulfate exposure. The design of the grade beam and caissons should be in accordance with the recommendations of the project structural engineer, and include the utilization of 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, embedded into low to highly expansive soil, it should be taken as a uniform 900 pounds per linear foot of caisson's depth, located above the creep zone.

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

Passive Resistance: Passive earth pressure of 300 psf per foot of depth per foot of caisson diameter, to a maximum value of 4,000 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 : 350 psf applied below the point of fixity (in formational soil) over the surface area of the shaft.

Tip capacity: 4,000 psf (ear of loose soil, bearing into dense unweathered bedrock).

DRIVEWAY/PARKING, FLATWORK, AND OTHER IMPROVEMENTS

Some of the onsite soil 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 developer notify any interested/affected parties of this long-term potential for distress. To reduce the likelihood of distress, the following recommendations are presented for all exterior flatwork:

1. Exterior concrete slabs should be cast entirely on properly compacted fill overlying dense, unweathered sedimentary bedrock. The subgrade area for concrete slabs should be compacted to achieve a minimum 90 percent relative compaction (sidewalks, patios), and 95 percent relative compaction (traffic pavements), and then be presoaked to 120 percent of the soils' optimum moisture content, to a depth of 18 inches below subgrade elevation. This moisture content should be maintained in the subgrade soils during concrete placement to promote uniform curing of the concrete and minimize the development of unsightly shrinkage cracks. If very low expansive soils (E.I. $\leq$ 20 or P.I. $\leq$ 14) are present, only optimum moisture content, or greater, is required and specific presoaking is not warranted. The moisture content of the subgrade should be proof tested within 72 hours prior to pouring concrete.

2. Concrete slabs should be cast over a non-yielding surface, consisting of a 4-inch layer of crushed rock, compacted aggregate base, gravel, or clean sand, that should be compacted and level prior to placing concrete. If very low expansive soils are present, the concrete may be cast directly upon properly prepared subgrade. The base layer or subgrade should be moisturized completely prior to placing concrete, to reduce loss of concrete moisture to the surrounding earth materials.

3. Exterior concrete slabs supporting pedestrian traffic only should be a minimum of 4 inches thick. Please refer to the "Preliminary Pavement Design/Construction."

4. In order to reduce unsightly cracking, the outer edges of flatwork to be bordered by landscaping should be provided with an 8-inch wide concrete cutoff shoulder (thickened edge) extending to a depth of at least 12 inches below the bottoms of the slabs to mitigate excessive infiltration of water under the flatwork. These thickened edges should be reinforced with two No. 4 bars, one at the top and one at the bottom.

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 and/or expansion joints to accommodate anticipated concrete shrinkage and expansion.

   In order to reduce the potential for unsightly cracks, slabs should be reinforced at mid-height with a minimum of No. 3 bars placed at 18 inches on center, in each direction. 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 6 feet. The slabs should be separated from the foundations and sidewalks with expansion joint filler material.

6. Surface and shrinkage cracking of the finish slabs may be reduced if a low slump and water-cement ratio are maintained during concrete placement. Excessive water added to concrete prior to placement is likely to cause shrinkage cracking, and should be avoided. Some concrete shrinkage cracking, however, is unavoidable.

7. No traffic should be allowed upon the newly placed concrete slabs until they have been properly cured to within 75 percent of design strength. Concrete compression strength should be a minimum of 2,500 psi.

8. Driveways, sidewalks, and patio slabs adjacent to the building should be separated from the building 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.

9. Planters and walls should not be tied to the building(s).

10. Overhang structures should be supported on the slabs, or structurally designed with continuous footings tied in at least two directions.

11. Any masonry landscape 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.

12. Utilities should be enclosed within a closed utilidor (vault) or designed with flexible connections to accommodate differential settlement and expansive soil conditions.

13. Positive site drainage should be maintained at all times. Finish grades should be provided with a minimum of 1 to 2 percent fall to the street, or other approved area, as indicated herein. 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 homeowner.

14. Air conditioning (A/C) units should be supported by 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-erosive outlet.

15. 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 imate and time of year, sulfate content of soils, corrosion potential of soils, and fertilizers used on site.

16. If perimeter, top of slope walls are to be considered, design and construction recommendations could be provided on request.

STORM WATER TREATMENT AND HYDROMODIFICATION MANAGEMENT

Infiltration Feasibility

As previously stated herein, the results of our testing indicate that soil conditions in the vicinity of the proposed permanent stormwater BMP/LID are conducive to no infiltration with an estimate reliable infiltration rate of 0.019 in/hr. The infiltration into the onsite earth materials is not recommended due to the potential adverse effects it could pose to existing offsite improvements.

The following geotechnical guidelines should be considered when designing the permanent stormwater BMP/LID:

- It is not good engineering practice to allow water to saturate soils, especially near slopes or settlement-sensitive improvements; however, the controlling agency/authority is now requiring this for OIRRS purposes on many projects.

- In order to reduce the lateral migration of the stored stormwater, impermeable liners should be installed along the sides and bottom of the basin. The impermeable liner should extend a few inches above the 100-year (Q_{100}) water level, and should be covered with at least 12 inches of soil that is free of angular particles. Impermeable liners should consist of a 30-mil polyvinyl chloride (PVC) membrane with the following minimum specifications:

  Specific Gravity (ASTM D792): 1.2 (g/cc, min.); Tensile Strength at Break (ASTM D882): 73 (lb/in-width, min); Elongation (ASTM D882): 380 (%, min); Modulus at 100 percent (ASTM D882): 32 (lb/in-width, min.); Tear Strength (ASTM D1004): 8 (lbs, min); Seam Strength (ASTM D882): 58.4 (lb/in, min); Seam Shear Strength (ASTM D882): 15 (lbs/in, min.); and Seam Peel Strength (ASTM D882) 2.6 (kN/m, min).

  The impermeable liner may require periodic maintenance, repair, and/or replacement over the design life of the proposed development.

- The perforated drain pipes used within the basins should be Schedule 40 or SDR 35, and should be sleeved with filter sock.

- Proposed settlement-sensitive improvements should be located below a 1:1 (horizontal:vertical [h:v]) plane projected up from the bottom of the basin's perimeter.

- With the exception of the subdrain, underground utilities/solid drain pipes within the BMPs should be backfilled with a minimum two (2)-sack mix of slurry.

- In order to reduce the potential for surficial instability of stormwater basin slopes, they should be constructed at gradients no steeper than 4:1 (h:v).

- Wherever possible, the stormwater BMP/LID should not be placed within a distance of H/2 from the toes of slopes (where H equals the height of slope).

- Wherever possible, the stormwater BMP should not be installed within 10 feet of a residential structure or wall.

- The landscape architect should be notified of the location of the proposed OIRRS. If landscaping is proposed within the OIRRS, consideration should be given to the type of vegetation chosen and their potential effect upon subsurface improvements (i.e., some trees/shrubs will have an effect on subsurface improvements with their extensive root systems). Over-watering landscape areas above, or adjacent to, the proposed stormwater BMP/LID could adversely affect performance of the system.

- Areas adjacent to, or within, the stormwater BMP/LID that are subject to inundation should be properly protected against scouring, undermining, and erosion, in accordance with the recommendations of the design engineer.

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 and/or further geotechnical studies may be warranted. It should be noted that structural and landscape plans were not available for review at this time.

DEVELOPMENT CRITERIA

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 planted slopes. Over-watering should be avoided, as it adversely affects site improvements, and causes perched groundwater conditions. Graded slopes constructed utilizing onsite materials would be erosive. Eroded debris may be minimized 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 minimize short-term erosion until vegetation is established. Plants selected for landscaping should be light weight, 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 allowing the establishment of a sparse plant cover. Utilizing 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 each owner. Over-steepening of slopes should be avoided during building construction activities and landscaping.

Drainage

Adequate lot surface drainage is a very important factor in reducing the likelihood of adverse performance of foundations, hardscape, and slopes. Surface drainage should be sufficient to prevent ponding of water anywhere on a pad, and especially near structures and tops of slopes. 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 within lots and common areas should be provided and maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond and/or seep into the ground. In general, the area within 5 feet around a structure should slope away from the structure. We recommend that unpaved lawn and landscape areas have a minimum gradient of 1 percent sloping away from structures, and whenever possible, should be above adjacent paved areas. Consideration should be given to avoiding construction of planters adjacent to structures (buildings, ancillary slabs, etc.). Pad drainage should be directed toward the street or other approved area(s). Although not a geotechnical requirement, roof gutters, down spouts, or other appropriate means may be utilized to control roof drainage. Down spouts, or drainage devices should outlet a minimum of 5 feet from structures 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 minimizing this effect could be provided upon request.

Toe of Slope Drains/Toe Drains

Where significant slopes intersect pad areas, surface drainage down the slope allows for some seepage into the subsurface materials, sometimes creating conditions causing or contributing to perched and/or ponded water. Toe of slope/toe drains may be beneficial in the mitigation of this condition due to surface drainage. The general criteria to be utilized by the design engineer for evaluating the need for this type of drain is as follows:

- Is there a source of irrigation above or on the slope that could contribute to saturation of soil at the base of the slope? Are the slopes hard rock and/or impermeable, or relatively permeable, or; do the slopes already have or are they proposed to have subdrains (i.e., stabilization fills, etc.)?

- Was the lot at the base of the slope overexcavated or is it proposed to be overexcavated? Overexcavated lots located at the base of a slope could accumulate subsurface water along the base of the fill cap.

- Are the slopes north facing? North facing slopes tend to receive less sunlight (less evaporation) relative to south facing slopes and are more exposed to the currently prevailing seasonal storm tracks.

- What is the slope height? It has been our experience that slopes with heights in excess of approximately 10 feet tend to have more problems due to storm runoff and irrigation than slopes of a lesser height.

- Do the slopes "toe out" into a residential lot or a lot where perched or ponded water may adversely impact its proposed use?

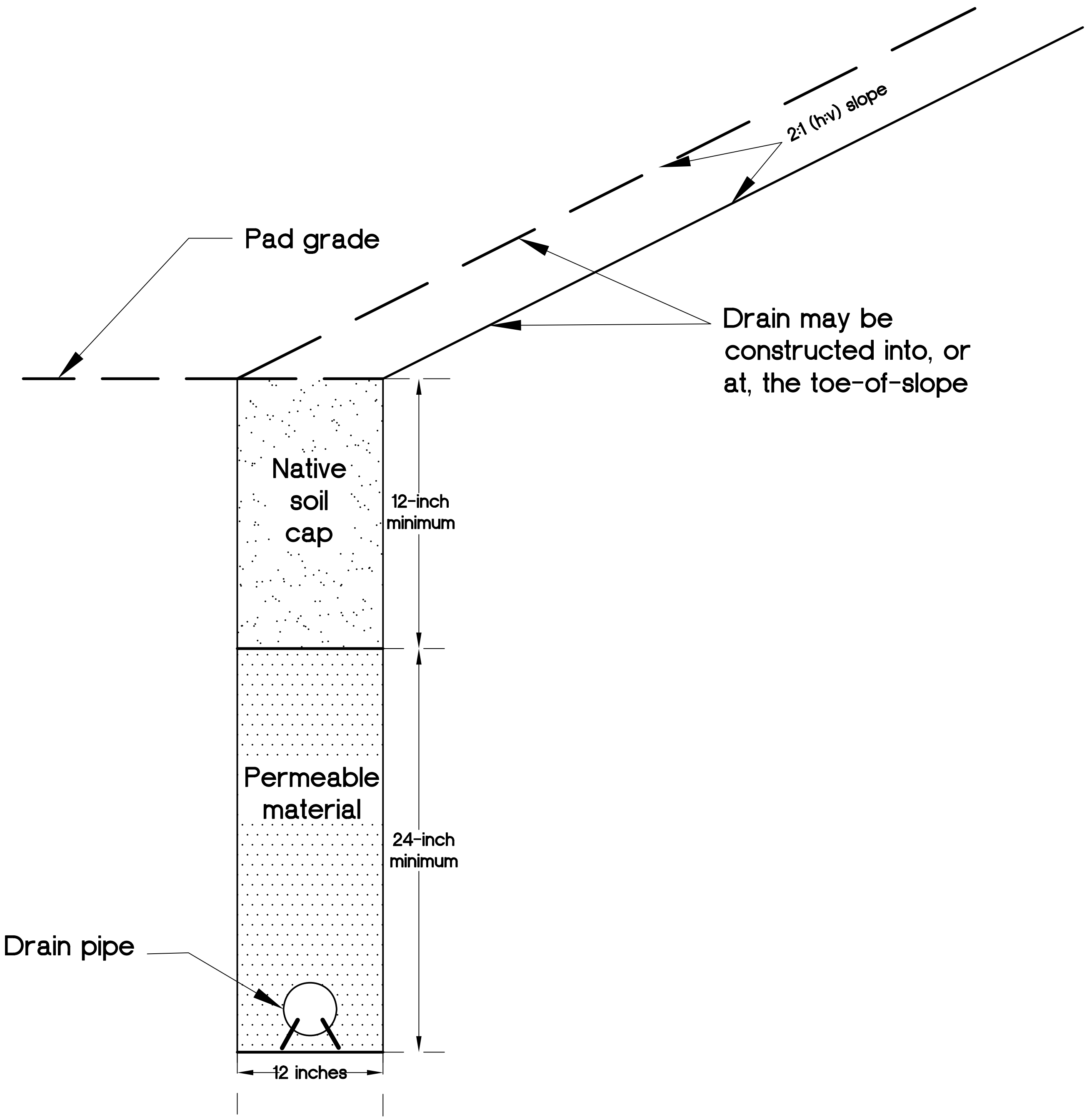
Based on these general criteria, the construction of toe drains may be considered by the design engineer along the toe of slopes, or at retaining walls in slopes, descending to the rear of such lots. Following are Detail 4 (Schematic Toe Drain Detail) and Detail 5 (Subdrain Along Retaining Wall Detail). Other drains may be warranted due to unforeseen conditions, homeowner irrigation, or other circumstances. Where drains are constructed during grading, including subdrains, the locations/elevations of such drains should be surveyed, and recorded on the final as-built grading plans by the design engineer. It is recommended that the above be disclosed to all interested parties, including homeowners and any homeowners association.

Erosion Control

Cut and fill slopes will be subject to surficial erosion during and after grading. Onsite earth materials have a moderate to high erosion potential. Consideration should be given to providing hay bales and silt fences for the temporary control of surface water, from a geotechnical viewpoint.

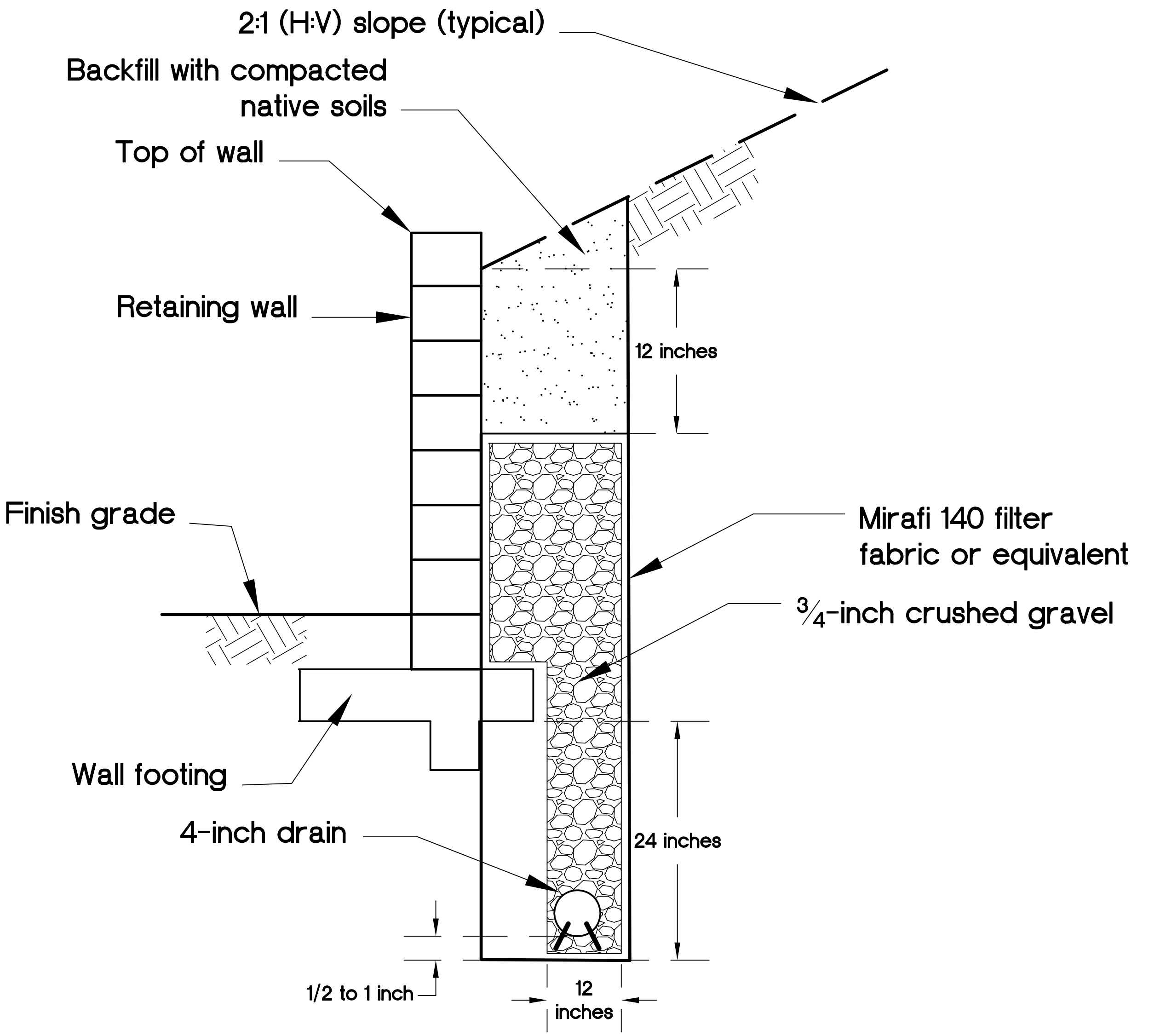
Landscape Maintenance and Open-Bottom Planter Location

Only the amount of irrigation necessary to sustain plant life should be provided. Over-watering the landscape areas will adversely affect proposed site improvements. We would recommend that any proposed open-bottom planters adjacent to proposed structures be eliminated for a minimum distance of 10 feet. As an alternative, closed-bottom type planters could be utilized. An outlet placed in the bottom of the planter, could be installed to direct drainage away from structures or any exterior concrete flatwork. If planters are constructed adjacent to structures, the sides and bottom of the



1. Soil cap compacted to 90 percent relative compaction.
2. Permeable material may be gravel wrapped in filter fabric (Mirafi 140N or equivalent).
3. 4-inch-diameter, perforated pipe (SDR-35 or equivalent) with perforations down.
4. Pipe to maintain a minimum 1 percent fall.
5. Concrete cut-off wall to be provided at transition to solid outlet pipe.
6. Solid outlet pipe to drain to approved area.
7. Cleanouts are recommended at each property line.

GeoSoils, Inc.	SCHEMATIC TOE DRAIN DETAIL	Detail 4



NOTES:

1. Soil cap compacted to 90 percent relative compaction.
2. Permeable material may be gravel wrapped in filter fabric (Mirafi 140N or equivalent).
3. 4-inch-diameter, perforated pipe (SDR-35 or equivalent) with perforations down.
4. Pipe to maintain a minimum 1 percent fall.
5. Concrete cut-off wall to be provided at transition to solid outlet pipe.
6. Solid outlet pipe to drain to approved area.
7. Cleanouts are recommended at each property line.
8. Effort to compact should be applied to drain rock.

GeoSoils, Inc.	SUBDRAIN ALONG RETAINING WALL DETAIL	Detail 5

planter should be provided with a moisture barrier to prevent penetration 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. Graded slope areas should be planted with drought resistant vegetation. 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. If the surface soils are processed for the purpose of adding amendments, they should be recompacted to 90 percent minimum relative compaction.

Gutters and Downspouts

As previously discussed in the drainage section, the installation of gutters and downspouts should be considered to collect roof water that may otherwise infiltrate the soils adjacent to the structures. If utilized, the downspouts should be drained into PVC collector pipes or other non-erosive devices (e.g., paved swales or ditches; below grade, solid tight-lined PVC pipes; etc.), that will carry the water away from the structure, to an appropriate outlet, in accordance with the recommendations of the design civil engineer. Downspouts and gutters are not a requirement; however, from a geotechnical viewpoint, provided that positive drainage is incorporated into project design (as discussed previously)

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. Perched groundwater conditions along zones of contrasting permeabilities may not be precluded from occurring in the future due to site irrigation, poor drainage conditions, or damaged utilities, and should be anticipated. Should perched groundwater conditions develop, this office could assess the affected area(s) and provide the appropriate recommendations to mitigate the observed groundwater conditions. Groundwater conditions may change with the introduction of irrigation, rainfall, or other factors.

Site Improvements

If in the future, any additional improvements (e.g., wall, enosures, etc.) are planned for the site, recommendations concerning the geological or geotechnical aspects of design and construction of said improvements could be provided upon request. Swimming pools/spas should not be constructed without a geotechnical review. 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, utility trench and retaining wall backfills, flatwork, etc. This information should be provided to all interested/affected parties.

Additional Grading

This office should be notified in advance of any fill placement, supplemental regrading of the site, or trench backfilling after rough grading has been completed. This includes completion of grading in the street, driveway approaches, driveways, parking areas, and utility trench and retaining wall backfills.

Footing Trench Excavation

All footing excavations should be observed by a representative of this firm subsequent to trenching and prior to concrete form and 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 materials are exposed within the footing excavation, a deeper footing or removal and recompaction of the subgrade materials would be recommended at that time. Footing trench spoil and any excess soils generated from utility trench excavations should be compacted to a minimum relative compaction of 90 percent (ASTM D 1557), if not removed from the site.

Trenching/Temporary Construction Backcuts

Considering the nature of the onsite earth materials, it should be anticipated that 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 [except as specifically superceded within the text of this report]), should be anticipated. All excavations should be observed by an engineering geologist or soil engineer from GSI, prior to workers entering the excavation or trench, and minimally conform to CAL-OSHA, state, and local safety codes. Should adverse conditions exist, appropriate recommendations would be offered at that time. The above recommendations should be provided to any contractors and/or subcontractors, or homeowners, etc., that may perform such work.

Utility Trench Backfill

1. All interior utility trench backfill should be brought to at least 2 percent above optimum moisture content and then compacted to obtain a minimum relative compaction of 90 percent of the laboratory standard (ASTM D 1557). As an alternative for shallow (12-inch to 18-inch) under-slab trenches, sand having a sand equivalent value of 30 or greater may be utilized and jetted or flooded into place. Observation, probing and testing should be provided to evaluate the desired results.

2. Exterior trenches adjacent to, and within areas extending below a 1:1 plane projected from the outside bottom edge of the footing, and all trenches beneath hardscape features and in slopes, should be compacted to at least 90 percent of the laboratory standard. Sand backfill, unless excavated from the trench, should

not be used in these backfill areas. Compaction testing and observations, along with probing, should be accomplished to evaluate the desired results.

3. All trench excavations should conform to CAL-OSHA, state, and local safety codes.

4. Utilities crossing grade beams, perimeter beams, or footings should either pass below the footing or grade beam utilizing a hardened collar or foam spacer, or pass through the footing or grade beam in accordance with the recommendations of the structural engineer.

Monitoring of Structures

1. The contractor should complete a written and photographic log of the existing building or other structures located within 100 feet or three times the depth of shoring (whichever is greater) prior to excavation and/or any shoring construction. A licensed surveyor should document all existing substantial cracks (i.e., greater than ⅛ inch horizontal or vertical separation) in the adjacent building and structures.

2. The contractor should document the existing condition of wall cracks in the existing building adjacent to the shoring wall prior to the start of shoring construction.

3. The contractor should monitor existing building walls and improvements for movement or cracking that may result from the adjacent excavation/shoring.

4. If excessive movement or visible cracking occurs, the shoring contractor should stop work and shore/reinforce the excavation, and contact the geotechnical engineer and/or Shoring Design Engineer, and the Building Official.

5. Monitoring of the existing building(s) or adjacent structures should be made at reasonable intervals as required by the registered design professional, subject to approval by the Building Official. Monitoring should be performed by a licensed surveyor.

6. Prior to excavation, or commencing shoring construction, a pre-construction meeting should take place between the contractor, Shoring Design Engineer, Surveyor, Geotechnical Engineer, and the Building Official to identify monitoring locations on existing buildings.

7. If in the opinion of the Building Official or Shoring Design Engineer, monitoring data indicate excessive movement or other distress, all excavation should cease until the Geotechnical Engineer and Shoring Design Engineer investigates the situation and makes recommendations for remedial actions or continuation.

8. All readings and measurements should be submitted to the Building Official and Shoring Design Engineer.

SUMMARY OF RECOMMENDATIONS REGARDING GEOTECHNICAL OBSERVATION AND TESTING

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

- During grading/recertification.
- During excavation.
- During placement of subdrains, toe drains, or other subdrainage devices, prior to placing fill and/or backfill.
- 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 presoaking/presaturation of building pads and other flatwork subgrade, before the placement of concrete, reinforcing steel, capillary break (i.e., sand, pea-gravel, etc.), or vapor retarders (i.e., visqueen, etc.).
- During retaining wall subdrain installation, prior to backfill placement.
- During placement of backfill for area drain, interior plumbing, utility line trenches, and retaining wall backfill.
- During slope construction/repair.
- When any unusual soil conditions are encountered during any construction operations, subsequent to the issuance of this report.
- When any improvements, such as flatwork, walls, etc., are constructed, 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 ear documentation of site work, and/or to comply with code requirements.

