Appendix E3. Geotechnical Evaluation

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GEOTECHNICAL EVALUATION OF MARJA ACRES
APN 207-101-35 & -37, 1910 EL CAMINO REAL
CARLSBAD, SAN DIEGO COUNTY, CALIFORNIA

FOR

NEW URBAN WEST, INC.
16935 WEST BERNARDO DRIVE, SUITE 260
SAN DIEGO, CALIFORNIA 92127

W.O. 6971-A-SC JULY 8, 2016

GeoSoils, Inc.

Geotechnical • Geologic • Coastal • Environmental

5741 Palmer Way • Carlsbad, California 92010 • (760) 438-3155 • FAX (760) 931-0915 • www.geosoilsinc.com

July 8, 2016

W.O. 6944-A-SC

New Urban West, Inc.
16935 West Bernardo Drive, Suite 260
San Diego, California 92127

Attention: Mr. Jason Han

Subject: Geotechnical Evaluation of Marja Acres, APN 207-101-35 & -37, 1910 El Camino Real, Carlsbad, San Diego County, California

Dear Mr. Han:

In accordance with your request and authorization, GeoSoils, Inc. (GSI) is pleased to present the results of our geotechnical evaluation at the subject site. The purpose of our study was to evaluate the geologic and geotechnical conditions at the site in order to develop preliminary recommendations for site earthwork and the design of foundations, walls, and pavements related to the proposed mixed use, residential/commercial construction at the property.

EXECUTIVE SUMMARY

Based upon our field exploration, geologic, and geotechnical engineering analysis, the proposed development appears feasible from a soils engineering and geologic viewpoint, provided that the recommendations presented in the text of this report are properly incorporated into the design and construction of the project. The most significant elements of our study are summarized below:

- A review of conceptual plans by SWC (2015), indicate that the site will be prepared for the construction of a mixed use project consisting of commercial buildings along El Camino Real, and residential units within the upper elevations of the site, with associated underground utilities, and other typical residential improvements. The buildings are anticipated to be supported by perimeter and isolated footing foundation systems.

- The project area is primarily underlain with a surficial deposits of undocumented fill, colluvium (topsoil), and alluvium, which is underlain by formational deposits of Quaternary older alluvium and Eocene-age sedimentary bedrock.

- Our evaluation indicates that the regional groundwater table currently occurs at a depth of approximately 14 to 17 feet below existing surface grades across the site (elevation of approximately 31.5 feet to 35 feet Mean Sea Level [MSL]). Historically, groundwater levels appear to generally be moderated by the adjacent Agua Hedionda Lagoon. Provided that the recommendations contained in this report are followed, regional groundwater is not expected to be a major factor in development of the site, but will need to be considered during planning and construction. Deep utilities, if planned, may encounter groundwater.

- Based on the presence of alluvium, and a relatively shallow groundwater table, an analysis of liquefaction and seismic settlement was performed. Our analyses indicate that the alluvial soils are generally not susceptible to surface deformations from liquefaction, and seismic settlement is anticipated to be within tolerable limits for a typical foundation system. The potential for the site to experience surface manifestation of liquefaction (sand boils, injection dikes, etc.), is considered low; however, this potential would increase around manholes or utility risers.

- Alluvium was also evaluated for hydrocollaspe potential. Our evaluation indicated that the potential for adverse settlement due to hydrocollaspe is low, provided that the recommended mitigative earthwork is incorporated into the design and construction oft eh project.

- Representative soil samples were evaluated as ranging from very low to highly expansive (Expansion index [E.I.] range of 17 to 128). As such, some onsite soils are considered detrimentally expansive, as defined in Section 1803.5.2 of the 2013 California Building Code ([2013 CBC], California Building Standards Commission [CBSC], 2013), and will require special design. Soil expansivity should be re-evaluated at the conclusion of grading and provide updated data for final foundation design.

- A representative sample of site soil was evaluated for corrosion potential. Test results indicate a "not applicable" sulfate exposure to concrete (per ACI 318-11), a slightly elevated chloride level, alkaline pH, and a severely corrosive environment to buried metal, when saturated. Consultation from a qualified corrosion engineer may be obtained based on the level of corrosion protection requirements by the project architect and structural engineer.

- Undocumented fill, surficial deposits of colluvium (topsoil), and near surface deposits of alluvium, are not considered suitable for the support of settlement-sensitive improvements or engineered fill in their existing state. Thus, these earth materials should be removed, moisture conditioned, and recompacted. Prior to placement of fill, the exposed removal bottom should be scarified, moisture conditions, and compacted to at least 90 percent relative compaction.

- It should be noted that the 2013 CBC (CBSC, 2013) indicates that remedial grading be performed across all areas under the purview of the grading permit, 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 and/or offsite. Thus, any settlement-sensitive improvements (walls, flatwork, etc.), constructed within this zone, may require deepened foundations, reinforcements, etc., or will retain some potential for settlement and associated distress. This will require proper disclosure to all interested/affected parties, should this condition exist at the conclusion of site grading.

- Support of the new structures should be provided entirely by compacted fill. Due to the potential differential settlements and soil expansion potential evaluated herein, a properly designed concrete slab on grade foundation system may be utilized, based on the available data, and assuming the recommended remedial grading. All foundations should be designed for expansive soil conditions, as warranted. Foundations underlain with up to 30 feet of compacted fill over formation should be designed to accommodate at least 1 inch of static differential settlement in a 40-foot span, without exhibiting distress to the superstructure. Thicker fill, or compacted fill placed on alluvial soil left in place, will require more onerous foundation design, and/or waiting periods prior to construction, as recommended herein.

- Graded slopes are anticipated to perform adequately, assuming proper construction and maintenance, under the prevailing climate.

- GSI's review indicates no known active faults are crossing the site, and the site is not located within an Alquist-Priolo Earthquake Fault Zone. The subject site is situated in a seismically active region. As is typical in southern California, the site may experience moderate to strong ground shaking should an earthquake occur on any of the regional active faults. The seismic acceleration values and design parameters provided herein should be utilized during the design of the proposed project.

- Adverse geologic structures that would preclude project feasibility were not encountered. However, the potentially liquefiable and compressible deposits of alluvium will require some mitigation during earthwork and/or specialized foundation design, as discussed herein.

- With respect to storm water infiltration system designs, undisturbed site soils and planned compacted fills are anticipated to belong within Hydrologic Soil Group D (no infiltration). Basins located within 10 feet of any structure or settlement/expansion sensitive improvement, will also need to be designed for no infiltration. It should be noted that the local groundwater table was evaluated to be as shallow as 14 feet below existing site grades. Infiltration is not feasible when the bottom of basins are located 10 feet, or less, from the water table.

- 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.

PROFESSIONAL GEOLOGIST
ROBERT G. CRISMAN
No. 1934
Certified Engineering Geologist
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
DAVID W. SKELLY
No. RCE 47857
Exp. 12-31-17
CIVIL
STATE OF CALIFORNIA

Robert G. Crisman
Engineering Geologist, CEG 1934

David W. Skelly
Civil Engineer, RCE 47857

RGC/JPF/DWS/jh

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ATTACHMENTS:

GEOTECHNICAL EVALUATION FOR MARJA ACRES, APN 207-101-35, & -37, 1910 EL CAMINO REAL CARLSBAD, SAN DIEGO COUNTY, CALIFORNIA

SCOPE OF SERVICES

The scope of our services has included the following:

1. Review of readily available published literature, aerial photographs, and maps of the vicinity (see Appendix A), including proprietary in-house geologic/geotechnical reports for other nearby sites.

2. Site reconnaissance mapping and the excavation of three (3) exploratory auger borings and ten (ten) exploratory test pits to evaluate the soil/bedrock profiles, sample representative earth materials, and delineate the horizontal and vertical extent of earth material units (see Appendix B).

3. General areal seismicity evaluation (see Appendix C).

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

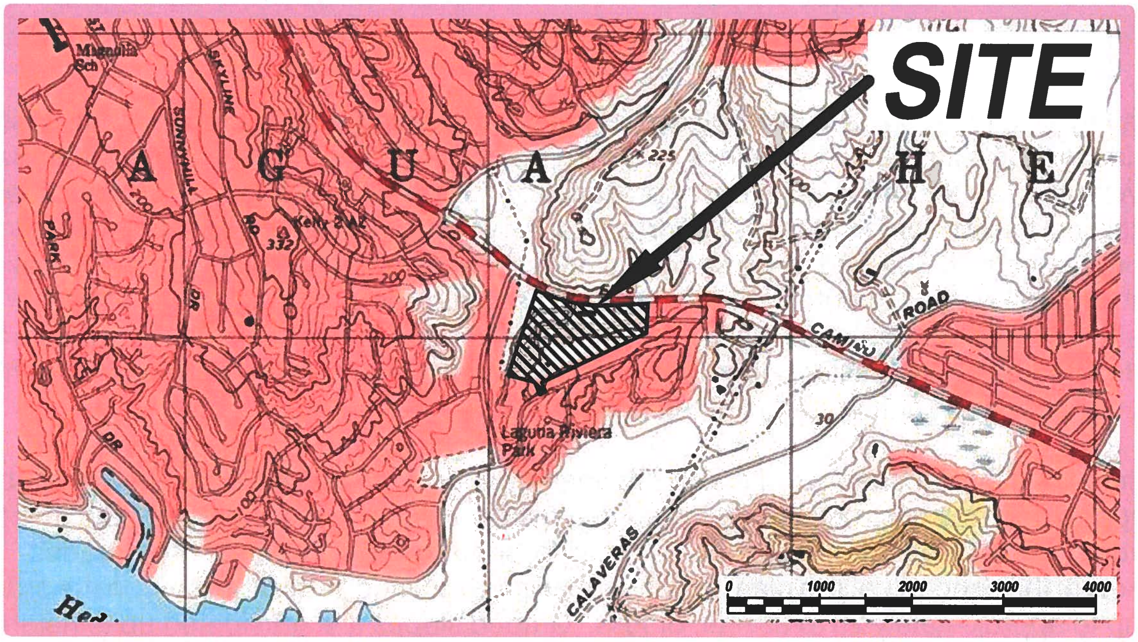
5. Analysis of field and laboratory data relative to the proposed development, including liquefaction and settlement evaluations (Appendix E).

6. Discussion of Infiltration feasibility and completion of City worksheet I-8 (Appendix F).

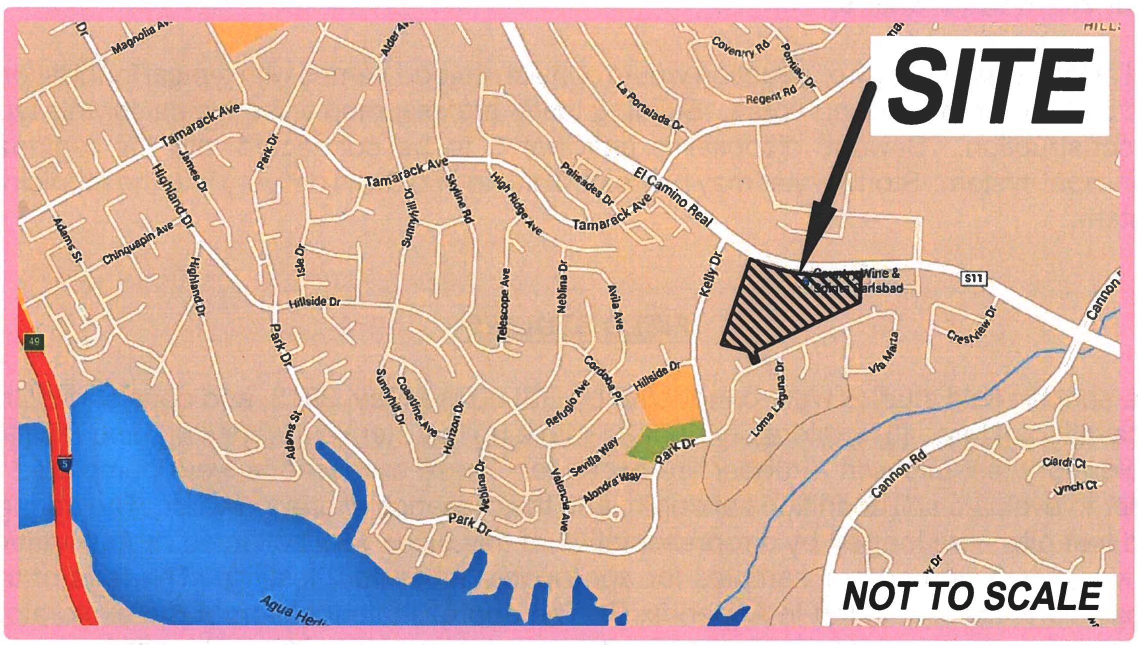
7. Appropriate engineering and geologic analyses of data collected, and the preparation of this summary report and accompaniments.

SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The site consists of an irregular shaped, approximately 20-acre property located on the west side of El Camino Real, and south of Kelly Drive, in the City of Carlsbad, San Diego County, California (see Figure 1, Site Location Map). Topographically, the site consists of a relatively flat lying upper "mesa" area within the western portion of the site. Along the eastern edge of the mesa, moderate slopes descend eastward toward a relatively flat lying "bottom," or alluviated area, located between the slope and the existing alignment of El Camino Real. Site drainage generally appears to be directed from the mesa onto offsite areas to the north, and eastward into the "bottom" area. Runoff within the bottom area appears to ultimately be directed via sheet flow and a small channel offsite to the north. Previous grading operations appear to have occurred within the bottom area, along the western edge of El Camino Real, and at the southernmost portion of the site, where



Base Map: TOPO!® ©2003 National Geographic, U.S.G.S. San Luis Rey Quadrangle, California -- San Diego Co., 7.5 Minute, dated 1997, current, 1999.



Base Map: Google Maps, Copyright 2016 Google, Map Data Copyright 2016 Google

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GSI GeoSoils, Inc.	W.O. 6971-A1-SC
SITE LOCATION MAP	
Figure 1	

existing fill appears to have been placed as part of an existing onsite, and adjacent, offsite residential development. Elevations range from approximately 110 feet above Mean Sea Level (MSL) on the mesa, down to approximately 46 feet MSL, within the "bottom" area, at the northeast corner of the site, for a total relief on the order of ±64 feet. Existing improvements to the site consist of a single-story commercial building and smaller outbuildings within the bottom area, and a single-family residential structure, sheds, and storage containers within the mesa. A communication facility (antennas) also appears to be located along the eastern edge of the mesa. Nursery facilities are also located within the northern and southern ends of the bottom area.

As indicated on a conceptual site plan provided by your office (SWA, 2015), the site will be prepared for the construction of a mixed use residential/commercial project. Cut and fill grading techniques are anticipated in order to bring the site to the desired grades. A review of the conceptual site plan, indicates maximum cut and fill thicknesses, on the order of up to approximately 25 to 35 feet, or less. Graded slopes are anticipated up to heights of approximately 25 feet at gradients of 2:1 (horizontal:vertical [h:v]) or flatter. Underground and street improvements are anticipated, along with bioretention basins and other storm water BMP's.

GSI anticipates that the construction would consist of wood frames with typical foundations and slab-on-grade ground floors. Building loads are assumed to be typical for this type of construction. Sewage disposal is anticipated to be connected into the regional, municipal system. Storm water may be treated onsite prior to its delivery into the municipal system.

FIELD STUDIES

Site-specific field studies were conducted by GSI during May, 2015, and consisted of the excavation of three (3) exploratory borings with a hollow stem auger drill rig, and ten (10) exploratory test pits with a rubber tire backhoe. These excavations were completed in order to evaluate soil conditions at depth, and the presence of groundwater. The borings and test pits were logged by a representative of this office who collected representative bulk and undisturbed soil samples for appropriate laboratory testing. The logs of the excavations are presented in Appendix B. The approximate location of the exploratory excavations are presented on the Geotechnical Map (see Plate 1), which uses a topographic survey completed by Aerotech Mapping (2016), as a base.

REGIONAL GEOLOGY

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 as 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 early Pleistocene time, a broad coastal plain was developed from the deposition of marine terrace deposits. During mid to late Pleistocene time, 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. Regional geologic mapping by Kennedy and Tan (2005, 1996) indicate the site is underlain with deposits of older Quaternary-age alluvium (formerly termed "terrace deposits") within the upper, mesa area, and younger, Quaternary-age alluvium overlying Eocene sediments at depth, within the bottom area.

SITE GEOLOGIC UNITS

General

The earth material units that were observed and/or encountered at the subject site generally consist of surficial deposits of undocumented fill, colluvium (topsoil), and alluvium, overlying formational deposits of Quaternary-age older alluvium, and Eocene-age sedimentary bedrock, belonging to the Santiago Formation, which occurs at depth throughout the site. A general description of each material type is presented as follows, from youngest to oldest. The general distribution of earth materials onsite is shown in plan view on Plate 1, and in cross section on Plate 2.

Undocumented Fill (Map Symbol - Afu)

Undocumented fill occurs locally throughout the upper, mesa area of the site as minor road embankments, and in the vicinity of existing telecommunication towers, with a thickness on the order of about 4 feet. Within the bottom area, located between El Camino Real and the base of the ascending, east facing slope, west of the existing commercial buildings/nurseries, undocumented fill appears to occur as a surficial layer of material, ranging in thickness from about 4 feet to 7 feet. An embankment of existing fill also occurs at the southeastern corner of the site, and appears to be supporting existing offsite residential development. Where encountered in our test excavations, undocumented fill varies from sandy clay to clayey sand, typically observed to be light brown to brown, slightly moist to moist, loose (clayey sand) and soft (sandy clay), and porous, with some plastic debris locally. Existing undocumented fill is generally considered to be potentially compressible in its existing state. As such, it should not be used for the support of settlement-sensitive improvements and/or any planned fill, unless adequately remediated. Where existing fills are supporting offsite residential structures, at the southeastern corner of the site, complete removal may not be possible. As such, a combination of partial removals and structural setbacks may be recommended, based on conditions exposed during grading.

Colluvium (Not Mapped)

A relatively thin surficial/near surface layer of colluvium (topsoil), ranging from about 1 to 5 feet in thickness, occurs within slope areas, and across the upper mesa area of the site, with thicker accumulations generally occurring within the lower reaches of swales developed on existing slopes. Where encountered, colluvium consists of brown to dark brown clayey sand and clay, typically observed to be moist and loose (clayey sand), and soft (clay), porous, locally desiccated (clay fraction), with few roots. Existing colluvial soil is considered potentially compressible in its existing state. As such, it should not be used for the support of settlement-sensitive improvements and/or any planned fill, unless adequately remediated.

Quaternary Alluvium (Map Symbol - Qal)

Quaternary-age deposits of alluvium were observed underlying undocumented fill within the "bottom" area of the site. Alluvial deposits were encountered to depths ranging from about 20 to 36 feet below existing surface grades, and appears to thicken to the north, along the long axis of the "bottom" area. Alluvium is anticipated to thin, and "pinch out" toward the based of the existing, east facing slope that descends toward the bottom area (see Plate 2, Geologic Cross Sections) and also thin towards El Camino Real, along the east side of the site.

Where encountered, these sediments generally consist of dark yellowish brown, grayish brown, and brown, interlayered clays, sandy clays, and clayey sands, typically observed to be moist, and loose (clayey sands), or soft (sandy clays/clays) within about 10 feet from existing surface grades, becoming wet to saturated, relatively dense and stiff with depth (with some exceptions).

The near surface zone of relatively loose and soft soil, at depths to about 10 feet below existing grades (including the overlying undocumented fill), is potentially compressible in its existing state. As such, it should not be used for the support of settlement-sensitive improvements and/or any planned fill, unless adequately remediated. Based on our observations and analysis, the underlying, relative denser deposits of alluvial soil may be left in place, provided that the recommendations presented herein are properly incorporated into the design and construction of the project.

Quaternary Older Alluvium (Map Symbol - Qoa)

Quaternary-age deposits of older alluvium were generally observed within the upper mesa area of the site and unconformably overly the older Eocene-age deposits of sedimentary bedrock. Based on test pit observations, a relatively flat lying contact between older alluvium and the underlying sedimentary bedrock occurs at an approximate elevation of 88 to 89 feet MSL. Based on a topographic high of about 110 feet MSL, the maximum thickness of older alluvium is on the order of about 21 to 22 feet. Where encountered, these sediments generally consist of interlayered gray brown, brown to light brown silty sands with clay, brown and olive brown clayey sands, and dark brown to olive brown clay. Older alluvium was typically observed to be moist, medium dense (sands), or very stiff (clays), with some carbonate mottling within near surface layers. Older alluvium is considered to be suitable bearing materials for the support of fills, or settlement-sensitive improvements.

Eocene Santiago Formation (Map Symbol - Tsa)

Eocene-age sedimentary bedrock, belonging to the Santiago Formation, underlies the entire site, beneath older alluvium within the upper mesa area, and beneath younger alluvium within the bottom area of the site, and forms the lower slopes below the contact with older alluvium. Where encountered, sedimentary bedrock consists of brown and light grayish yellow sandstone and yellowish brown to light grayish brown clayey sandstone, typically observed to be slightly moist to moist within slope areas, becoming wet to saturated at depth within the bottom area light gray and very dark gray, wet and dense, granitic rock, breaking to a silty sand upon excavation. A paleosol, with an observed thickness of 2 to 4 feet, and consisting of a very dark gray, moist and stiff clay with many gravel size carbonate nodules, appears to be developed within the Santiago Formation, at the contact with the overlying older alluvium locally. The Santiago Formation is considered to be suitable bearing materials for the support of fills, or settlement-sensitive improvements. The paleosol will require remediation, in the form of removal/recompaction where it is within about 7 feet of planned grade.

Geologic Structure

Quaternary-age deposits of older alluvium are generally massive to thickly bedded sediments, with poorly developed sub-horizontal orientation. The contact with the underlying Santiago Formation appears to be sub horizontal, and daylights within the existing east facing slope at a approximate elevation of about 88 to 89 feet MSL.

Bedding orientations within the Santiago Formation were not observed onsite. However, proprietary information from nearby sites indicate thin to thick bedding, and a general, regional trend of northwesterly to westerly to southwesterly shallow inclinations, ranging from near horizontal to about 8 degrees. Jointing within the Santiago Formation was typically highly inclined (65 to 89 degrees), predominately trending north to northwest, and to a lessor extent highly inclined to the northeast.

GROUNDWATER

Groundwater was encountered in borings located within the bottom area of the site, primarily as a perched water table within existing alluvium overlying sedimentary bedrock. Depths to groundwater encountered in these alluvial areas ranged from approximately 14 to 17 feet below existing grades, or at approximate elevations of about 31½ feet MSL (north) to 35 feet MSL (south). The local groundwater gradient appears to be from south to north, towards Agua Hedionda Lagoon.

Surface signs of water wells were not observed onsite during our site reconnaissance. In addition, there are no water wells reported within the site, as listed in a website for United States Geological Survey database (2016) and the California Department of Water Resources (2016). State of California regional groundwater maps indicate no permitted water wells existing within the subject site; therefore, a discussion of historic groundwater levels is not readily available. However, based on the relatively close proximity to relatively constant water levels associated with the coastline and adjacent lagoon, and relative low soil permeabilities, groundwater levels are considered to have remained relatively constant, from a historic perspective, and fluctuate with precipitation. Nearby, groundwater has fluxtuated between elevations ranging from ± 27 to 42½ feet MSL.

Regional groundwater should not adversely affect site development, provided that the recommendations presented in this report are properly incorporated into the design and construction of the project. The low lying portions of the site, particularly underlain by alluvium, may encounter groundwater at a shallow depth. These observations reflect site conditions at the time of our field evaluation and do not preclude changes in local groundwater conditions in the future from heavy irrigation or precipitation. In general, perched groundwater conditions, along zones of contrasting permeabilities, discontinuities, or fill lifts, may not be precluded from occurring in the future due to site irrigation, poor drainage conditions, or damaged utilities. Perched groundwater should be anticipated to occur after development, and may require additional mitigation when it manifests itself. Subdrains are typically used to control subsurface water in natural drainages that are proposed to be filled, and are recommended herein.

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, 2016). 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.

MASS WASTING/LANDSLIDE SUSCEPTIBILITY

The existing, east and north facing natural slopes, that descend from the mesa area, were evaluated by visual observation, exploratory test pits, and literature review, for the presence of landslide deposits. Landslide deposits were not noted within the site during field work, or during a review of Kennedy and Tan (2005), Tan and Giffen (1995), or Tan and Kennedy (1996) and no evidence of landslide deposits, and/or geomorphology indicative of landsliding (i.e., humocky topography, scarps, lobate soil deposits, etc.) was noted within these slopes during site work. Tan and Giffen (1995) evaluated the area as "generally susceptible" based on slope perceived to be near their stability limits due to weak materials or slope gradient. However, as indicated, landslide deposits were not noted on this site. Furthermore, typical site earthwork should mitigate any potential slope instability, should these conditions be encountered during site grading.

FAULTING AND REGIONAL SEISMICITY

Regional Faults

Our review indicates that there are no known active faults crossing the project and the site is not within an Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). However, the site is situated in an area of active faulting. The Rose Canyon fault zone is closest known active fault to the site (located at a distance of approximately 6.3 miles [10.1 kilometers]), and should have the greatest effect on the site in the form of strong ground shaking, should the design earthquake occur. The location of the Rose Canyon fault and other major faults relative to the site is shown on the "California Fault Map" in Appendix C. The possibility of ground acceleration, or shaking at the site, may be considered as approximately similar to the southern California region as a whole.

Local Faulting

Although active faults lie within several miles of the site, no local active faulting was noted in our review, nor observed to specifically transect the site during the field investigation. Additionally, a review of available regional geologic maps does not indicate the presence of local active faults crossing the specific project site. However, an "inactive" fault transects the site, as shown on the City of Carlsbad "Fault Location and Seismically Induced Ground Shaking Map" (Leighton &Associates, et al., 1992).

Seismicity

It is our understanding that site-specific seismic design criteria from the 2013 California Building Code ([2013 CBC], California Building Standards Commission [CBSC], 2013), are to be utilized for foundation design. Much of the 2013 CBC relies on the American Society of Civil Engineers (ASCE) Minimum Design Loads for Buildings and Other Structures (ASCE Standard 7-10). The seismic design parameters provided herein are based on the 2013 CBC.

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 0.56g. The computer printouts of pertinent portions of the EQFAULT program are included within Appendix C.

Historical site seismicity was evaluated with the acceleration-attenuation relation of Bozorgnia, Campbell, and Niazi (1999), and the computer program EQSEARCH (Blake, 2000b, updated to January 2015). This program performs a search of the historical earthquake records for magnitude 5.0 to 9.0 seismic events within a 100-kilometer radius, between the years 1800 through January 2015. Based on the selected acceleration-attenuation relationship, a peak horizontal ground acceleration is estimated, which may have affected the site during the specific event listed. Based on the available data and the attenuation relationship used, the estimated maximum (peak) site acceleration during the period 1800 through January 2015 was about 0.26g. 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 C.

For the evaluation of liquefaction potential and settlement onsite, and in general accordance with California Department of Conservation (2008), a probabilistic seismic hazards analysis was performed using a PSHA Interactive Deaggregation computer program provided by the USGS (2014). Based on a review of these data, and considering the relative seismic activity of the southern California region, probabilistic horizontal site accelerations (PHSA's) of 0.23g and 0.44 were evaluated. These values correspond to a 10 percent, and 2 percent, respectively, probability of exceedance in 50 years. For other design aspects of site foundation design and construction, a probabilistic seismic hazards analysis was performed using the computer program "Seismic Design Maps," provided by the United States Geologic Survey (USGS, 2014).

Seismic Shaking Parameters

Based on the site conditions, the following table summarizes the updated site-specific design criteria obtained from the 2013 CBC (CBSC, 2013), Chapter 16 Structural Design, Section 1613, Earthquake Loads. The computer program "U.S. Seismic Design Maps, provided by the United States Geologic Survey (USGS, 2014) was utilized for design (http://geohazards.usgs.gov/designmaps/us/application.php). The short spectral response utilizes a period of 0.2 seconds.

2013 CBC SEISMIC DESIGN PARAMETERS		
PARAMETER	**VALUE**	**2013 CBC/ASCE REFERENCE**
Risk Category	II	Table 1604.5
Site Class	D	Section 1613.3.2/ASCE 7-10 (p. 203-205)
Spectral Response - (0.2 sec), S_s	1.093	Section 1613.3.1 Figure 1613.3.1(1)
Spectral Response - (1 sec), S_1	0.420	Section 1613.3.1 Figure 1613.3.1(2)
Site Coefficient, F_a	1.063	Table 1613.3.3(1)
Site Coefficient, F_v	1.580	Table 1613.3.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec), S_{MS}	1.162	Section 1613.3.3 (Eqn 16-37)
Maximum Considered Earthquake Spectral Response Acceleration (1 sec), S_{M1}	0.664	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	0.774	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.443	Section 1613.3.4 (Eqn 16-40)
PGA_M	0.457 g	ASCE 7-10 (Eqn 11.8.1)
Seismic Design Category	D	Section 1613.3.5/ASCE 7-10 (Table 11.6-1 or 11.6-2)

GENERAL SEISMIC PARAMETERS	
PARAMETER	**VALUE**
Distance to Seismic Source (Rose Canyon)	6.3 mi (10.1 km)[(1)]
Upper Bound Earthquake (Rose Canyon)	$M_W = 7.2$[(2)]
Probabilistic Horizontal Site Acceleration ([PHSA] 2% probability of exceedance in 50 years)	0.44g
Probabilistic Horizontal Site Acceleration ([PHSA] 10% probability of exceedance in 50 years)	0.23g
[(1)] - From 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 2013 CBC (CBSC, 2013) and regular maintenance and repair following locally significant seismic events (i.e., M_w5.5) will likely be necessary, as is the case in all of southern California.

SECONDARY SEISMIC HAZARDS

Liquefaction

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]).

The susceptibility of a site to liquefaction 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. About three of these five concurrent conditions have the potential to occur and/or exist on the site. Thus, it would appear that liquefaction potential may be a significant hazard to any proposed development. Given the intended development, the potential site accelerations, the relatively low density granular soils (i.e., Unified Soil Classification System [USCS] soil types SP, SM, and SC) occurring within the upper 100 feet of the soil profile, and the relatively high elevation of the groundwater, GSI has performed a liquefaction analysis for the proposed development.

The condition of liquefaction has two principal effects. One is the volumetric strain or "consolidation" of loose sediments with resultant settlement of the ground surface. The other effect is lateral sliding. Significant permanent lateral movement generally occurs only when there is significant differential loading, such as fill or natural ground slopes within susceptible materials. Plans do not indicate any fill slopes, however to the east of the property is a flood control channel and planned water retention near the top of this channel on the eastern development area. Therefore, there is some potential for lateral spreading to affect the site along the eastern property line near the flood control channel that GSI has evaluated herein.

The evaluation of whether or not surface manifestation of liquefaction, such as sand boils, ground fissures, and cracking, etc., will occur at a site was made using Special Publication 117A "Guidelines for Analyzing and Mitigating Liquefaction in California" (CGS, 2008), as well as ASCE 7-10. Based on the thickness of the potentially liquefiable layer, the minimum to maximum thickness of the non-liquefiable soil, depth of bedrock (about 20 to 36 feet), and ground acceleration for the design earthquake, an evaluation of these potentially liquefiable soils was made. Our evaluation indicates a very low potential for surface manifestation to occur onsite, except around manholes or utility risers, where that potential would increase.

Seismic-Induced Settlement, Liquefaction and Densification

Seismic-induced ground motions from earthquakes can result in the volumetric strain caused by the excess pore pressures generated in saturated soils. This volumetric strain, in the absence of lateral flow or spreading, can result in manifestation of settlement once excess pore pressure is reduced. The same volumetric strain may also occur in unsaturated earth materials above the water table in relatively dry (well below optimum moisture as defined by ASTM D 1557) to moist, loose to medium dense granular (sandy) soils. This potential for seismic densification of dry to moist, unmitigated alluvial soil below the fill and above the groundwater is significantly lower in magnitude of settlement than that below the water table. A quantitative discussion of "dynamic settlement" is presented in a later section of this report.

Lateral Spreading

Lateral spread phenomenon is described as the lateral movement of stiff, surficial, mostly intact blocks of sediment or compacted fill displaced downslope towards a free face along a shear zone that has formed within the liquefied sediment. The resulting ground deformation typically has extensional fissures at the head of the failure, shear deformations along the side margins, and compression or buckling of the soil at the toe. The extent of lateral displacement typically ranges from less than an inch to several feet. Two types of lateral spread can occur: 1) lateral spread towards a free face (e.g., river/creek channel or embankment); and 2) lateral spread down a gentle ground slope where a free face is absent. Factors such as earthquake magnitude, distance from the seismic energy source, thickness of the liquefiable layers, and the fines content and particle size of those sediments also correlate with ground displacement. Since no free face occurs on this project, no adjacent lake is present, the only lateral spread that may possibly occur would be low magnitude and due to slight elevation variations already within the established vicinity. The margins of the developed/undeveloped portions of the site, along its northern edge, may exhibit some lateral movement toward the natural or undeveloped areas.

Surface Manifestation of Secondary Seismic Events

The evaluation of whether or not surface manifestation of liquefaction, such as sand boils, ground fissures, foundation tilt and cracking, etc., will occur at a site is made using guidelines contained in Special Publication 117A "Guidelines for Evaluating and Mitigating Seismic Hazards in California" (California Department of Conservation, California Geological Survey [CGS], 2008). Based on the thickness of the potentially liquefiable layer(s), the thickness of the non-liquefiable soil cover, and ground acceleration for the design earthquake, an evaluation of these "liquefied" soils was made. Due to the depth of the groundwater surface (design basis elevation of approximately 12 to 14 feet below the existing ground surface), the potential for sand boils is considered low depending on the depth of groundwater, and the potential for surface soil settlement is also considered low to moderate. Utilities that are embedded in unmitigated alluvial soils, if planned, may be impacted on a moderate level. Sand boils may occur if deep (> 7 to 8 feet) manholes or utility vaults are embedded into or near the groundwater saturated alluvium. This may be mitigated through design and planning.

Planned new fills (planned and remedial grading) in areas underlain with alluvium are anticipated to be relatively thick, locally in excess of 25 to 35 feet. On this site, the thickness of the potentially liquefiable layers is less than the overlying unsaturated alluvium and densified fill soils. Therefore, it is our opinion that the potential for surface manifestation of liquefaction at the site, in the event of the design earthquake, is considered low, based on the assumed design. Furthermore, based upon our analyses, it appears that a non-liquefiable soil cover consisting of remediated alluvium, and new fill will generally mitigate the surface manifestation of liquefaction and densification. The potential for distress/deformation from surface manifestations of liquefaction should be mitigated to levels similar to the existing nearby residential developments, provided our recommendations are properly adhered to during design and construction.

Other Secondary Seismic Hazards

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

- Subsidence
- Ground Lurching or Shallow Ground Rupture
- Tsunami
- Seiche

LABORATORY TESTING

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

Classification

Soils were visually 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 of the onsite soils are provided on the Test Pit Logs in Appendix B.

Moisture-Density Relations

The field moisture contents and dry unit weights were determined for selected undisturbed samples in the laboratory in general accordance with ASTM D 2937. The dry unit weight was determined in pounds per cubic foot (pcf), and the field moisture content was determined as a percentage of the dry unit weight. The results of these tests are indicated on the Boring Log in Appendix B.

Laboratory Standard

The maximum density and optimum moisture content was evaluated for the major soil type encountered in the borings, in general accordance with the laboratory standard, ASTM D 1557. The moisture-density relationships obtained for this soil are shown on the following table:

LOCATION	SOIL TYPE	MAXIMUM DENSITY (PCF)	OPTIMUM MOISTURE CONTENT (%)
TP-1 @ 6'	Dark Yellowish Brown, Clayey SAND	124.0	11.5

Expansion Index

Representative samples of near-surface soils were tested for expansivity in general accordance with ASTM D 4829, and were classified accordingly. The laboratory test results are presented in the following table.

LOCATION	EXPANSION INDEX	EXPANSION POTENTIAL*
TP-1 @ 6'	17	Very Low
TP-4 @ 1-1½'	128	High
TP-8 @ 5½'	76	Medium
Per ASTM 4829, E.I. = 0-20, very low; 21-50, low; 51-90, medium; 91-130, high; > 130, very high		

Atterberg Limits

Tests were performed to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. The test results are presented in Appendix D, and the table below:

LOCATION	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
TP-4 @ 1-1½'	70	23	47
TP-8 @ 5½'	49	19	30

Particle-Size Analysis

A particle-size evaluation was performed on representative soil samples in general accordance with ASTM D 422-63. The grain-size distribution curves are presented in Appendix D.

Consolidation Testing

Consolidation testing was performed on three (3) relatively undisturbed soil samples in general accordance with ASTM test method D 2435. The consolidation test results are presented in Appendix D.

Direct Shear

A strain-controlled direct shear test was performed on remolded and relatively undisturbed samples collected from the borings in general conformance with the ASTM D 3080 test method. The test results are summarized below and presented in Appendix D.

SAMPLE LOCATION AND DEPTH (FT)	PRIMARY		RESIDUAL	
	COHESION (PSF)	FRICTION ANGLE (DEGREES)	COHESION (PSF)	FRICTION ANGLE (DEGREES)
TP-1 @ 6'	47	32	29	32

Saturated Resistivity, pH, and Soluble Sulfates, and Chlorides

GSI conducted sampling of onsite earth materials for general soil corrosivity and soluble sulfates, and chlorides testing. The testing included evaluation of soil pH, soluble sulfates, chlorides, and saturated resistivity. Test results are presented in Appendix D and the following table:

SAMPLE LOCATION AND DEPTH (FT)	pH	SATURATED RESISTIVITY (ohm-cm)	SOLUBLE SULFATES (% by wt.)	SOLUBLE CHLORIDES (ppm)
TP-1, TP-4, TP-5 composite	9.22	1200	0.0237	112

Corrosion Summary

Laboratory testing indicates that the tested sample of the onsite soil is alkaline with respect to soil acidity/alkalinity, is severely corrosive to exposed, buried metals when saturated, present a "not applicable" per ACI 318-11 for sulfate exposure to concrete, and chloride levels appear somewhat elevated, but below action levels (Caltrans, 2003). Reinforced concrete mix design for foundations, slab-on-grade floors, and pavements should minimally conform to "Exposure Class C1" in Table 4.2.1 of ACI 318-11, as concrete would likely be exposed to moisture. It should be noted that GSI does not consult in the field of corrosion engineering. The client and project architect should agree on the level of corrosion protection required for the project and seek consultation from a qualified corrosion consultant as warranted.

PRELIMINARY SETTLEMENT ANALYSIS

GSI has estimated the potential magnitudes of total settlement, differential settlement, and angular distortion for the site. The analyses were based on laboratory test results and upon a review of proprietary data from an adjacent site. Site specific conditions affecting settlement potential include depositional environment, grain size and lithology of sediments, cementing agents, stress history, moisture history, material shape, density, void ratio, etc.

Ground settlement should be anticipated due to primary consolidation and secondary compression of the left-in-place alluvium and compacted fills. The total amount of settlement, and time over which it occurs, is dependent upon various factors, including material type, depth of fill, depth of removals, initial and final moisture content, and in-place density of subsurface materials.

Post-Grading Settlement of Compacted Fill Overlying Formational Soil/Bedrock

Compacted fills, to the thicknesses anticipated, and placed on suitable formational soil, are not generally prone to excessive settlement. Based on our analysis, total settlements (static and seismic) on the order of 2 inches or less, for fills up to approximately 30 feet in thickness, and up to 3 inches for planned fills up to approximately 50 feet in thickness, should be anticipated. These anticipated settlements are for fills overlying bedrock, or dense formational earth materials. Differential settlements are anticipated to be 1 inch in 40 feet for fills up to 30 feet, and 2 inches in 40 feet for fills up to about 50 feet thick. Settlement estimates do not include effects of slope creep that may occur to improvements placed within the slope setback zone as defined by the 2013 CBC.

Post-Grading Settlement of Compacted Fill Overlying Alluvium

Where these materials are left in place, settlement of the underlying saturated alluvium is anticipated due to the weight of added planned fills. The magnitude of this settlement will vary with the proposed fill heights (i.e., measured from existing grades), and the thickness, texture, and compressibility of the underlying, left-in-place saturated alluvium. Due to the predominantly fine grained texture of the alluvial soils onsite, settlement of the alluvial soil will occur over time. Should conditions result in leaving alluvium in place, total static settlements on the order of up to 5 inches should be anticipated within areas of the deepest fill/thickest alluvial section left in place. When a sufficient waiting period is applied, (i.e., at least 2 to 4 months), an adequate amount of settlement may occur to allow for a less onerous design. Total settlement may be revised, dependant on conditions exposed during grading, results of settlement monitoring, and the actual amount of alluvial material removed and left in place. Total settlements are summarized in the foundation section of this report.

Monitoring

Areas where alluvial soil is left-in-place should be monitored and the settlement values revised based on actual field data. Monitoring should include the measurement of any horizontal and vertical movements of the fill. Locations and type of surface monitoring devices should be selected as soon as the total fill thickness is placed. Alternatively, settlement monitoring may be of the subsurface type and placed at the fill/alluvium contact. The program of monitoring should be agreed upon between the project team, the site surveyor and the Geotechnical Engineer of Record, prior to excavation.

For a survey monitoring system, an accuracy of a least 0.01 foot should be required. Reference points should be installed, and read, immediately after the completion of grading in the area of concern.

The frequency of readings will depend upon the results of previous readings and the rate of construction. Weekly readings could be assumed initially, with the frequency adjusted, based on the previous set of readings. The reading should be plotted by the Surveyor and then reviewed by the Geotechnical Engineer.

For grading adjacent to exiting streets that are to remain, pre-construction surveys including photo documentation of existing conditions should be performed.

Dynamic Settlement of Fill Over Alluvium

The magnitude of potential seismic settlement was computed using various methods within the LiquefyPro program in general accordance with Special Publication 117A "Guidelines for Evaluating and Mitigating Seismic Hazards in California" (California Department of Conservation, California Geological Survey [CGS], 2008). This includes the recommended use of a Factor-of-Safety (FOS) = 1.3, and site accelerations evaluated as on the order of equivalent to PGAm, and/or a 10 percent probability of occurrence in 50 years. Based upon the assumed current design configuration (i.e., ground improvement through remedial earthwork and fill loading) and the results of our seismic settlement analysis, the total ground settlement, across a typical lot, during the design basis seismic event is anticipated to be on the order of ½ to 1½ inch, with a potential differential settlement of approximately ¾ inch over 40 feet horizontally (i.e., angular distortion approximately 1/960). The area of highest potential for differential settlement is near the northern boundary of the site, and where any buildings straddle the contact between left in place alluvium and formation. This level of deformation should be considered in foundation design and planning, in addition to foundation settlement under static loading conditions. This anticipated seismic-induced settlement may be mitigated by foundation type, grading and/or ground modification. The current analysis is included in Appendix E.

Settlement Due to Structural Loads

The settlement of the structures supported on strip and/or spread footings founded on compacted fill will depend on the actual footing dimensions, the thickness and compressibility of compacted fill below the bottom of the footing, and the imposed structural loads. Provided the thickness of compacted fill below the bottom of the footing is at least equal to the width of the footing, and based on a maximum allowable bearing pressure of 3,000 pounds per square foot (psf), provided in this report, total settlement of less than ½ inch should be anticipated. The majority of this settlement should occur as the building loads are applied during construction. Differential settlement between the lightest and heaviest loaded footings may occur if the foundation is of the conventional type and is anticipated to be on the order of ¼ inch.

Summary of Settlement Analysis

A summary of design settlements for foundations are presented in a later section of this report regarding foundation design/construction.

SUBSIDENCE

Subsidence is a phenomenon whereby a lowering of the ground surface occurs as a result of a number of processes. These include dynamic loading during grading, fill loading, fault activity, or fault creep, as well as groundwater withdrawal.

An analysis of fill loading is presented in the previous section. Ground subsidence (consolidation), due to vibrations, would depend on the equipment being used, the weight of the equipment, repetition of use, and the dynamic effects of the equipment. Most of these factors cannot be evaluated and may be beyond ordinary estimating possibilities. However, it is anticipated that any additional settlement from processes other that fill loading would be relatively minor (on the order of 1 inch or less, which should occur during grading), and should not significantly affect site development. The effect of fill loading on alluvial soil has been evaluated in the previous section.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on our field exploration, laboratory testing, and geotechnical engineering analysis, it is our opinion that the subject site is suitable for the proposed residential 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 and improvements are:

- Earth materials characteristics and depth to competent bearing material.
- On-going expansion and corrosion potential of site soils.
- Erosiveness of site earth materials.
- Settlement potential.
- Relatively high groundwater table and potential for perched water during and following site development.
- Potential for storm water infiltration.
- 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 verified or modified in writing by this office. Foundation design parameters are considered preliminary until the foundation design, layout, and structural loads are provided to this office for review.

EARTHWORK CONSTRUCTION RECOMMENDATIONS

General

All earthwork should conform to the guidelines presented in the 2013 CBC (CBSC, 2013), the requirements of the City of Carlsbad, and the General Earthwork and Grading Guidelines presented in Appendix G, except where specifically superceded in the text of this report. Prior to earthwork, a GSI representative should be present at the preconstruction meeting to provide additional earthwork guidelines, if needed, and review the earthwork schedule. This office should be notified in advance of any fill placement, supplemental regrading of the site, or backfilling underground utility trenches and retaining walls after rough earthwork has been completed. This includes grading for driveway approaches, driveways, and exterior hardscape.

During earthwork construction, all site preparation and the general grading procedures of the contractor should be observed and the fill selectively tested by a representative(s) of GSI. If unusual or unexpected conditions are exposed in the field, they should be reviewed by this 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 save working environment for our field staff who are onsite. GSI does not consult in the area of safety engineering.

Demolition/Grubbing

1. Vegetation and any miscellaneous debris should be removed from the areas of proposed grading.

2. Any existing subsurface structures uncovered during the recommended removal should be observed by GSI so that appropriate remedial recommendations can be provided.

3. Cavities or loose soils remaining after demolition and site clearance should be cleaned out and observed by the soil engineer. The cavities should be replaced 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.

4. Onsite septic systems (if encountered) should be removed in accordance with San Diego County Department of Environmental Health standards/guidelines.

Treatment of Existing Ground

1. Removals should consist of all surficial deposits of undocumented fill, colluvium, and near surface alluvium within the "bottom" area." Removals depths for undocumented fill within existing slope areas, and within the upper "mesa" area are anticipated to be on the order of 2 to 5 feet, with locally deeper removals possible where paleosols developed within the Santiago Formation are within 7 feet of planned finished grades. Within the bottom area, removals consisting of all undocumented fill, and the upper portion of existing alluvium, should be completed to at least 10 feet below existing grades. These soils may be re-used as fill, provided that the soil is cleaned of any deleterious material and moisture conditioned, and compacted to a minimum 90 percent relative compaction per ASTM D 1557. Removals should be completed throughout the site, and minimally at least 5 feet beyond the limits of any settlement-sensitive improvement, or to a distance equal to the depth of removal, whichever is greater.