OTHER DESIGN PROFESSIONALS/CONSULTANTS

The design civil engineer, structural engineer, post-tension designer, 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. Please note that the recommendations contained herein are not intended to preclude the transmission of water or vapor through the slab or any foundation. The structural engineer/foundation and/or slab designer should provide recommendations to not allow water or 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, as appropriate.

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 and/or improvement's designer should confirm to GSI and the governing agency, in writing, that the proposed foundations and/or improvements can tolerate the amount of differential settlement and/or expansion characteristics and other design criteria specified herein.

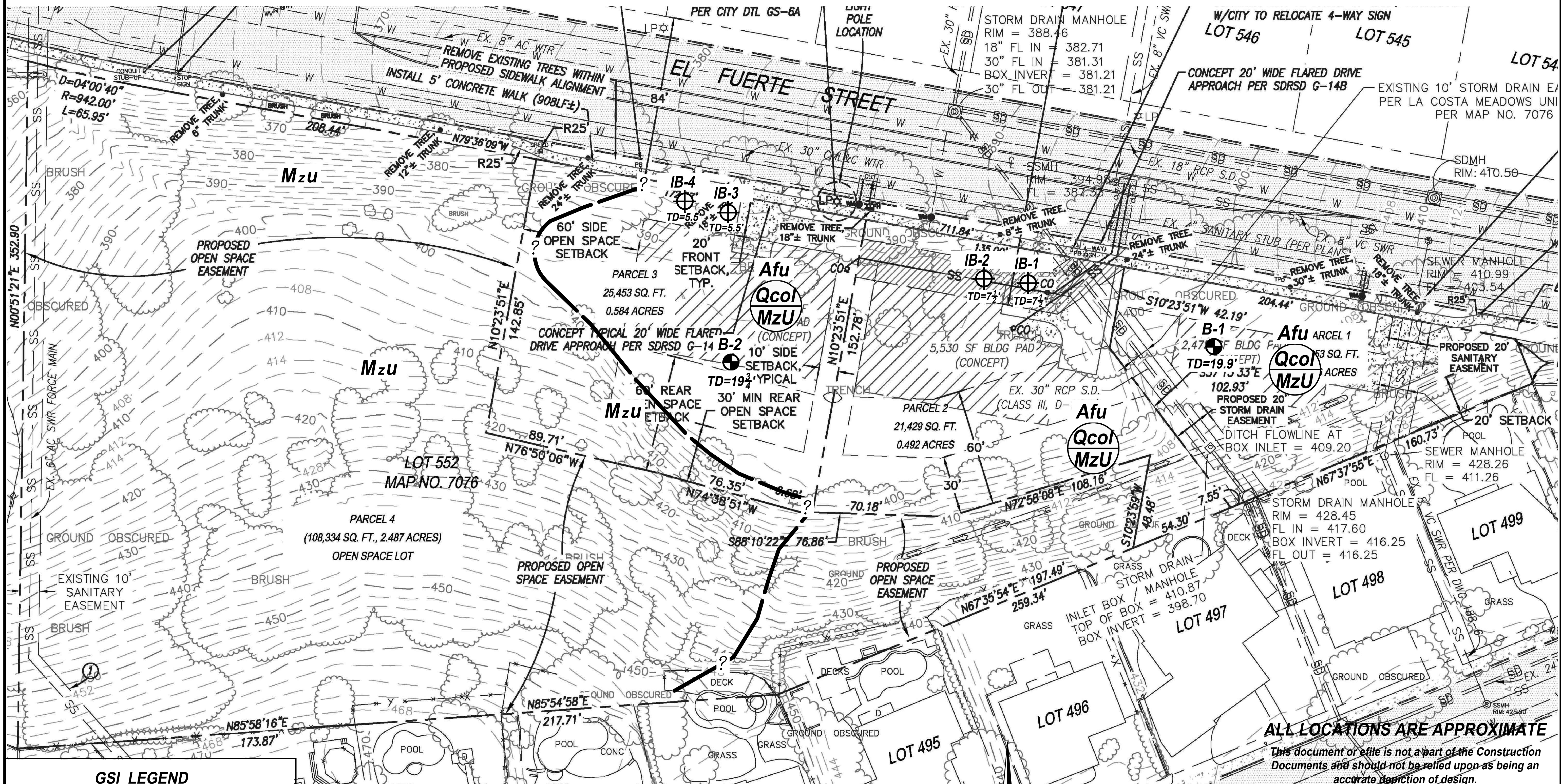
PLAN REVIEW

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 and/or further geotechnical studies may be warranted.

LIMITATIONS

The materials encountered on the project site and utilized 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.



GSI LEGEND

- **Afu** – ARTIFICIAL FILL – UNDOCUMENTED
- **Qcol** – QUATERNARY-AGE COLLUVIUM, CIRCLED WHERE BURIED
- **Mzu** – MESOZOIC-AGE METASEDIMENTARY AND METAVOLCANIC ROCKS, UNDIVIDED, CIRCLED WHERE BURIED
- ? – APPROXIMATE LOCATION OF GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN
- B-2, TD=19.9' – APPROXIMATE LOCATION OF HAND AUGER BORING
- IB-4, TD=7½' – APPROXIMATE LOCATION OF EXPLORATORY TEST PIT

BASE MAP FROM:

ADVANCED CIVIL ENGINEERING PLLC
Advanced Civil Engineering, PLLC
Serving the Intermountain West
Phone: (801) 884 6010
Fax: (801) 341 8528
www.advancedcivilengineering.com
(professional limited liability company)
CIVIL / PLANNING / CONTRACT ADMINISTRATION

DESIGN BY: BKS
DRAWN BY: BKS
REVIEWED BY:
PLOT SCALE: 1" = 40'
FILE NAME: EL FUERTE BASE
PROJECT NO.: 6026

SHEET NO. C102 OF 8

N

GRAPHIC SCALE
50 0 25 50 100
1" = 50'

GeoSoils, Inc.

GEOTECHNICAL MAP

Plate 1

W.O. 7795-A-SC | DATE: 04/20 | SCALE: 1" = 50'

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

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APPENDIX B

EXPLORATION 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.

BORING LOG

PROJECT: C2 FUERTE HOLDINGS
El Fuerte View

W.O. 7795-A-SC *BORING* B-1 *SHEET* 1 *OF* 1

DATE EXCAVATED 2-24-20 *LOGGED BY:* MK *APPROX. ELEV.:* ±407.6

SAMPLE METHOD: Modified Cal. Sampler, 140 lbs @ 30" Drop

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
0	■			SC				**ARTIFICIAL FILL - UNDOCUMENTED:** @ 0' CLAYEY SAND, brown, damp, loose to medium dense; fine to coarse grained.
		I	44	SC	106.5	16.3		@ 3' becomes moist, medium dense.
5		I	76	SC				@ 5' CLAYEY SAND, olive brown, moist, medium dense; fine grained.
10		I	49	CL	100.7	18.9		**COLLUVIUM:** @ 10' SANDY CLAY, dark brown, moist, stiff; fine to coarse grained.
15		I	54	CH				**METASEDIMENTARY AND METAVOLCANIC ROCKS, UNDIVIDED (WEATHERED):** @ 13½' METASEDIMENTARY ROCK TO SILTY CLAY, reddish brown to pale gray, moist, fractured, medium hard; fine grained.
		I	50-5"	CH				**METASEDIMENTARY AND METAVOLCANIC ROCKS, UNDIVIDED:** @ 17' METASEDIMENTARY ROCK TO SILTY CLAY, reddish brown to pale gray, moist, hard; fine grained.
20								Total Depth = 19.9' No Groundwater or Caving Encountered Backfilled 2-24-20
25								
30								

Standard Penetration Test
Undisturbed, Ring Sample
Groundwater
Seepage

GeoSoils, Inc.

BORING LOG

PROJECT: C2 FUERTE HOLDINGS
El Fuerte View

W.O. 7795-A-SC BORING B-2 SHEET 1 OF 1

DATE EXCAVATED 2-24-20 LOGGED BY: MK APPROX. ELEV.: ±397.6

SAMPLE METHOD: Modified Cal. Sampler, 140 lbs @ 30" Drop

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
0				SC				**ARTFIICIAL FILL - UNDOCUMENTED:** @ 0' CLAYEY SAND, brown, moist, loose to medium dense; fine grained.
			75					@ 3' Becomes medium dense to dense.
5			55	CL	104.3	21.4		@ 4½' Encountered asphalt.
								COLLUVIUM: @ 5½' SANDY CLAY, dark brown, moist, fine to coarse grained.
10			50-5½"	CH	99.1	21.9		**METASEDIMENTARY AND METAVOLCANIC ROCKS, UNDIVIDED - (WEATHERED):** @ 9½' METASEDIMENTARY ROCK TO SILTY CLAY, reddish brown to pale gray, moist, fractured, very dense; fine grained to coarse grained with traces of crystals.
15			50-5½"	CH	110.01	14.7		**METASEDIMENTARY AND METAVOLCANIC ROCKS , UNDIVIDED:** @ 13' METASEDIMENTARY ROCK TO SILTY CLAY, play gray to light brown to pink, moist, hard; fine grained.
			50-3"	CH	104.1	22.5		
20								Total Depth = 19¾' No Groundwater or Caving Encountered Backfilled 2-24-20
25								
30								

Standard Penetration Test
Undisturbed, Ring Sample
Groundwater
Seepage

APPENDIX C

INFILTRATION DATA

Percolation Test Data Sheet							
Project:	El Fuente View		Project No:	7795-A-SC		Date:	02-24-20
Test Hole No:		1B-1	Tested By:	MK			
Depth of Test Hole, D_T:		90.0"	USCS Soil Classification:		CL		
Test Hole Dimensions (inches)					Length	Width	
Diameter (if round)=		~8"	Sides (if rectangular)=				
Sandy Soil Criteria Test*							
Trial No.	02-25-20 Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	8:13	8:38	25	16	16.25	0.25	N
2	8:39	9:04	25	16.25	16.50	0.25	N

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Other wise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	ΔD Change in Water Level (in.)	Percolation Rate (min./in.)
1	9:31	10:01	30	15.75	16.0	0.25	
2	10:02	10:32	30	16.0	16.25	0.25	
3	10:34	11:04	30	16.25	16.5	0.25	
4	11:05	11:35	30	16.5	16.75	0.25	
5	11:36	12:06	30	16.75	16.75	0	
6	12:08	12:38	30	16.75	17.0	0.25	
7	12:40	1:10	30	17.0	17.0	0	
8	1:11	1:41	30	17.0	17.25	0.25	
9	1:42	2:12	30	17.25	17.25	0	
10	2:13	2:43	30	17.25	17.25	0	
11							
12							
13							
14							
15							

COMMENTS:

Table 5 – Sample Test Data Form for Percolation Test

Percolation Test Data Sheet

Project:	El Fuerte View		Project No:	7795-A-SC		Date:	02-24-20
Test Hole No:		1B-2	Tested By:	MK			
Depth of Test Hole, D_T:		90.0"	USCS Soil Classification:		CL		
Test Hole Dimensions (inches)					Length	Width	
Diameter (if round)=		~8"	Sides (if rectangular)=				

Sandy Soil Criteria Test*

Trial No.	02-25-20 Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	8:15	8:40	25	15	15.25	0.25	N
2	8:41	9:06	25	15.25	15.5	0.25	N

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Other wise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	ΔD Change in Water Level (in.)	Percolation Rate (min./in.)
1	9:34	10:04	30	15.0	15.25	0.25	
2	10:05	10:35	30	15.25	15.5	0.25	
3	10:36	11:06	30	15.5	15.75	0.25	
4	11:08	11:38	30	15.75	15.75	0	
5	11:40	12:10	30	15.75	16.0	0.25	
6	12:12	12:42	30	16.0	16.0	0	
7	12:43	1:13	30	16.0	16.25	0.25	
8	1:14	1:44	30	16.25	16.25	0	
9	1:45	2:15	30	16.25	16.5	0.25	
10	2:16	2:46	30	16.5	16.5	0	
11							
12							
13							
14							
15							

COMMENTS:

Table 5 – Sample Test Data Form for Percolation Test

Percolation Test Data Sheet

Project:	El Fuerte View		Project No:	7795-A-SC		Date:	02-24-26
Test Hole No:		1B-3	Tested By:	MK			
Depth of Test Hole, D_T:		66.0"	USCS Soil Classification:		CH		
Test Hole Dimensions (inches)					Length	Width	
Diameter (if round)=		~8"	Sides (if rectangular)=				

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	8:21	8:46	25	17.5	17.75	0.25	N
2	8:47	9:12	25	17.75	18.0	0.25	N

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Other wise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	ΔD Change in Water Level (in.)	Percolation Rate (min./in.)
1	10:08	10:38	30	17.0	17.25	0.25	
2	10:39	11:09	30	17.25	17.75	0.50	
3	11:11	11:41	30	17.75	18.25	0.50	
4	11:42	12:12	30	18.25	18.50	0.25	
5	12:13	12:43	30	16.0	16.25	0.25	
6	12:44	1:14	30	16.25	16.75	0.50	
7	1:15	1:45	30	16.75	17.0	0.25	
8	1:46	2:06	30	15.0	15.25	0.25	
9	2:07	2:47	30	15.25	15.5	0.25	
10	2:48	3:18	30	15.5	15.75	0.25	
11							
12							
13							
14							
15							

COMMENTS:

Table 5 – Sample Test Data Form for Percolation Test

Percolation Test Data Sheet

Project:	El Fuerte View		Project No:	7795-A-SC		Date:	02-24-20
Test Hole No:		IB-4	Tested By:	MK			
Depth of Test Hole, D_T:		66.0"	USCS Soil Classification:		CH		
Test Hole Dimensions (inches)					Length	Width	
Diameter (if round)=		~8"	Sides (if rectangular)=				

Sandy Soil Criteria Test*

Trial No.	02-25-20 Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	8:24	8:49	25	18.25	18.50	0.25	N
2	8:50	9:15	25	18.50	19.00	0.50	N

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Other wise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	ΔD Change in Water Level (in.)	Percolation Rate (min./in.)
1	10:10	10:40	30	17.0	17.25	0.25	
2	10:41	11:11	30	17.25	17.75	0.5	
3	11:12	11:42	30	17.75	18	0.25	
4	11:44	12:14	30	16.25	16.75	0.5	
5	12:15	12:45	30	16.75	17.0	0.25	
6	12:47	1:17	30	17.0	17.5	0.5	
7	1:18	1:48	30	13.75	14.25	0.5	
8	1:49	2:19	30	14.25	14.50	0.25	
9	2:20	2:50	30	14.50	14.75	0.25	
10	2:52	3:22	30	14.75	15.0	0.25	
11							
12							
13							
14							
15							

COMMENTS:

Table 5 – Sample Test Data Form for Percolation Test

Project:	El Fuerte View	W.O. Number:	7795-A-SC
Test Hole No.:	IB-1	Date Excavated:	2/24/2020

Percolation Rate to Infiltration Rate Conversion

$$* \text{ Infiltration Rate } (I_t) = \frac{\Delta H\ \pi r^2\ 60}{\Delta t(\pi r^2 + 2\pi r H_{avg})} = \frac{\Delta H\ 60\ r}{\Delta t(r+2H_{avg})}$$

Where:

I_t = tested infiltration rate, inches/hour
ΔH = change in head over the time interval, inches
Δt = time interval, minutes
r = effective radius of test hole
H_{avg} = average head over the time interval, inches

		Δt	Init Level	Fnl Level	ΔH	H_{avg}	I_t
Infiltration Test Numbers	IB-1 @ 7.5 ft.	30	72 3/4	72 3/4	0	72 3/4	0.000

* Conversion per the "Porchet Method" (RCFWCD, 2011)

Project:	El Fuerte View	W.O. Number:	7795-A-SC
Test Hole No.:	IB-2	Date Excavated:	2/24/2020

Percolation Rate to Infiltration Rate Conversion

* Infiltration Rate (I_t) = $$\frac{\Delta H\ \pi r^2\ 60}{\Delta t(\pi r^2 + 2\pi r H_{avg})} = \frac{\Delta H\ 60\ r}{\Delta t(r+2H_{avg})}$$

Where:

I_t = tested infiltration rate, inches/hour
ΔH = change in head over the time interval, inches
Δt = time interval, minutes
r = effective radius of test hole
H_{avg} = average head over the time interval, inches

		Δt	**Init Level**	**Fnl Level**	**ΔH**	**H_{avg}**	**I_t**
Infiltration Test Numbers	IB-2 @ 7.5 ft.	30	73 1/2	73 1/2	0	73 1/2	0.000

* Conversion per the "Porchet Method" (RCFWCD, 2011)

Project:	El Fuerte View	W.O. Number:	7795-A-SC
Test Hole No.:	IB-3	Date Excavated:	2/24/2020

Percolation Rate to Infiltration Rate Conversion

* Infiltration Rate (I_t) = $\dfrac{\Delta H\ \pi r^2\ 60}{\Delta t(\pi r^2 + 2\pi r H_{avg})}$ = $\dfrac{\Delta H\ 60\ r}{\Delta t(r+2H_{avg})}$

Where:

I_t = tested infiltration rate, inches/hour
ΔH = change in head over the time interval, inches
Δt = time interval, minutes
r = effective radius of test hole
H_{avg} = average head over the time interval, inches

		Δt	Init Level	Fnl Level	ΔH	H_{avg}	I_t
Infiltration Test Numbers	IB-3 @ 5.5 ft.	30	50 1/2	50 1/4	1/4	50 3/8	0.019

* Conversion per the "Porchet Method" (RCFWCD, 2011)

Project:	El Fuerte View	W.O. Number:	7795-A-SC
Test Hole No.:	IB-4	Date Excavated:	2/24/2020

Percolation Rate to Infiltration Rate Conversion

* Infiltration Rate (I_t) = $\dfrac{\Delta H\ \pi r^2\ 60}{\Delta t(\pi r^2 + 2\pi r H_{avg})}$ = $\dfrac{\Delta H\ 60\ r}{\Delta t(r+2H_{avg})}$

Where:

I_t = tested infiltration rate, inches/hour
ΔH = change in head over the time interval, inches
Δt = time interval, minutes
r = effective radius of test hole
H_{avg} = average head over the time interval, inches

		Δt	Init Level	Fnl Level	ΔH	H_{avg}	I_t
Infiltration Test Numbers	IB-4 @ 5.5 ft.	30	51 1/4	51	1/4	51 1/8	0.019

* Conversion per the "Porchet Method" (RCFWCD, 2011)

Worksheet C.4-1: Categorization of Infiltration Feasibility Condition

Categorization of Infiltration Feasibility Condition		**Worksheet C.4-1 Basin # 1**	
Part 1 - Full Infiltration Feasibility Screening Criteria **Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?**			
Criteria	**Screening Question**	**Yes**	**No**
1	**Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X
Provide basis: USDA data indicates that the soil in this basin is the Huerhuero Loam, belonging to Hydrologic Soils Group (HSG) D, with infiltration rates (K_{SAT}) ranging from 0.00 to 0.06 inches/hour. Our field data supports the USDA data. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
2	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X
Provide basis: Infiltration is less than 0.5 inches per hour. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			

<table>
<tr><th colspan="4">Worksheet C.4.1 Page 2 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>3</td><td>Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensible evaluation of the factors presented in Appendix C.3.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis:

Infiltration is less than 0.5 inches per hour.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>4</td><td>Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as a change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis:

Infiltration is less than 0.5 inches per hour.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>Part 1 Result*</td><td colspan="2">In the answers to rows 1-4 are “Yes” a full infiltration design is potentially feasible. The feasibility screening category is Full Infiltration

If any answer from row 1-4 is “No”, infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a “full infiltration” design.
Proceed to Part 2</td><td>proceed to part 2</td></tr>
</table>

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

Worksheet C.4.1 Page 3 of 4			
Part 2 - Partial Infiltration vs. No Infiltration Feasibility Screening Criteria **Would infiltration of water in an appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?**			
Criteria	**Screening Question**	**Yes**	**No**
5	**Do soil and geologic conditions allow for infiltration in any appreciable rate or volume?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X
Provide basis: USDA data indicates that the soil in this basin is the Huerhuero Loam, belonging to Hydrologic Soils Group (HSG) D, with design infiltration rates ranging from 0.00 to 0.01 inches/hour, based on testing. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
6	**Can infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X
Provide basis: Onsite bedding is unfavorable to slope stability. Water weakens the rock strength, and the performance of the onsite sediments is sensitive to moisture variation. Given the cemented and clayey nature of the bedrock, mounding and lateral and vertical migration of water will occur, increasing soil moisture transmission rates, adversely affecting slope stability, both onsite and offiste. Deformations from such could distress improvements within the aforementioned adjacent properties. It is not good engineering practice to discharge water into the subsurface for these conditions. Accordingly, infiltration is not recommended. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			

<table>
<tr><th colspan="4">Worksheet C.4.1 Page 4 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>7</td><td>Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis:

Due to the shallow, clayey, and dense nature of the bedrock underlying the site, such sediments will be susceptible to seasonal perched groundwater conditions, causing saturation, distress to proposed and existing improvements, and slope instability, both onsite and offsite. GSI is not aware of any pollutants that would be transmitted onsite or offsite. Design infiltration rates range from 0.00 to 0.01 inches/hr

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>8</td><td>Can infiltration be allowed without violating downstream water rights? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:

Water rights are considered a legal matter, and typically do not fall within the purview of geotechnical engineering. GSI is not aware of any downstream water rights issues of concern on the adjoining properties. Further, given the low infiltration rate of onsite soils, it does not appear that infiltration should significantly affect downstream water rights, from a geotechnical perspective. Drainage appears to be directed offsite and collected within the municipal system. The depth to regional groundwater is greater than 20 feet below the bottom of the anticipated basin elevation.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>Part 2 Result*</td><td colspan="2">If all answers from row 5-8 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration.

If any answer from row 5-8 is no, then infiltration of any volume is considered to be infeasible within the drainage area. The feasibility screening category is No Infiltration.</td><td>No Infiltration</td></tr>
</table>

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

Worksheet D.5-1: Factor of Safety and Design Infiltration Rate Worksheet

Factor of Safety Infiltration Rate Worksheet			Worksheet D.5-1		
Factor Criteria		**Factor Description**	**Assigned Weight (w)**	**Factor Value (v)**	**Product (p) p = w x v**
A	Suitability Assessment	Soil assessment methods	0.25	2	0.5
		Predominant soil texture	0.25	1	0.25
		Site soil variability	0.25	2	0.50
		Depth to groundwater/impervious layer	0.25	1	0.25
		Suitability Assessment Safety Factor, $S_A = \Sigma p$			1.50
B	Design	Level of pretreatment/expected sediment loads	0.5		
		Redundancy/resiliency	0.25		
		Compaction during construction	0.25		
		Design Safety Factor, $S_B = \Sigma p$			
Combined Safety Factor, $S_{total} = S_A \times S_B$				Assume 2.0 min	
Observed Infiltration Rate, inch/hr, $K_{observed}$ (corrected for test-specific bias)					
Design Infiltration Rate, in/hr, $K_{design} = K_{observed} / S_{total}$				0.00 to 0.01	
Supporting Data					
Briefly describe infiltration test and provide reference to test forms:					

*Design Criteria has been left blank due to the fact that design plans for an infiltration basin have not been created yet.