2. Where removals are completed to less than 4 feet below finish subgrade elevations, or the thickness of planned fills with removals is less than 4 feet, the building pad should be undercut to provide a minimum 4-foot thick fill cap.

3. Plan cut lots, cut/fill transition lots, or lots where the as-built fill thickness is less than 4 feet, should be undercut to provide a minimum 4-foot thick fill cap.

4. Where the maximum as-built fill thickness is greater than 3 times the minimum fill thickness within a given lot, the lot should be undercut in order to provide a minimum fill thickness that is at least ⅓ the maximum fill thickness.

5. Where existing fills supporting offsite residential structures near the southeastern corner of the site, the existing fill should be removed above a 1:1 projection down and away from the property line to where the projection encounters suitable earth material, and the existing fill heavily benched during subsequent fill placement. Structural setbacks may be necessary, and subsequently recommended, based on final development plans, including planned building locations.

6. Subsequent to the above removals/overexcavation, the exposed bottom should be scarified to a depth of at least 8 inches, brought to at least optimum moisture content, and recompacted to a minimum relative compaction of 90 percent of the laboratory standard, prior to any fill placement. If properly cross ripped and processed, this processed zone may be included in the overall minimum fill cap thickness.

7. Localized deeper removals may be necessary due to unforeseen subterranean structures, septic systems, etc. The project soils engineer/geologist should observe all removal areas during the grading.

8. Existing fill and removed natural ground materials may be reused as compacted fill provided that major concentrations of vegetation and miscellaneous debris are removed from the site, prior to or during fill placement.

Fill Suitability

Existing earth materials onsite should generate relatively fine grained, granular fill material. However, oversize material (i.e., greater than 12 inches in long dimension) may be present, and/or generated locally, as part of demolition operations for the existing structure(s).

Any soil import should be evaluated by this office prior to importing in order to assure compatibility with the onsite site soils and the recommendations presented in this report. Import soils, if used, should be relatively sandy and low expansive (i.e., expansion index less than 50).

Fill Placement

1. Subsequent to ground preparation, fill materials should be brought to at least optimum moisture content, placed in thin 6- to 8-inch lifts, and mechanically compacted to obtain a minimum relative compaction of 90 percent of the laboratory standard.

2. Fill materials should be cleansed of major vegetation and debris prior to placement.

3. Any import materials should be observed and deemed suitable by the soils engineer prior to placement on the site. Foundation designs may be altered if import materials have a greater expansion value than the onsite materials encountered in this investigation.

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:

Existing Artificial Fill .... 5% to 10% shrinkage
Colluvium .... 3% to 8% shrinkage
Alluvium .... 10% to 15% shrinkage
Older Alluvium .... 2% to 3% shrinkage or bulk
Santiago Formation .... 2% to 3% shrinkage or bulk

It should be noted that the above factors are estimates only, based on preliminary data. Alluvium may achieve higher shrinkage if organics or clay content is higher than anticipated. 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.

Fill Drainage

Slope subdrainage may be recommended for any perimeter fill slope, based on conditions exposed during site grading. Due to the anticipated contrast in permeability between the earth materials onsite, subdrains may be necessary, and subsequently recommended. Schematic details of subdrains are provided in Appendix G.

Slope Considerations and Slope Design

Graded Slopes

All slopes should be designed and constructed in accordance with the minimum requirements of City/County, the 2013 CBC (CBSC, 2013), and the recommendations in Appendix G. Slopes constructed with sand fractions of the terrace deposits or Santiago Formations are anticipated to have erosion and surficial instability issues if left unplanted, and without engineered surface drainage control, and as such, will require periodic and regular maintenance.

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. Temporary slopes, up to a maximum height of ±20 feet, may be excavated at a 1:1 (h:v) gradient, or flatter, provided groundwater and/or running sands are not exposed. In the case of groundwater, CAL-OSHA and/or OSHA requirements for Type "C" soils will apply. Construction materials or soil stockpiles should not be placed 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.

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 planned 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 additions 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.

Expansive Soils

Current laboratory testing indicates that the onsite soils exhibit an expansion index(E.I.) values ranging from about 17 (very low) to 128 (high), with a plasticity index (P.I.) evaluated between 30 and 47. As such, site soil meets the criteria of detrimentally expansive soils as defined in Section 1803.5.2 of the 2013 CBC. Foundation systems constructed within the influence of detrimentally expansive soils (i.e., E.I. > 20 and P.I. ≥ 15) will require specific design to resist expansive soil effects per Sections 1808.6.1 or 1808.6.2 of the 2013 CBC, and should be reviewed by the project structural engineer.

The following foundation construction recommendations are intended to support planned improvements underlain by at least 7 feet of non-detrimentally expansive soils (i.e., E.I.<21 and P.I. <15), and are considered "minimum" values/criteria, based on the presence of expansive soil onsite. As indicated, foundations underlain by expansive soils will require specific design to mitigate expansive soil effects as required in Sections 1808.6.1 or 1808.6.2 of the 2013 CBC.

Preliminary Foundation Design

1. The foundation systems should be designed and constructed in accordance with guidelines presented in the 2013 CBC.

2. An allowable bearing value of 2,000 pounds per square foot (psf) may be used for the design of footings that maintain a minimum width of 12 inches and a minimum depth of 12 inches (below the lowest adjacent grade) and are founded entirely into properly compacted, engineered fill. This value may be increased by 20 percent for each additional 12 inches in footing depth to a maximum value of 2,500 psf. These values may be increased by one-third when considering short duration seismic or wind loads. Isolated pad footings should have a minimum dimension of at least 24 inches square and a minimum embedment of 24 inches below the lowest adjacent grade into properly engineered fill. Foundation embedment depth

excludes concrete slabs-on-grade, and/or slab underlayment. Foundations should not simultaneously bear on formational soils and engineered fill.

3. For foundations deriving passive resistance from 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 2013 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.

8. Footings for structures adjacent to retaining walls should be deepened so as to extend below a 1:1 projection from the heel of the wall. Alternatively, walls may be designed to accommodate structural loads from buildings or appurtenances as described in the "Retaining Wall" section of this report.

Foundation Settlement Summary

Designing residential foundations for the existing soil conditions, the estimated settlement and angular distortion values that an individual structure could be subjected to should be evaluated by a qualified structural engineer. In addition, significant site improvements such as retaining walls, sound walls, spas, pools, or other settlement-sensitive improvements should be evaluated by a structural engineer given the site conditions and geotechnical parameters expressed in this report. The levels of angular distortion were evaluated on a 40-foot length assumed as the minimum dimension of buildings; if, from a structural standpoint, a decreased or increased length over which the differential settlement is assumed to occur is justified, this change should be incorporated into the design. This also applies to the other site improvements previously discussed.

The proposed residential structures should be designed in accordance with the following Table, on a preliminary basis:

TABLE - SETTLEMENT DESIGN SUMMARY		
AS-BUILT CONDITIONS	**TOTAL SETTLEMENT STATIC AND SEISMIC (Inches)**	**DESIGN DIFFERENTIAL SETTLEMENT (STATIC AND SEISMIC)**
Compacted Fills less than 30 feet thick and underlain with suitable formational soil/bedrock	2	1 inch in 40 feet
Compacted Fills greater than 30 and less than 50 feet thick and underlain with suitable formational soil/bedrock	3	1½ inches in 40 feet
Compacted fills less than 30 feet in thickness overlying alluvial soil left in place	3-5	2 inches in 40 feet (1 inch is 40 feet after a 60 to 90 day wait period)
Compacted fills greater than 30 feet in thickness overlying alluvial soil left in place	5-6	2½ to 3 inches in 40 feet (1 inch in 40 feet after a 90 to 120 day wait period)
NOTE: Settlement potential in areas underlain with alluvial soil may be reduced when an appropriate waiting period is incorporated into construction or alternative grading techniques are used, and/or settlement monitoring indicates otherwise.		

These settlement estimates do not include top of slope deformation (within code setback zones) if improvements are planned in these areas, and fills that have been saturated due to utility leaks, pool leaks, or excessive landscape irrigation. Post-construction settlement of the fill should be mitigated by proper foundation design, provided the design parameters, provided herein and in previous reports for the site, are properly utilized in final design of the residential foundation systems. Seismic settlement of fill over bedrock may be significantly reduced by increasing compaction standards of fill soils. In addition to the above, the structural engineer should also consider estimated settlements due to short duration seismic loading and applicable load combinations, as required by the City/County and/or the 2013 CBC.

PRELIMINARY FOUNDATION CONSTRUCTION RECOMMENDATIONS

Current laboratory testing indicates that some onsite soils meet the criteria of detrimentally expansive soils as defined in Section 1803.5.2 of the 2013 CBC. The following foundation construction recommendations are presented as a minimum criteria from a soils engineering viewpoint, where the planned improvements are underlain by at least 7 feet, and perhaps more (as determined during grading), of non-detrimentally expansive soils (i.e., E.I.<21 and P.I. <15). Should foundations be underlain by expansive soils, they will require specific design to mitigate expansive soil effects as required in Sections 1808.6.1 or 1808.6.2 of the 2013 CBC.

1. Exterior and interior footings should be founded into engineered fill at a minimum depth of 18 inches below the lowest adjacent grade, and a minimum width of 12 or 15 inches, for the planned one- or two-story floor load structures, respectively.

Isolated, exterior column and panel pads, or wall footings, should be at least 24 inches, square, and founded at a minimum depth of 24 inches into properly engineered fill. 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 column footings, and perimeter wall footings, should be tied together via grade beams 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 two No.4 reinforcing bars at the top, and two 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.

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 5 inches is recommended. Recommendations for floor slab underlayment are presented in a later section of this report.

5. Concrete slabs should be minimally reinforced with No. 3 reinforcement bars placed at 18-inch on centers, in two horizontally perpendicular directions (i.e., long axis and short axis), and as determined by the structural engineer/slab designer.

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. Specific slab subgrade pre-soaking is recommended for these soil conditions. Prior to the placement of underlayment sand and vapor retarder, GSI recommends that the slab subgrade materials be moisture conditioned to at least optimum moisture content to a minimum depth of 12 inches for very low expansive soil conditions; to at least 2 percent over optimum moisture content (or 1.2 times) to a depth of 18 inches, for medium expansive soils; and 3 percent over optimum moisture content (or 1.3 times) to a depth of 24 inches, for highly expansive soils. Slab subgrade pre-soaking should be evaluated by the geotechnical consultant within 72 hours of 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 conform to "Exposure Class C1" in Table 4.3.1 of ACI 318-11 since concrete would likely be exposed to moisture.

Stiffened Slabs

All foundations supported by expansive soils (as defined per Section 1803.5.3 of the 2013 CBC), shall be in compliance with Section 1808.6 of the 2013 CBC (CBSC, 2013), and the findings of this report.

For a typical slab designed with interior ribs, or stiffeners, the slab should minimally be at least 5 inches thick. The ribs should be provided in both transverse and longitudinal directions. The interior rib spacing and depth should be provided by the project structural engineer. The perimeter beams, however, should be embedded at least 18 inches for medium expansive, and 24 inches for soils with high expansion potential, and in consideration of the building type. The embedment depth should be measured downward from the lowest adjacent grade surface to the bottom of the beam.

Structural Mat Foundations - Design/Construction

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 is assumes that the bearing soils will consist of engineered fills with an average relative compaction of 90 percent of the laboratory (ASTM D 1557), overlying dense formational earth materials.

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

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 (PI) to be used in mat foundation design for various expansive soil conditions are presented in the following table.

LOW EXPANSION (E.I. = 0-50)	MEDIUM EXPANSION (E.I. = 51-90)	HIGH EXPANSION (E.I. = 91-130)
K_S = 100 pci/inch, PI < 15	K_S = 85 pci/inch, PI = 25	K_S = 70 pci/inch, PI = 35

Reinforcement bar sizing and spacing for mat slab foundations should be provided by the structural engineer. Mat slabs may be uniform thickness foundations (UTF) or may incorporate the use of edge footings for moisture cut-off barriers as recommended herein for post-tension foundations. Edge footings should be a minimum of 6 inches thick. The bottom of the edge footing should be designed to resist tension, using reinforcement per the structural engineer. The need and arrangement of interior grade beams (stiffening beams) will be in accordance with the structural consultant's recommendations. The recommendations for a mat type of foundation assume that the soils below the slab are compacted fill overlying dense, unweathered formational earth materials. The parameters herein are to mitigate the effects of expansive soils and should be modified to mitigate the effects of the total and differential settlements reported earlier in this report.

GSI recommends that the slab subgrade materials be moisture conditioned per recommendations presented in the previous section on general foundation construction.

In order to mitigate the effects from post-development perched water and to impede water vapor transmission, structural mats, shall be in accordance with Table 4.2.1 of the ACI (2011) per the 2013 CBC (CBSC, 2013), for low permeability concrete (i.e., a maximum water-cement ratio of 0.50). Recommendations for slab underlayment and soil moisture transmission considerations are presented in a later section of this report.

Nuisance cracking may be lessened by the addition of engineered reinforcing fibers in the concrete and careful control of water/cement ratios. For below grade structures (garages, etc.) epoxy-coated reinforcing bars should be considered and are dependent on the structural consultant's waterproofing and corrosion specialists' recommendations.

<u>Post-Tension Slab Foundations</u>

Post-tension (PT) slab foundation may also be used to support structures overlying expansive soils. PT slab foundations should be designed in accordance with 2013 CBC (CBSC, 2013), the criteria for the expansive soil conditions prevalent onsite, and per the PTI Method (3rd Edition).

The following table presents foundation design parameters for post-tensioned slab foundations relative to a specific range of soil expansion potential in accordance with the 2013 CBC and the PTI Method (3rd Edition).

Correction Factor in Integration	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
Plasticity Index (P.I.)*	15-45
* The effective plasticity index should be evaluated for the upper 7 to 15 feet of earth materials.	

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

POST-TENSION FOUNDATION DESIGN			
DESIGN PARAMETER[3]	EXPANSION POTENTIAL		
	VERY LOW TO LOW	MEDIUM	HIGH
e_m center lift	9.0 feet	8.7 feet	8.5 feet
e_m edge lift	5.2 feet	4.5 feet	4.0 feet
y_m center lift	0.4 inches	0.50 inches	0.66 inches
y_m edge lift	0.7 inch	1.3 inch	1.7 inch
Bearing Value [1]	1,000 psf [1]	1,000 psf [1]	1,000 psf [1]
Lateral Pressure	250 psf	175 psf	150 psf
Subgrade Modulus (k)	100 pci/inch	85 pci/inch	70 pci/inch
Minimum Perimeter Footing Embedment [2]	12 inches	18 inches	24 inches

[1] Internal bearing values within the perimeter of the post-tension slab 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] As measured below the lowest adjacent compacted subgrade surface (not including slab underlayment layer thickness).
[3] Post-tension slab design should also be evaluated with respect to the potential differential settlements provided in this report.
Note: The use of open bottomed raised planters adjacent to foundations will require more onerous design parameters.

The parameters are considered minimums and may not be adequate to represent all expansive soils/drainage conditions such as adverse drainage and/or improper landscaping and maintenance. The above parameters are applicable provided the structure has positive drainage that is maintained away from the structure. 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 owners. 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.

GSI recommends that the slab subgrade materials be moisture conditioned per recommendations presented in the previous section regarding general foundation construction.

Corrosion and Concrete Mix

Upon completion of grading, laboratory testing should be performed of site materials for corrosion to concrete and corrosion to steel. Additional comments may be obtained from a qualified corrosion engineer at that time.

SOIL MOISTURE TRANSMISSION 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, 2016). 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:

- Concrete slabs should have increased thickness.
- Concrete slab underlayment should consist of a 15-mil vapor retarder, or equivalent, with all laps sealed per the 2013 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.

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

- Concrete slabs, including the garage areas, shall be underlain by 2 inches of clean, 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.

- The vapor retarder shall 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. As discussed previously, GSI indicated this layer of import sand may be eliminated below the vapor retarder, if laboratory testing indicates that the slab subgrade soil have a sand equivalent (SE) of 30 or greater.

- Concrete should have a maximum water/cement ratio of 0.50. This does not supercede Table 4.2.1 of Chapter 4 of the ACI (2011) for corrosion or other corrosive requirements. Additional concrete mix design recommendations should be provided by the structural consultant and/or waterproofing specialist. Concrete finishing and workablity should be addressed by the structural consultant and a waterproofing specialist.

- 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.

- The owner(s) 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.

- 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.

WALL DESIGN PARAMETERS

Conventional Retaining Walls

The design parameters provided below assume that either non expansive soils (typically Class 2 permeable filter material or Class 3 aggregate base) or native onsite materials (up to and including an E.I. of 20) are used to backfill any retaining walls. The type of backfill (i.e., select or native), should be specified by the wall designer, and clearly shown on the plans. Building walls, below grade, should be water-proofed. To reduce the potential for site retaining walls to suffer efflorescence staining, they may also be water-proofed. The foundation system for the proposed retaining walls should be designed in accordance with the recommendations presented in this and preceding sections of this report, as appropriate. Recommendations for specialty walls (i.e., crib, earthstone, geogrid, etc.) can be provided upon request, and would be based on site specific conditions.

Preliminary Retaining Wall Foundation Design

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

Minimum Footing Embedment - 18 inches below the lowest adjacent grade (excluding landscape layer [upper 6 inches]).

Minimum Footing Width - 24 inches

Allowable Bearing Pressure - An allowable bearing pressure of 2,500 pcf 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 18 inches into approved engineered fill overlying dense formational materials. This pressure 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 silty to clayey sand fill.

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 105 pcf and 115 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 retaining wall footings near the perimeter of the site will likely need to be deepened into unweathered formation for adequate vertical and lateral bearing support. All retaining wall footing setbacks from slopes should comply with Figure 1808.7.1 of the 2013 CBC. GSI recommends a minimum horizontal setback distance of 7 feet as measured from the bottom, outboard edge of the footing to the slope face.

Restrained Walls

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. The design should include any applicable surcharge loading. For areas of male or re-entrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall (2H) laterally from the corner.

Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls up to 10 feet high. Design parameters for walls less than 3 feet in height may be superceded by County of San Diego regional standard design. 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 <u>do not</u> 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 backfill for light truck and cars traffic. This does not include the surcharge of parked vehicles which should be evaluated at a higher surcharge to account for the effects of seismic loading. Equivalent fluid pressures for the design of cantilevered retaining walls are provided in the following table:

SURFACE SLOPE OF RETAINED MATERIAL (HORIZONTAL:VERTICAL)	EQUIVALENT FLUID WEIGHT P.C.F. (SELECT BACKFILL)[2]	EQUIVALENT FLUID WEIGHT P.C.F. (NATIVE BACKFILL)[3]
Level[1]	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 50, SE $\geq$ 30, P.I. < 15, E.I. < 21, and $\leq$ 15% passing No. 200 sieve.

Seismic Surcharge

For engineered retaining walls, GSI recommends that the walls be evaluated for a seismic surcharge (in general accordance with 2013 CBC requirements), should walls be within 6 feet of ingress/egress areas. 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 (excluding 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 an inverted triangular distribution using 15H. 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° - $\phi/2$ plane away from the back of the wall. The 15H seismic surcharge is derived from the formula:

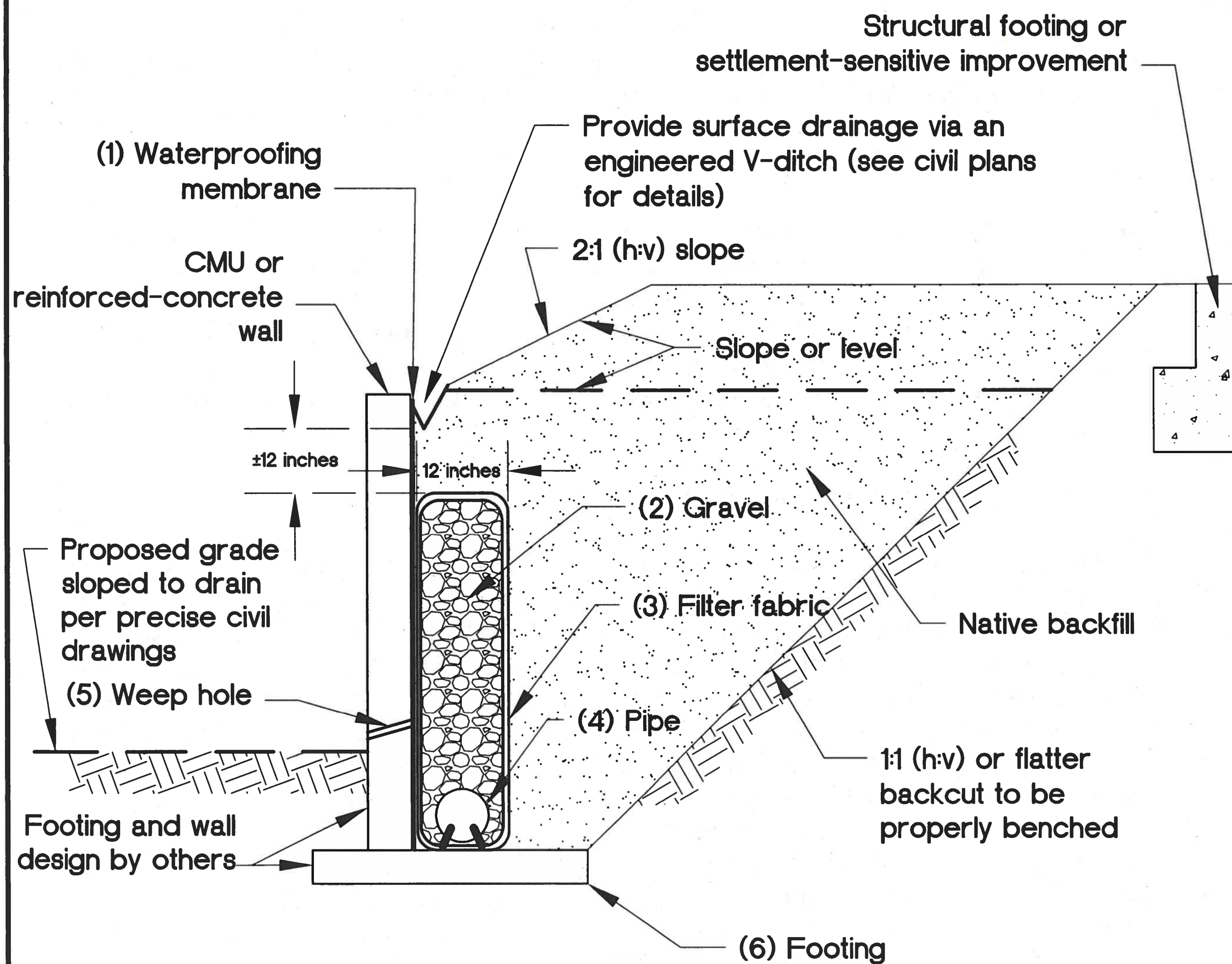
$$P_h = 3/8 \bullet a_h \bullet \gamma_t H$$

Where:	P_h	=	Seismic increment
	a_h	=	Probabilistic horizontal site acceleration with a percentage of "g"
	γ_t	=	total unit weight (115 to 125 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 back drainage 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. For native backfill that has up to medium expansion potential, continuous Class 2 permeable drain materials should be used behind the wall. This material should be continuous (i.e., full height) behind the wall, and it should be constructed in accordance with the enclosed Detail 1 (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). Materials with an E.I. potential of greater than 50 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).

Drain 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 higher than 2 feet, 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.



(1) Waterproofing membrane.

(2) Gravel: Clean, crushed, 3/4 to 1½ inch.

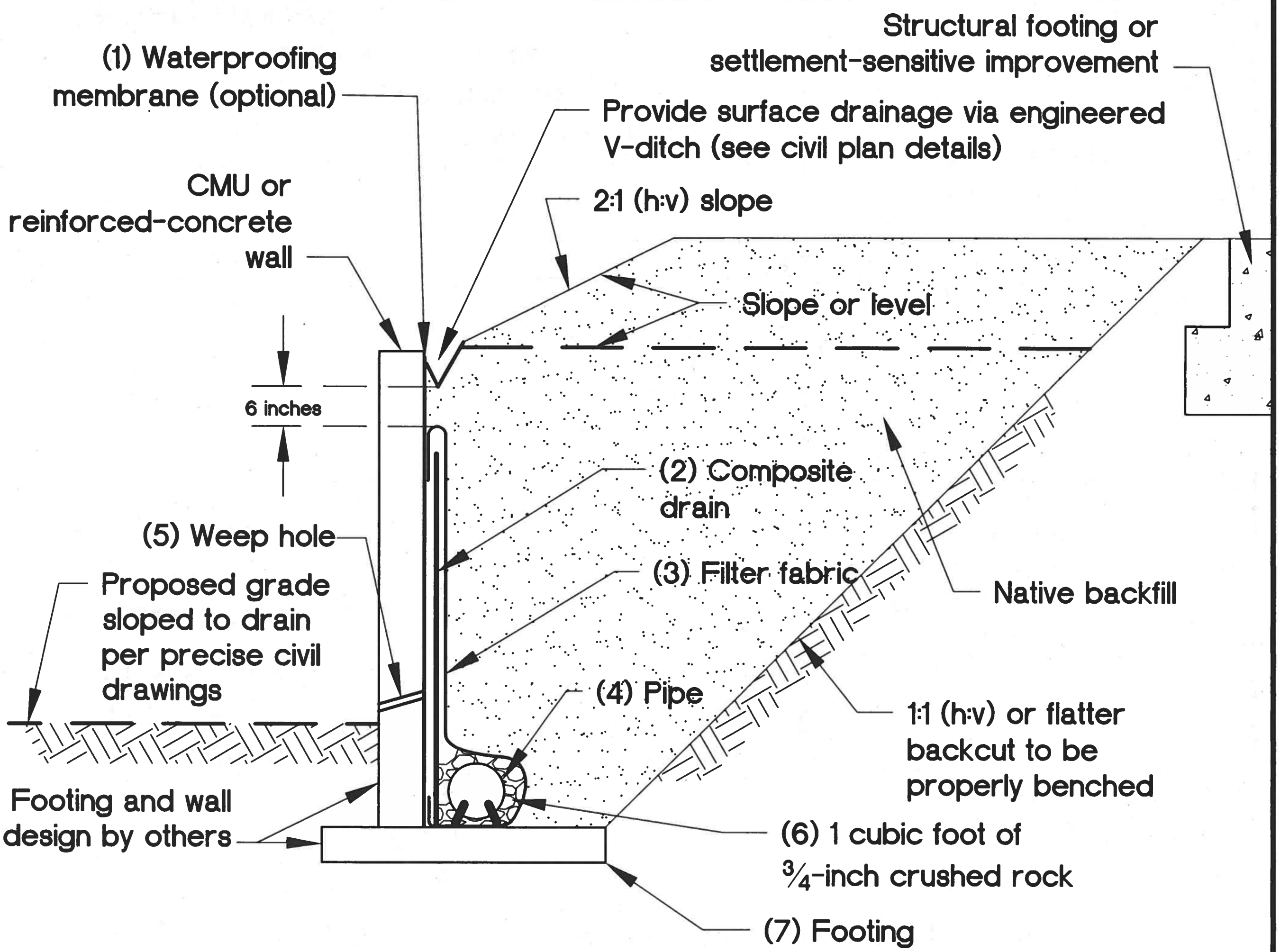
(3) Filter fabric: Mirafi 140N or approved equivalent.

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

(5) Weep 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.

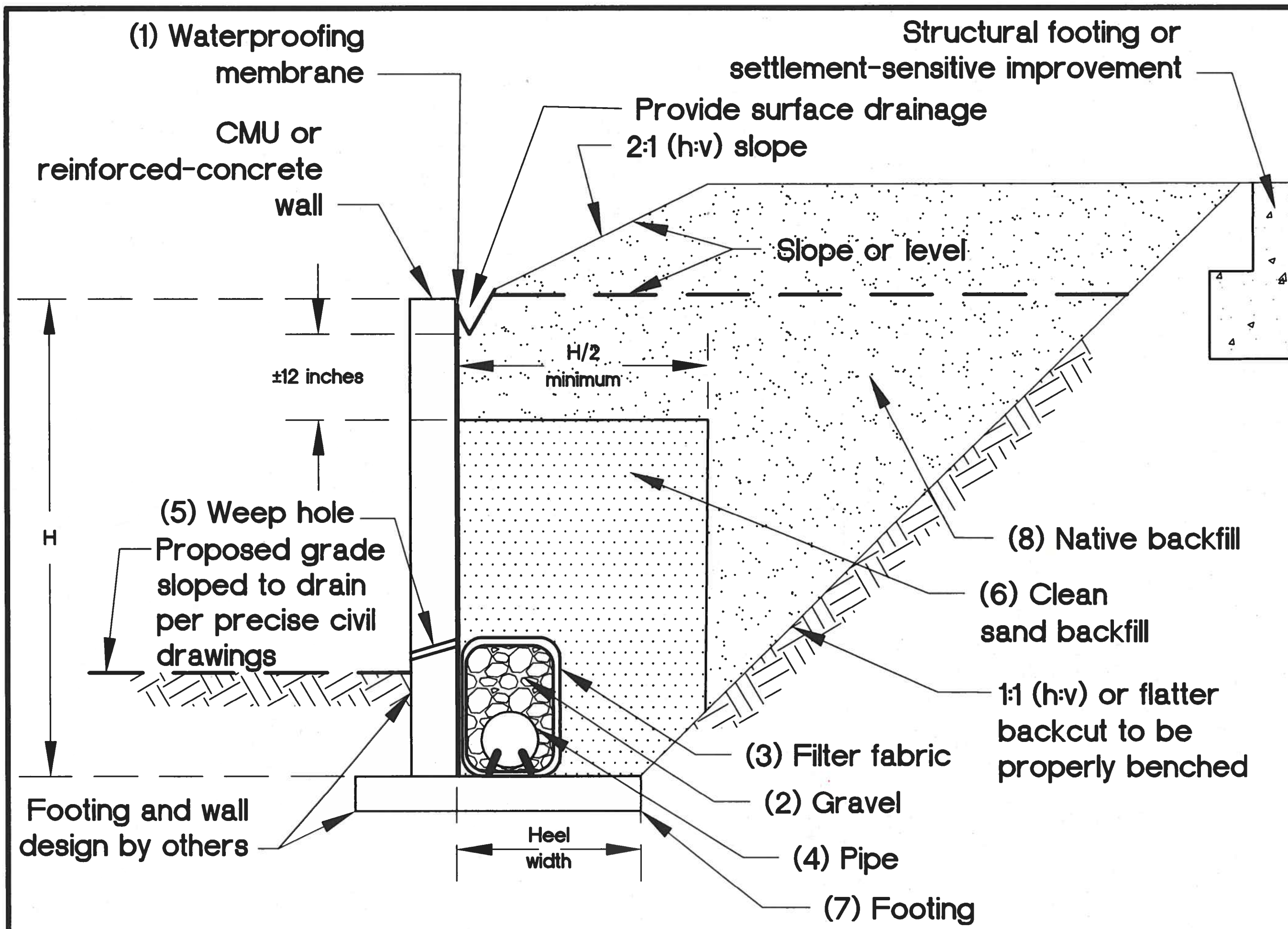
(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

(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, ¾ to 1½ 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, ¾ to 1½ inch.

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

(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient 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. ≥ 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

Wall/Retaining Wall Footing Transitions

Site walls are anticipated to be founded on footings designed in accordance with the recommendations in this report. Although not anticipated, should wall footings transition from cut to fill, the civil designer may specify either:

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 a 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 native formational 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.

PRELIMINARY PAVEMENT DESIGN

Based on an estimated resistance value (R-value) for onsite soils, the following preliminary pavement sections are provided, in accordance with the respective value of the traffic index (T.I.). Pavement subgrade should not be allowed to be saturated during placement and should have detailed positive drainage to extend pavement life.

PRELIMINARY ASPHALTIC CONCRETE PAVEMENTS (ACP)				
TRAFFIC AREA	TRAFFIC INDEX(1)	SUBGRADE R-VALUE	A.C. THICKNESS (INCHES)	AGGREGATE BASE THICKNESS(2) (INCHES)
Residential Street, per City "Design Standards"	5.0	15 (est.)	4.0	8.0

(1) The type of street appropriate for the traffic index to be evaluated by the traffic engineer.
(2) Denotes Class 2 Aggregate Base (R-value >78, SE >25), or equivalent at 95 percent relative compaction.

All subgrade (upper 6 inches) should be compacted to at least 95 percent relative compaction (per ASTM D 1557) prior to base paving. Aggregate base should be compacted to at least 95 percent relative compaction (per ASTM D 1557). All pavement construction should minimally be performed in general accordance with industry standards and properly transitioned. Final pavement design should be based on the actual design traffic index for a given street area, and R-value testing performed at the conclusion of grading.

Pavement design is based on the use of aggregate base conforming to specifications presented in Section 26 of the "Caltrans Standard Specifications" (Caltrans, 1995). Any existing concrete and asphalt may be used as aggregate road base onsite provided that the materials produced conform to specifications for either "crushed miscellaneous base" or "processed miscellaneous base" per Section 200 of the "Greenbook" Standard Specifications for Public Works Construction" (Public Works Construction, Inc., 2003).

DRIVEWAY/PARKING, FLATWORK, AND OTHER IMPROVEMENTS

The effects of expansive soils are cumulative, and typically occur over the lifetime of any improvements. On relatively level areas, when the soils are allowed to dry, the dessication 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 important that the homeowner be aware of this long-term potential for distress. To reduce the likelihood of distress, the following recommendations are presented for all exterior flatwork:

1. 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 2 to 3 percentage points above (or 125 percent of) the soils' optimum moisture content, to a depth of 18 inches below subgrade elevation. If very low expansive soils 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, gravel, or clean sand, that should be compacted and level prior to pouring concrete. If very low expansive soils are present, the rock or gravel or sand may be deleted. The layer or subgrade should be wet-down completely prior to pouring concrete, to minimize loss of concrete moisture to the surrounding earth materials.

3. Exterior slabs (sidewalks, patios, etc.) should be a minimum of 4 inches thick.

4. Driveway and parking area slabs and approaches should be at least 5½ inches thick. A thickened edge (12 inches) should also be considered adjacent to all landscape areas, to help impede infiltration of landscape water under the slab(s).

All pavement construction should minimally be performed in general accordance with industry standards and properly transitioned.

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.

6. 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. If subgrade soils within the top 7 feet from finish grade are very low expansive soils (i.e., E.I. ≤20), then 6x6-W1.4xW1.4 welded-wire mesh may be substituted for the rebar, provided the reinforcement is placed on chairs, at slab mid-height. The exterior slabs should be scored or saw cut, ½ to ⅜ inches deep, often enough so that no section is greater than 10 feet by 10 feet. For sidewalks or narrow slabs, control joints should be provided at intervals of every 6 feet. The slabs should be separated from the foundations and sidewalks with expansion joint filler material.

7. No traffic should be allowed upon the newly poured 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 for sidewalks and patios, and a minimum 3,250 psi for traffic pavements.

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

9. Planters and walls should not be tied to the structure.

10. Overhang structures should be supported on the slabs, or structurally designed with continuous footings tied in at least two directions. If very low expansion soils are present, footings need only be tied in one direction.

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 grade on the lot should provide a minimum of 1 to 2 percent fall to the street, 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 climate and time of year, sulfate content of soils, corrosion potential of soils, and fertilizers used on site.

STORM WATER TREATMENT AND HYDROMODIFICATION MANAGEMENT

Infiltration Feasibility

In accordance with the BMP Design Manual (County, 2016), the infiltration feasibility for this site was evaluated. A review of the United States Department of Agriculture database (USDA; 1973, 2016) indicates that the site is underlain with clays, loamy clays, and loamy fine sands with K_{sat} rates ranging from 0.00 to 0.57 inches/hour. Based on our site specific subsurface exploration, the site appears to be underlain predominantly with "clay," or the "clay loam" as referred to in USDA (2015). Based on a review of USDA (2016), the majority of site soils fall into Hydrologic subgroup "D."

Based on our review and analysis (see Appendix F), full infiltration does not appear feasible. Partial infiltration may be feasible for areas of undisturbed soil, located no closer than 10 feet of any structure. For hydromodification structures located within 10 feet of a residential structure, storm water treatment and hydromodification management should be designed for no infiltration. An additional discussion of infiltration feasibility is presented in Appendix F, which contains a Categorization of infiltration feasibility condition, Worksheet I-8, provided by the City (2016). It should be noted that the infiltration rates evaluated are for undisturbed, native soils. Infiltration rates for compacted fills will be substantially less. Compacted fills are considered as belonging to Hydrologic Soil Group "D" (no infiltration).

Onsite Infiltration-Runoff Retention Systems

General design criteria regarding the use of onsite infiltration-runoff retention systems (OIRRS) are presented below.

Should onsite infiltration-runoff retention systems (OIRRS) be planned for Best Management Practices (BMP's) or Low Impact Development (LID) principles for the project, some guidelines should/must be followed in the planning, design, and construction of such systems. Such facilities, if improperly designed or implemented without consideration of the geotechnical aspects of site conditions, can contribute to flooding, saturation of bearing materials beneath site improvements, slope instability, and possible concentration and contribution of pollutants into the groundwater or storm drain and/or utility trench systems.

A key factor in these systems is the infiltration rate (often referred to as the percolation rate) which can be ascribed to, or determined for, the earth materials within which these systems are installed. Additionally, the infiltration rate of the designed system (which may include gravel, sand, mulch/topsoil, or other amendments, etc.) will need to be considered. The project infiltration testing is very site specific, any changes to the location of the proposed OIRRS and/or estimated size of the OIRRS, may require additional infiltration testing. Locally, relatively impermeable formations include the underlying formational (Santiago) bedrock, which is anticipated to have relatively very low vertical infiltration rate.

Some of the methods which are utilized for onsite infiltration include percolation basins, dry wells, bio-swale/bio-retention, permeable pavers/pavement, infiltration trenches, filter boxes and subsurface infiltration galleries/chambers. Some of these systems are constructed using native and import soils, perforated piping, and filter fabrics while others employ structural components such as stormwater infiltration chambers and filters/separators. Every site will have characteristics which should lend themselves to one or more of these methods; but, not every site is suitable for OIRRS. In practice, OIRRS are usually initially designed by the project design civil engineer. Selection of methods should include (but should not be limited to) review by licensed professionals including the geotechnical engineer, hydrogeologist, engineering geologist, project civil engineer, landscape architect, environmental professional, and industrial hygienist. Applicable governing agency requirements should be reviewed and included in design considerations. The following geotechnical guidelines should be considered when designing onsite infiltration-runoff retention systems:

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

- Wherever possible, infiltration systems should not be installed within ±50 feet of the tops of slopes steeper than 15 percent or within H/3 from the tops of slopes (where H equals the height of slope).

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

- Wherever possible, infiltration systems should not be installed within 10 feet of a residential structure.

- The landscape architect should be notified of the location of the proposed OIRRS. If landscaping is proposed <u>within</u> 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 OIRRS could adversely affect performance of the system. Soil chemical amendment could alter soil chemistry, which may affect soil corrosion and permeability.

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

- If subsurface infiltration galleries/chambers are proposed, the appropriate size, depth interval, and ultimate placement of the detention/infiltration system should be evaluated by the design engineer, and be of sufficient width/depth to achieve optimum performance, based on the infiltration rates provided. In addition, proper debris filter systems will need to be utilized for the infiltration galleries/chambers. Debris filter systems will need to be self cleaning and periodically and regularly maintained on a regular basis. Provisions for the regular and periodic maintenance of any debris filter system is recommended and this condition should be disclosed to all interested/affected parties.

- Where infiltration systems are located within setback areas noted above, impermeable liners and subdrains should be used along the bottom of bioretention swales/basins located within the influence of slopes and structures. Impermeable liners used in conjunction with bioretention basins should consist of a 30-mil polyvinyl chloride (PVC) membrane that is covered by a minimum of 12 inches of clean soil, free from rocks and debris, with a maximum 4:1 (h:v) slope inclination, or flatter, and meets the following minimum specifications:

    Specific Gravity (ASTM D792): 1.2 (g/cc, min.); Tensile (ASTM D882): 73 (lb/in-width, min); Elongation at Break (ASTM D882): 380 (%, min); Modulus (ASTM D882): 32 (lb/in-width, min.); and Tear Strength

(ASTM D1004): 8 (lb/in, min); Seam Shear Strength (ASTM D882) 58.4 (lb/in, min); Seam Peel Strength (ASTM D882) 15 (lb/in, min).

- Subdrains should consist of at least 4-inch diameter Schedule 40 or SDR 35 drain pipe with perforations oriented down. The drain pipe should be sleeved with a filter sock.

Based on the existing, and potential as-built soil conditions, GSI strongly recommends that any required storm water treatment BMP is provided with impermeable liners, and subdrains should be used along the bottom of bioretention swales/basins located within the influence of planned improvements to direct subsurface water to a suitable outlet or sump pump.

In practice, storm water BMP's are usually initially designed by the project design civil engineer. Selection of methods should include (but should not be limited to) review by licensed professionals including the geotechnical engineer, hydrogeologist, engineering geologist, project civil engineer, landscape architect, environmental professional, and industrial hygienist. Applicable governing agency requirements should be reviewed and included in design considerations.

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 all interested/affected parties. Over-steepening of slopes should be avoided during building construction activities and landscaping.

Drainage

Adequate 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 mitigate ponding of water anywhere on the property, and especially near structures and tops of slopes. 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 the property 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 tops of slopes, and not allowed to pond and/or seep into the ground. In general, site drainage should conform to Section 1804.3 of the 2013 CBC. Consideration should be given to avoiding construction of planters adjacent to structures (buildings, pools, spas, etc.). Building 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.

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

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 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., pools, spas, 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. Pools and/or spas should not be constructed without specific design and construction recommendations from GSI, and this construction recommendation should be provided to all interested/affected parties. 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.

Tile Flooring

Tile flooring can crack, reflecting cracks in the concrete slab below the tile, although small cracks in a conventional slab may not be significant. Therefore, the designer should consider additional steel reinforcement for concrete slabs-on-grade where tile will be placed. The tile installer should consider installation methods that reduce possible cracking of the tile such as slipsheets. Slipsheets or a vinyl crack isolation membrane (approved by the Tile Council of America/Ceramic Tile Institute) are recommended between tile and concrete slabs on grade.

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, if not removed from the site.

Trenching/Temporary Construction Backcuts

Considering the nature of the onsite earth materials, and the potential for encountering groundwater at depth, 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. 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.

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 or other subdrainage devices, prior to placing fill and/or backfill.
- After excavation of building footings, retaining wall footings, 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 homeowner improvements, such as flatwork, spas, pools, walls, etc., are constructed, prior to construction.
- A report of geotechnical observation and testing should be provided at the conclusion of each of the above stages, in order to provide concise and clear documentation of site work, and/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 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.

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.

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

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APPENDIX B

BORING AND TEST PIT LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM

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

CONSISTENCY OR RELATIVE DENSITY

CRITERIA

Standard Penetration Test

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

Standard Penetration Test

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

3" 3/4" #4 #10 #40 #200 U.S. Standard Sieve

Unified Soil Classification	Cobbles	Gravel		Sand			Silt or Clay
		coarse	fine	coarse	medium	fine	

MOISTURE CONDITIONS

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

MATERIAL QUANTITY

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

OTHER SYMBOLS

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

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

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

W.O. 6971-A1-SC
Marja Acres, LLC
4901 El Camino Real
Logged By: RGC
May 12, 2016

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-1	109' MSL	0-½	SC				**COLLUVIUM:** CLAYEY SAND, brown to dark brown, slightly moist, few roots, cultivated.
		½-1½	CH				CLAY, dark brown, moist, soft; few roots.
		1½-2½	SC				CLAYEY SAND, mottled olive gray brown and brown, moist, medium dense; abundant carbonate mottlings, porous.
		2½-4	CL				**OLDER ALLUVIUM:** CLAY, olive brown and strong brown, moist, very stiff; sub-horizontal basal contact.
		4-9	SC				CLAYEY SAND, brown, slightly moist, dense; few carbonate filled random fractures.
		9-11	SM				SILTY SAND, gray brown, moist, medium dense to dense.
							Total Depth = 11' No Groundwater/Caving Encountered Backfilled 5-12-2016

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-2	95' MSL	0-4	CL				**UNDOCUMENTED FILL:** SANDY CLAY, brown to dark brown, moist, soft; some plastic debris, porous.
		4-7	CH				**COLLUVIUM:** CLAY, dark brown, moist, soft to firm; porous, some carbonate mottling, porous.
		7-9	SM/SC	Ring @ 7	11.1	113.7	**OLDER ALLUVIUM:** SILTY SAND with CLAY, light brown, moist, medium dense; thickly bedded (sub-horizontal)
							Total Depth = 9' No Groundwater/Caving Encountered Backfilled 5-12-2016
TP-3	98' MSL	0-3½	CL				**UNDOCUMENTED FILL:** SANDY CLAY, dark brown, moist, soft; some plastic debris, porous.
		3½-6	CH				**COLLUVIUM:** CLAY, dark brown to dark gray brown, moist, soft; porous.
		6-8	SM				**OLDER ALLUVIUM:** SILTY SAND, brown, moist, medium dense; thick sub-horizontal bedding.
							Total Depth = 8' No Groundwater/Caving Encountered Backfilled 5-12-2016

GeoSoils, Inc.

W.O. 6971-A1-SC
Marja Acres, LLC
4901 El Camino Real
Logged By: RGC
May 12, 2016

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-4	105' MSL	0-1½	CH	Bulk @ 1			**COLLUVIUM:** CLAY, dark brown, moist, soft; porous, few roots.
		1½-3	CL				SANDY CLAY, mottled brown and olive brown, moist, loose to medium dense; highly fractured, few carbonate and manganese cuttings on fracture faces.
		3-5½	CL				**PARALIC DEPOSITS:** CLAYEY SAND to SANDY CLAY, Dark Brown, moist, medium dense/stiff; manganese coatings on fracture faces, sub-horizontal basal contact.
		5½-6	SM				SILTY SAND, brown, moist, medium dense.
		6-8	CL				CLAYEY SAND, brown and olive brown, moist, medium dense.
		8-10	SM	Bulk @ 9			SILTY SAND with CLAY.
							Total Depth = 10' No Groundwater/Caving Encountered Backfilled 5-12-2016

GeoSoils, Inc.