APPENDIX D

SEISMICITY ANALYSIS

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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: 7795-A-SC

DATE: 05-06-2020

JOB NAME: EL FUERTE

CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: C:\Program Files\EQFAULT1\CGSFLTE.DAT

SITE COORDINATES:
   SITE LATITUDE: 33.1115
   SITE LONGITUDE: 117.2431

SEARCH RADIUS: 62.14 mi

ATTENUATION RELATION: 12) Bozorgnia Campbell Niazi (1999) Hor.-Soft 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: 1   Campbell SHR: 0
   COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: C:\Program Files\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	7.7(12.4)	7.2	0.490	X
NEWPORT-INGLEWOOD (Offshore)	11.4(18.3)	7.1	0.344	IX
ELSINORE (TEMECULA)	22.7(36.6)	6.8	0.147	VIII
ELSINORE (JULIAN)	22.7(36.6)	7.1	0.179	VIII
CORONADO BANK	22.8(36.7)	7.6	0.250	IX
ELSINORE (GLEN IVY)	37.3(60.0)	6.8	0.088	VII
EARTHQUAKE VALLEY	38.6(62.2)	6.5	0.069	VI
SAN JOAQUIN HILLS	41.4(66.6)	6.6	0.098	VII
PALOS VERDES	42.1(67.7)	7.3	0.110	VII
SAN JACINTO-ANZA	45.5(73.3)	7.2	0.094	VII
SAN JACINTO-SAN JACINTO VALLEY	47.4(76.3)	6.9	0.073	VII
SAN JACINTO-COYOTE CREEK	48.8(78.6)	6.6	0.058	VI
ELSINORE (COYOTE MOUNTAIN)	52.0(83.7)	6.8	0.062	VI
NEWPORT-INGLEWOOD (L.A.Basin)	52.2(84.0)	7.1	0.076	VII
CHINO-CENTRAL AVE. (Elsinore)	52.6(84.7)	6.7	0.081	VII
WHITTIER	56.5(91.0)	6.8	0.057	VI
SAN JACINTO - BORREGO	61.1(98.3)	6.6	0.046	VI

****

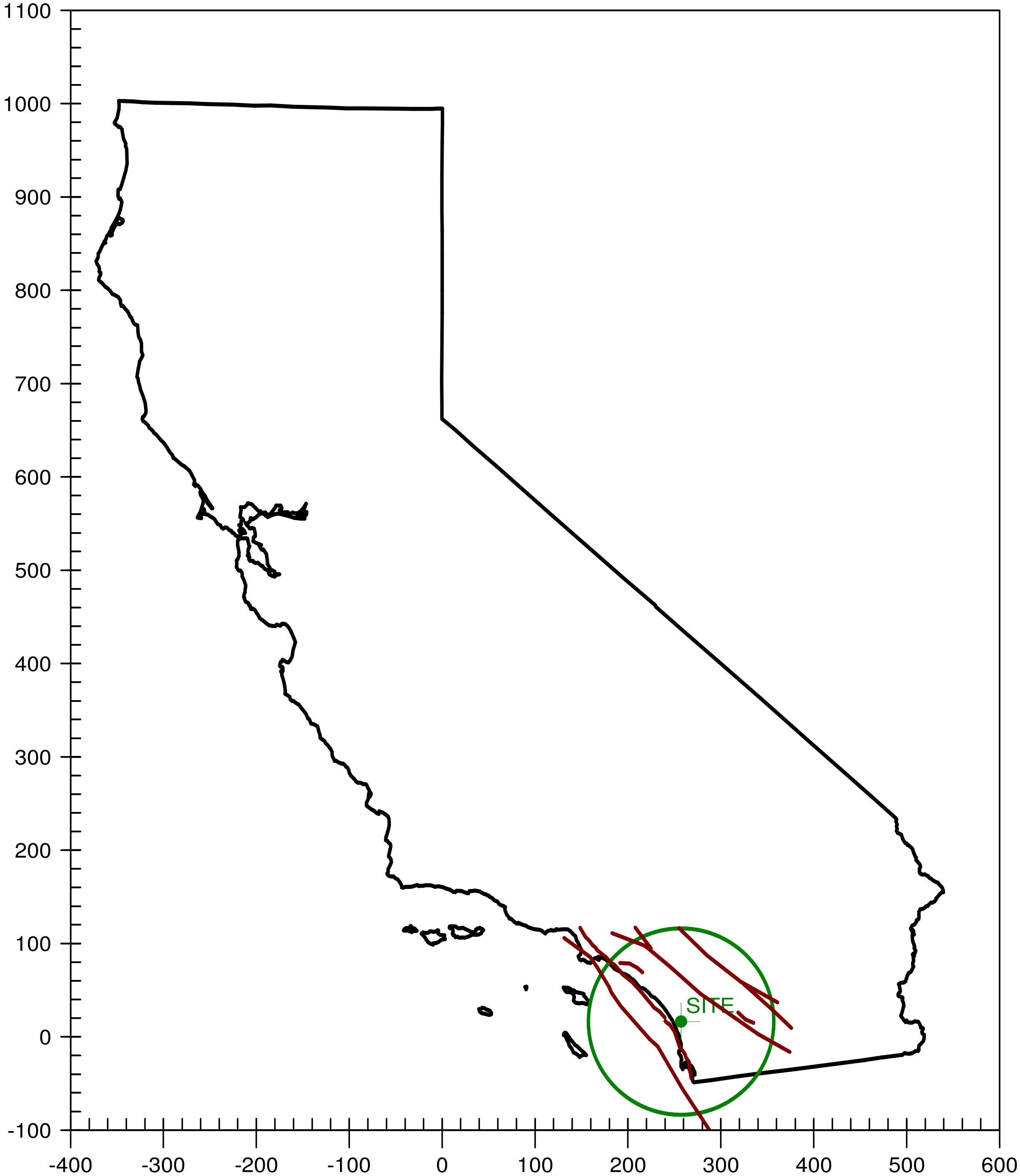
-END OF SEARCH- 17 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ROSE CANYON FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 7.7 MILES (12.4 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.4897 g

CALIFORNIA FAULT MAP

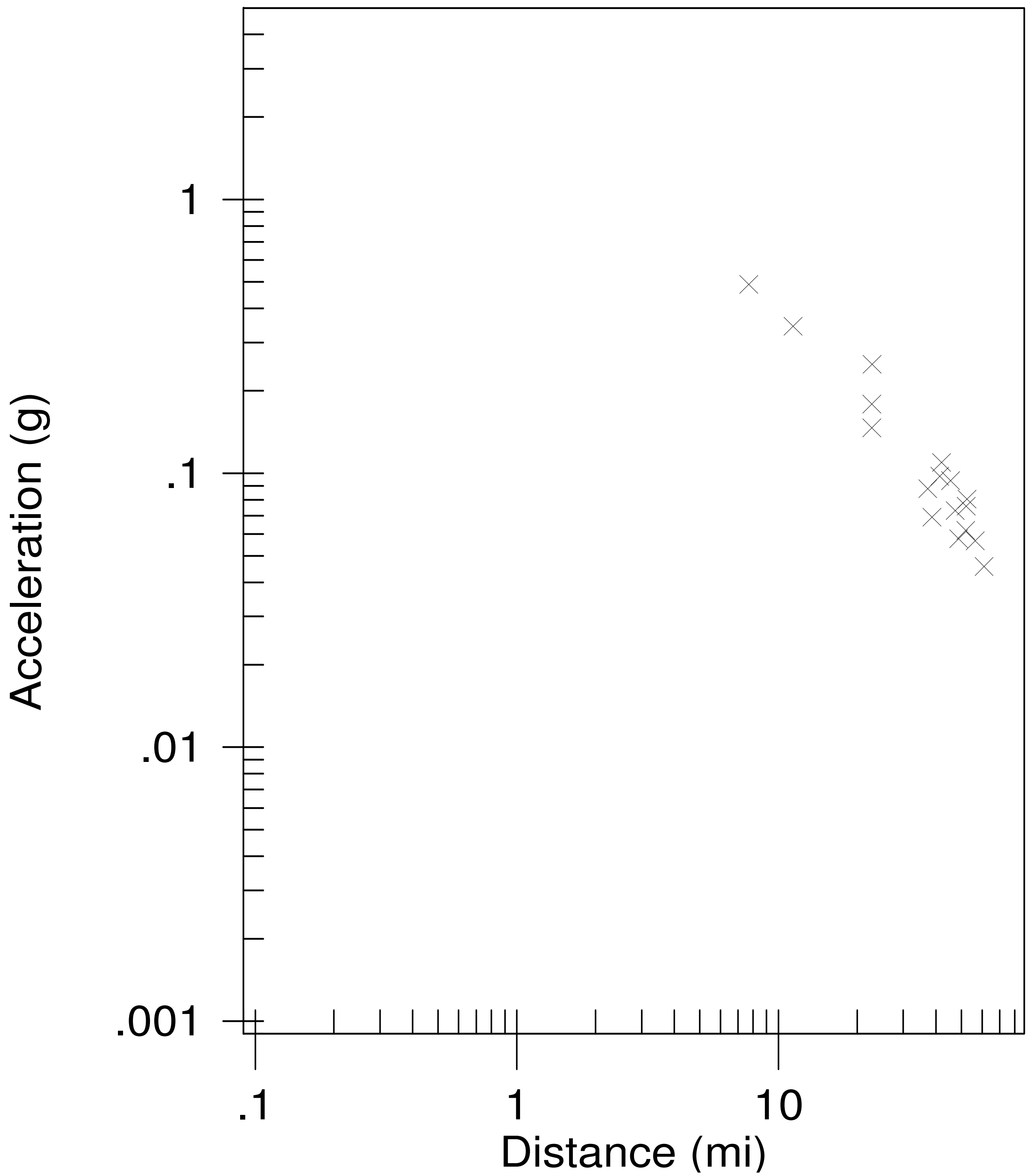
El Fuerte



W.O. 7795-A-SC
PLATE D-3

MAXIMUM EARTHQUAKES

El Fuerte



W.O. 7795-A-SC
PLATE D-4

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*                       *
*    E Q S E A R C H    *
*                       *
*     Version 3.00      *
*                       *
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ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 7795-A-SC

DATE: 05-06-2020

JOB NAME: EL FUERTE

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

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MAGNITUDE RANGE:
   MINIMUM MAGNITUDE:  4.00
   MAXIMUM MAGNITUDE:  9.00

SITE COORDINATES:
   SITE LATITUDE:  33.1115
   SITE LONGITUDE:  117.2431

SEARCH DATES:
           START DATE:   1800
           END DATE:   2020

SEARCH RADIUS:
           62.1 mi
           100.0 km

ATTENUATION RELATION:  12) Bozorgnia Campbell Niazi (1999) Hor.-Soft Rock-Cor.
   UNCERTAINTY (M=Median, S=Sigma): S       Number of Sigmas:  1.0
   ASSUMED SOURCE TYPE:  DS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
   SCOND:   0  Depth Source:  A
   Basement Depth:  5.00 km     Campbell SSR:  1     Campbell SHR:  0
   COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km):  3.0
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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	0.442	X	8.4(13.5)
MGI	33.2000	117.0000	07/20/1923	7 0 0.0	0.0	4.00	0.059	VI	15.3(24.7)
MGI	33.0000	117.0000	12/29/1914	10 0 0.0	0.0	4.00	0.056	VI	16.0(25.8)
MGI	33.0000	117.0000	09/21/1856	730 0.0	0.0	5.00	0.095	VII	16.0(25.8)
DMG	33.0000	117.0000	03/03/1906	2025 0.0	0.0	4.50	0.072	VII	16.0(25.8)
DMG	33.2670	117.0170	06/07/1935	1633 0.0	0.0	4.00	0.053	VI	16.9(27.2)
DMG	32.8500	117.4830	02/23/1943	92112.0	0.0	4.00	0.040	V	22.8(36.7)
MGI	32.8000	117.1000	05/25/1803	0 0 0.0	0.0	5.00	0.066	VI	23.0(37.1)
MGI	33.1000	116.8000	06/22/1918	557 0.0	0.0	4.00	0.035	V	25.6(41.3)
GSP	33.0700	116.8000	12/04/1991	071057.5	15.0	4.20	0.039	V	25.8(41.5)
MGI	32.7000	117.2000	05/20/1920	1330 0.0	0.0	4.00	0.031	V	28.5(45.9)
DMG	32.7000	117.2000	05/27/1862	20 0 0.0	0.0	5.90	0.091	VII	28.5(45.9)
MGI	32.7000	117.2000	09/08/1915	742 0.0	0.0	4.00	0.031	V	28.5(45.9)
MGI	32.7000	117.2000	04/19/1906	028 0.0	0.0	4.30	0.037	V	28.5(45.9)
T-A	33.5000	117.0700	12/29/1880	7 0 0.0	0.0	4.30	0.036	V	28.6(46.1)
DMG	33.5000	117.0000	08/08/1925	1013 0.0	0.0	4.50	0.038	V	30.3(48.7)
PAS	32.6790	117.1510	06/18/1985	32228.7	5.7	4.00	0.029	V	30.3(48.8)
PAS	32.9470	117.7360	01/15/1989	153955.2	6.0	4.20	0.032	V	30.7(49.4)
T-A	32.6700	117.1700	10/21/1862	0 0 0.0	0.0	5.00	0.049	VI	30.8(49.5)
T-A	32.6700	117.1700	12/00/1856	0 0 0.0	0.0	5.00	0.049	VI	30.8(49.5)
T-A	32.6700	117.1700	04/15/1865	840 0.0	0.0	4.30	0.034	V	30.8(49.5)
T-A	32.6700	117.1700	05/24/1865	0 0 0.0	0.0	5.00	0.049	VI	30.8(49.5)
T-A	32.6700	117.1700	01/25/1863	1020 0.0	0.0	4.30	0.034	V	30.8(49.5)
DMG	33.2000	116.7200	05/12/1930	172548.5	0.0	4.20	0.032	V	30.8(49.6)
DMG	33.4540	116.8980	07/29/1936	142252.8	10.0	4.00	0.029	V	30.9(49.7)
DMG	33.4560	116.8960	06/16/1938	55916.9	10.0	4.00	0.029	V	31.1(50.0)
DMG	33.2000	116.7000	01/01/1920	235 0.0	0.0	5.00	0.047	VI	32.0(51.5)
DMG	33.5000	116.9170	11/04/1935	355 0.0	0.0	4.50	0.035	V	32.8(52.7)
DMG	32.8000	116.8000	10/23/1894	23 3 0.0	0.0	5.70	0.068	VI	33.5(53.9)
MGI	32.8000	116.8000	08/14/1927	1448 0.0	0.0	4.60	0.036	V	33.5(53.9)
USG	33.0170	117.8170	07/14/1986	11112.6	10.0	4.12	0.028	V	33.8(54.5)
USG	33.0170	117.8170	07/16/1986	1247 3.7	10.0	4.11	0.028	V	33.8(54.5)
PAS	32.9700	117.8030	07/14/1986	03246.2	10.0	4.00	0.026	V	33.8(54.5)
GSP	32.9700	117.8100	04/04/1990	085439.3	6.0	4.00	0.026	V	34.2(55.1)
PAS	32.6270	117.3770	06/29/1983	8 836.4	5.0	4.60	0.035	V	34.3(55.3)
GSP	32.9850	117.8180	06/21/1995	211736.2	6.0	4.30	0.030	V	34.4(55.4)
PAS	32.9450	117.8060	09/07/1984	11 313.4	6.0	4.30	0.030	V	34.5(55.6)
PAS	32.6150	117.1520	10/29/1986	23815.3	14.6	4.10	0.027	V	34.7(55.8)
DMG	33.1000	116.6330	02/08/1952	174028.0	0.0	4.00	0.025	V	35.3(56.8)
PAS	32.9860	117.8440	10/01/1986	201218.6	6.0	4.00	0.025	V	35.8(57.7)
PAS	32.9450	117.8310	07/29/1986	81741.8	10.0	4.10	0.026	V	35.9(57.8)
PAS	32.9900	117.8490	07/13/1986	14 133.0	12.0	4.60	0.033	V	36.1(58.0)
PAS	32.9330	117.8410	07/29/1986	81741.6	10.0	4.30	0.028	V	36.7(59.1)

MGI	33.5000	116.8000	06/02/1917	435 0.0	0.0	4.00	0.024	V	37.1(59.6)
MGI	33.5000	116.8000	11/26/1916	17 5 0.0	0.0	4.00	0.024	V	37.1(59.6)
MGI	33.5000	116.8000	03/30/1918	16 5 0.0	0.0	4.60	0.033	V	37.1(59.6)
MGI	33.5000	116.8000	05/31/1917	435 0.0	0.0	4.00	0.024	V	37.1(59.6)
MGI	33.1000	116.6000	02/05/1922	1915 0.0	0.0	4.00	0.024	IV	37.2(59.9)
MGI	33.1000	116.6000	08/19/1917	710 0.0	0.0	4.00	0.024	IV	37.2(59.9)
MGI	33.1000	116.6000	05/11/1915	1145 0.0	0.0	4.00	0.024	IV	37.2(59.9)
MGI	33.1000	116.6000	02/09/1920	220 0.0	0.0	4.00	0.024	IV	37.2(59.9)
MGI	33.1000	116.6000	03/04/1915	1250 0.0	0.0	4.00	0.024	IV	37.2(59.9)
MGI	33.1000	116.6000	05/28/1917	1017 0.0	0.0	4.00	0.024	IV	37.2(59.9)

EARTHQUAKE SEARCH RESULTS

Page 2

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]
MGI	33.1000	116.6000	08/10/1921	19 6 0.0	0.0	4.00	0.024	IV	37.2(59.9)
MGI	33.1000	116.6000	08/10/1921	2151 0.0	0.0	4.00	0.024	IV	37.2(59.9)
MGI	33.1000	116.6000	02/16/1915	1330 0.0	0.0	4.00	0.024	IV	37.2(59.9)
DMG	33.4880	116.7770	06/12/1959	11 313.0	5.7	4.00	0.024	IV	37.4(60.2)
PAS	32.9710	117.8700	07/13/1986	1347 8.2	6.0	5.30	0.047	VI	37.6(60.4)
MGI	33.2000	116.6000	10/12/1920	1748 0.0	0.0	5.30	0.047	VI	37.7(60.6)
PAS	33.4200	116.6980	06/05/1978	16 3 3.9	11.9	4.40	0.029	V	38.0(61.1)
MGI	33.0000	116.6000	06/11/1917	354 0.0	0.0	4.00	0.023	IV	38.0(61.2)
DMG	33.1500	116.5830	12/02/1935	319 0.0	0.0	4.00	0.023	IV	38.3(61.6)
DMG	33.4500	116.6830	04/25/1955	25515.0	0.0	4.00	0.022	IV	39.9(64.2)
DMG	32.8000	117.8330	01/24/1942	214148.0	0.0	4.00	0.022	IV	40.4(65.0)
DMG	33.4830	116.7000	12/28/1948	125341.0	0.0	4.00	0.022	IV	40.5(65.2)
PAS	33.0330	117.9440	02/22/1983	21830.4	10.0	4.30	0.025	V	40.9(65.8)
DMG	33.7000	117.1000	06/11/1902	245 0.0	0.0	4.50	0.027	V	41.5(66.7)
DMG	33.7000	117.4000	05/15/1910	1547 0.0	0.0	6.00	0.065	VI	41.6(67.0)
DMG	33.7000	117.4000	05/13/1910	620 0.0	0.0	5.00	0.036	V	41.6(67.0)
DMG	33.7000	117.4000	04/11/1910	757 0.0	0.0	5.00	0.036	V	41.6(67.0)
DMG	33.1100	116.5230	01/24/1957	205449.9	3.9	4.60	0.029	V	41.6(67.0)
MGI	32.7000	116.7000	03/21/1918	2325 0.0	0.0	4.00	0.021	IV	42.4(68.2)
DMG	33.4670	116.6330	02/20/1934	1035 0.0	0.0	4.00	0.021	IV	42.9(69.1)
PAS	33.1380	116.5010	10/10/1984	212258.9	11.6	4.50	0.026	V	42.9(69.1)
DMG	33.1670	116.5000	06/23/1932	23037.1	0.0	4.00	0.020	IV	43.1(69.4)
DMG	33.1670	116.5000	06/23/1932	22552.7	0.0	4.00	0.020	IV	43.1(69.4)
DMG	33.6820	117.5530	07/05/1938	18 655.7	10.0	4.50	0.026	V	43.2(69.6)
DMG	33.7380	117.1870	04/27/1962	91232.1	5.7	4.10	0.021	IV	43.4(69.8)
DMG	33.6990	117.5110	05/31/1938	83455.4	10.0	5.50	0.046	VI	43.4(69.8)
DMG	32.7170	117.8330	11/06/1950	205546.0	0.0	4.40	0.025	V	43.7(70.3)
DMG	33.4000	116.5670	02/04/1953	43616.0	0.0	4.30	0.023	IV	43.8(70.5)
DMG	33.5450	117.8070	10/27/1969	1316 2.3	6.5	4.50	0.026	V	44.2(71.1)
DMG	33.4170	116.5670	12/22/1950	2 536.0	0.0	4.00	0.020	IV	44.4(71.4)
DMG	33.7170	117.5070	08/06/1938	22 056.0	10.0	4.00	0.020	IV	44.5(71.6)
DMG	33.5080	116.6310	08/11/1967	05711.4	10.7	4.10	0.021	IV	44.7(71.9)
DMG	33.7170	117.5170	06/19/1935	1117 0.0	0.0	4.00	0.020	IV	44.7(71.9)
DMG	33.7330	117.4670	10/26/1954	162226.0	0.0	4.10	0.021	IV	44.8(72.1)
DMG	33.7250	117.4980	01/03/1956	02548.9	13.7	4.70	0.028	V	44.8(72.1)
DMG	33.1670	116.4670	08/01/1960	193930.0	0.0	4.20	0.022	IV	45.0(72.5)
DMG	33.7100	116.9250	09/23/1963	144152.6	16.5	5.00	0.033	V	45.2(72.7)
DMG	33.4670	116.5830	03/27/1937	528 0.0	0.0	4.00	0.019	IV	45.3(72.9)
DMG	33.4670	116.5830	01/04/1938	029 0.0	0.0	4.50	0.025	V	45.3(72.9)

DMG	33.4670	116.5830	03/26/1937	2124 0.0	0.0	4.00	0.019	IV	45.3(72.9)
DMG	33.4670	116.5830	03/27/1937	742 0.0	0.0	4.50	0.025	V	45.3(72.9)
PAS	33.5580	116.6670	06/15/1982	234921.3	12.2	4.80	0.029	V	45.3(72.9)
PAS	32.7590	117.9060	10/18/1976	172753.1	13.8	4.20	0.021	IV	45.5(73.2)
DMG	33.5330	116.6330	09/21/1942	7 754.0	0.0	4.00	0.019	IV	45.7(73.5)
DMG	33.1000	116.4500	11/23/1953	1339 7.0	0.0	4.30	0.022	IV	45.9(73.8)
DMG	33.7480	117.4790	06/22/1971	104119.0	8.0	4.20	0.021	IV	46.0(74.0)
DMG	33.0970	116.4440	08/18/1959	215221.3	17.3	4.30	0.022	IV	46.2(74.4)
DMG	33.7500	117.0000	04/21/1918	223225.0	0.0	6.80	0.099	VII	46.3(74.4)
DMG	33.7500	117.0000	06/06/1918	2232 0.0	0.0	5.00	0.032	V	46.3(74.4)
DMG	33.5060	116.5850	05/21/1967	144234.4	19.4	4.70	0.027	V	46.7(75.2)
DMG	33.6500	116.7500	09/05/1950	191956.0	0.0	4.80	0.028	V	46.8(75.3)
PAS	33.7010	116.8370	08/22/1979	2 136.3	5.0	4.10	0.020	IV	46.9(75.6)
GSP	33.6320	116.7190	07/19/1999	220927.5	14.0	4.20	0.021	IV	47.0(75.6)

EARTHQUAKE SEARCH RESULTS

Page 3

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]
GSP	33.6500	116.7400	12/02/1989	231647.8	14.0	4.20	0.021	IV	47.2(75.9)
DMG	33.4000	116.5000	10/11/1918	4 0 0.0	0.0	4.00	0.019	IV	47.3(76.1)
DMG	33.0020	116.4360	07/02/1957	65638.5	12.8	4.10	0.019	IV	47.3(76.1)
PAS	32.7140	117.9100	10/18/1976	172652.6	15.1	4.20	0.020	IV	47.4(76.3)
DMG	33.0000	116.4330	06/04/1940	1035 8.3	0.0	5.10	0.033	V	47.5(76.4)
DMG	33.1170	116.4170	10/21/1940	64933.0	0.0	4.50	0.024	IV	47.8(76.9)
DMG	33.1170	116.4170	06/04/1940	103656.0	0.0	4.00	0.018	IV	47.8(76.9)
DMG	33.1670	116.4170	07/10/1938	18 6 0.0	0.0	4.00	0.018	IV	47.9(77.1)
DMG	33.1670	116.4170	12/05/1939	173352.0	0.0	4.00	0.018	IV	47.9(77.1)
DMG	33.1670	116.4170	10/14/1935	1550 0.0	0.0	4.00	0.018	IV	47.9(77.1)
DMG	33.4200	116.4900	03/29/1937	17 316.8	10.0	4.00	0.018	IV	48.4(77.9)
PAS	33.5200	116.5580	08/02/1975	014 7.7	13.4	4.70	0.026	V	48.6(78.1)
DMG	32.5830	117.8000	04/19/1939	741 0.0	0.0	4.50	0.023	IV	48.7(78.4)
GSP	33.1100	116.4000	04/01/1984	071702.3	11.0	4.00	0.018	IV	48.8(78.5)
DMG	33.5340	116.5610	09/23/1956	112441.9	12.2	4.30	0.021	IV	49.0(78.8)
DMG	33.3330	116.4330	02/12/1954	94428.0	0.0	4.50	0.023	IV	49.2(79.2)
PAS	33.4840	116.5130	08/11/1976	152455.5	15.4	4.30	0.021	IV	49.4(79.4)
DMG	33.3680	116.4440	03/25/1937	232026.7	10.0	4.00	0.018	IV	49.4(79.5)
DMG	33.8000	117.0000	12/25/1899	1225 0.0	0.0	6.40	0.070	VI	49.6(79.7)
PAS	32.7560	117.9880	01/12/1975	212214.8	15.3	4.80	0.027	V	49.7(79.9)
DMG	33.2670	116.4000	06/06/1940	2321 4.0	0.0	4.00	0.018	IV	49.9(80.3)
DMG	33.1830	116.3830	10/14/1949	02925.0	0.0	4.10	0.018	IV	50.0(80.4)
DMG	33.4830	116.5000	02/15/1951	104759.0	0.0	4.80	0.026	V	50.0(80.4)
DMG	33.4830	116.5000	02/15/1951	104957.0	0.0	4.80	0.026	V	50.0(80.4)
PAS	33.5010	116.5130	02/25/1980	104738.5	13.6	5.50	0.039	V	50.0(80.4)
PDP	33.5080	116.5140	10/31/2001	075616.6	15.0	5.10	0.031	V	50.2(80.8)
DMG	33.5000	116.5000	09/30/1916	211 0.0	0.0	5.00	0.029	V	50.6(81.4)
DMG	33.8330	117.4000	06/05/1940	82727.0	0.0	4.00	0.017	IV	50.6(81.5)
DMG	32.9670	116.3830	10/31/1942	15 758.0	0.0	4.00	0.017	IV	50.8(81.7)
DMG	33.0380	116.3610	02/26/1957	211652.2	0.0	4.10	0.018	IV	51.3(82.5)
DMG	33.5000	116.4830	02/23/1941	183614.0	0.0	4.50	0.022	IV	51.4(82.7)
MGI	33.8000	116.9000	04/29/1918	2 0 0.0	0.0	4.00	0.017	IV	51.5(82.8)
MGI	33.8000	116.9000	06/14/1918	1024 0.0	0.0	4.00	0.017	IV	51.5(82.8)
MGI	33.8000	116.9000	04/23/1918	1415 0.0	0.0	4.00	0.017	IV	51.5(82.8)
MGI	33.8000	116.9000	12/18/1920	1726 0.0	0.0	4.00	0.017	IV	51.5(82.8)