W.O. 6971-A1-SC
Marja Acres, LLC
4901 El Camino Real
Logged By: RGC
May 12, 2016

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-5	86' MSL	0-1	CL				**COLLUVIUM:** SANDY CLAY, dark brown, moist, soft; porous, few roots along basal contact.
		1-2	SM				**SANTIAGO FORMATION:** SANDSTONE, brown and light grayish yellow, slightly moist, loose/dense.
		2-7	SP/SM	Bulk/Ring @ 2	10.3	112.6	SANDSTONE, light grayish yellow, moist, dense; fine grained. Bedding: N30°W, 2-3° SW.
							Total Depth = 8' No Groundwater/Caving Encountered Backfilled 5-13-2016
TP-6	98' MSL	0-1½	CH				**COLLUVIUM:** CLAY, dark grayish brown, moist, soft; porous, few roots.
		1½-3	CL				SANDY CLAY, dark brown, moist, firm to stiff; porous.
		3-12	SC				**OLDER ALLUVIUM:** CLAYEY SAND, grayish brown, olive brown, moist, medium dense.
		12-14	CL				CLAY, dark brown, moist, stiff.
		14-15½	SP				SAND, brown, moist, loose.
							Total Depth = 15½' No Groundwater/Caving Encountered Backfilled 5-12-2016

GeoSoils, Inc.

W.O. 6971-A1-SC
Marja Acres, LLC
4901 El Camino Real
Logged By: RGC
May 12, 2016

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-7	86' MSL	0-1½	CL				**COLLUVIUM:** SANDY CLAY, brown, moist, soft; porous, few roots.
		1½-3	CH				CLAY, dark brown, moist, stiff; porous.
		3-7	CH	Bulk @ 4			**PALEOSOL/COLLUVIUM?** CLAY, very dark gray, moist, stiff; randomly fractured with up to ¾ inch carbonate modules.
		7-9	SC				**SANTIAGO FORMATION:** CLAYEY SANDSTONE, grayish brown, moist, medium dense; slightly weathered.
							Total Depth = 9' No Groundwater/Caving Encountered Backfilled 5-12-2016
TP-8	99' MSL	0-1	CL/CH				**COLLUVIUM:** SANDY CLAY, dark brown, moist, soft; porous, few roots.
		1-2½	CH				CLAY, very dark brown, moist, soft.
		2½-3½	SC				**OLDER ALLUVIUM:** CLAYEY SAND, brown, moist, medium dense.
		3½-5½	SM				SILTY SAND, brown, wet, medium dense.
		5½-8½	CL	Bulk @ 5½			CLAY, grayish brown to olive brown, moist, stiff.
		8½-10	SM				SAND with SILT, brown, moist, medium dense.
							Total Depth = 10' No Groundwater/Caving Encountered Backfilled 5-12-2016

GeoSoils, Inc.

W.O. 6971-A1-SC
Marja Acres, LLC
4901 El Camino Real
Logged By: RGC
May 12, 2016

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-9	84' MSL	0-2	CL				**COLLUVIUM:** SANDY CLAY, dark brown, moist, soft; porous, few roots.
		2-5	CH				CLAY, very dark grayish brown, moist, stiff; blocky structure, abundant caliche stringers and coatings on blocky faces.
		5-7	SC				**OLDER ALLUVIUM:** CLAYEY SAND, brown and light olive bornw, moist, medium dense.
	Note:2- to 3-foot thick fill embankment immediate northwest of pit.						Total Depth = 7' No Groundwater/Caving Encountered Backfilled 5-12-2016
TP-10		0-2	CH				**COLLUVIUM:** SANDY CLAY, dark brown, moist, soft.
		2-3½	SC				**OLDER ALLUVIUM:** CLAYEY SAND, light olive brown, moist, medium dense.
		3½-5½	CH				**PALEOSOL:** CLAY, very dark grayish brown, moist, stiff; blocky, abundant carbonates on blocky ped faced and nodules.
		5½-7	SM				**SANTIAGO FORMATION:** SILTY SANDSTONE, yellowish brown, moist, medium dense.
							Total Depth = 7' No Groundwater/Caving Encountered Backfilled 5-12-2016

GeoSoils, Inc.

BORING LOG

W.O. 6971-A1-SC

PROJECT: MARJA ACRES, LLC
4901 El Camino Real

BORING B-1 SHEET 1 OF 2

DATE EXCAVATED 5-23-16

SAMPLE METHOD: Hollow Stem Auger

Approx. Elevation: ±47' MSL

Standard Penetration Test

Undisturbed, Ring Sample

Groundwater

Seepage

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
0				SC				**UNDOCUMENTED FILL:** @ 0' CLAYEY SAND, light brown to brown, slightly moist, loose.
4				SC				**QUATERNARY ALLUVIUM:** @ 4' CLAYEY SAND, brown, moist, loose; fine laminations of very dark gray SILT.
5			18		95.6	11.1	40.4	
9				SP				@ 9' SAND, grayish brown, slightly moist, loose.
10			11		101.3	20.5	86.0	
10½				CL				@ 10½' SANDY CLAY, dark yellowish brown, wet, soft.
15			25	SC	106.7	18.2	87.5	@ 15' CLAYEY SAND, brown, wet, loose.
15½				CH				@ 15½' CLAY, dark grayish brown, moist to wet, stiff; few carbonate filaments. @ 15½' Groundwater encountered.
20			22	CL	107.7	19.0	94.2	@ 20' SANDY CLAY, grayish brown, wet, stiff.
25			15	SC	104.6	21.2	96.8	@ 25' CLAYEY SAND, dark yellowish brown to yellowish brown, saturated, loose.

BORING LOG

GeoSoils, Inc.

W.O. 6971-A1-SC

PROJECT: MARJA ACRES, LLC
4901 El Camino Real

BORING B-1 SHEET 2 OF 2

DATE EXCAVATED 5-23-16

SAMPLE METHOD: Hollow Stem Auger

Approx. Elevation: ±47' MSL

Standard Penetration Test

Undisturbed, Ring Sample

Groundwater

Seepage

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
30		Ring	30	CL	110.3	19.0	100	@ 30' CLAY, very dark grayish brown, wet, very stiff.
35		Ring	45	SC	105.6	22.3	100	**SANTIAGO FORMATION:** @ 35' CLAYEY SANDSTONE, yellowish brown to brown, saturated, dense.
40		Ring	39		103.1	22.0	96.5	@ 40' CLAYEY SANDSTONE, light brownish gray, wet, dense.
41								Total Depth = 41' Groundwater Encountered @ 15½' No Caving Encountered Backfilled 5-23-2016

GeoSoils, Inc.

BORING LOG

W.O. 6971-A1-SC

PROJECT: MARJA ACRES, LLC
4901 El Camino Real

BORING B-2 SHEET 1 OF 2

DATE EXCAVATED 5-23-16

SAMPLE METHOD: Hollow Stem Auger

Approx. Elevation: ±46' MSL

Standard Penetration Test

Undisturbed, Ring Sample

Groundwater

Seepage

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
0				SC				**UNDOCUMENTED FILL:** @ 0' CLAYEY SAND, brown, slightly moist, loose; few gravels.
5		Ring	14		110.4	15.6	83	
7				SC				**QUATERNARY ALLUVIUM:** @ 7' CLAYEY SAND, brown, moist, loose.
10		Ring	7		111.3	18.2	99	@ 10' As per 7'.
11				CH				@ 11' CLAY, very dark gray, moist, soft.
14 ▽								@ 14' Groundwater encountered.
15		Ring	15		105.5	18.5	86	@ 15' As per 11'.
16				SC				@ 16' CLAYEY SAND, dark yellowish brown, saturated, loose.
20		SPT	22	CL/CH				@ 20' CLAY, very dark gray, saturated, stiff.
25		Ring	32		106.6	19.9	100	@ 25' CLAY, very dark grayish brown, brown, wet, stiff.

GeoSoils, Inc.

BORING LOG

W.O. 6971-A1-SC

PROJECT: MARJA ACRES, LLC
4901 El Camino Real

BORING B-2 SHEET 2 OF 2

DATE EXCAVATED 5-23-16

SAMPLE METHOD: Hollow Stem Auger

Approx. Elevation: ±46' MSL

Standard Penetration Test

Undisturbed, Ring Sample

Groundwater

Seepage

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
30		SPT	22	CL/CH				@ 30' As per 25'.
35		Ring	25	CH	107.6	19.5	96	**PALEOSOL?** @ 35' CLAY, very dark gray, wet, stiff.
36				SP				**WEATHERED SANTIAGO FORMATION:** @ 36' SANDSTONE, brown, saturated, medium dense.
40		SPT	22					@ 40' SAND with CLAY, brown, saturated, medium dense.
45		SPT	24					@ 45' SAND, grayish brown, saturated, medium dense.
50		SPT	33	SP				**SANTIAGO FORMATION:** @ 50' SANDSTONE with CLAY, light grayish brown, wet, dense.

Total Depth = 51½'
Groundwater Encountered @ 14'
No Caving Encountered
Backfilled 5-23-2016

BORING LOG

GeoSoils, Inc.

W.O. 6971-A1-SC

PROJECT: MARJA ACRES, LLC
4901 El Camino Real

BORING B-3 SHEET 1 OF 1

DATE EXCAVATED 5-23-16

SAMPLE METHOD: Hollow Stem Auger

Approx. Elevation: ±52' MSL

Standard Penetration Test

Undisturbed, Ring Sample

Groundwater

Seepage

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
0								**ASPHALT PAVEMENT:** @ 0' ASPHALT, 4½ inches over 2 inches of "DG."
1				SC				**UNDOCUMENTED FILL:** @ ½' CLAYEY SAND, brown and gray brown, moist, loose to medium dense.
5	X	X	8	SM	104.2	6.7	30	**QUATERNARY ALLUVIUM:** @ 5' SAND with SILT, light brown, dry, loose.
6	X			CH				@ 6' CLAY, very dark gray, moist, soft.
10		X	32	SC	112.3	16.7	94	@ 10' CLAYEY SAND to SANDY CLAY, very dark yellowish brown, slightly moist, medium dense/stiff.
15		X	24		107.3	18.8	92	@ 15' CLAYEY SAND, dark grayish brown, moist, medium dense/stiff.
17								@ 17' Groundwater encountered.
20		X	31	SC	108.2	19.9	100	**SANTIAGO FORMATION:** @ 20' CLAYEY SANDSTONE, grayish brown, saturated, medium dense.
25		X	33	SM	109.4	18.3	95	@ 25' SILTY SANDSTONE, brownish gray, wet, medium dense to dense.
26								Total Depth = 26' Groundwater Encountered @ 17' No Caving Encountered Backfilled 5-23-2016

APPENDIX C

SEISMICITY

TEST.OUT

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DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 6971-A1-SC

DATE: 06-27-2016

JOB NAME: Marja Acres

CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: C:\EQ\EQFAULT\CGSFLTE.DAT

SITE COORDINATES:
   SITE LATITUDE: 33.1500
   SITE LONGITUDE: 117.3080

SEARCH RADIUS: 62.4 mi

ATTENUATION RELATION: 11) Bozorgnia Campbell Niazi (1999) Hor.-Pleist. Soil-Cor.
   UNCERTAINTY (M=Median, S=Sigma): S   Number of Sigmas: 1.0
   DISTANCE MEASURE: cdist
   SCOND: 1
   Basement Depth: .01 km   Campbell SSR: 0   Campbell SHR: 0
   COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: C:\EQ\EQFAULT\CGSFLTE.DAT

MINIMUM DEPTH VALUE (km): 3.0

TEST.OUT

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	6.3(10.1)	7.2	0.564	X
NEWPORT-INGLEWOOD (Offshore)	7.3(11.7)	7.1	0.494	X
CORONADO BANK	22.1(35.6)	7.6	0.261	IX
ELSINORE (TEMECULA)	23.2(37.3)	6.8	0.146	VIII
ELSINORE (JULIAN)	23.2(37.4)	7.1	0.178	VIII
ELSINORE (GLEN IVY)	34.2(55.0)	6.8	0.098	VII
SAN JOAQUIN HILLS	36.9(59.4)	6.6	0.112	VII
PALOS VERDES	37.7(60.7)	7.3	0.125	VII
EARTHQUAKE VALLEY	42.1(67.8)	6.5	0.064	VI
SAN JACINTO-ANZA	45.9(73.8)	7.2	0.095	VII
SAN JACINTO-SAN JACINTO VALLEY	46.7(75.1)	6.9	0.075	VII
NEWPORT-INGLEWOOD (L.A.Basin)	47.7(76.7)	7.1	0.085	VII
CHINO-CENTRAL AVE. (Elsinore)	48.8(78.5)	6.7	0.089	VII
SAN JACINTO-COYOTE CREEK	51.0(82.0)	6.6	0.056	VI
WHITTIER	52.6(84.7)	6.8	0.062	VI
ELSINORE (COYOTE MOUNTAIN)	56.2(90.5)	6.8	0.058	VI
SAN JACINTO-SAN BERNARDINO	60.1(96.7)	6.7	0.050	VI

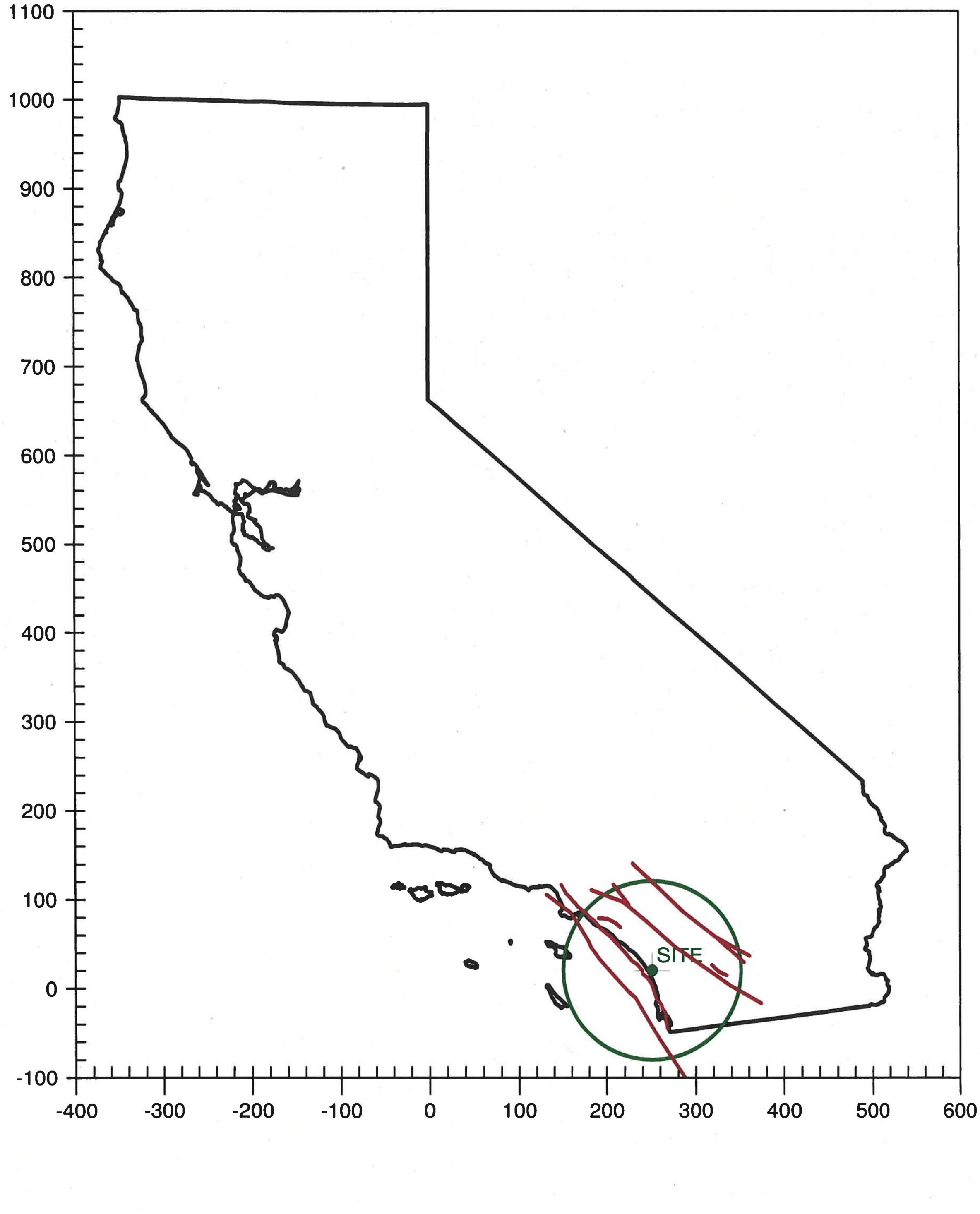
-END OF SEARCH- 17 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ROSE CANYON FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 6.3 MILES (10.1 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.5643 g

CALIFORNIA FAULT MAP

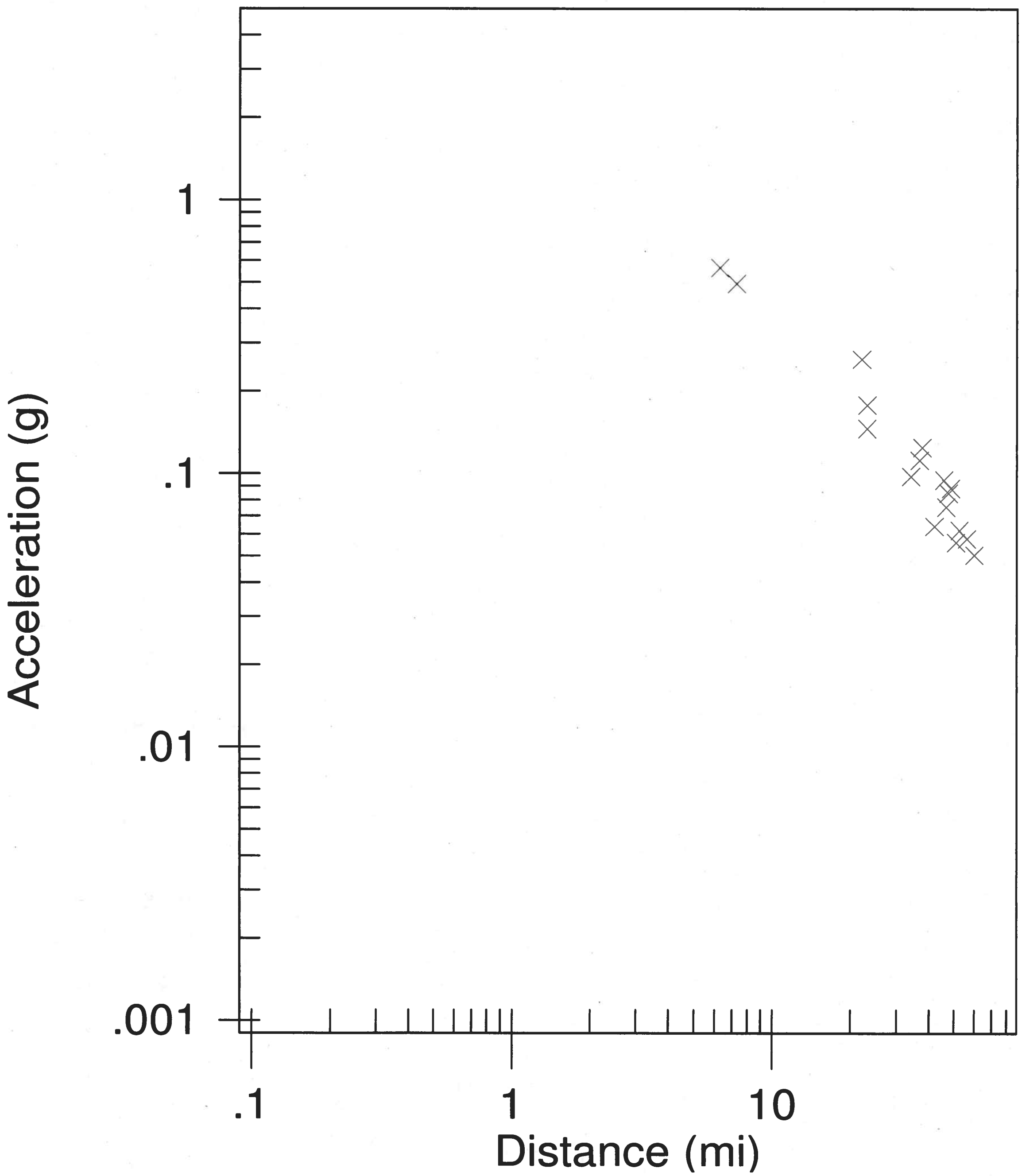
Marja Acres



W.O. 6971-A-SC
PLATE C-3

MAXIMUM EARTHQUAKES

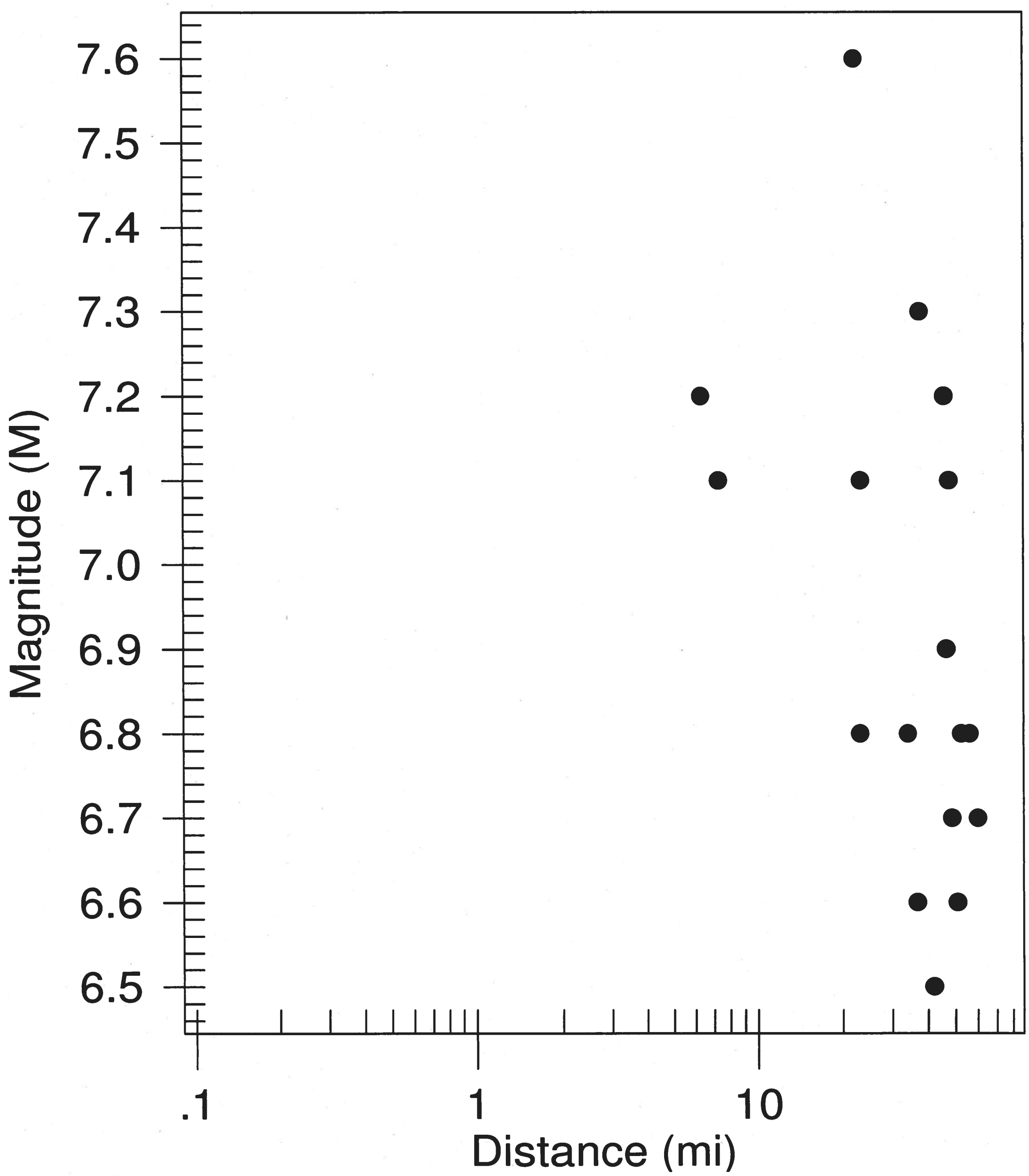
Marja Acres



W.O. 6971-A-SC
PLATE C-4

EARTHQUAKE MAGNITUDES & DISTANCES

Marja Acres



W.O. 6971-A-SC
PLATE C-5

TEST.OUT

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ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 6971-A1-SC