GSP	33.6200	117.9000	04/07/1989	200730.2	13.0	4.50	0.022	IV	51.6(83.1)
DMG	33.1210	116.3490	05/25/1971	10 252.9	8.0	4.10	0.018	IV	51.7(83.2)
DMG	33.8000	117.6000	09/16/1903	1210 0.0	0.0	4.00	0.017	IV	51.8(83.3)
MGI	33.8000	117.6000	04/22/1918	2115 0.0	0.0	5.00	0.028	V	51.8(83.3)
DMG	33.4170	116.4170	01/02/1943	141118.0	0.0	4.50	0.022	IV	52.1(83.9)
DMG	33.4260	116.4210	03/25/1937	20 4 8.3	10.0	4.00	0.017	IV	52.2(84.0)
PAS	33.4580	116.4340	02/12/1979	44842.3	3.9	4.20	0.018	IV	52.5(84.4)
DMG	33.4670	116.4330	05/12/1939	1925 2.2	0.0	4.50	0.021	IV	52.8(85.0)
DMG	33.2830	116.3500	04/13/1949	75336.0	0.0	4.10	0.017	IV	52.9(85.2)
DMG	33.5670	117.9830	04/17/1934	1833 0.0	0.0	4.00	0.016	IV	53.0(85.3)
DMG	33.5670	117.9830	07/07/1937	1112 0.0	0.0	4.00	0.016	IV	53.0(85.3)
PAS	33.4830	116.4380	07/02/1988	02658.2	12.6	4.00	0.016	IV	53.1(85.4)
PAS	33.4710	118.0610	02/27/1984	101815.0	6.0	4.00	0.016	IV	53.3(85.8)
DMG	33.5750	117.9830	03/11/1933	518 4.0	0.0	5.20	0.031	V	53.3(85.8)
GSP	33.5100	116.4500	02/18/1990	155259.9	9.0	4.10	0.017	IV	53.4(85.9)
DMG	33.5010	116.4290	02/23/1971	0 739.2	8.0	4.20	0.018	IV	54.1(87.1)
DMG	33.3430	116.3460	04/28/1969	232042.9	20.0	5.80	0.044	VI	54.2(87.3)
DMG	33.0530	116.3060	04/02/1967	201538.6	1.0	4.30	0.019	IV	54.4(87.5)

EARTHQUAKE SEARCH RESULTS

Page 4

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	33.6170	117.9670	03/11/1933	154 7.8	0.0	6.30	0.060	VI	54.4(87.6)
DMG	33.9000	117.2000	12/19/1880	0 0 0.0	0.0	6.00	0.049	VI	54.5(87.7)
GSP	32.7260	118.0680	12/27/2000	002714.1	6.0	4.10	0.017	IV	54.7(88.1)
DMG	33.2000	116.3000	05/12/1930	414 0.0	0.0	4.00	0.016	IV	54.9(88.3)
DMG	33.2910	116.3170	03/19/1966	142156.0	10.9	4.00	0.016	IV	54.9(88.4)
GSP	33.3990	116.3540	07/26/1997	031456.0	11.0	4.80	0.024	IV	55.0(88.6)
PAS	33.5080	118.0710	11/20/1988	53928.7	6.0	4.50	0.020	IV	55.1(88.6)
GDP	33.8060	117.7150	03/07/2000	002028.2	11.0	4.00	0.016	IV	55.1(88.7)
DMG	33.6000	118.0000	03/11/1933	217 0.0	0.0	4.50	0.020	IV	55.2(88.8)
DMG	33.6000	118.0000	03/11/1933	231 0.0	0.0	4.40	0.019	IV	55.2(88.8)
DMG	33.3670	118.1500	04/16/1942	72833.0	0.0	4.00	0.016	IV	55.3(88.9)
MGI	33.7000	117.9000	07/08/1902	945 0.0	0.0	4.00	0.016	IV	55.5(89.4)
PAS	32.6250	118.0090	07/11/1981	215029.4	5.0	4.30	0.018	IV	55.7(89.6)
MGI	32.6000	116.5000	05/03/1918	425 0.0	0.0	4.00	0.016	IV	55.7(89.7)
PAS	33.4600	116.3700	09/07/1984	175730.3	15.2	4.10	0.016	IV	55.8(89.9)
DMG	33.6000	118.0170	12/25/1935	1715 0.0	0.0	4.50	0.020	IV	55.9(90.0)
DMG	33.3150	116.3050	04/09/1968	1831 3.8	12.6	4.70	0.022	IV	56.0(90.1)
DMG	33.3000	116.3000	01/04/1940	8 711.0	0.0	4.00	0.016	IV	56.0(90.1)
DMG	33.7670	117.8170	08/22/1936	521 0.0	0.0	4.00	0.016	IV	56.0(90.2)
DMG	32.9610	116.2900	08/25/1971	23 033.0	8.0	4.00	0.015	IV	56.1(90.3)
DMG	33.5610	118.0580	01/15/1937	183547.0	10.0	4.00	0.015	IV	56.3(90.6)
DMG	33.3330	116.3000	08/05/1933	2331 0.0	0.0	4.40	0.019	IV	56.6(91.0)
DMG	33.3330	116.3000	08/06/1933	332 0.0	0.0	4.70	0.022	IV	56.6(91.0)
DMG	33.6170	118.0170	03/14/1933	19 150.0	0.0	5.10	0.027	V	56.7(91.2)
DMG	33.6170	118.0170	03/15/1933	111332.0	0.0	4.90	0.024	V	56.7(91.2)
DMG	33.6170	118.0170	10/02/1933	1326 1.0	0.0	4.00	0.015	IV	56.7(91.2)
DMG	32.6800	118.0770	10/28/1973	22 0 2.7	8.0	4.50	0.020	IV	56.8(91.4)
DMG	33.5170	118.1000	03/22/1941	82240.0	0.0	4.00	0.015	IV	56.8(91.4)
DMG	32.9520	116.2790	09/13/1973	173039.8	8.0	4.80	0.023	IV	56.9(91.5)
DMG	33.8000	116.7000	08/11/1911	2340 0.0	0.0	4.50	0.020	IV	56.9(91.6)
DMG	33.8000	116.7000	08/11/1911	1820 0.0	0.0	4.00	0.015	IV	56.9(91.6)

DMG	33.6590	117.9810	10/20/1961	20 714.5	6.1	4.00	0.015	IV	56.9(91.6)
DMG	32.9900	116.2680	11/08/1958	132044.1	2.4	4.10	0.016	IV	57.0(91.8)
DMG	33.0430	116.2600	08/22/1961	231933.6	12.1	4.40	0.019	IV	57.1(91.8)
DMG	33.6650	117.9790	10/20/1961	214240.7	7.2	4.00	0.015	IV	57.1(91.9)
DMG	33.2350	116.2660	04/09/1968	93833.0	5.2	4.00	0.015	IV	57.1(91.9)
DMG	33.9330	117.3670	10/24/1943	02921.0	0.0	4.00	0.015	IV	57.2(92.0)
DMG	33.6540	117.9940	10/20/1961	194950.5	4.6	4.30	0.018	IV	57.2(92.1)
MGI	33.8000	117.8000	11/07/1926	1948 0.0	0.0	4.60	0.021	IV	57.3(92.3)
MGI	33.8000	117.8000	11/09/1926	1535 0.0	0.0	4.60	0.021	IV	57.3(92.3)
MGI	33.8000	117.8000	05/19/1917	719 0.0	0.0	4.00	0.015	IV	57.3(92.3)
MGI	33.8000	117.8000	05/20/1917	945 0.0	0.0	4.00	0.015	IV	57.3(92.3)
MGI	33.8000	117.8000	05/19/1917	635 0.0	0.0	4.00	0.015	IV	57.3(92.3)
MGI	33.8000	117.8000	11/10/1926	1723 0.0	0.0	4.60	0.021	IV	57.3(92.3)
MGI	33.8000	117.8000	11/04/1926	2238 0.0	0.0	4.60	0.021	IV	57.3(92.3)
DMG	33.6170	118.0330	05/21/1938	944 0.0	0.0	4.00	0.015	IV	57.4(92.3)
DMG	32.9230	116.2720	10/14/1969	131842.7	10.0	4.50	0.019	IV	57.7(92.9)
DMG	33.4000	116.3000	02/09/1890	12 6 0.0	0.0	6.30	0.056	VI	58.0(93.3)
DMG	33.0500	116.2380	08/23/1961	1 047.8	11.9	4.70	0.021	IV	58.3(93.8)
GSP	32.6810	118.1090	06/20/1997	043540.5	6.0	4.70	0.021	IV	58.3(93.9)
DMG	33.6800	117.9930	11/20/1961	85334.7	4.4	4.00	0.015	IV	58.4(94.0)
DMG	32.9500	116.2500	11/14/1951	2355 3.0	0.0	4.10	0.016	IV	58.6(94.2)
DMG	33.2790	116.2490	01/07/1966	191023.0	-1.7	4.00	0.015	IV	58.6(94.3)

EARTHQUAKE SEARCH RESULTS

Page 5

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]
PAS	32.9050	116.2610	12/25/1975	71852.3	3.6	4.00	0.015	IV	58.6(94.3)
DMG	33.0330	116.2330	09/20/1961	5 410.0	0.0	4.00	0.015	IV	58.7(94.4)
DMG	33.2000	116.2330	04/05/1942	92039.0	0.0	4.00	0.015	IV	58.7(94.5)
DMG	33.6710	118.0120	10/20/1961	223534.2	5.6	4.10	0.015	IV	58.8(94.6)
DMG	33.8540	117.7520	10/04/1961	22131.6	4.3	4.10	0.015	IV	59.0(95.0)
DMG	33.0190	116.2250	08/20/1969	152957.2	0.6	4.00	0.015	IV	59.3(95.4)
DMG	33.0210	116.2230	01/13/1963	23938.9	13.0	4.20	0.016	IV	59.4(95.5)
DMG	32.6800	116.3540	01/21/1970	1124 0.4	8.0	4.10	0.015	IV	59.5(95.8)
GSP	32.6850	118.1380	06/20/1997	053855.0	6.0	4.20	0.016	IV	59.7(96.0)
PAS	32.3020	116.8810	08/19/1978	931 5.7	19.8	4.10	0.015	IV	59.7(96.1)
PAS	33.0580	116.2110	03/22/1982	85328.6	4.6	4.50	0.019	IV	59.8(96.3)
DMG	33.3330	116.2360	10/05/1962	1529 2.6	13.9	4.10	0.015	IV	60.1(96.8)
DMG	33.8000	116.6000	09/10/1931	436 0.0	0.0	4.00	0.014	IV	60.3(97.0)
DMG	33.4080	116.2610	03/25/1937	1649 1.8	10.0	6.00	0.044	VI	60.3(97.0)
DMG	32.7180	118.1720	04/28/1938	6 728.0	10.0	4.50	0.018	IV	60.3(97.0)
DMG	33.3330	116.2330	06/09/1942	5 633.0	0.0	4.00	0.014	IV	60.3(97.1)
DMG	33.3100	116.2240	05/22/1968	132655.4	7.5	4.40	0.018	IV	60.4(97.3)
DMG	33.2000	116.2000	05/28/1892	1115 0.0	0.0	6.30	0.053	VI	60.6(97.5)
DMG	32.8670	118.2500	02/13/1952	151337.0	0.0	4.70	0.020	IV	60.7(97.7)
MGI	33.8000	117.9000	05/22/1902	740 0.0	0.0	4.30	0.017	IV	60.8(97.8)
DMG	32.7500	118.2000	06/25/1939	149 0.0	0.0	4.50	0.018	IV	60.8(97.9)
DMG	33.6830	118.0500	03/11/1933	1250 0.0	0.0	4.40	0.017	IV	61.0(98.2)
DMG	33.6830	118.0500	03/11/1933	658 3.0	0.0	5.50	0.032	V	61.0(98.2)
DMG	33.9960	117.2700	02/17/1952	123658.3	16.0	4.50	0.018	IV	61.1(98.3)
DMG	33.9500	117.5830	04/11/1941	12024.0	0.0	4.00	0.014	IV	61.1(98.3)
DMG	33.6170	118.1170	01/20/1934	2117 0.0	0.0	4.50	0.018	IV	61.3(98.6)
T-A	32.2500	117.5000	01/13/1877	20 0 0.0	0.0	5.00	0.024	IV	61.3(98.7)

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DMG |34.0000|117.2500|11/01/1932| 445 0.0|  0.0| 4.00| 0.014 | IV | 61.3( 98.7)
DMG |34.0000|117.2500|07/23/1923| 73026.0|  0.0| 6.25| 0.051 | VI | 61.3( 98.7)
DMG |34.0000|117.2830|11/07/1939|1852 8.4|  0.0| 4.70| 0.020 | IV | 61.4( 98.8)
DMG |33.2370|116.1900|04/14/1968|125558.7| 10.8| 4.30| 0.016 | IV | 61.5( 98.9)
DMG |32.7000|116.3000|02/24/1892| 720 0.0|  0.0| 6.70| 0.068 | VI | 61.6( 99.1)
MGI |34.0000|117.4000|05/22/1907| 652 0.0|  0.0| 4.60| 0.019 | IV | 62.0( 99.8)
DMG |33.7500|118.0000|11/16/1934|2126 0.0|  0.0| 4.00| 0.014 | IV | 62.0( 99.8)

******************************************************************************
-END OF SEARCH-   246 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH:   1800  TO  2020

LENGTH OF SEARCH TIME:   221  years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 8.4 MILES (13.5 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 6.8

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.442 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
  a-value=  2.713
  b-value=  0.685
  beta-value=  1.576


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TABLE OF MAGNITUDES AND EXCEEDANCES:
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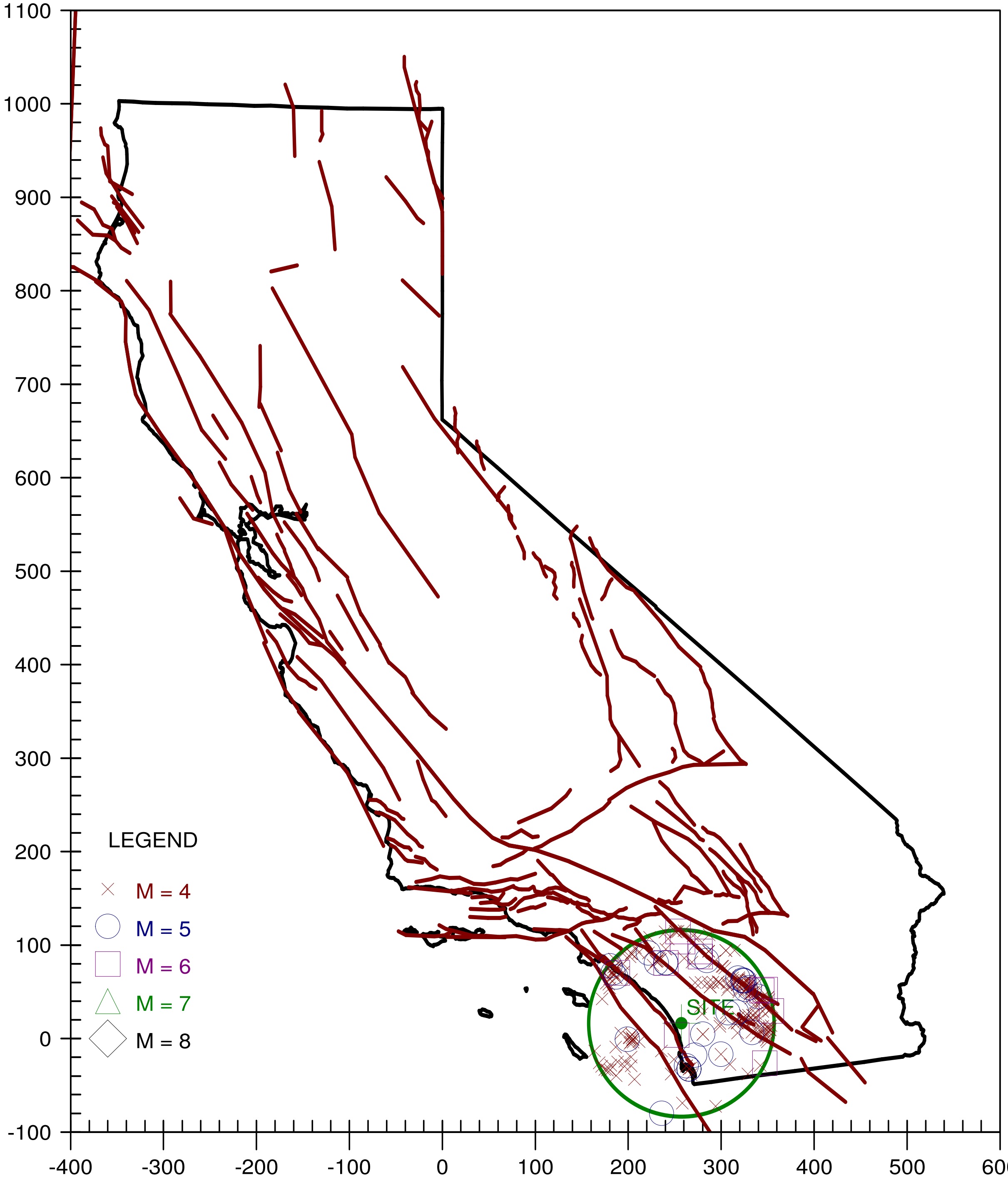
```

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	246	1.11312
4.5	90	0.40724
5.0	36	0.16290
5.5	17	0.07692
6.0	11	0.04977
6.5	3	0.01357

EARTHQUAKE EPICENTER MAP

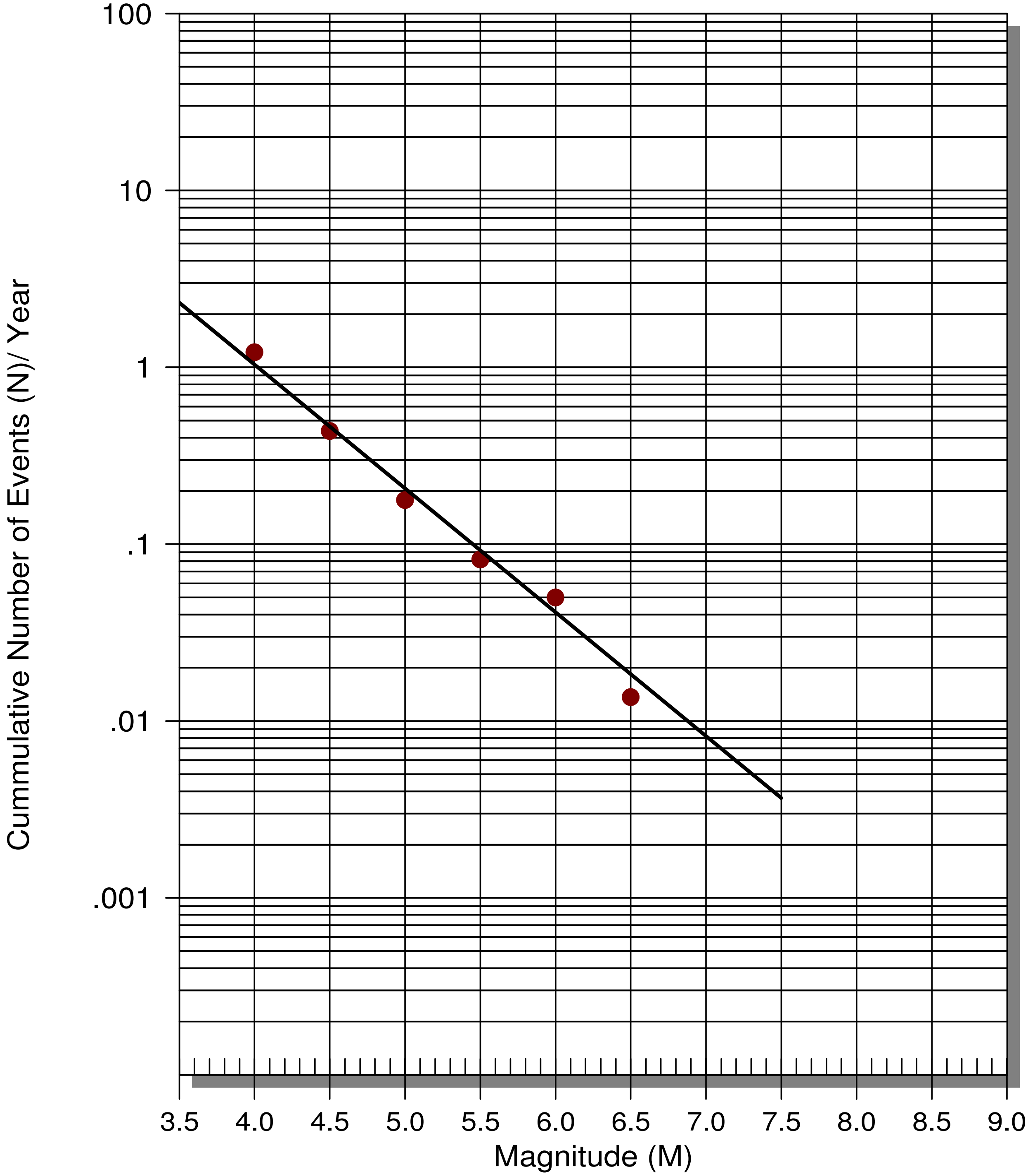
El Fuerte



W.O. 7795-A-SC
PLATE D-12

EARTHQUAKE RECURRENCE CURVE

El Fuerte

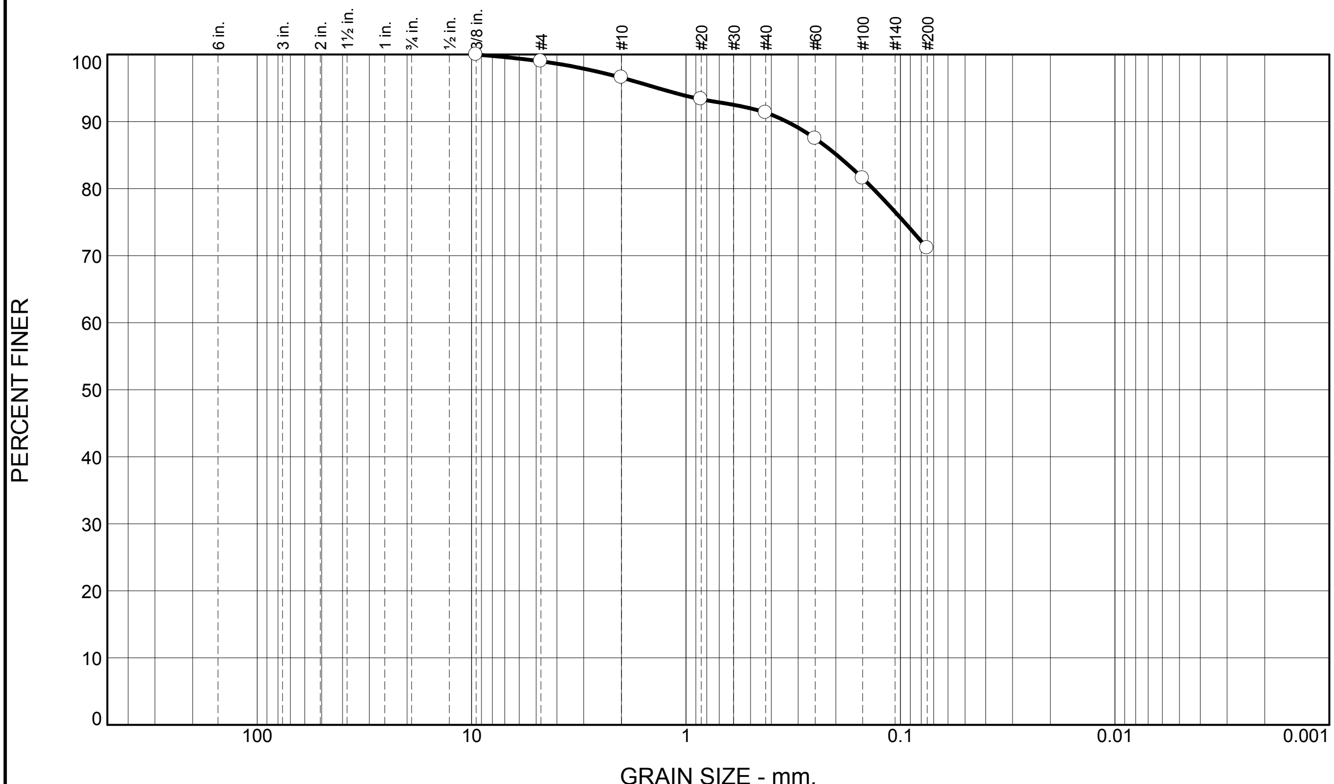


W.O. 7795-A-SC
PLATE D-13

APPENDIX E

LABORATORY DATA

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.0	2.4	5.3	20.2	71.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375	100.0		
#4	99.0		
#10	96.6		
#20	93.3		
#40	91.3		
#60	87.5		
#100	81.5		
#200	71.1		

* (no specification provided)

Soil Description

Yellowish Brown Sandy Clay

Atterberg Limits

PL= 16 LL= 65 PI= 49

Coefficients

D_{90}= 0.3389 D_{85}= 0.1979 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CH AASHTO= A-7-6(34)

Remarks

Source of Sample: B-2 **Depth:** 5-10
Sample Number: B-2 **Date:** 3-9-20

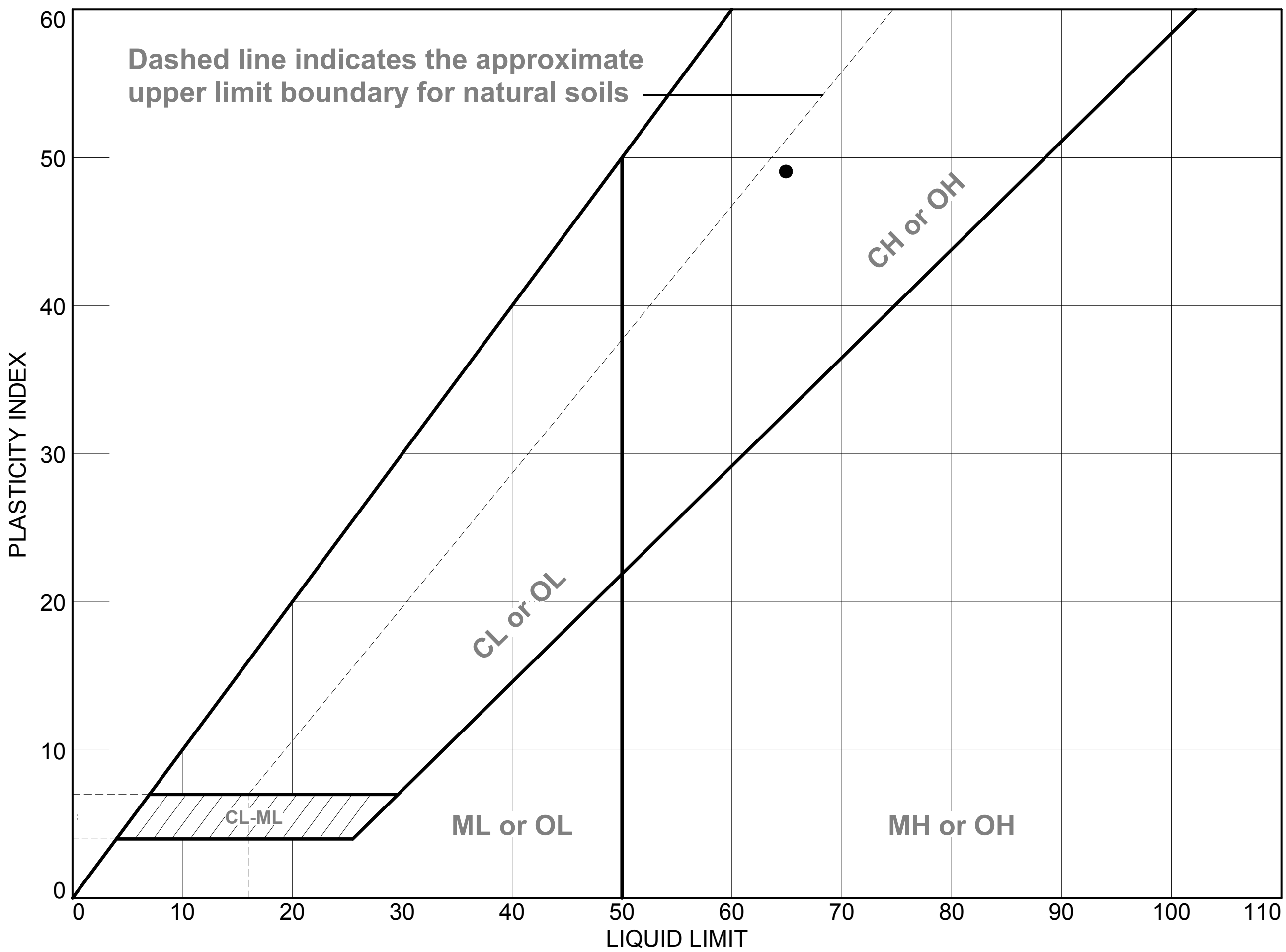
GeoSoils, Inc.

Client: C2 Fuerte Holdings, LLC
Project: 6450 El Fuerte St.

Project No: 7795-A-SC **Figure** E-1

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

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-2	B-2	5-10	9.0	16	65	49	CH

GeoSoils, Inc.

Client: C2 Fuerte Holdings, LLC
Project: 6450 El Fuerte St.

Project No.: 7795-A-SC **Figure** E-2

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

SoilCor
CORROSION & THERMAL SCIENCES
42184 Remington Ave, Temecula CA 92590
ph (951) 795-3135 • fx (951) 894-2683

Work Order No.: 20B9141
Client: GeoSoils, Inc.
Project No.: 7795-A-SC
Project Name: El Fuerte
Report Date: March 10, 2020

Laboratory Test(s) Results Summary

The subject soil sample was processed with the U.S. Standard No. 10 Sieve and tested for pH (ASTM G 51-95 2012), Soil Resistivity (ASTM G 57-06 2012), Sulfate Ion Content (ASTM D 516-16) and Chloride Ion Content (ASTM D 512-12B). The test results follow:

Sample Identification	pH (H+)	As Rec'd Resistivity (ohm-cm)	Saturated Resistivity (ohm-cm)	Sulfate Content (mg/L)	Chloride Content (mg/L)
B-2 @ 5-10ft	5.3	1,400	380	590	530

*ND=No Detection

We appreciate the opportunity to serve you. Please do not hesitate to contact us with any questions or clarifications regarding these results or procedures.

Ahmet K. Kaya, Laboratory Manager

APPENDIX F

GENERAL EARTHWORK, GRADING GUIDELINES AND PRELIMINARY CRITERIA

GENERAL EARTHWORK, GRADING GUIDELINES, AND PRELIMINARY CRITERIA

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 Code 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 conuded prior to placing fill. In-place existing fill, soil, 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 ods, 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 utilized 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 disosed 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.

PRELIMINARY OUTDOOR POOL/SPA DESIGN RECOMMENDATIONS

The following preliminary recommendations are provided for consideration in pool/spa design and planning. Actual recommendations should be provided by a qualified geotechnical consultant, based on site specific geotechnical conditions, including a subsurface investigation, differential settlement potential, expansive and corrosive soil potential, proximity of the proposed pool/spa to any slopes with regard to slope creep and lateral fill extension, as well as slope setbacks per Code, and geometry of the proposed improvements. Recommendations for pools/spas and/or deck flatwork underlain by expansive soils, or for areas with differential settlement greater than ¼-inch over 40 feet horizontally, will be more onerous than the preliminary recommendations presented below. The 1:1 (h:v) influence zone of any nearby retaining wall site structures should be delineated on the project civil drawings with the pool/spa. This 1:1 (h:v) zone is defined as a plane up from the lower-most heel of the retaining structure, to the daylight grade of the nearby building pad or slope. If pools/spas or associated pool/spa improvements are constructed within this zone, they should be re-positioned (horizontally or vertically) so that they are supported by earth materials that are outside or below this 1:1 plane. If this is not possible given the area of the building pad, the owner should consider eliminating these improvements or allow for increased potential for lateral/vertical deformations and associated distress that may render these improvements unusable in the future, unless they are periodically repaired and maintained. The conditions and recommendations presented herein should be disclosed to all homeowners and any interested/affected parties.

General

1. The equivalent fluid pressure to be used for the pool/spa design should be 60 pounds per cubic foot (pcf) for pool/spa walls with level backfill, and 75 pcf for a 2:1 sloped backfill condition. In addition, backdrains should be provided behind pool/spa walls subjacent to slopes.

2. Passive earth pressure may be computed as an equivalent fluid having a density of 150 pcf, to a maximum lateral earth pressure of 1,000 pounds per square foot (psf).

3. An allowable coefficient of friction between soil and concrete of 0.30 may be used with the dead load forces.

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

5. 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.

6. 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 pool/spa in cross section and plan view may affect the performance of the pool, from a geotechnical standpoint. Pools and spas should also be designed in accordance with the latest adopted Code. Minimally, the bottoms of the pools/spas, should maintain a distance 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.