DATE: 06-27-2016

JOB NAME: Marja Acres

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

MAGNITUDE RANGE:
MINIMUM MAGNITUDE: 5.00
MAXIMUM MAGNITUDE: 9.00

SITE COORDINATES:
SITE LATITUDE: 33.1500
SITE LONGITUDE: 117.3080

SEARCH DATES:
START DATE: 1800
END DATE: 2016

SEARCH RADIUS:
62.4 mi
100.4 km

ATTENUATION RELATION: 11) Bozorgnia Campbell Niazi (1999) Hor.-Pleist. Soil-Cor.
UNCERTAINTY (M=Median, S=Sigma): S Number of Sigmas: 1.0
ASSUMED SOURCE TYPE: SS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
SCOND: 1 Depth Source: A
Basement Depth: .01 km Campbell SSR: 0 Campbell SHR: 0
COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km): 3.0

EARTHQUAKE SEARCH RESULTS

Page 1

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	33.0000	117.3000	11/22/1800	2130 0.0	0.0	6.50	0.263	IX	10.4(16.7)
MGI	33.0000	117.0000	09/21/1856	730 0.0	0.0	5.00	0.053	VI	20.6(33.2)
MGI	32.8000	117.1000	05/25/1803	0 0 0.0	0.0	5.00	0.040	V	27.0(43.4)
DMG	32.7000	117.2000	05/27/1862	20 0 0.0	0.0	5.90	0.059	VI	31.7(51.0)
T-A	32.6700	117.1700	10/21/1862	0 0 0.0	0.0	5.00	0.032	V	34.1(54.9)
T-A	32.6700	117.1700	12/00/1856	0 0 0.0	0.0	5.00	0.032	V	34.1(54.9)
T-A	32.6700	117.1700	05/24/1865	0 0 0.0	0.0	5.00	0.032	V	34.1(54.9)
PAS	32.9710	117.8700	07/13/1986	1347 8.2	6.0	5.30	0.037	V	34.8(56.0)
DMG	33.2000	116.7000	01/01/1920	235 0.0	0.0	5.00	0.031	V	35.3(56.8)
DMG	32.8000	116.8000	10/23/1894	23 3 0.0	0.0	5.70	0.043	VI	38.1(61.3)
DMG	33.7000	117.4000	05/13/1910	620 0.0	0.0	5.00	0.028	V	38.3(61.7)
DMG	33.7000	117.4000	04/11/1910	757 0.0	0.0	5.00	0.028	V	38.3(61.7)
DMG	33.7000	117.4000	05/15/1910	1547 0.0	0.0	6.00	0.051	VI	38.3(61.7)
DMG	33.6990	117.5110	05/31/1938	83455.4	10.0	5.50	0.036	V	39.7(63.8)
MGI	33.2000	116.6000	10/12/1920	1748 0.0	0.0	5.30	0.031	V	41.1(66.1)
DMG	33.7100	116.9250	09/23/1963	144152.6	16.5	5.00	0.024	V	44.5(71.6)
DMG	33.7500	117.0000	04/21/1918	223225.0	0.0	6.80	0.073	VII	45.1(72.5)
DMG	33.7500	117.0000	06/06/1918	2232 0.0	0.0	5.00	0.024	IV	45.1(72.5)
MGI	33.8000	117.6000	04/22/1918	2115 0.0	0.0	5.00	0.022	IV	47.9(77.1)
DMG	33.8000	117.0000	12/25/1899	1225 0.0	0.0	6.40	0.052	VI	48.3(77.7)
DMG	33.5750	117.9830	03/11/1933	518 4.0	0.0	5.20	0.024	V	48.7(78.4)
DMG	33.6170	117.9670	03/11/1933	154 7.8	0.0	6.30	0.047	VI	49.8(80.2)
GSP	33.5290	116.5720	06/12/2005	154146.5	14.0	5.20	0.024	V	49.9(80.2)
GSG	33.4200	116.4890	07/07/2010	235333.5	14.0	5.50	0.028	V	50.8(81.8)
DMG	33.0000	116.4330	06/04/1940	1035 8.3	0.0	5.10	0.022	IV	51.7(83.1)
PAS	33.5010	116.5130	02/25/1980	104738.5	13.6	5.50	0.027	V	51.9(83.5)
GSP	33.5080	116.5140	10/31/2001	075616.6	15.0	5.10	0.022	IV	52.0(83.8)
DMG	33.6170	118.0170	03/14/1933	19 150.0	0.0	5.10	0.022	IV	52.1(83.8)
DMG	33.9000	117.2000	12/19/1880	0 0 0.0	0.0	6.00	0.037	V	52.2(83.9)
DMG	33.5000	116.5000	09/30/1916	211 0.0	0.0	5.00	0.020	IV	52.5(84.5)
DMG	33.6830	118.0500	03/11/1933	658 3.0	0.0	5.50	0.025	V	56.4(90.8)
DMG	33.3430	116.3460	04/28/1969	232042.9	20.0	5.80	0.030	V	57.1(91.9)
DMG	33.7000	118.0670	03/11/1933	51022.0	0.0	5.10	0.019	IV	57.9(93.2)
DMG	33.7000	118.0670	03/11/1933	85457.0	0.0	5.10	0.019	IV	57.9(93.2)
DMG	34.0000	117.2500	07/23/1923	73026.0	0.0	6.25	0.038	V	58.8(94.6)
MGI	34.0000	117.5000	12/16/1858	10 0 0.0	0.0	7.00	0.062	VI	59.7(96.1)
DMG	33.4000	116.3000	02/09/1890	12 6 0.0	0.0	6.30	0.038	V	60.7(97.7)
DMG	33.7500	118.0830	03/11/1933	230 0.0	0.0	5.10	0.018	IV	60.9(98.0)
DMG	33.7500	118.0830	03/11/1933	323 0.0	0.0	5.00	0.017	IV	60.9(98.0)
DMG	33.7500	118.0830	03/13/1933	131828.0	0.0	5.30	0.021	IV	60.9(98.0)
DMG	33.7500	118.0830	03/11/1933	2 9 0.0	0.0	5.00	0.017	IV	60.9(98.0)
DMG	33.7500	118.0830	03/11/1933	910 0.0	0.0	5.10	0.018	IV	60.9(98.0)
DMG	33.9500	116.8500	09/28/1946	719 9.0	0.0	5.00	0.017	IV	61.2(98.5)

GSG |33.9530|117.7610|07/29/2008|184215.7| 14.0| 5.30| 0.020 | IV | 61.3(98.6)

****

-END OF SEARCH- 44 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 2016

LENGTH OF SEARCH TIME: 217 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 10.4 MILES (16.7 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.263 g

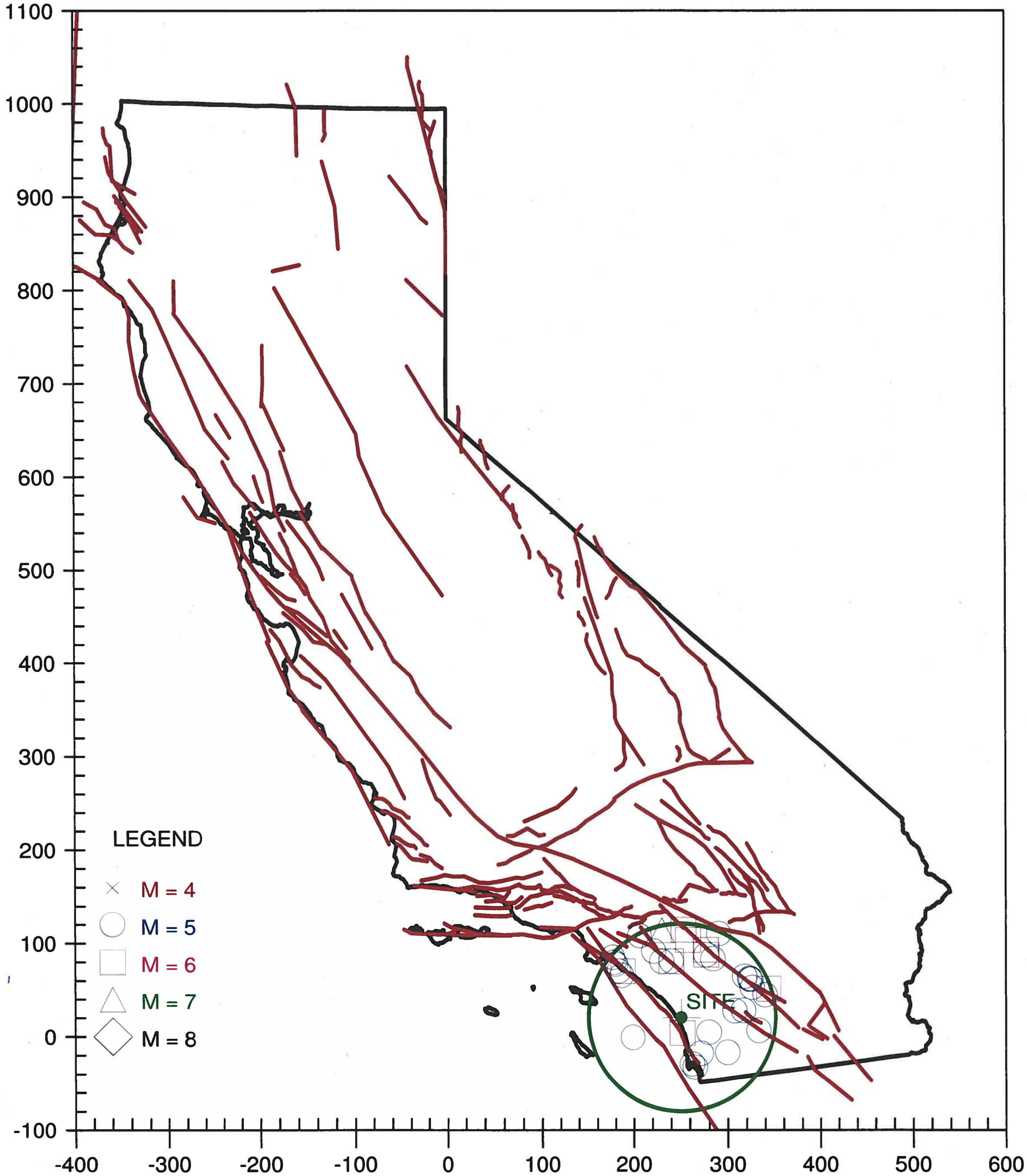
COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
a-value= 0.899
b-value= 0.364
beta-value= 0.837

TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	44	0.20276
4.5	44	0.20276
5.0	44	0.20276
5.5	16	0.07373
6.0	9	0.04147
6.5	3	0.01382
7.0	1	0.00461

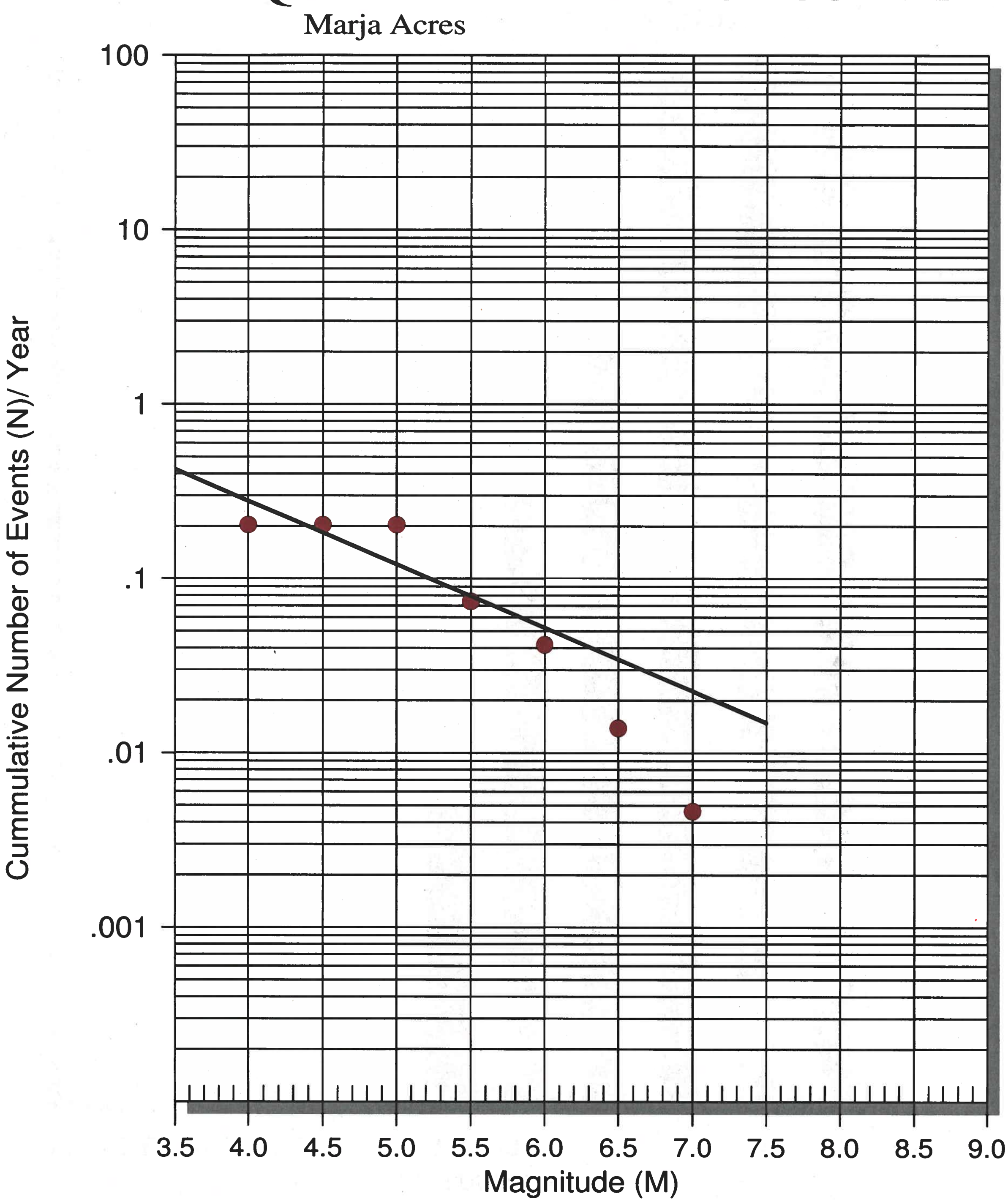
EARTHQUAKE EPICENTER MAP

Marja Acres

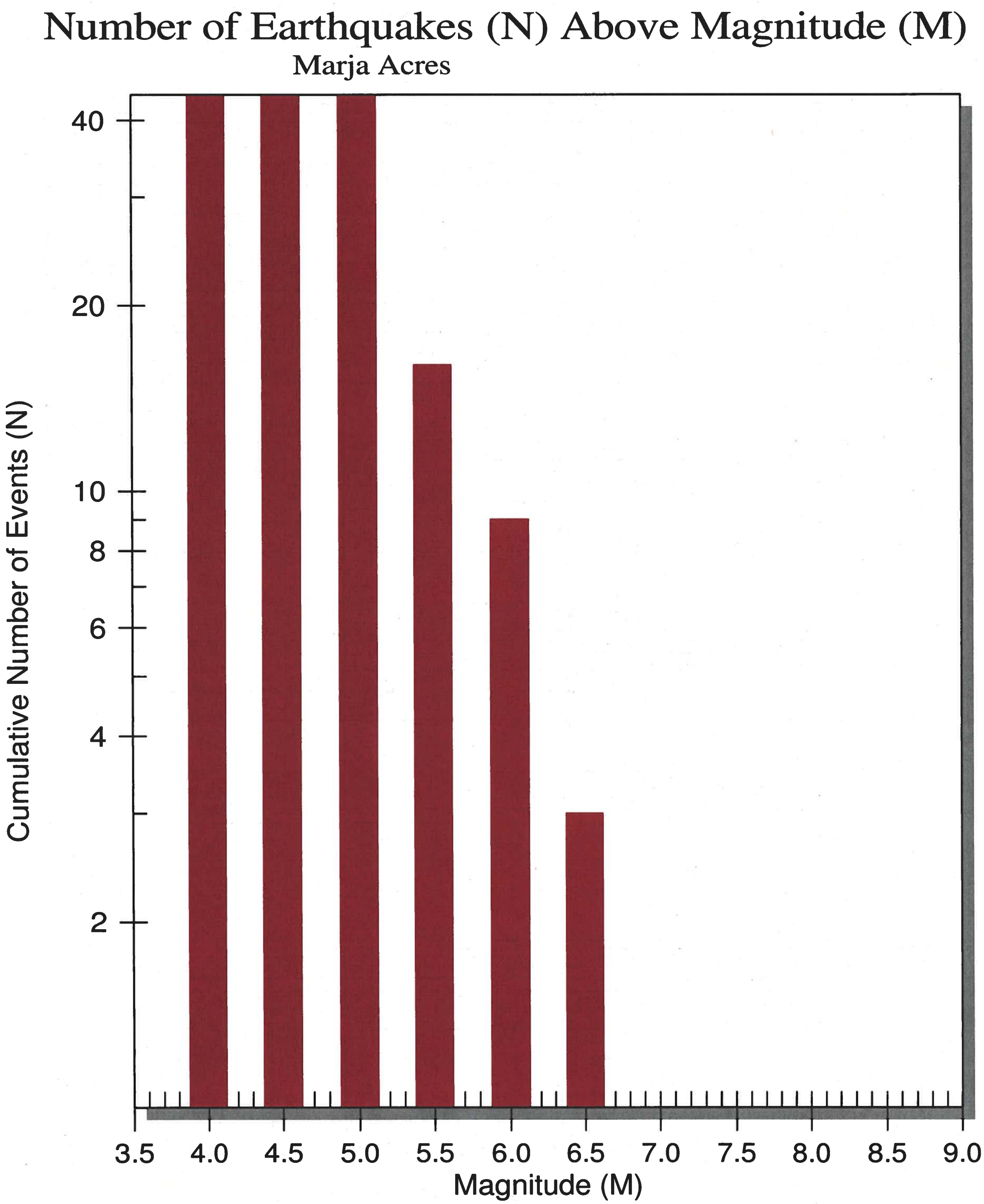


W.O. 6971-A-SC
PLATE C-9

EARTHQUAKE RECURRENCE CURVE



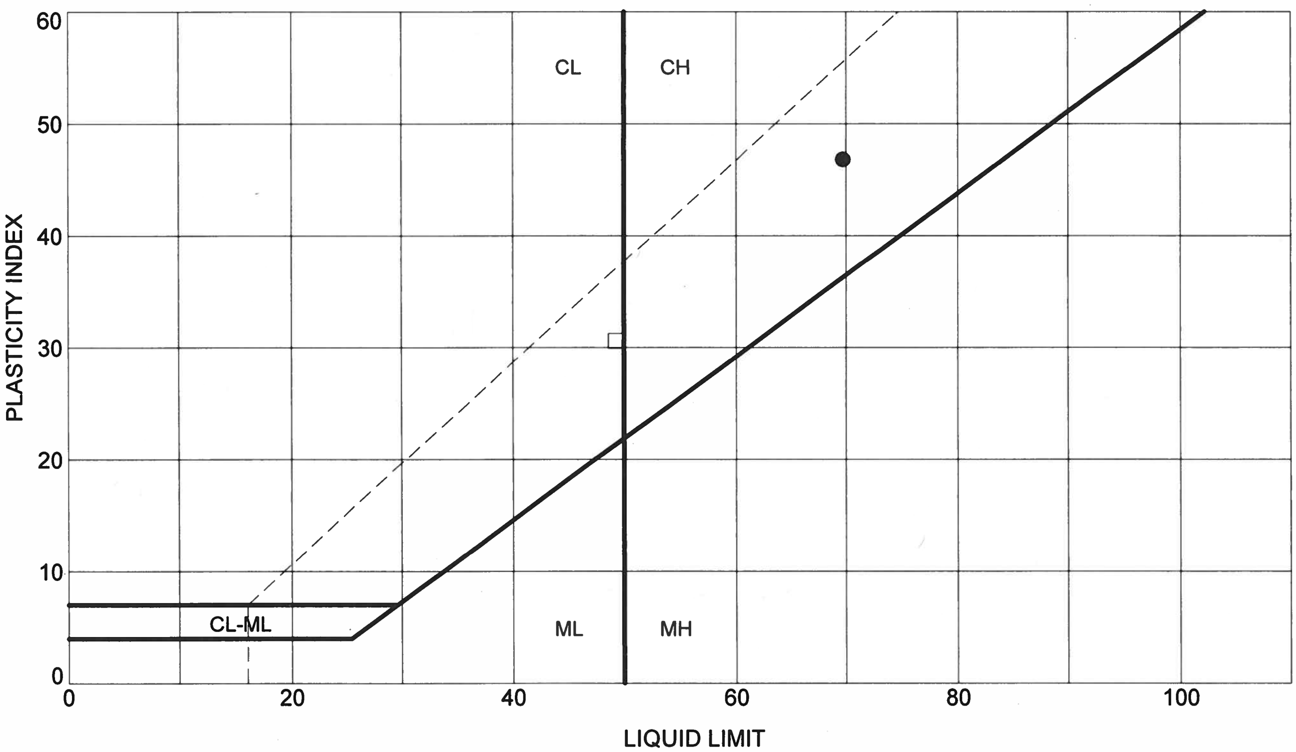
W.O. 6971-A-SC
PLATE C-10



W.O. 6971-A-SC
PLATE C-11

APPENDIX D

LABORATORY TESTING



	Sample	Depth/El.	LL	PL	PI	Fines	USCS CLASSIFICATION
●	TP-4	0.0	70	23	47		Clay w/ Sand
□	TP-8	5.5	49	19	30		Sandy Clay