7. The soil beneath the pool/spa bottom should be uniformly moist with the same stiffness throughout. If a fill/cut transition occurs beneath the pool/spa bottom, the cut portion should be overexcavated to a minimum depth of 48 inches, and replaced with compacted fill, such that there is a uniform blanket that is a minimum of 48 inches below the pool/spa shell. If very low expansive soil is used for fill, the fill should be placed at a minimum of 95 percent relative compaction, at optimum moisture conditions. This requirement should be 90 percent relative compaction at over optimum moisture if the pool/spa is constructed within or near expansive soils. The potential for grading and/or re-grading of the pool/spa bottom, and attendant potential for shoring and/or slot excavation, needs to be considered during all aspects of pool/spa planning, design, and construction.

8. If the pool/spa is founded entirely in compacted fill placed during rough grading, the deepest portion of the pool/spa should correspond with the thickest fill on the lot.

9. Hydrostatic pressure relief valves should be incorporated into the pool and spa designs. A pool/spa under-drain system is also recommended, with an appropriate outlet for discharge.

10. All fittings and pipe joints, particularly fittings in the side of the pool or 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 prevent water from seeping 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 minimally be 4 inches thick, and reinforced with No. 3 reinforcing bars at 18 inches on-center. All slab reinforcement should be supported to ensure proper mid-slab positioning during the placement of concrete. Wire mesh reinforcing is specifically not recommended. Deck slabs should not be tied to the pool/spa structure. Pre-moistening and/or pre-soaking of the slab subgrade is recommended, to a depth of 12 inches (optimum moisture content), or 18 inches (120 percent of the soil's optimum moisture content, or 3 percent over optimum moisture content, whichever is greater), for very low to low, and medium expansive soils, respectively. This moisture content should be maintained in the subgrade soils during concrete placement to promote uniform curing of the concrete and minimize the development of unsightly shrinkage cracks. Slab underlayment should consist of a 1- to 2-inch leveling course of sand (S.E.>30) and a minimum of 4 to 6 inches of ass 2 base compacted to 90 percent. 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 oser than H/3 or 7 feet (whichever is greater) from the slope face, in order to reduce, but not eliminate, this potential.

14. Pool/spa bottom or deck slabs should be founded entirely on competent bedrock, <u>or</u> properly compacted fill. Fill should be compacted to achieve a minimum 90 percent relative compaction, as discussed above. Prior to pouring concrete, subgrade soils below the pool/spa decking should be throughly watered to achieve a moisture content that is at least 2 percent above optimum moisture content, to a depth of at least 18 inches below the bottom of slabs. This moisture content should be maintained in the subgrade soils during concrete placement to promote uniform curing of the concrete and minimize the development of unsightly shrinkage cracks.

15. In order to reduce unsightly cracking, the outer edges of pool/spa decking to be bordered by landscaping, and the edges immediately adjacent to the pool/spa,

should be underlain by an 8-inch wide concrete cutoff shoulder (thickened edge) extending to a depth of at least 12 inches below the bottoms of the slabs to mitigate excessive infiltration of water under the pool/spa deck. These thickened edges should be reinforced with two No. 4 bars, one at the top and one at the bottom. Deck slabs may be minimally reinforced with No. 3 reinforcing bars placed at 18 inches on-center, in both directions. All slab reinforcement should be supported on chairs to ensure proper mid-slab positioning during the placement of concrete.

16. Surface and shrinkage cracking of the finish slab may be reduced if a low slump and water-cement ratio are maintained during concrete placement. Concrete utilized 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 concrete shrinkage cracking, however, is unavoidable.

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

18. Considering the nature of the onsite earth materials, it should be anticipated that 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 representative of the geotechnical consultant, including the project geologist and/or geotechnical engineer, prior to workers entering the excavation or trench, and <u>minimally</u> conform to Cal/OSHA ("Type B" soils may be assumed), state, and local safety codes. Should adverse conditions exist, appropriate recommendations should be offered at that time by the geotechnical consultant. GSI does not consult in the area of safety engineering and the safety of the construction crew is the responsibility of the pool/spa builder.

19. It is imperative that adequate provisions for surface drainage are incorporated by the homeowners into their overall improvement scheme. 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.

20. Regardless of the methods employed, once the pool/spa is filled with water, should it be emptied, there exists some potential that if emptied, significant distress may occur. Accordingly, once filled, the pool/spa should not be emptied unless evaluated by the geotechnical consultant and the pool/spa builder.

21. For pools/spas built within (all or part) of the Code setback and/or geotechnical setback, as indicated in the site geotechnical documents, special foundations are recommended to mitigate the affects of creep, lateral fill extension, expansive soils and settlement on the proposed pool/spa. Most municipalities or County reviewers

do not consider these effects in pool/spa plan approvals. As such, where pools/spas are proposed on 20 feet or more of fill, medium or highly expansive soils, or rock fill with limited "cap soils" and built within Code setbacks, or within the influence of the creep zone, or lateral fill extension, the following should be considered during design and construction:

> OPTION A: Shallow foundations with or without overexcavation of the pool/spa "shell," such that the pool/spa is surrounded by 5 feet of very low to low expansive soils (without irreducible parties greater that 6 inches), and the pool/spa walls oser to the slope(s) are designed to be free standing. GSI recommends a pool/spa under-drain or blanket system (see attached Typical Pool/Spa Detail). The pool/spa builders and owner in this optional construction technique should be generally satisfied with pool/spa performance under this scenario; however, some settlement, tilting, cracking, and leakage of the pool/spa is likely over the life of the project.

> OPTION B: Pier supported pool/spa foundations with or without overexcavation of the pool/spa shell such that the pool/spa is surrounded by 5 feet of very low to low expansive soils (without irreducible parties greater than 6 inches), and the pool/spa walls oser to the slope(s) are designed to be free standing. The need for a pool/spa under-drain system may be installed for leak detection purposes. Piers that support the pool/spa should be a minimum of 12 inches in diameter and at a spacing to provide vertical and lateral support of the pool/spa, in accordance with the pool/spa designers recommendations current applicable Codes. The pool/spa builder and owner in this second scenario construction technique should be more satisfied with pool/spa performance. This construction will reduce settlement and creep effects on the pool/spa; however, it will not eliminate these potentials, nor make the pool/spa "leak-free."

22. The temperature of the water lines for spas and pools may affect the corrosion properties of site 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.

23. All pool/spa utility trenches should be compacted to 90 percent of the laboratory standard, under the full-time observation and testing of a qualified geotechnical consultant. Utility trench bottoms should be sloped away from the primary structure on the property (typically the residence).

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

25. The pool/spa or associated 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.

26. The geotechnical consultant should review and approve all aspects of pool/spa and flatwork design 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.

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

28. Any changes in 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.

29. Disclosure should be made to homeowners and builders, contractors, and any interested/affected parties, that pools/spas built within about 15 feet of the top of a slope, and/or H/3, where H is the height of the slope (in feet), <u>will</u> 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 esthetically pleasing. The same is true with decking, flatwork and other improvements in this zone.

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

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

32. The information and recommendations discussed above should be provided to any contractors and/or subcontractors, or homeowners, interested/affected parties, etc., that may perform or may be affected by such work.

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 <u>prime</u> 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 vehie when on site, the other is to be placed atop the spoil pile on all test pits.

Flashing Lights: All vehies stationary in the grading area shall use rotating or flashing amber beacons, or strobe lights, on the vehie during all field testing. While operating a vehie in the grading area, the emergency flasher on the vehie 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 earance</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 vehie 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 vehie 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 vehiCe 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.

Trench and Vertical Excavation

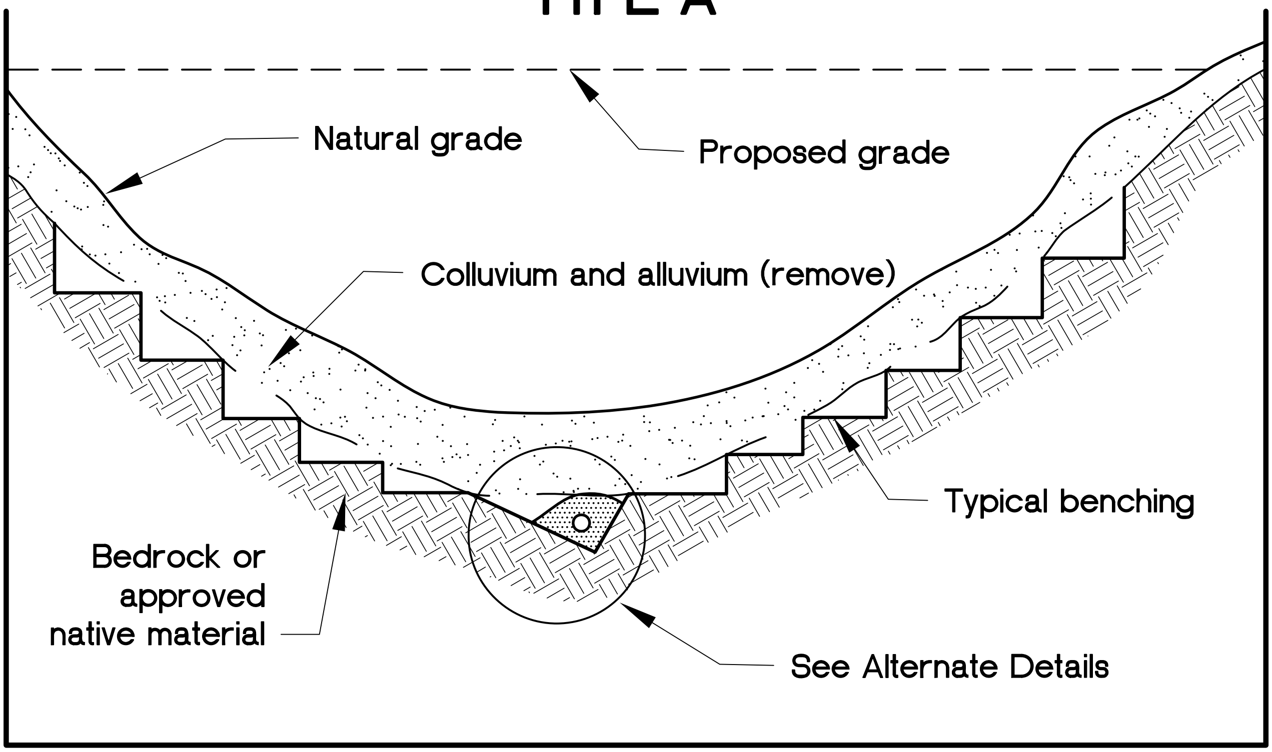
It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Our personnel are directed not to enter any excavation or vertical cut which: 1) is 5 feet or deeper unless shored or laid back; 2) displays any evidence of instability, has any loose rock or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

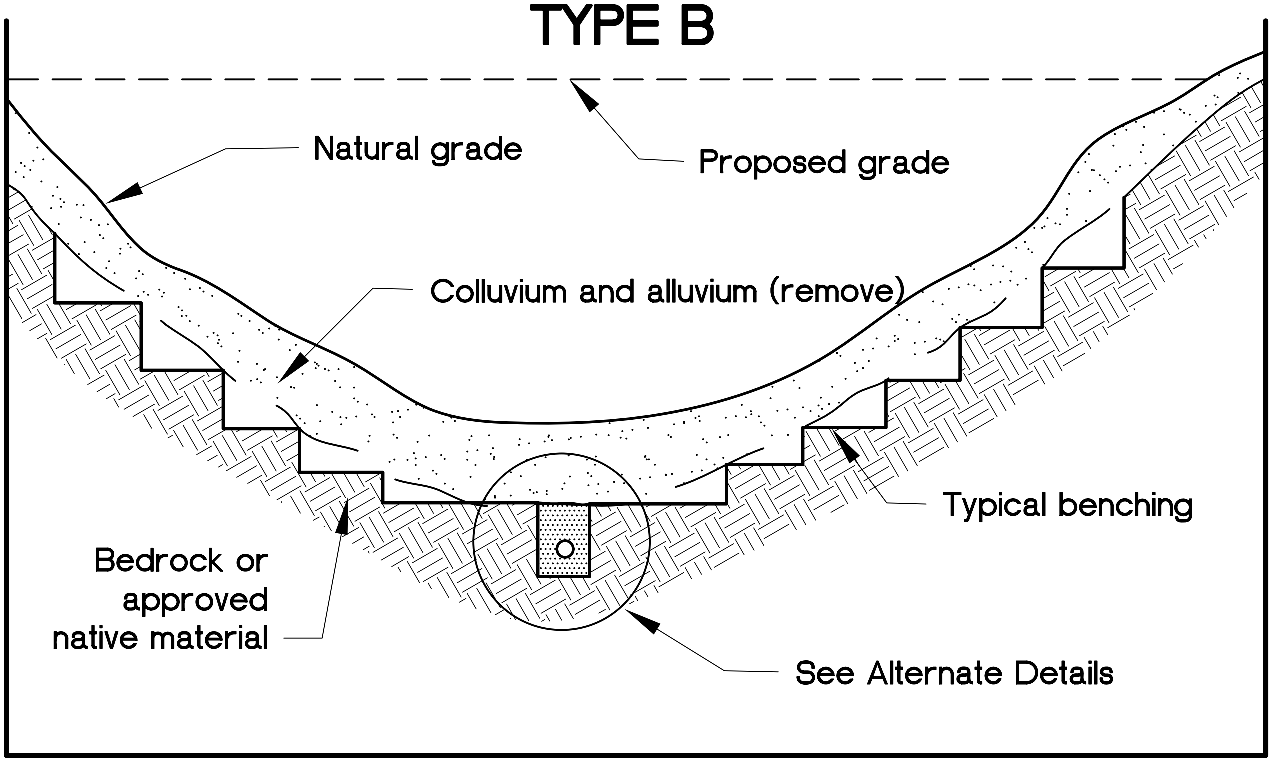
If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraw and notify his/her supervisor. The contractor’s representative will be contacted in an effort to affect a solution. All backfill not tested due to safety concerns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

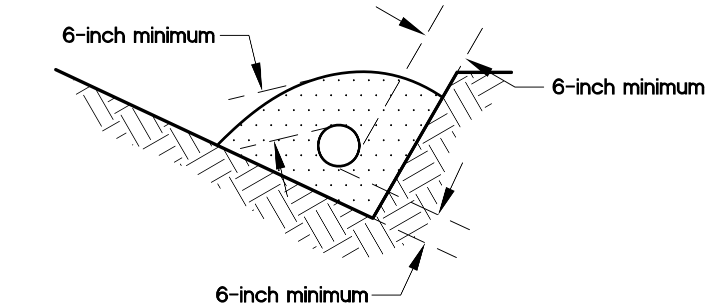


TYPE B

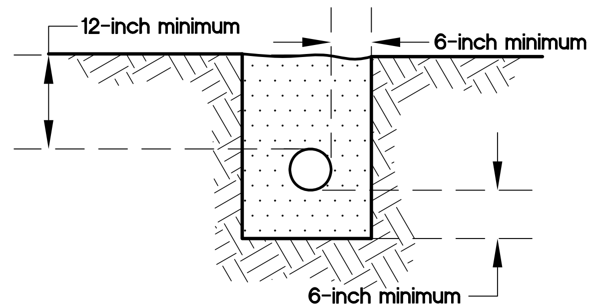


Selection of alternate subdrain details, location, and extent of subdrains should be evaluated by the geotechnical consultant during grading.

GeoSoils, Inc.	CANYON SUBDRAIN DETAIL	Plate F-1



A-1



B-1

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

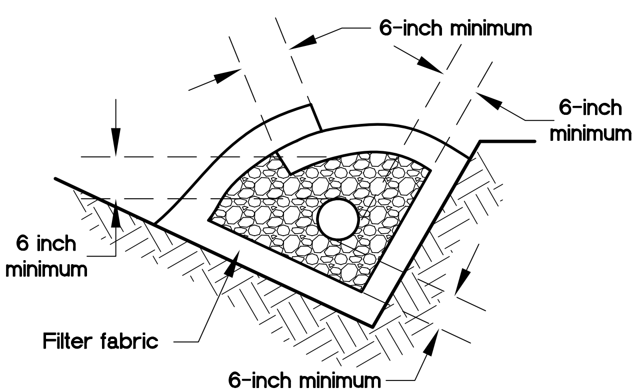
Perforated pipe: 6-inch-diameter ABS or PVC pipe or approved substitute with minimum 8 perforations (¼-inch diameter) per lineal foot in bottom half of pipe (ASTM D-2751, SDR-35, or ASTM D-1527, Schd. 40).

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

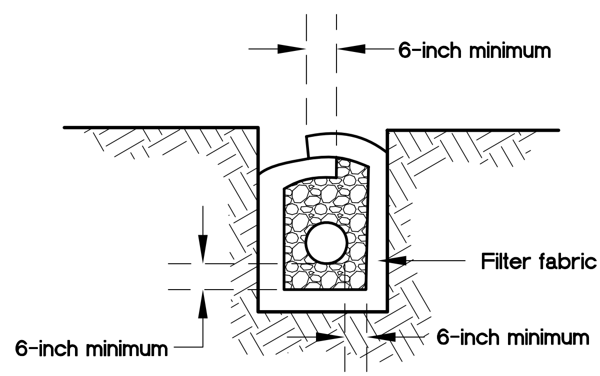
FILTER MATERIAL

Sieve Size	Percent Passing
1 inch	100
¾ inch	90-100
⅜ 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

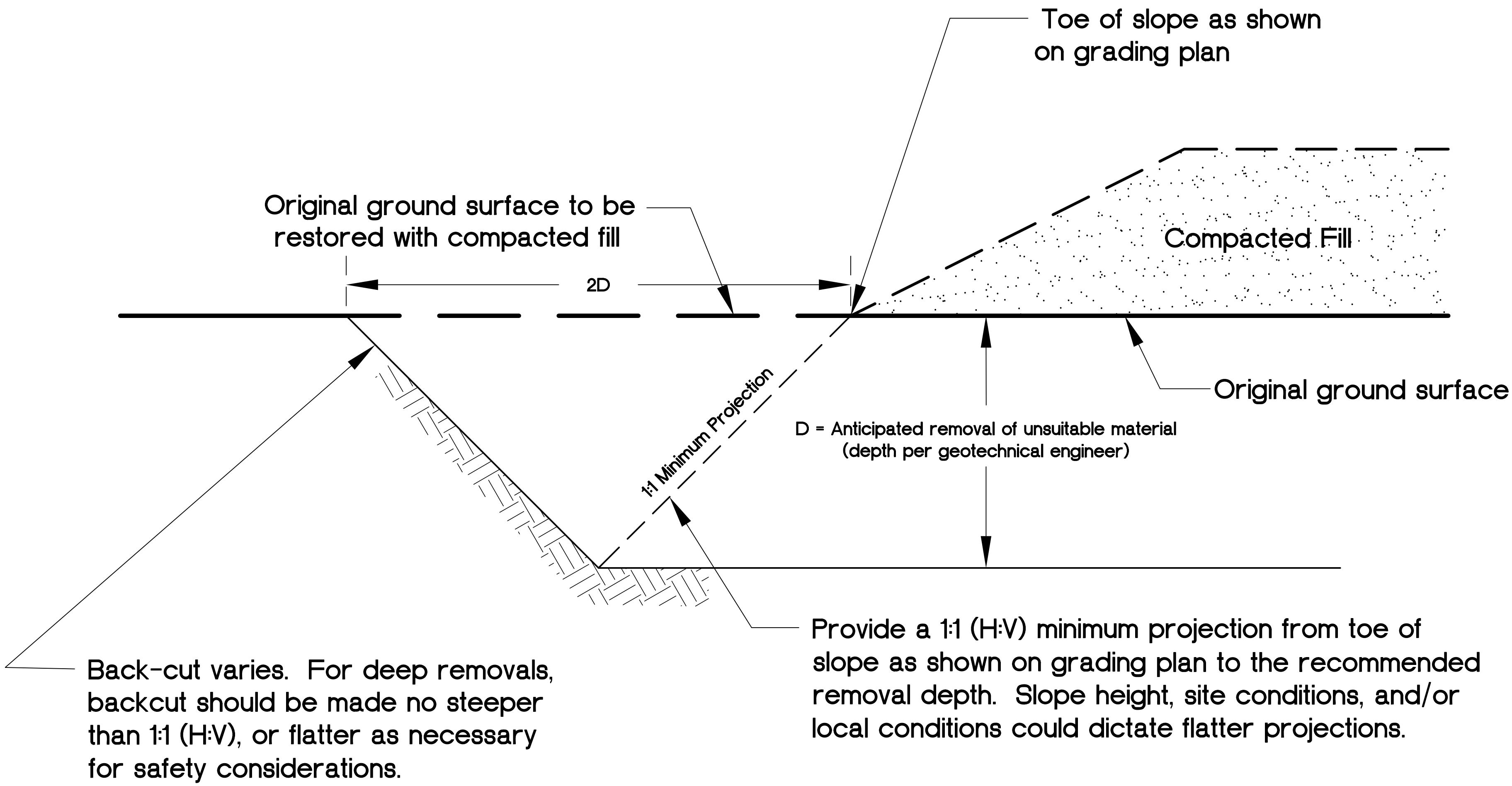


B-2

Gravel Material: 9 cubic feet per lineal foot.
Perforated Pipe: See Alternate 1
Gravel: Clean ¾-inch rock or approved substitute.
Filter Fabric: Mirafi 140 or approved substitute.

ALTERNATE 2: PERFORATED PIPE, GRAVEL, AND FILTER FABRIC

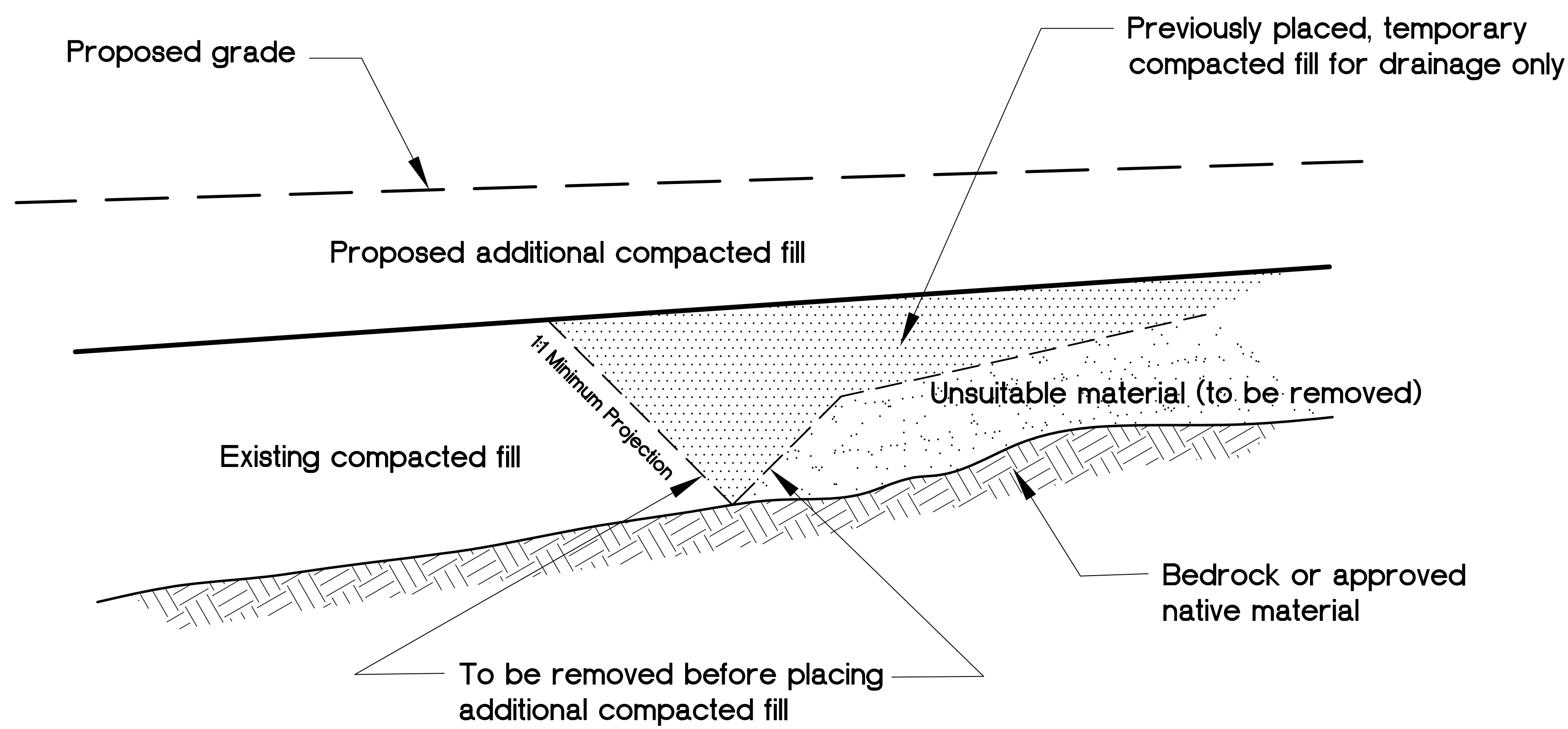
GeoSoils, Inc.	CANYON SUBDRAIN ALTERNATE DETAILS	Plate F-2



GeoSoils, Inc.

FILL SLOPE TOEING OUT ON FLAT ALLUVIATED CANYON DETAIL

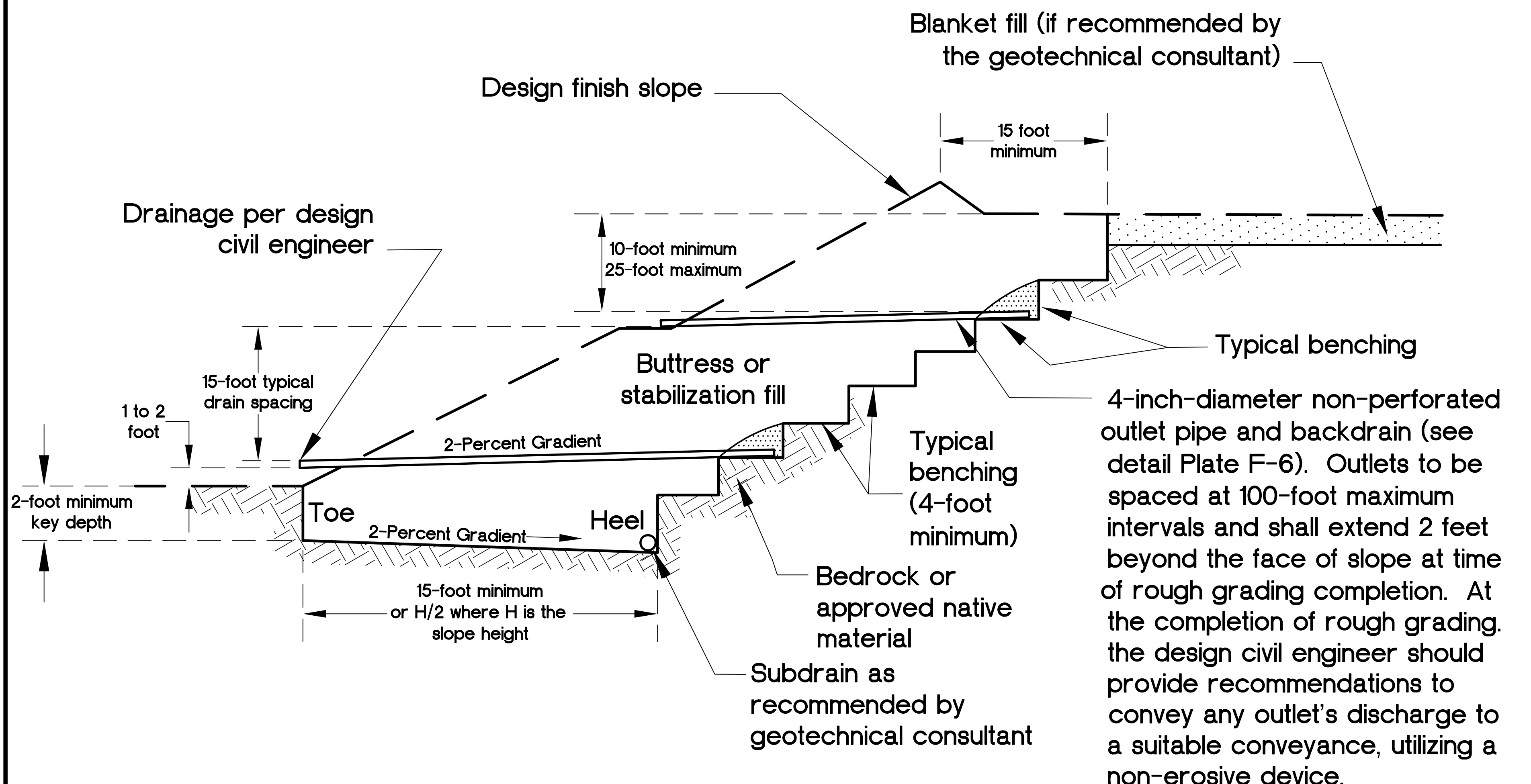
Plate F-3



GeoSoils, Inc.

REMOVAL ADJACENT TO EXISTING FILL ADJOINING CANYON FILL DETAIL

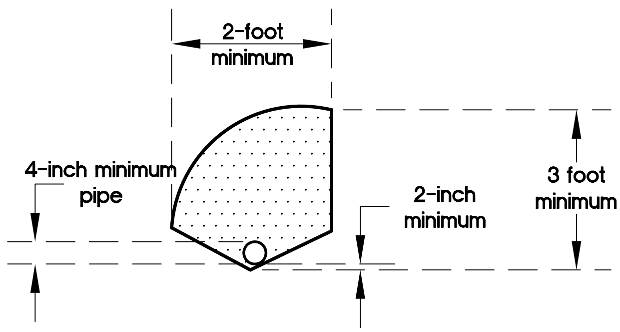
Plate F-4

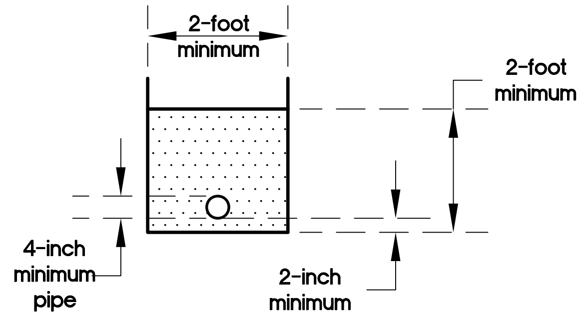


GeoSoils, Inc.

TYPICAL STABILIZATION / BUTTRESS FILL DETAIL

Plate F-5





Filter Material: Minimum of 5 cubic feet per lineal foot of pipe or 4 cubic feet per lineal feet of pipe when placed in square cut trench.

Alternative in Lieu of Filter Material: Gravel may be encased in approved filter fabric. Filter fabric shall be Mirafi 140 or equivalent. Filter fabric shall be lapped a minimum of 12 inches in all joints.

Minimum 4-Inch-Diameter Pipe: ABS-ASTM D-2751, SDR 35; or ASTM D-1527 Schedule 40, PVC-ASTM D-3034, SDR 35; or ASTM D-1785 Schedule 40 with a crushing strength of 1,000 pounds minimum, and a minimum of 8 uniformly-spaced perforations per foot of pipe. Must be installed with perforations down at bottom of pipe. Provide cap at upstream end of pipe. Slope at 2 percent to outlet pipe. Outlet pipe to be connected to subdrain pipe with tee or elbow.

Notes:

1. Trench for outlet pipes to be backfilled and compacted with onsite soil.
2. Backdrains and lateral drains shall be located at elevation of every bench drain. First drain located at elevation just above lower lot grade. Additional drains may be required at the discretion of the geotechnical consultant.

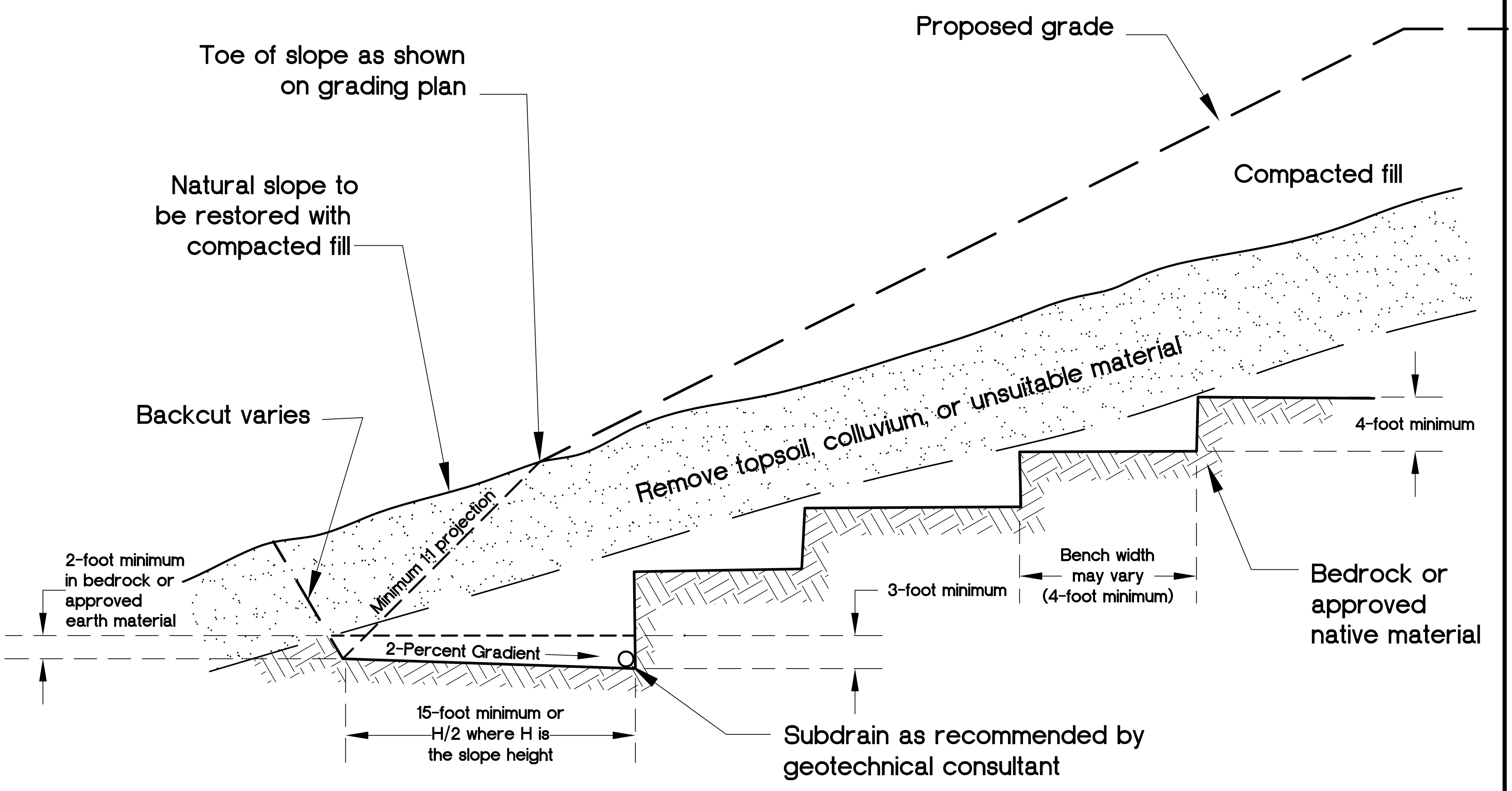
Filter Material shall be of the following specification or an approved equivalent.

Sieve Size	Percent Passing
1 inch	100
$\frac{3}{4}$ inch	90-100
$\frac{3}{8}$ inch	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

Gravel shall be of the following specification or an approved equivalent.

Sieve Size	Percent Passing
$1\frac{1}{2}$ inch	100
No. 4	50
No. 200	8

GeoSoils, Inc. | TYPICAL BUTTRESS SUBDRAIN DETAIL | Plate F-6



NOTES:

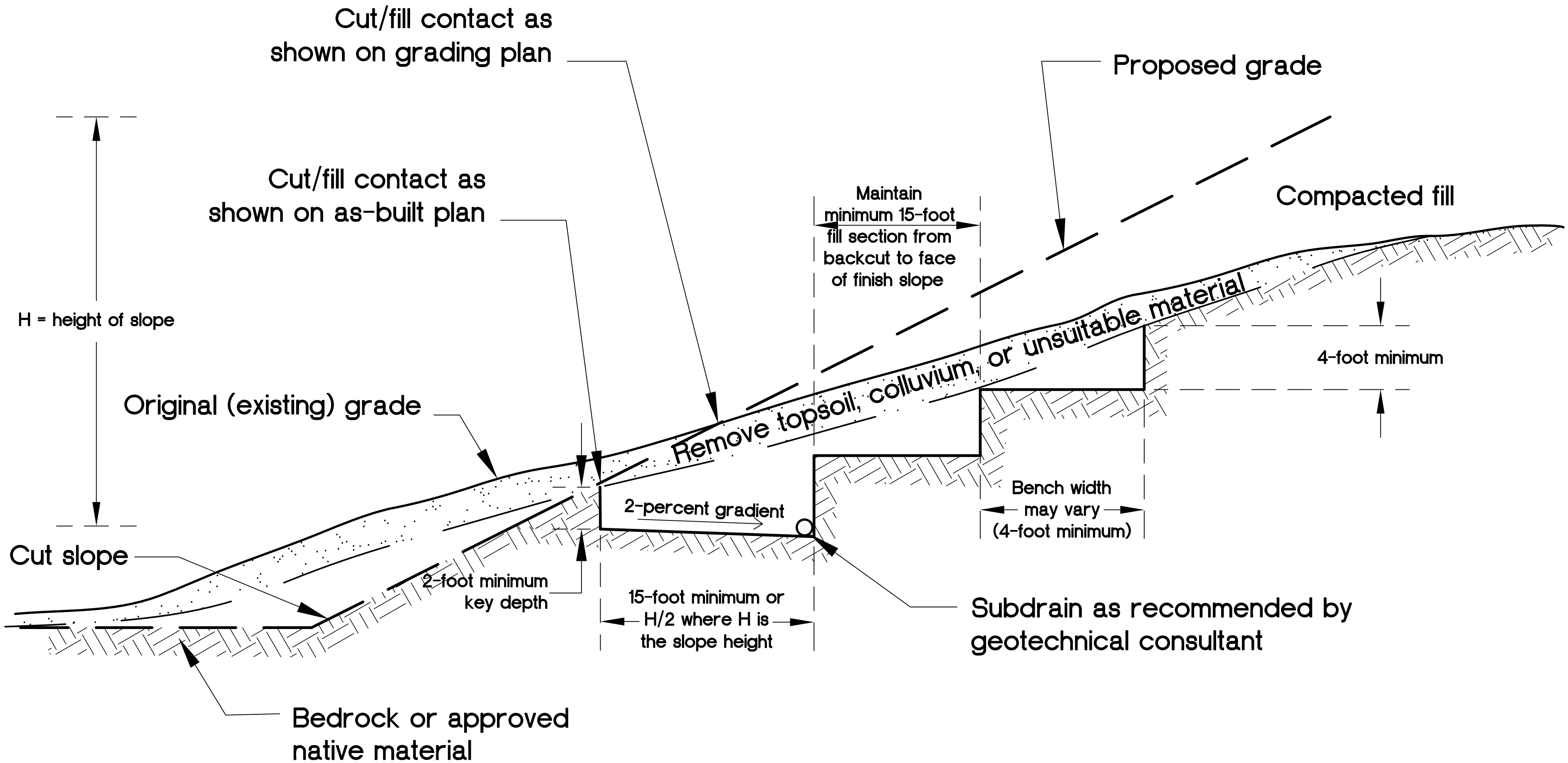
1. Where the natural slope approaches or exceeds the design slope ratio, special recommendations would be provided by the geotechnical consultant.

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

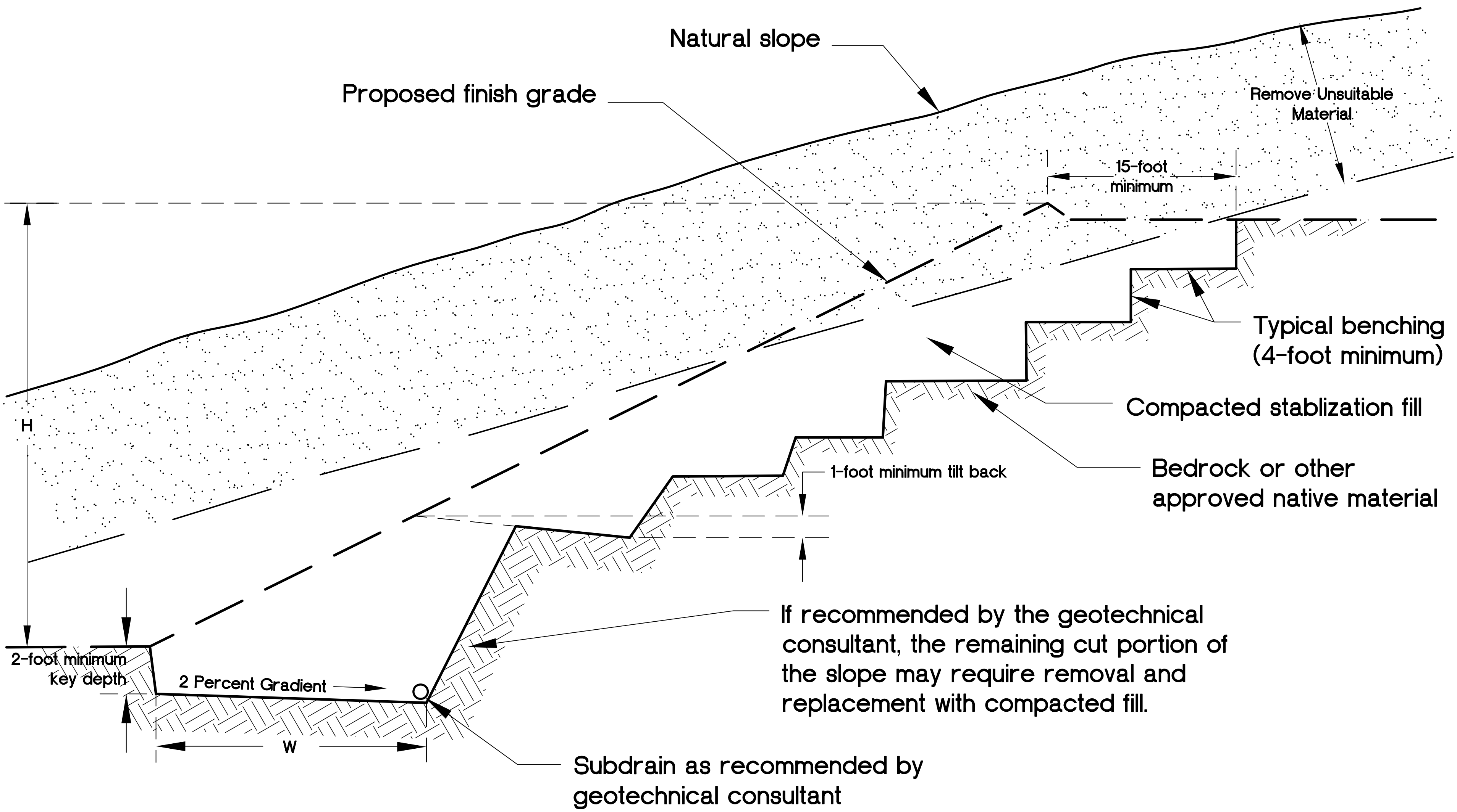


NOTE: The cut portion of the slope should be excavated and evaluated by the geotechnical consultant prior to construction of the fill portion.

GeoSoils, Inc.

FILL OVER CUT DETAIL

Plate F-8

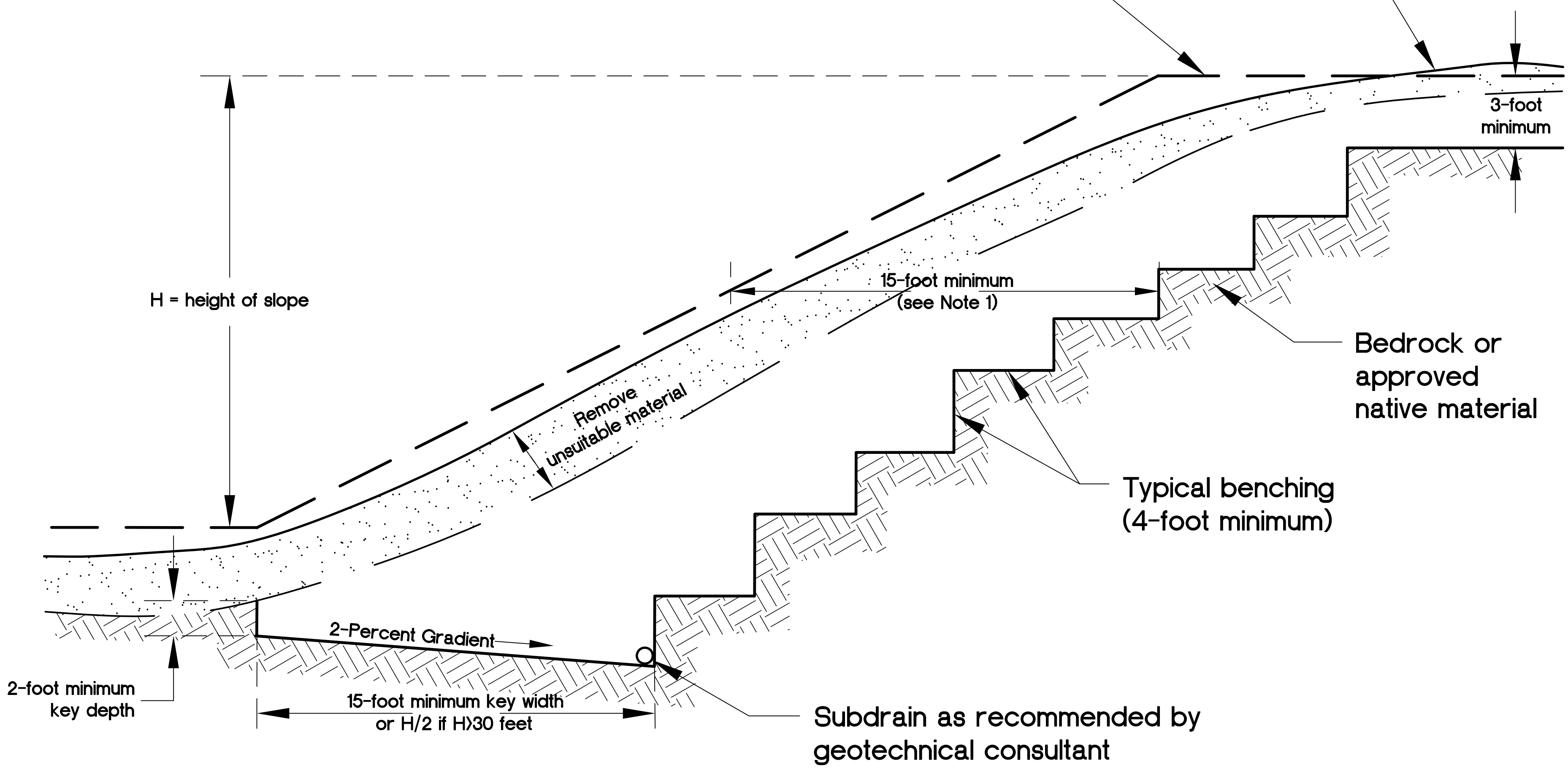


NOTES:

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



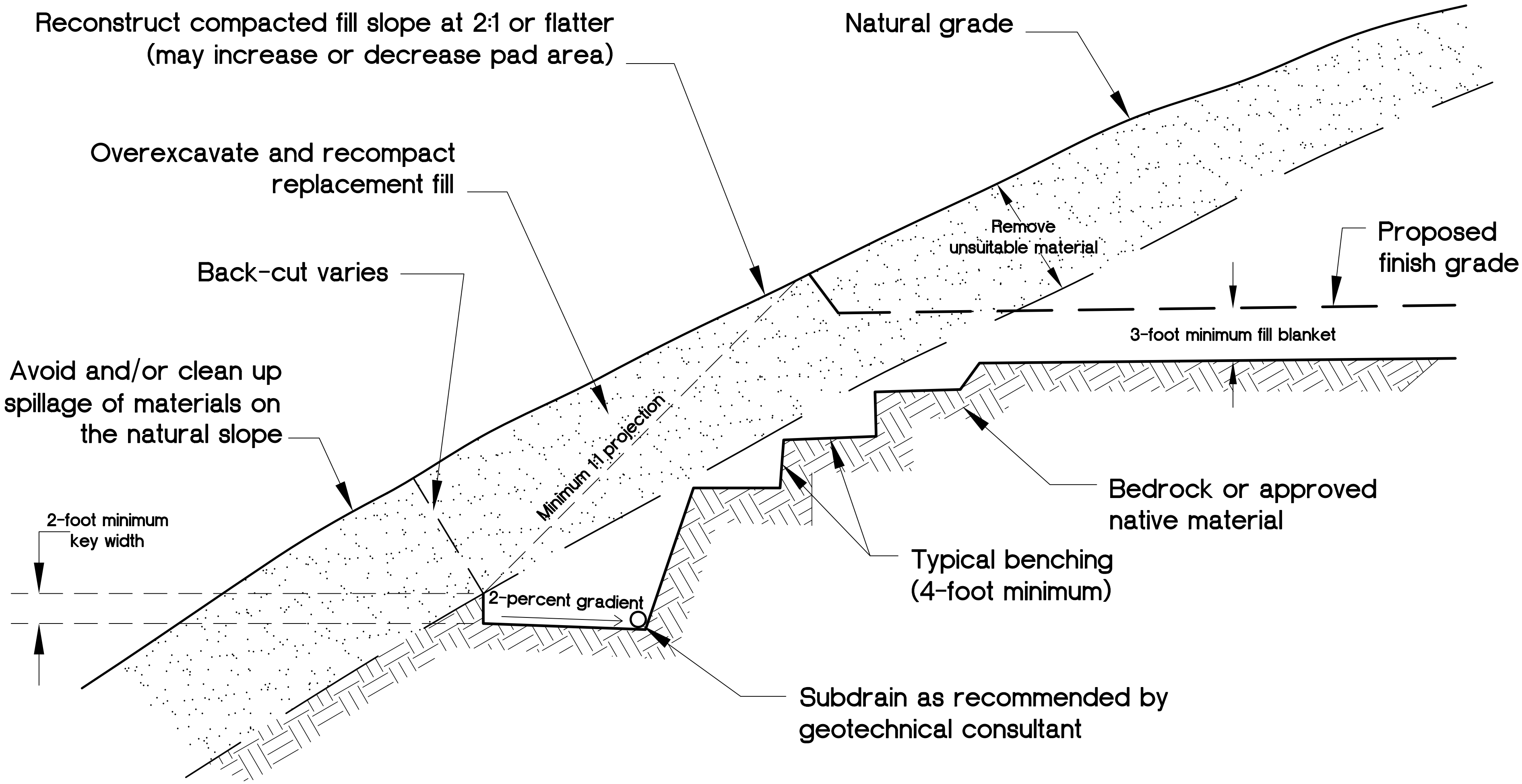
NOTES:

1. 15-foot minimum to be maintained from proposed finish slope face to backcut.
2. The need and disposition of drains will be evaluated by the geotechnical consultant based on field conditions.
3. Pad overexcavation and recompaction should be performed if evaluated to be necessary by the geotechnical consultant.

GeoSoils, Inc.

SKIN FILL OF NATURAL GROUND DETAIL

Plate F-10



NOTES:

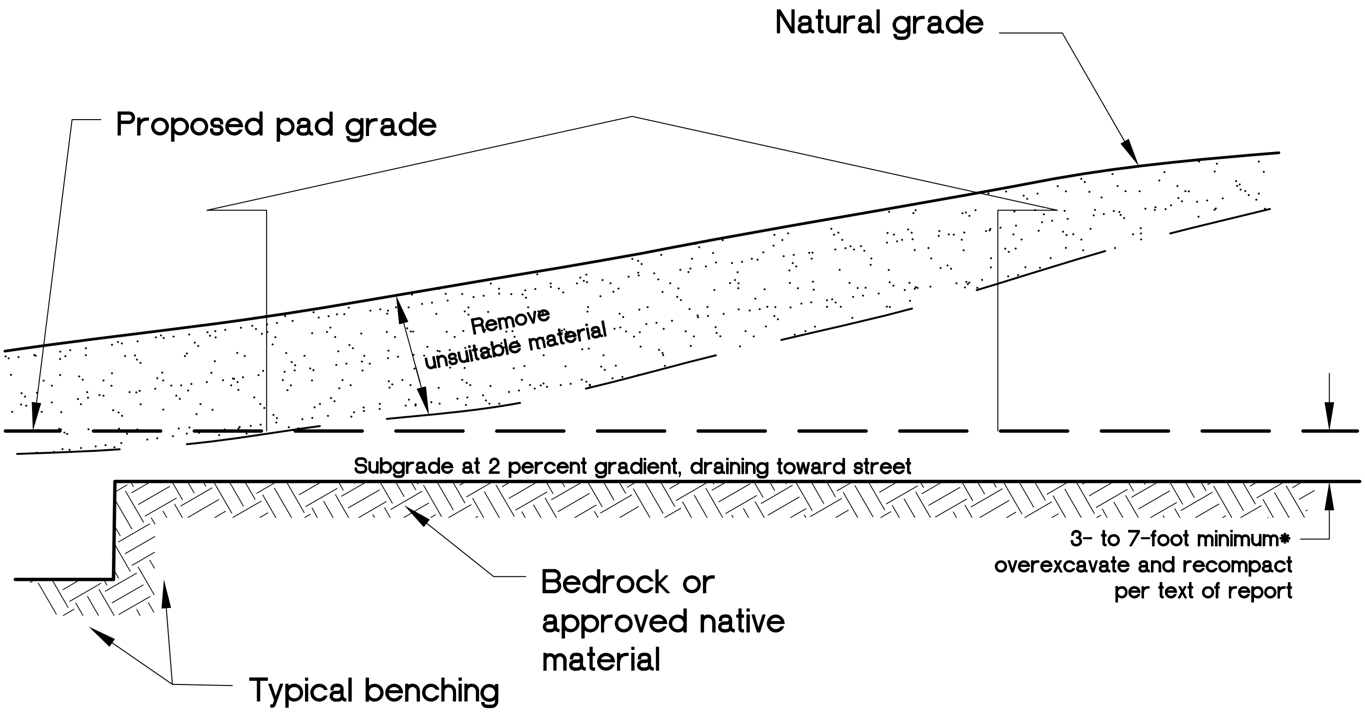
1. Subdrain and key width requirements will be evaluated based on exposed subsurface conditions and thickness of overburden.

2. Pad overexcavation and recompaction should be performed if evaluated necessary by the geotechnical consultant.

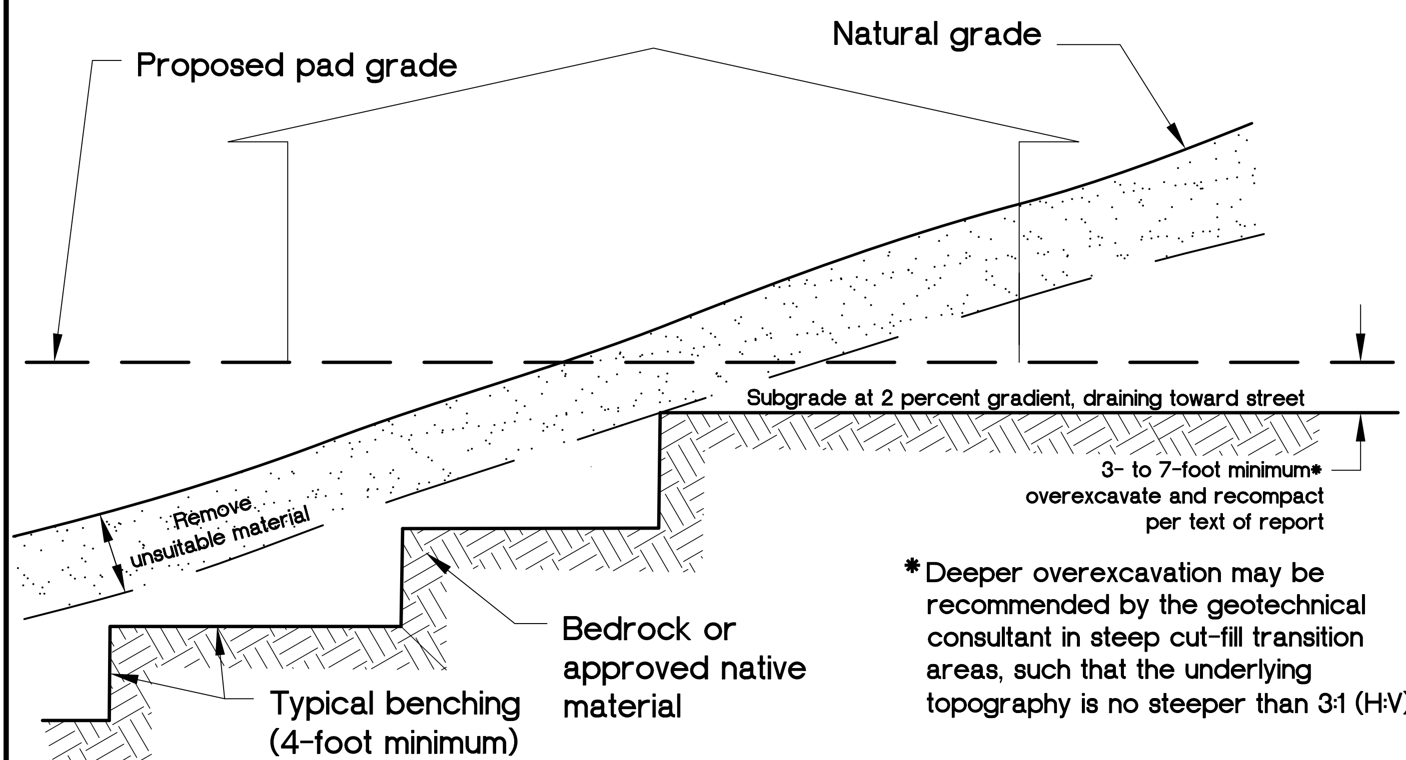
GeoSoils, Inc.

DAYLIGHT CUT LOT DETAIL

Plate F-11



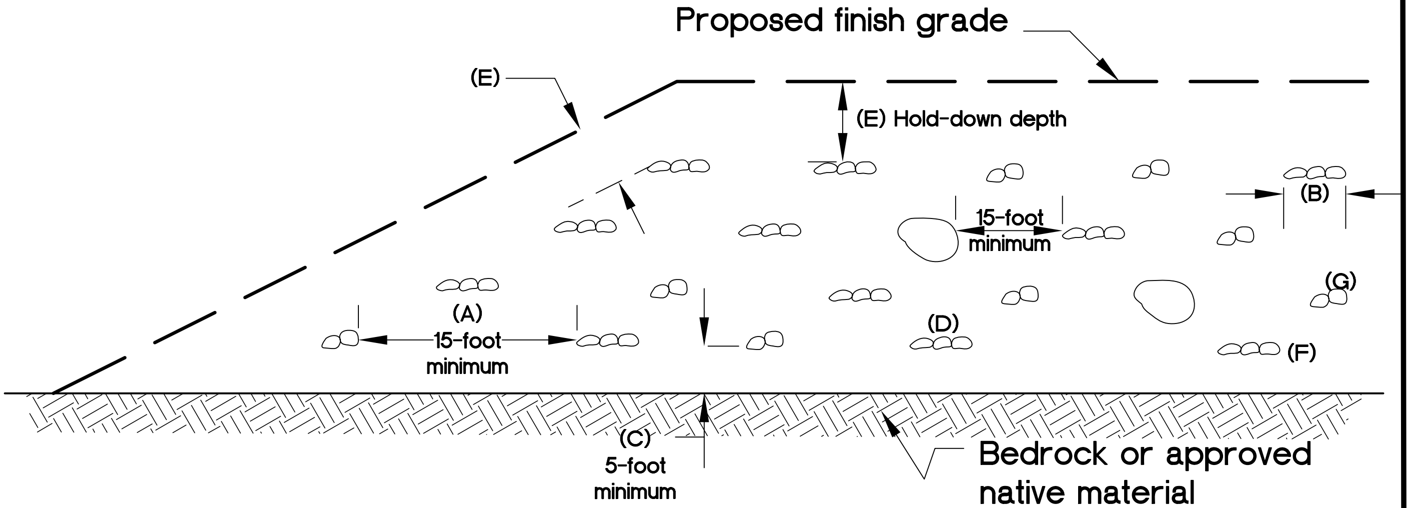
CUT LOT OR MATERIAL-TYPE TRANSITION



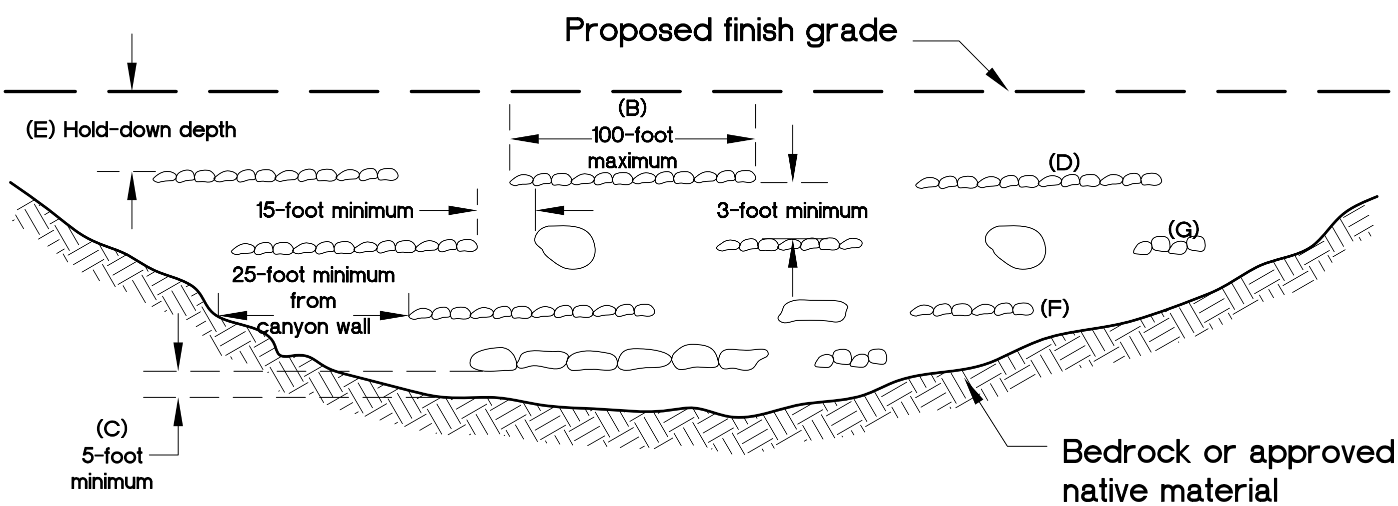
CUT-FILL LOT (DAYLIGHT TRANSITION)

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.

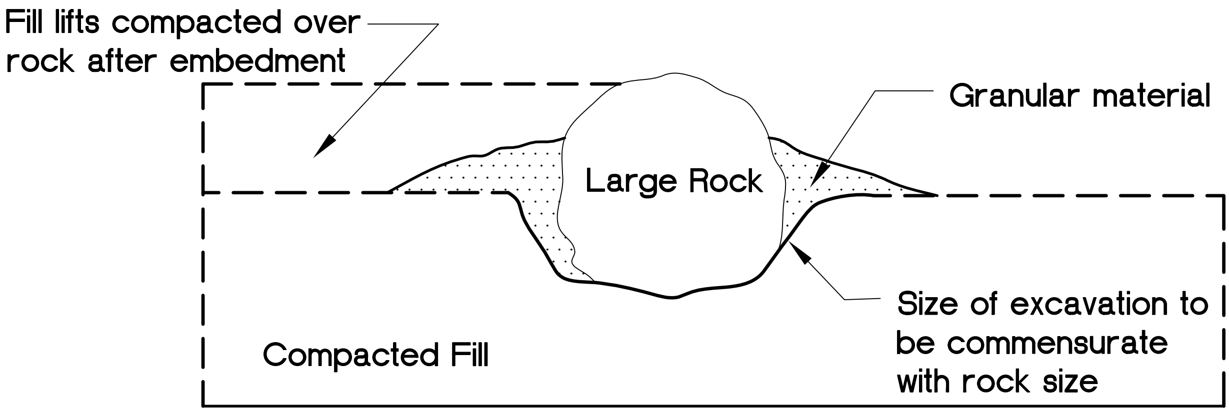
F. All fill over and around rock windrow shall be compacted to at least 90 percent relative compaction or as recommended.

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.

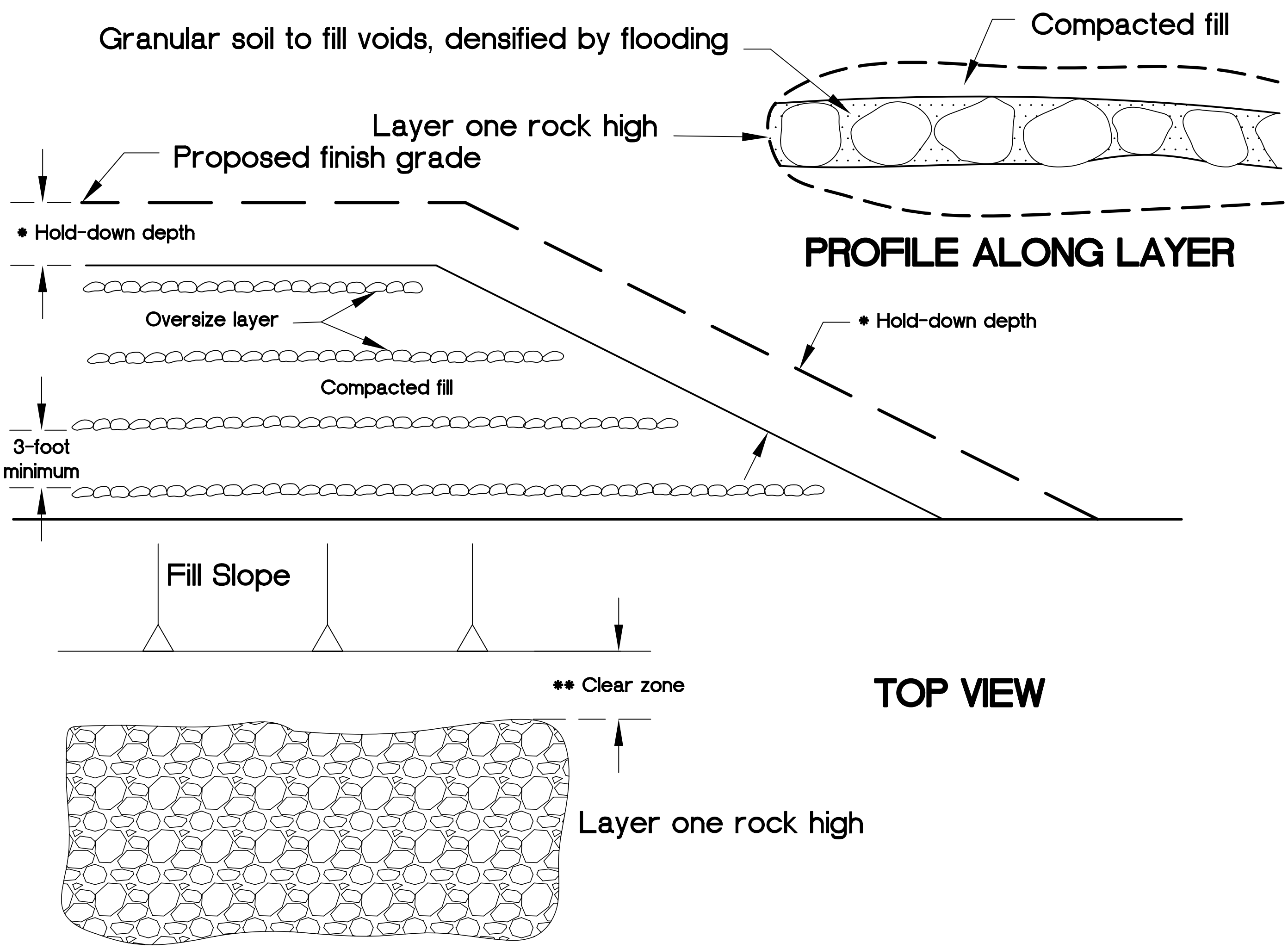
VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE
ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED

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

ROCK DISPOSAL PITS

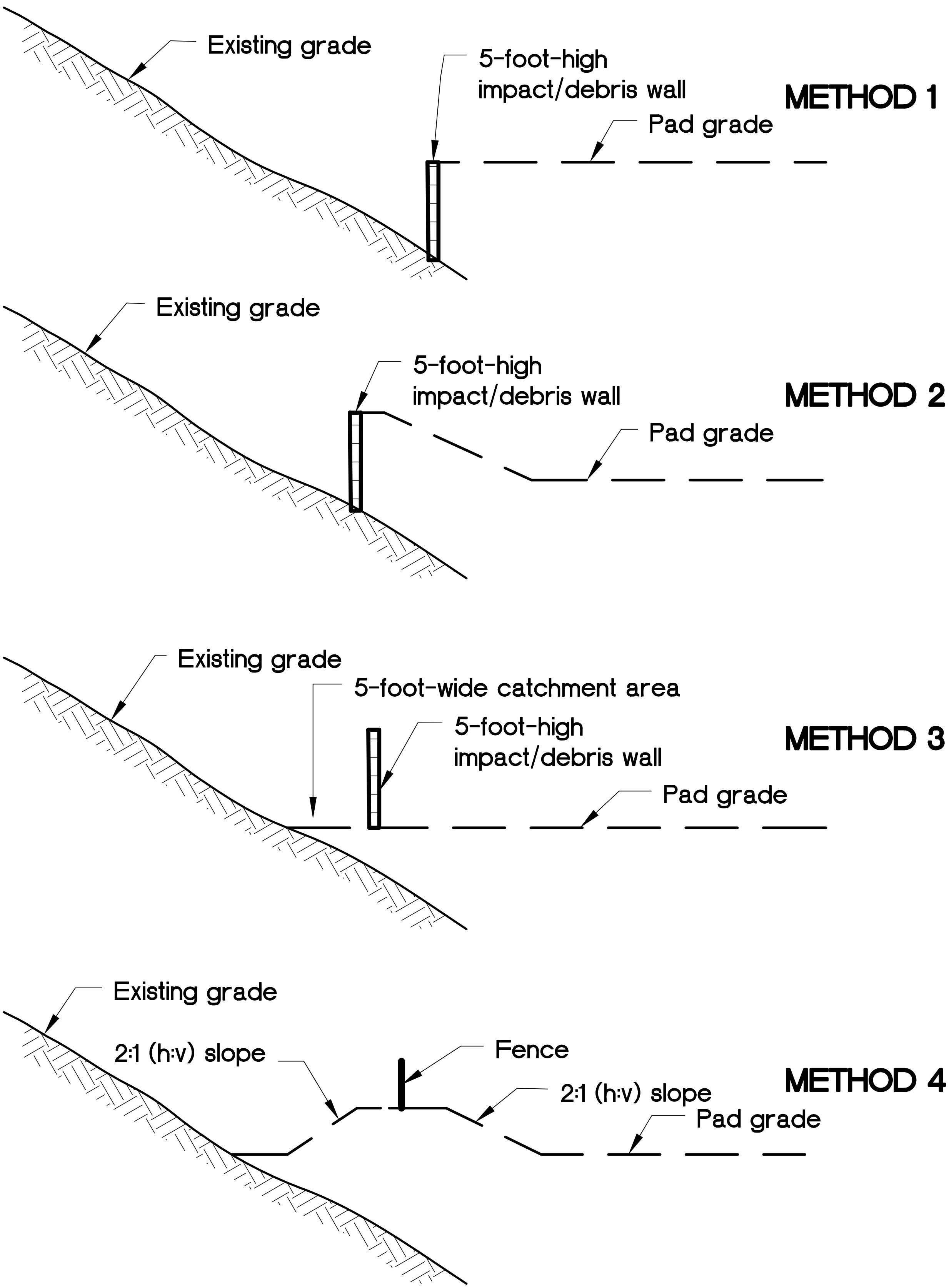


ROCK DISPOSAL LAYERS



* 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

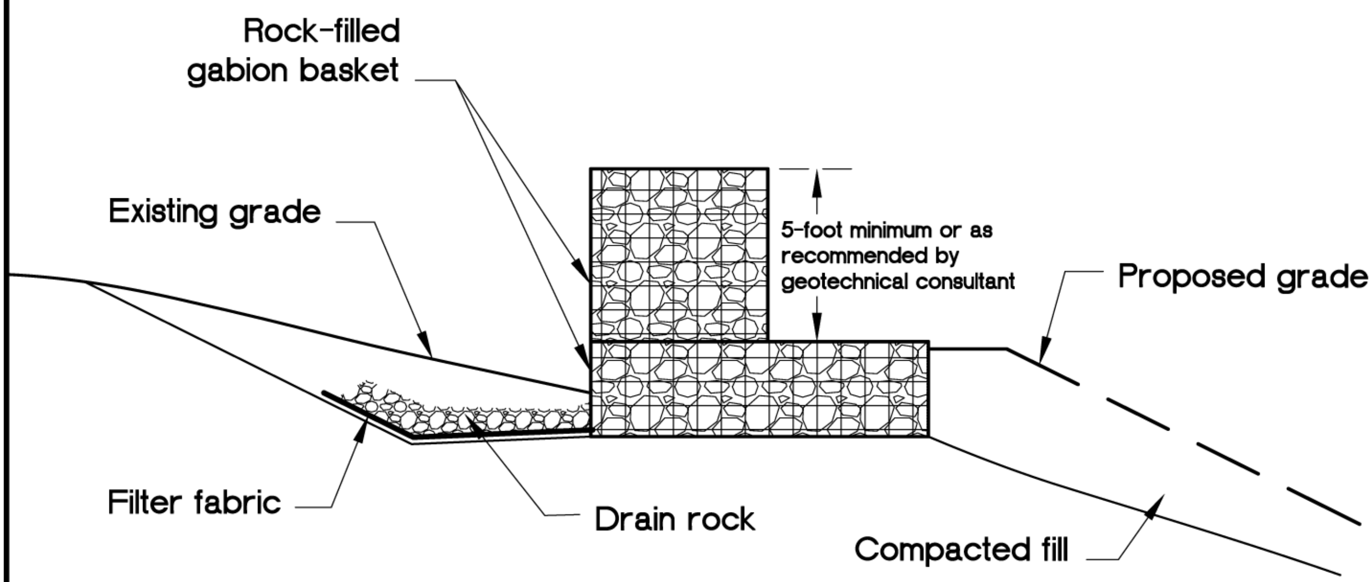


NOT TO SCALE

GeoSoils, Inc.

DEBRIS DEVICE CONTROL METHODS DETAIL

Plate F-15

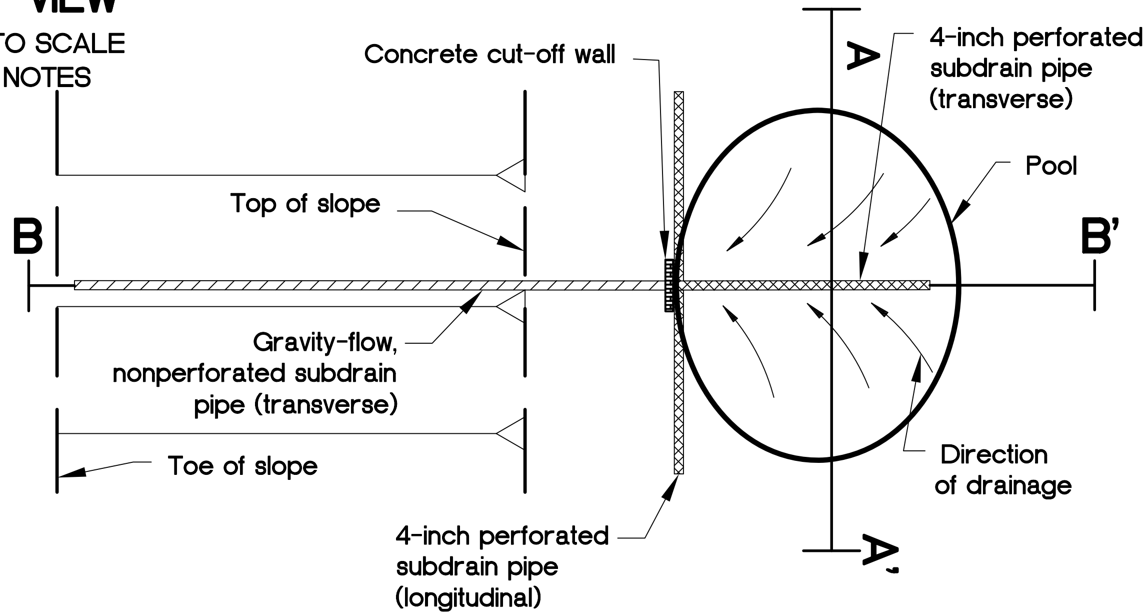


Gabion impact or diversion wall should be constructed at the base of the ascending slope subject to rock fall. Walls need to be constructed with high segments that sustain impact and mitigate potential for overtopping, and low segment that provides channelization of sediments and debris to desired depositional area for subsequent clean-out. Additional subdrain may be recommended by geotechnical consultant.

From GSA, 1987

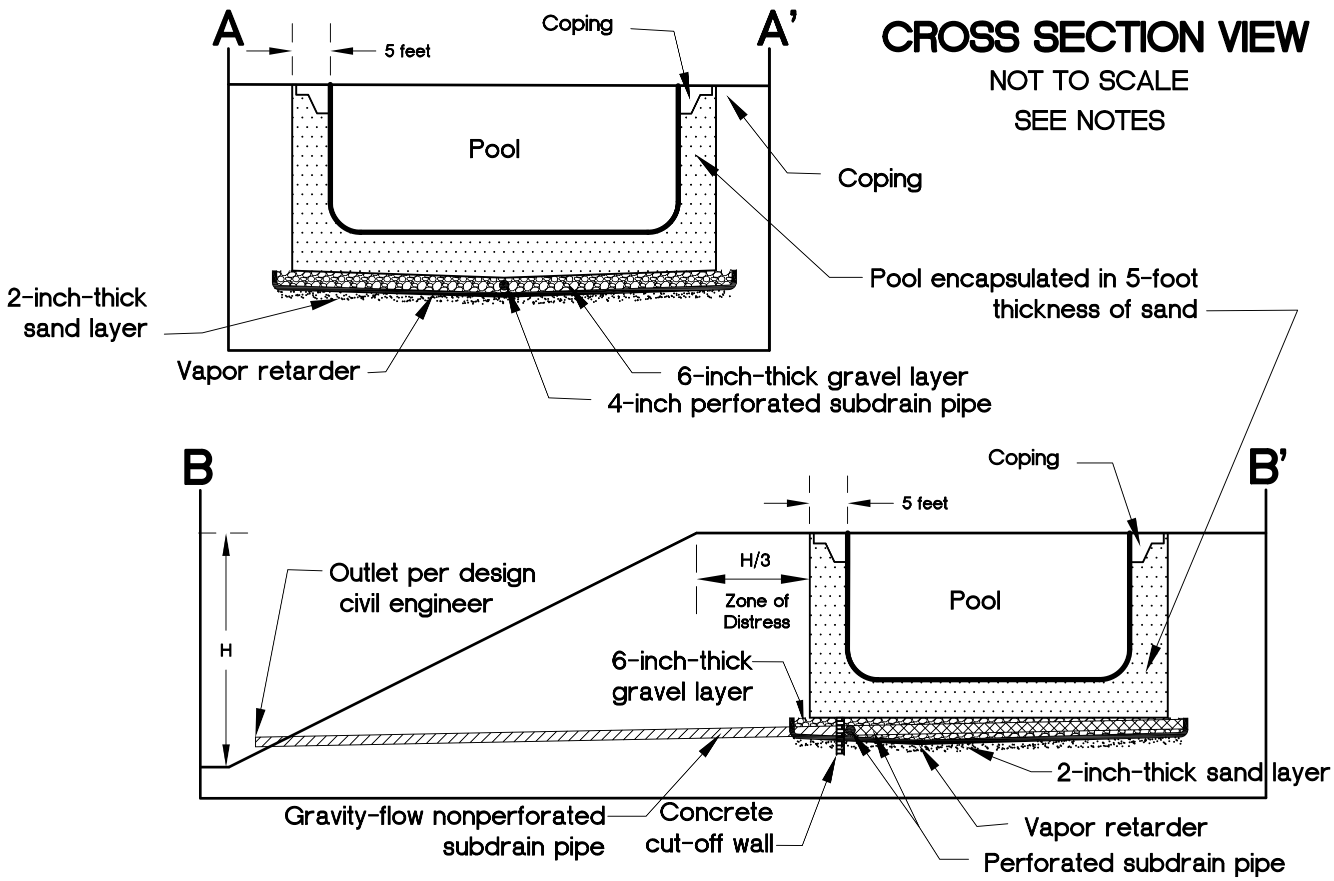
MAP VIEW

NOT TO SCALE
SEE NOTES



CROSS SECTION VIEW

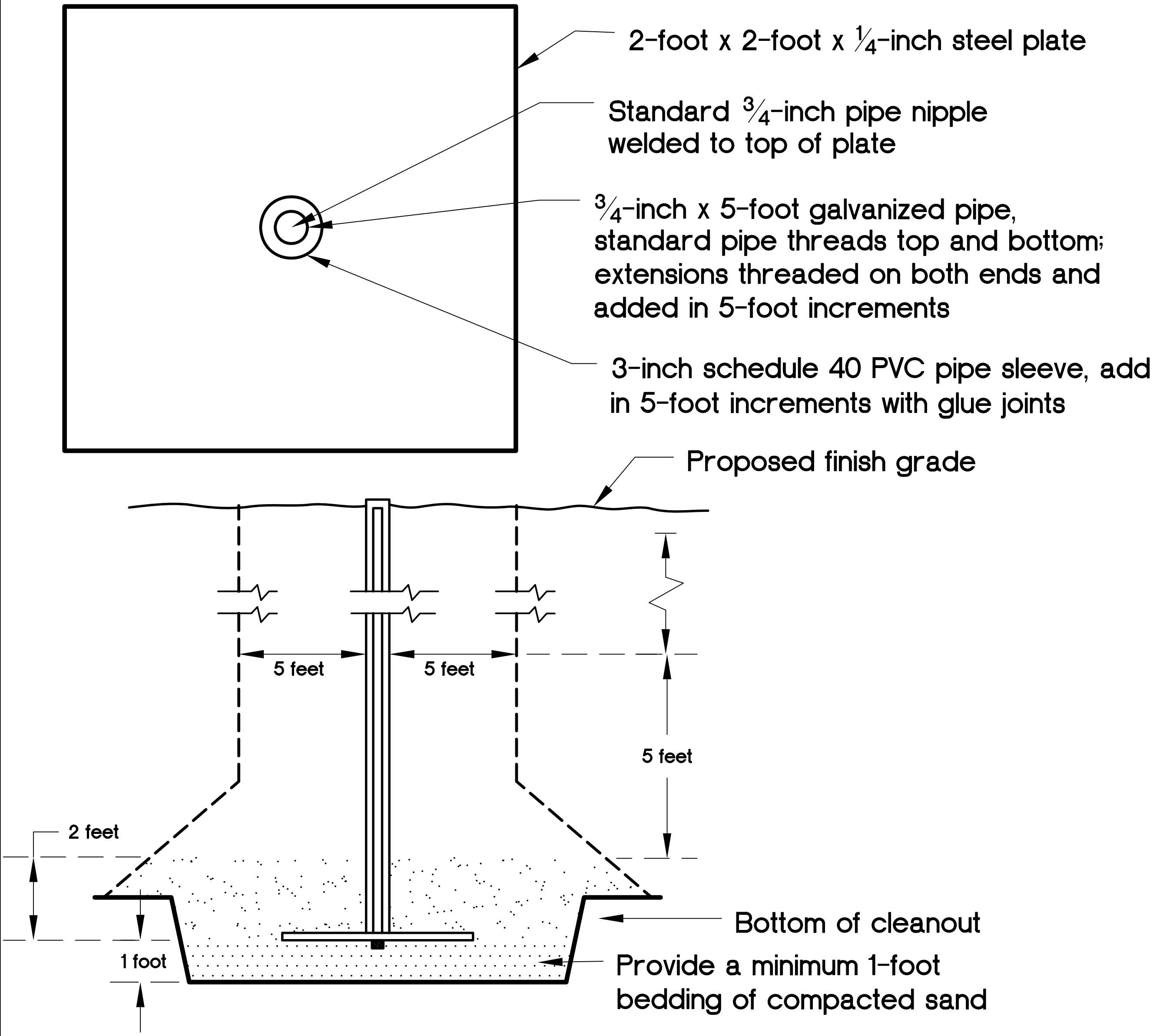
NOT TO SCALE
SEE NOTES



NOTES:

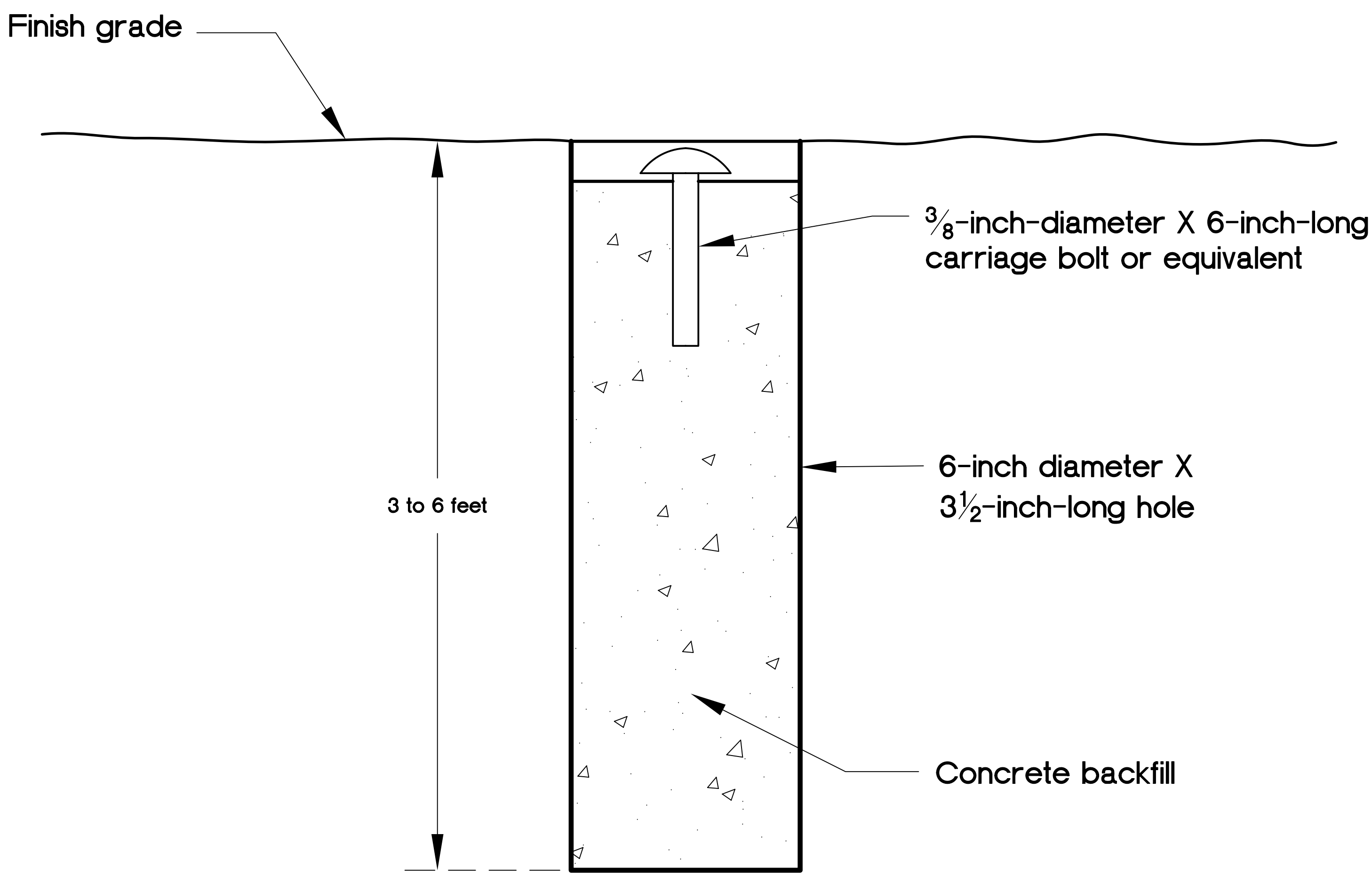
1. 6-inch-thick, clean gravel (¾ to 1½ inch) sub-base encapsulated in Mirafi 140N or equivalent, underlain by a 15-mil vapor retarder, with 4-inch-diameter perforated pipe longitudinal connected to 4-inch-diameter perforated pipe transverse. Connect transverse pipe to 4-inch-diameter nonperforated pipe 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



NOTES:

1. Locations of settlement plates should be clearly marked and readily visible (red flagged) to equipment operators.
2. Contractor should maintain clearance of a 5-foot radius of plate base and withiin 5 feet (vertical) for heavy equipment. Fill within clearance area should be hand compacted to project specifications or compacted by alternative approved method by the geotechnical consultant (in writing, prior to construction).
3. After 5 feet (vertical) of fill is in place, contractor should maintain a 5-foot radius equipment clearance from riser.
4. Place and mechanically hand compact initial 2 feet of fill prior to establishing the initial reading.
5. In the event of damage to the settlement plate or extension resulting from equipment operating within the specified clearance area, contractor should immediately notify the geotechnical consultant and should be responsible for restoring the settlement plates to working order.
6. An alternate design and method of installation may be provided at the discretion of the geotechnical consultant.

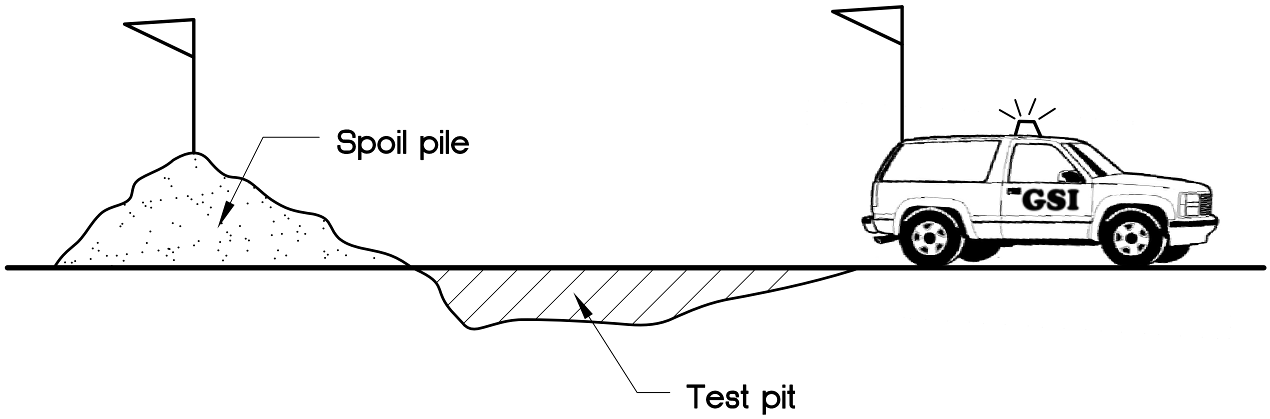


GeoSoils, Inc.

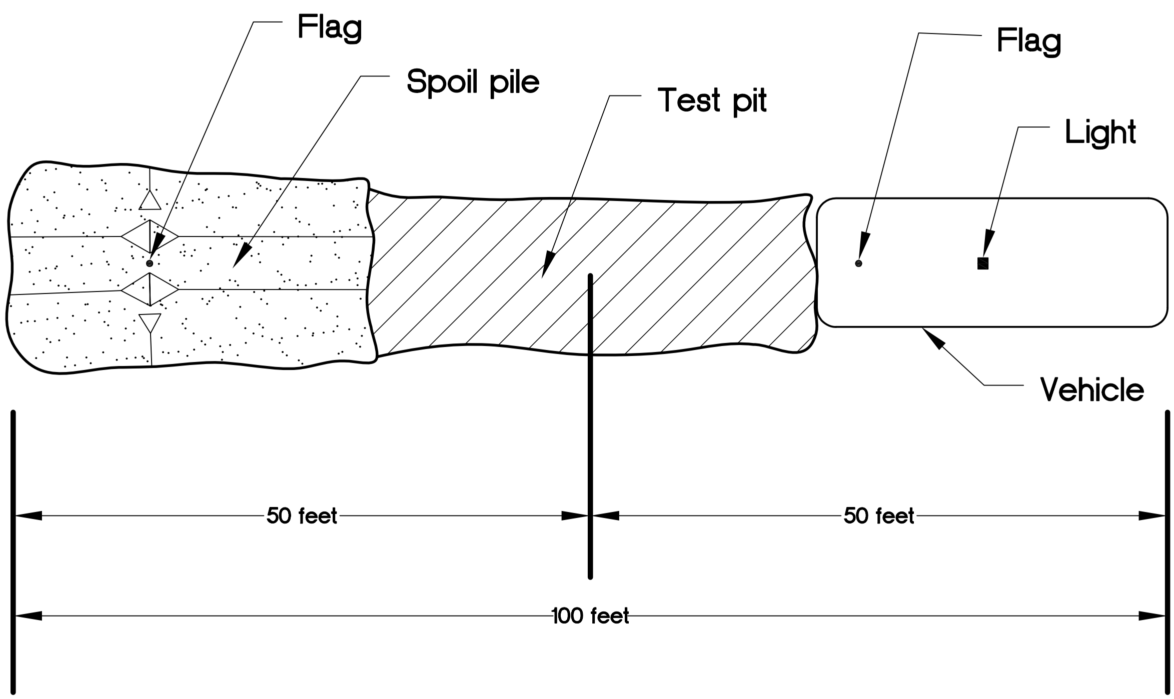
TYPICAL SURFACE SETTLEMENT MONUMENT

Plate F-19

SIDE VIEW



TOP VIEW



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