US ATTERBERG LIMITS 6971 MURRIETA.GPJ US LAB.GDT 7/7/16

GeoSoils, Inc.
GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

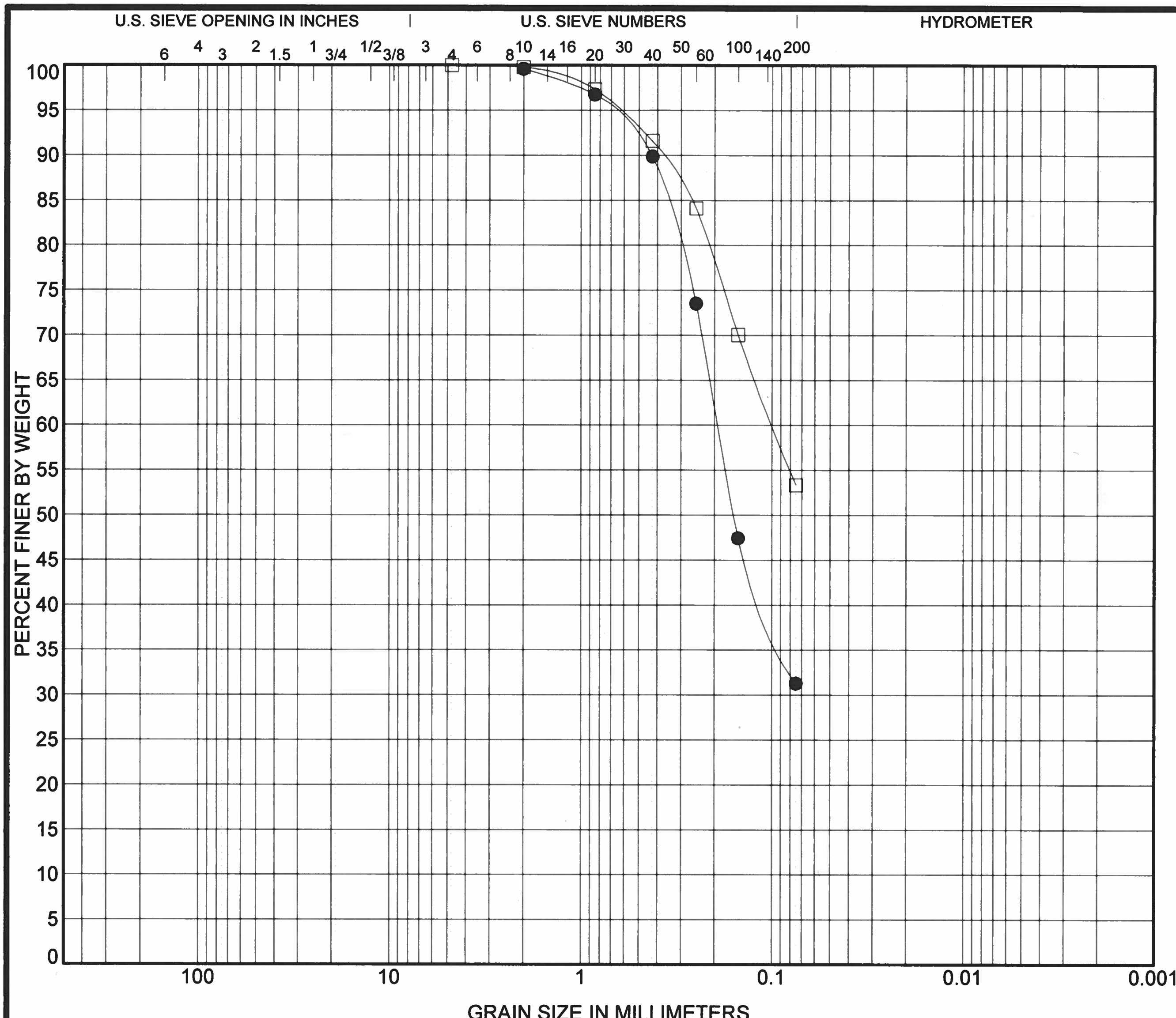
ATTERBERG LIMITS' RESULTS

Project: NUWI

Number: 6971-A-SC

Date: July 2016

Plate: D - 1



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample	Depth	Range	Visual Classification/USCS CLASSIFICATION	LL	PL	PI	Cc	Cu
●	B-1	15.0		Clayey Sand					
□	B-2	25.0		Clay w/ Sand					

	Sample	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-1	15.0	2	0.192			0.0	68.3	31.3	
□	B-2	25.0	4.75	0.099			0.0	46.7	53.3	

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

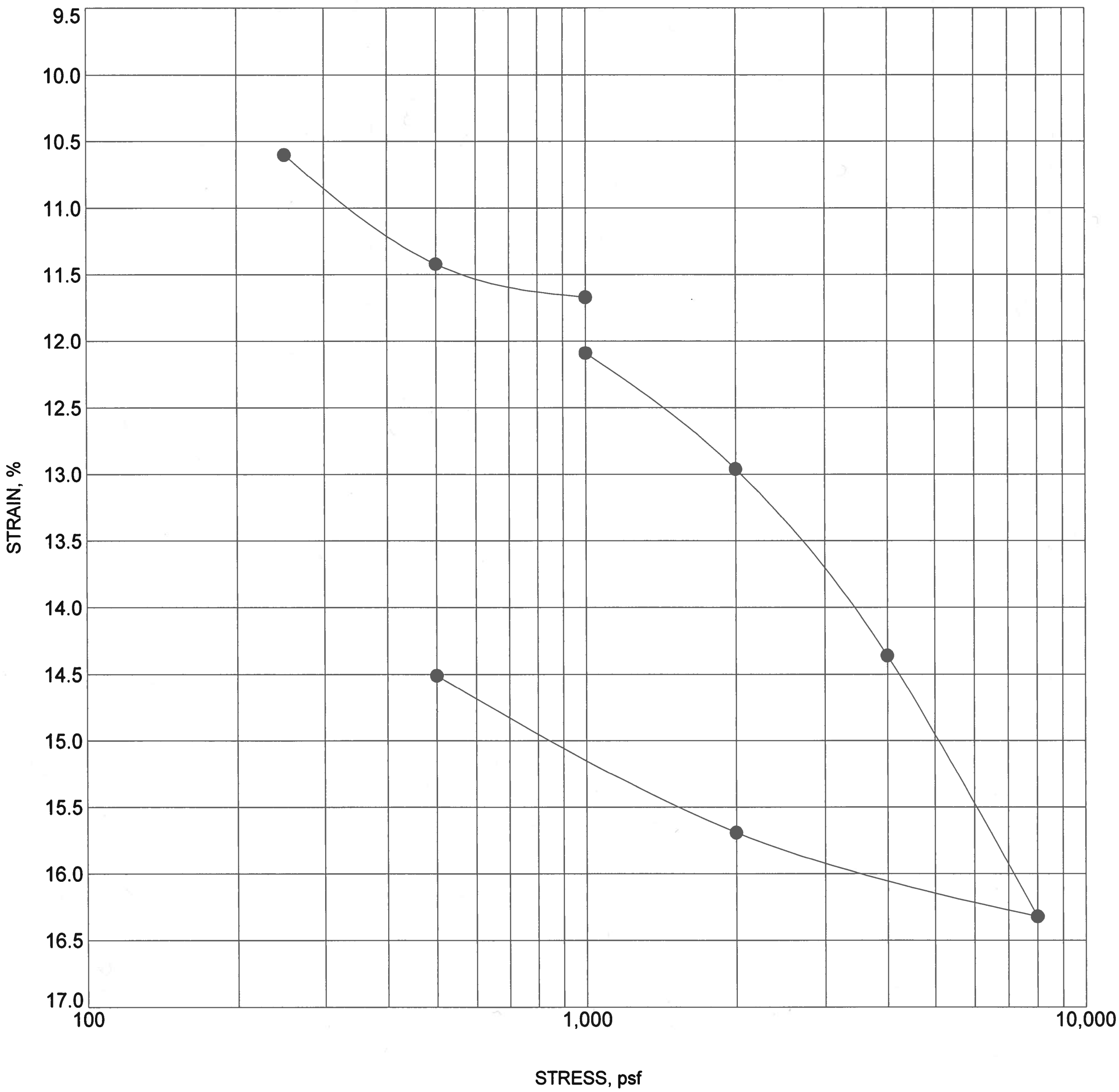
GRAIN SIZE DISTRIBUTION

Project: NUWI

Number: 6971-A-SC

Date: July 2016

Plate: D - 2



	Sample	Depth/El.	Visual Classification	γ_d Initial	MC Initial	MC Final	H20
●	B-1	5.0	Clayey Sand	100.0	15.6	24.5	1000

Stress at which water was added: 1000 psf
Strain Difference: _ _ _ _ 0.42%

US CONSOL STRAIN 6971 MURRIETA.GPJ US LAB.GDT 7/7/16

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

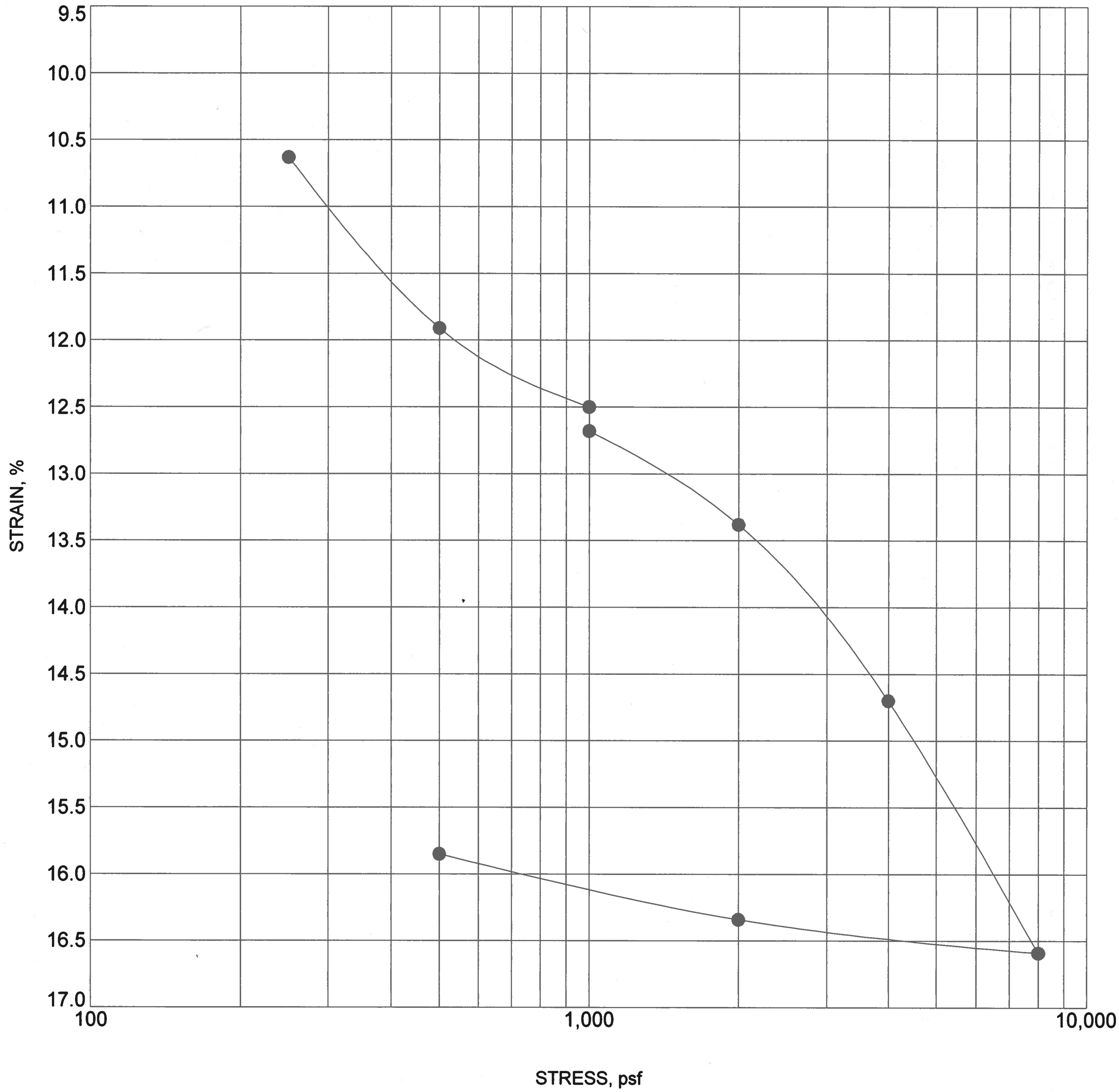
CONSOLIDATION TEST

Project: NUWI

Number: 6971-A-SC

Date: July 2016

Plate: D - 3



	Sample	Depth/El.	Visual Classification	γ_d Initial	MC Initial	MC Final	H20
●	B-2	15.0	Sandy Clay	104.7	18.5	18.7	1000

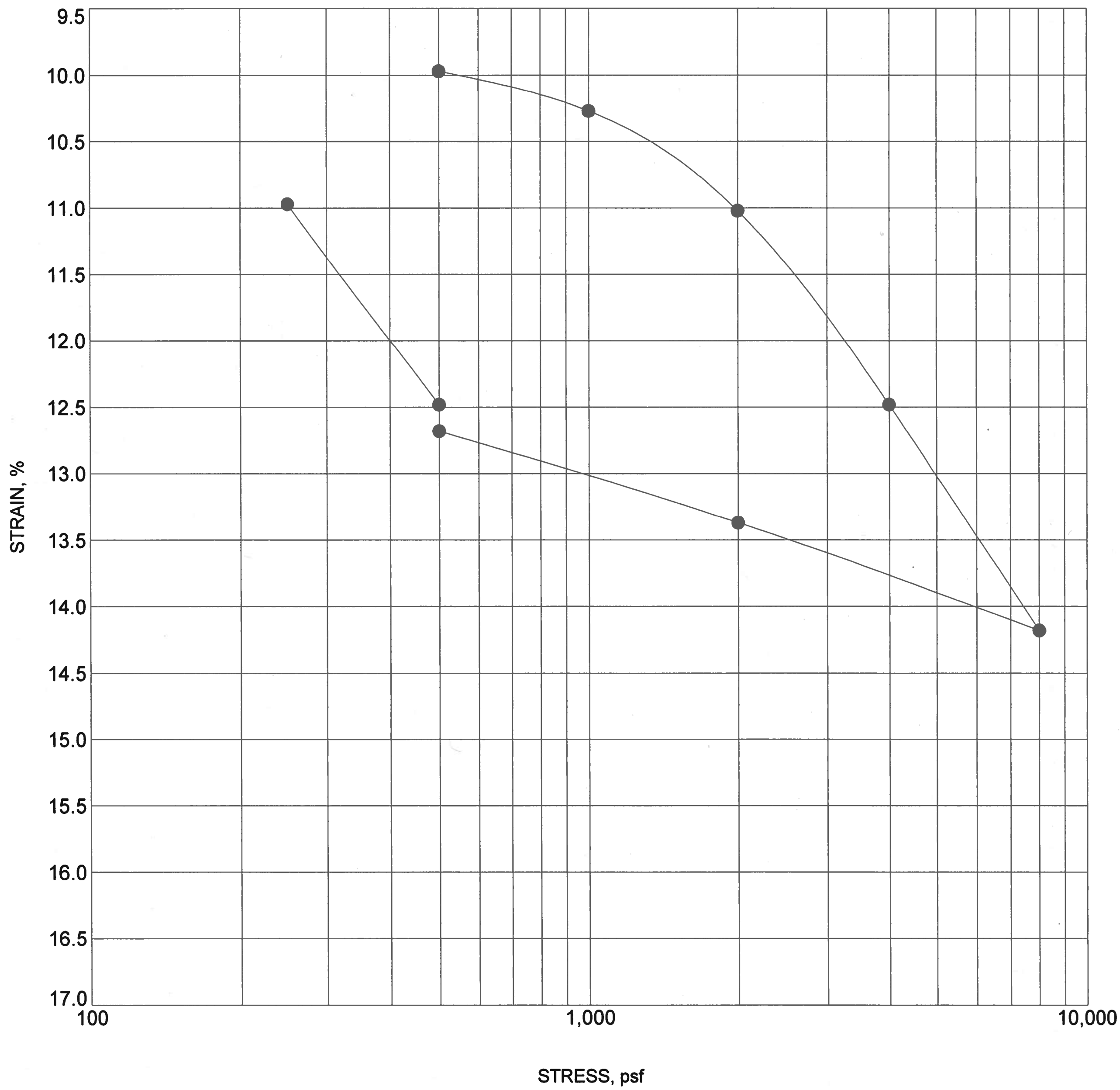
Stress at which water was added: 1000 psf
Strain Difference: _ _ _ _ 0.18%

US CONSOL STRAIN 6971 MURRIETA.GPJ US LAB.GDT 7/7/16

GeoSoils, Inc.
GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

CONSOLIDATION TEST

Project: NUWI
Number: 6971-A-SC
Date: July 2016
Plate: D - 4



	Sample	Depth/El.	Visual Classification	γ_d Initial	MC Initial	MC Final	H20
●	B-2	25.0	Clay w/ Sand	109.4	19.9	21.8	500

Stress at which water was added: 500 psf
Strain Difference: _ _ _ -2.51%

GeoSoils, Inc.
GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

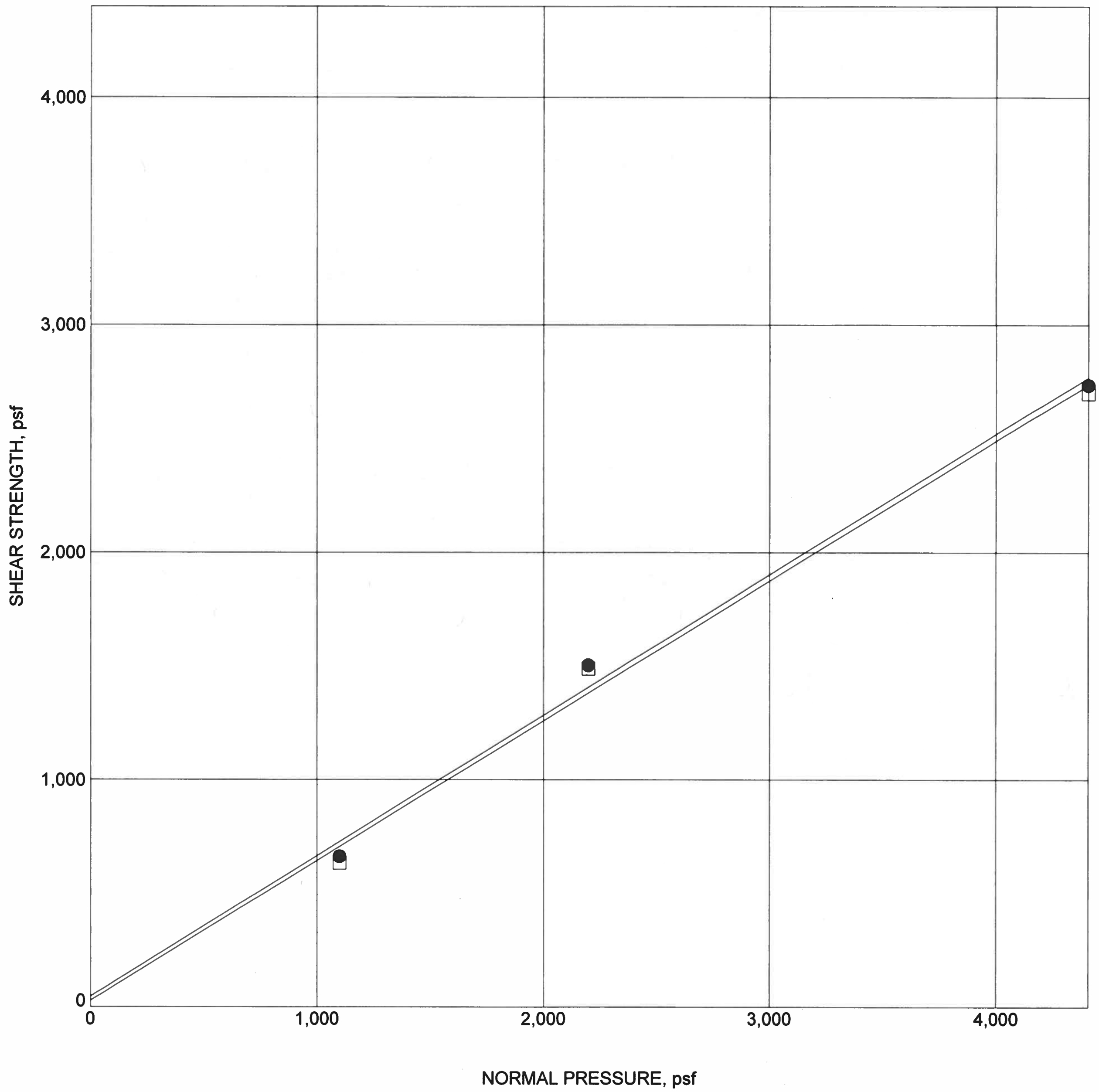
CONSOLIDATION TEST

Project: NUWI

Number: 6971-A-SC

Date: July 2016

Plate: D - 5



	Sample	Depth/El.	Range	Classification	Primary/Residual	Sample Type	γ_d	MC%	c	ϕ
●	TP-1	6.0		Clayey Sand	Primary Shear	Remolded	111.5	11.5	47	32
□	TP-1	6.1			Residual Shear	Remolded	111.5	11.5	29	32
					Reshear Shear	Remolded				
					Reshear Shear	Remolded				

Note: Sample Innundated Prior To Test

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

DIRECT SHEAR TEST

Project: NUWI
Number: 6971-A-SC
Date: July 2016
Plate: D - 6

SUMMARY OF LABORATORY TEST DATA

GeoSoils, Inc.
5741 Palmer Way, Suite D
Carlsbad, CA 92010

QCI Project No.: 16-029-005g
Date: May 26, 2016
Summarized by: KA

W.O. 6971-A-SC
Project Name: NUWI
Client: N/A

Corrosivity Test Results

Sample ID	Sample Depth (ft)	pH CT-532 (643)	Chloride CT-422 (ppm)	Sulfate CT-417 % By Weight	Resistivity CT-532 (643) (ohm-cm)
Composite TP-1, TP-4, TP-5	N/A	9.22	112	0.0237	1200

W.O. 6971-A-SC
PLATE D-7

APPENDIX E

LIQUEFACTION ANALYSIS

SEISMIC INDUCED LIQUEFACTION ANALYSIS

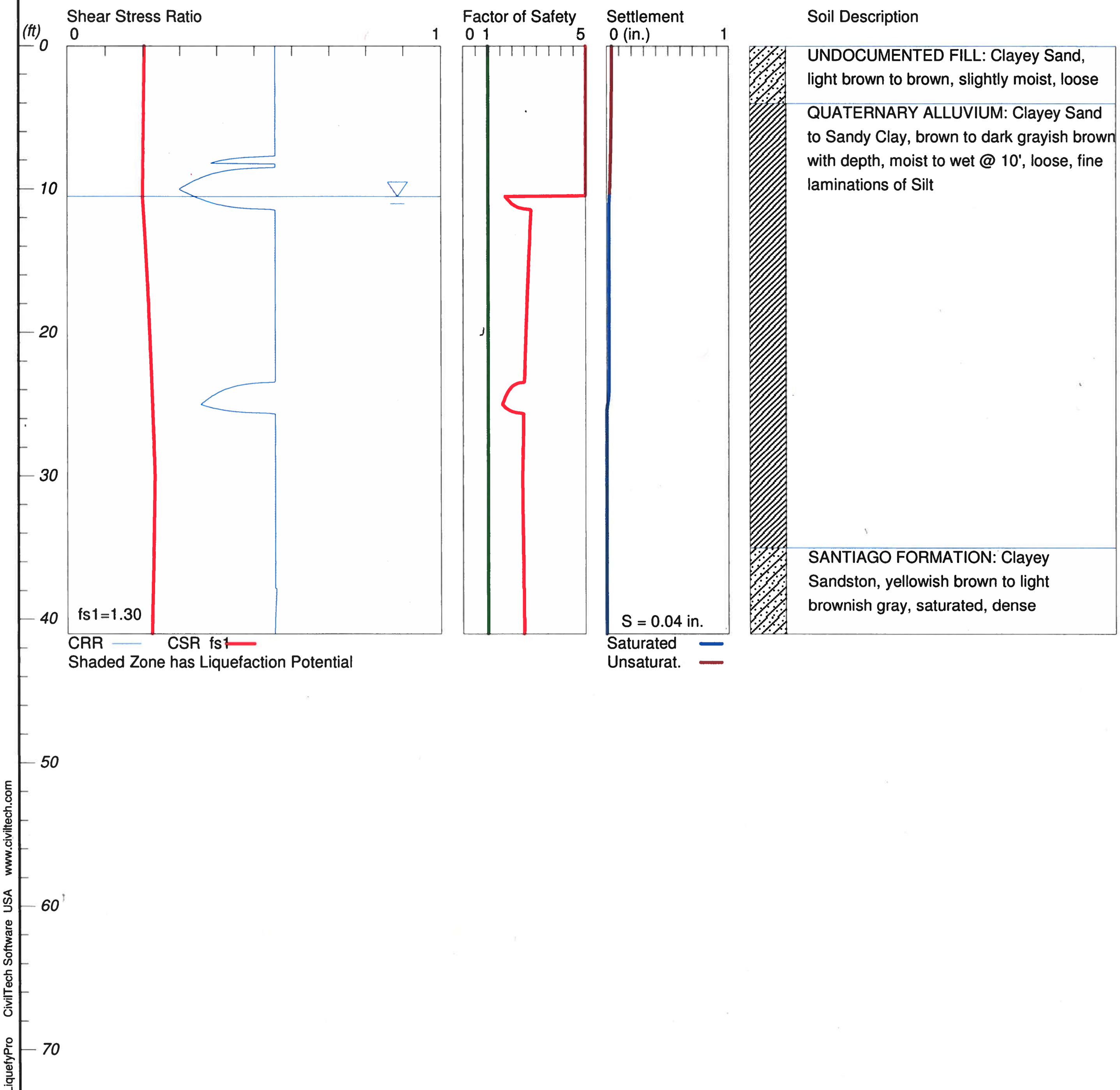
6971 Marja Acres, LLC, FOS = 1.3

Hole No.=B-1 Water Depth=10.5 ft Surface Elev.=47

Ground Improvement of Fill=25 ft

Magnitude=7.2

Acceleration=0.24g



Ishihara/Yoshimine; Stark/Olson et al

Plate E-1

GeoSoils, Inc.

SEISMIC INDUCED LIQUEFACTION ANALYSIS

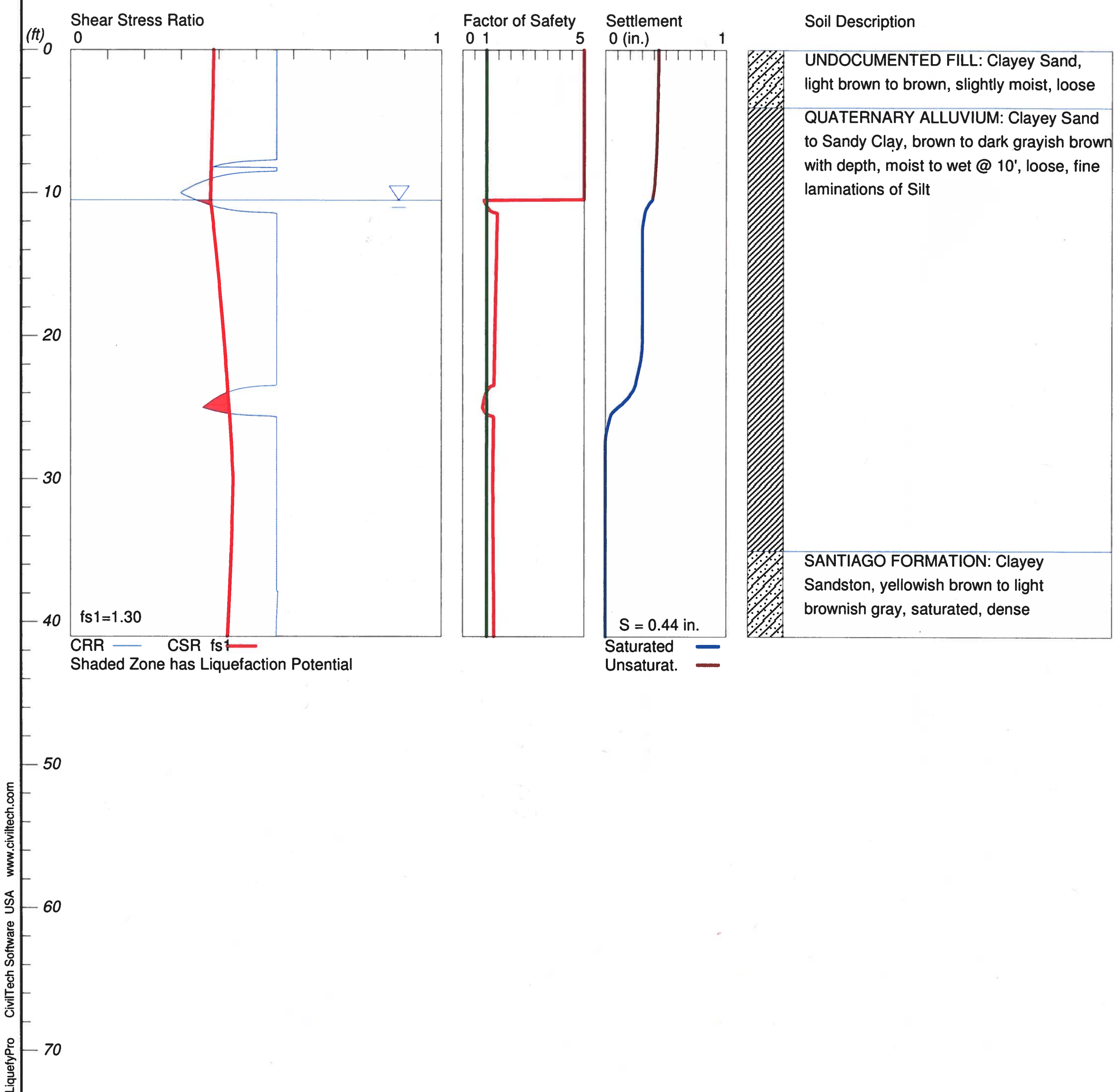
6971 Marja Acres, LLC, FOS = 1.3

Hole No.=B-1   Water Depth=10.5 ft   Surface Elev.=47

Ground Improvement of Fill=25 ft

Magnitude=7.2

Acceleration=0.457g



Ishihara/Yoshimine; Stark/Olson et al

Plate E-2

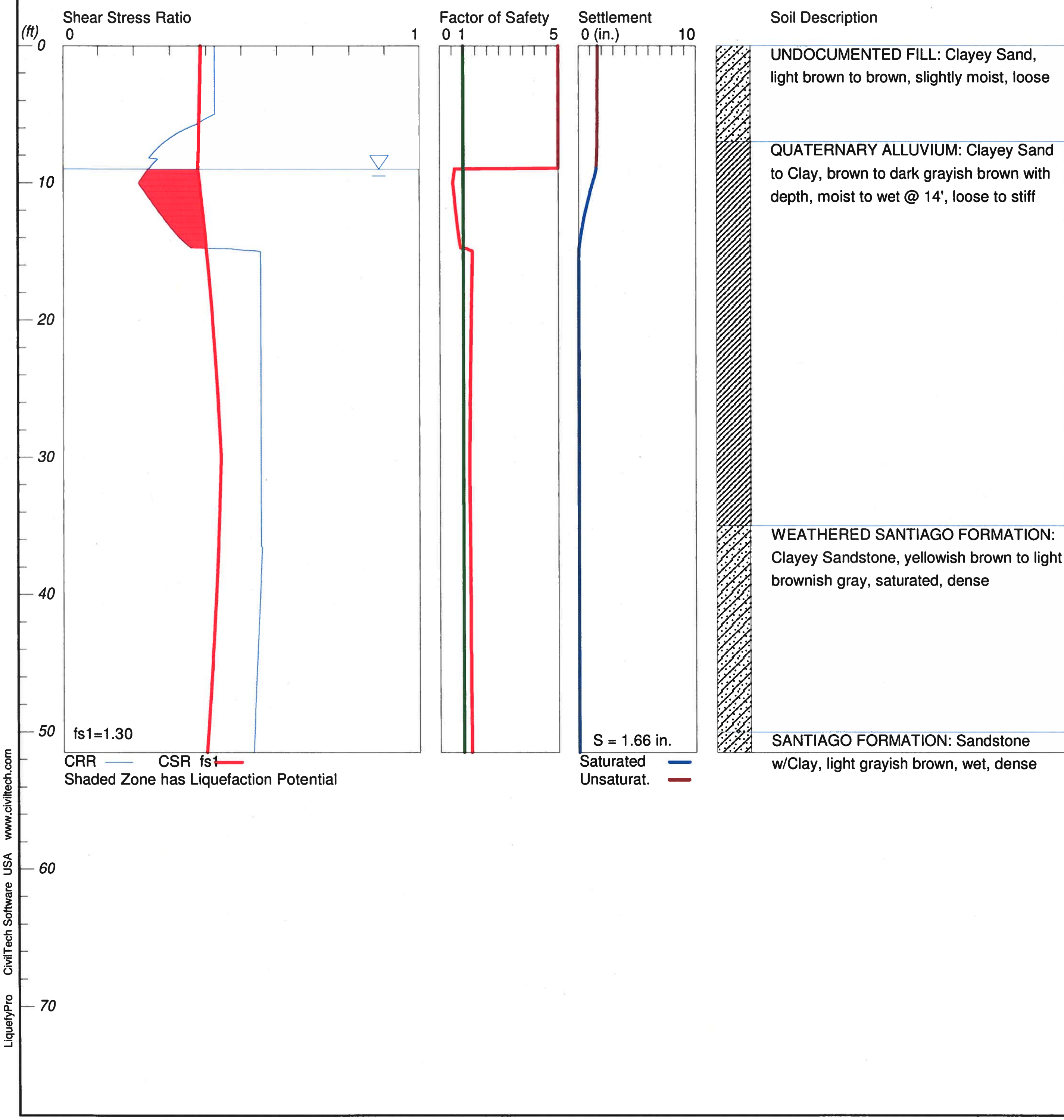
GeoSoils, Inc.

SEISMIC INDUCED LIQUEFACTION ANALYSIS

6971 Marja Acres, LLC, FOS = 1.3

Hole No.=B-2 Water Depth=9 ft Surface Elev.=46
Ground Improvement of Fill=25 ft

Magnitude=7.2
Acceleration=0.457g



Ishihara/Yoshimine; Stark/Olson et al **Plate E-3**

GeoSoils, Inc.

SEISMIC INDUCED LIQUEFACTION ANALYSIS

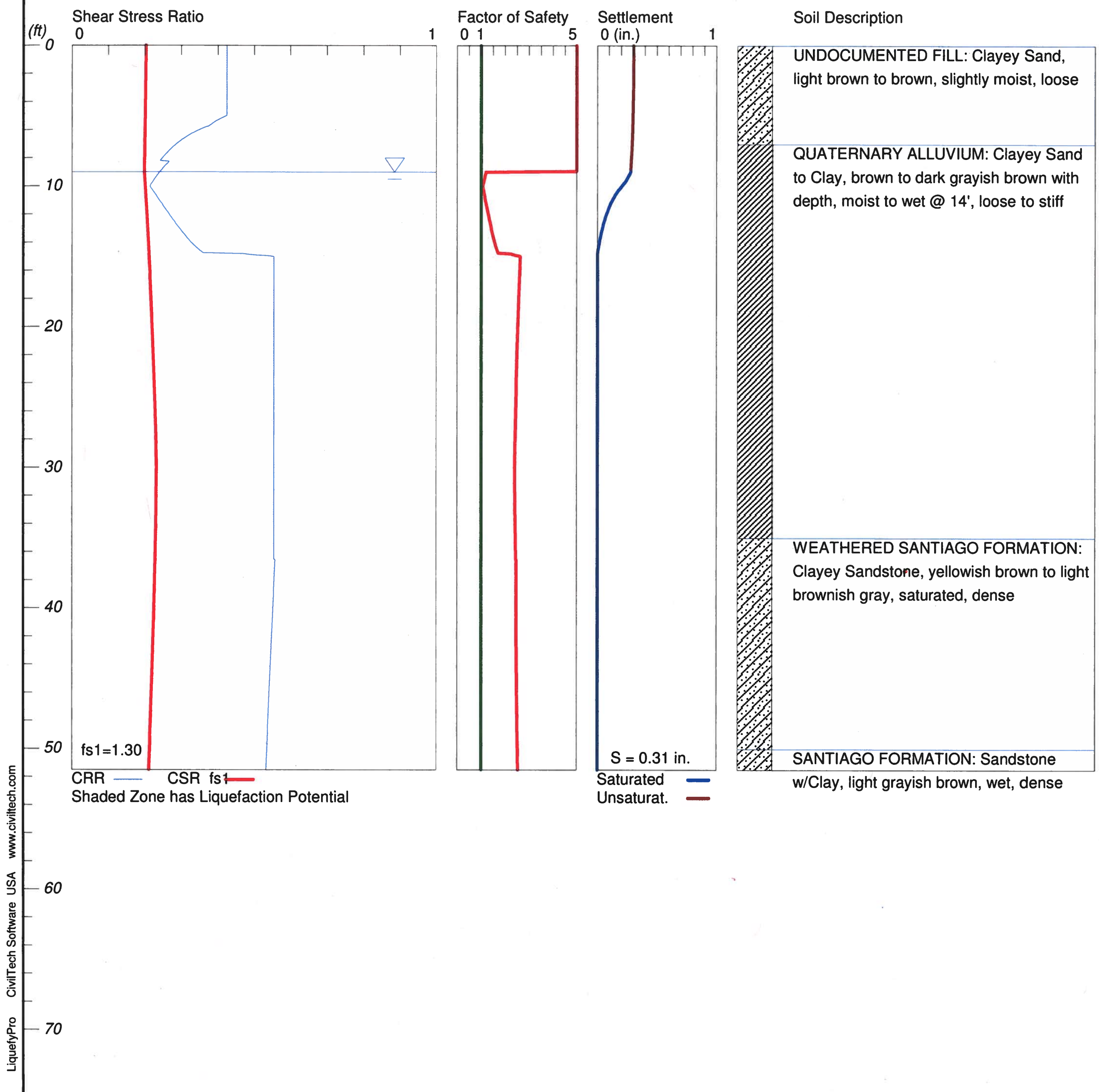
6971 Marja Acres, LLC, FOS = 1.3

Hole No.=B-2 Water Depth=9 ft Surface Elev.=46

Ground Improvement of Fill=25 ft

Magnitude=7.2

Acceleration=0.24g



Ishihara/Yoshimine; Stark/Olson et al

Plate E-4

GeoSoils, Inc.

APPENDIX F

STORM WATER INFILTRATION FEASIBILITY

Worksheet Form I-8: Categorization of Infiltration Feasibility Condition

Categorization of Infiltration Feasibility Condition			**Form I-8**
Part 1 - Full Infiltration Feasibility Screening Criteria **Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?**			
Criteria	**Screening Question**	**Yes**	**No**
1	**Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X
Provide basis: The United States Department of Agriculture (USDA) has evaluated the infiltration rate of natural surface soils as ranging from 0.00 to about 0.57 inches/hr (Hydrologic Soil Groups C and D). Furthermore, future development will result in the removal/recompaction of a natural surface soils, and/or the presence of cut areas exposing relatively dense formational soils consisting of clays, sandy clays, and sandstone. The resultant infiltration rates for these much denser (and proportionally less permeable) formational materials would be expected to be very near, or below the rate evaluated by the USDA. Artificial fill, created through removal/recompaction of onsite soils would also be considered to be of a similar, very low permeability. See text for other related discussions and references. 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: The limited permeability of formational soils will tend to result in the lateral migration of water and saturated conditions at, or near the surface, increasing the potential for distress to foundations, floor slabs, etc. Onsite soils are expansive. Saturation of some onsite soils can likely generate adverse uplift pressures on floor slabs, or lightly loaded foundation. There is an increased potential for the creation of perched groundwater (mounding) conditions along zones of contrasting permeabilities, including shallow cut/fill contacts, and transitions between relatively clayey and sandy formational materials, and shallow groundwater in low lying areas. Due to the likelihood of strong permeability contrasts between formation and fill, utility trenches can potentially act as french drains and provide conduits for the movement of excessive moisture beneath the structure(s). Graded slopes in close proximity to infiltration areas can become saturated, losing soil strength and becoming more susceptible to slope instability and failure. See text for other related discussions and references. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			

Worksheet C.4.1 Page 2 of 4			
Criteria	**Screening Question**	**Yes**	**No**
3	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensible evaluation of the factors presented in Appendix C.3.		X
Provide basis: While this study did no include an environmental assessment, visual observation did not indicate the presence of potential contaminants. The infiltration rate is generally less than 0.5 inches per hour. The regional groundwater table is considered a factor in the development of this site, the creation of a shallow "perched" water table can occur through infiltration. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
4	**Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as 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.		X
Provide basis: The infiltration rate is generally anticipated to be less than 0.5 inches per hour. The site currently drains offsite and no runoff appears to be retained onsite. The regional groundwater table is considered a factor in the development of this site. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
Part 1 Result*	If 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		

* 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.

From "Engineering standards, Volume 5, Carlsbad BMP Design Manual, Appendices, dated February 2016

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: No. Onsite soils are typically fine grained and clayey. The United States Department of Agriculture (USDA) has evaluated the infiltration rate of natural surface soils in the vicinity to be as low as 0.00. Subsequent development of the site will likely result in the removal/recompaction/densification of a natural surface soils, or the exposure of denser and less permeable formational soils at depth. The resultant infiltration rates for these much denser formational materials would be expected to be very near, or below the rate evaluated by the USDA. See text of report for other related discussions and references. 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: No. The limited permeability of the relatively dense, fine grained formational soils will tend to result in the lateral migration of water and saturated conditions at, or near the surface, increasing the potential for distress to foundations, floor slabs, etc. Onsite soils are expansive, saturation of onsite soils may generate adverse uplift pressures on floor slabs, or lightly loaded foundation. There is an increased potential for the creation of perched groundwater (mounding) conditions along zones of contrasting permeabilities, including shallow cut/fill contacts, and transitions between clayey and sandy formational materials within the sedimentary bedrock, and shallow groundwater in low lying areas. Due to the strung permeability contrast between formation and fill, utility trenches can potentially act as french drains and provide conduits for the movement of excessive moisture beneath the structure(s). Graded slopes in close proximity to infiltration areas can become saturated, losing soil strength and becoming more susceptible to slope instability and failure. See text of report for other related discussions and references. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			

Worksheet C.4.1 Page 4 of 4			
Criteria	**Screening Question**	**Yes**	**No**
7	**Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.		X
Provide basis: While the regional groundwater table is not considered a factor in the development of this site, the creation of a shallow "perched" water table can occur and increase the potential for distress to the structure(s) due to water vapor transmission through foundations, slabs, and any resultant corrosive effects on metal conduit in trenches. See text of report for other related discussions and references. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
8	**Can infiltration be allowed without violating downstream water rights?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: The site currently drains offsite and no runoff appears to be retained onsite. The regional groundwater table is not considered a factor in the development of this site. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
Part 2 Result*	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.**	**No Infiltration**	

* 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.

APPENDIX G

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

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

SITE PREPARATION

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

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

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

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

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

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

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

COMPACTED FILLS

Any earth materials imported or excavated on the property may be 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 disclosed to all interested/affected parties. Once approved by the governing agency, the hold-down depth for oversized rock (i.e., greater than 12 inches) in fills on this project is provided as 10 feet, unless specified differently in the text of this report. The governing agency may require that these materials need to be deeper, crushed, or reduced to less than 12 inches in maximum dimension, at their discretion.

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

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

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

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

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

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

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

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

1. An extra piece of equipment consisting of a heavy, short-shanked sheepsfoot should be used to roll (horizontal) parallel to the slopes continuously as fill is placed. The sheepsfoot roller should also be used to roll perpendicular to the

slopes, and extend out over the slope to provide adequate compaction to the face of the slope.

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

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

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

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

SUBDRAIN INSTALLATION

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

EXCAVATIONS

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

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

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

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

COMPLETION

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

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

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 Class 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 closer 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, or 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. All 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 minimally conform to Cal/OSHA ("Type C" 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 particles greater that 6 inches), and the pool/spa walls closer 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 particles greater than 6 inches), and the pool/spa walls closer 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 prime responsibility of the contractor; however, everyone must be safety conscious and responsible at all times. To achieve our goal of avoiding accidents, cooperation between the client, the contractor, and GSI personnel must be maintained.

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

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

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

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

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

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

Test Pits Location, Orientation, and Clearance

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

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

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

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

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

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

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

Trench and Vertical Excavation

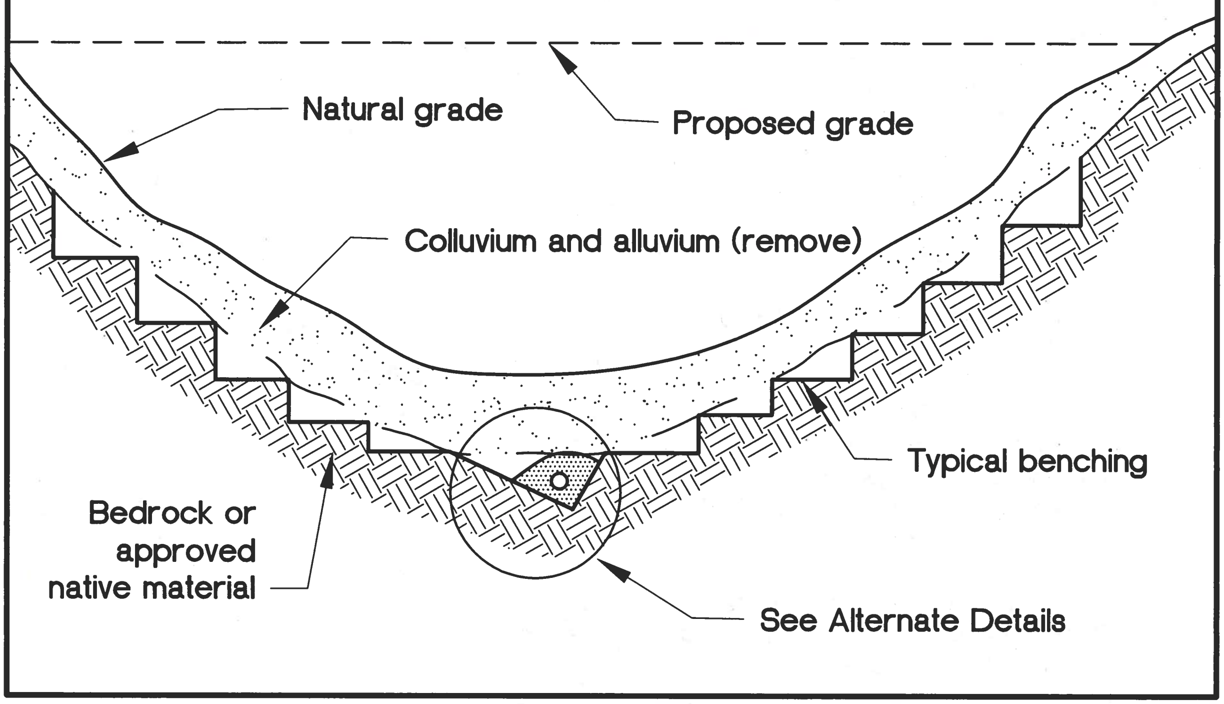
It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Our personnel are directed not to enter any excavation or vertical cut which: 1) is 5 feet or deeper unless shored or laid back; 2) displays any evidence of instability, has any loose rock or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

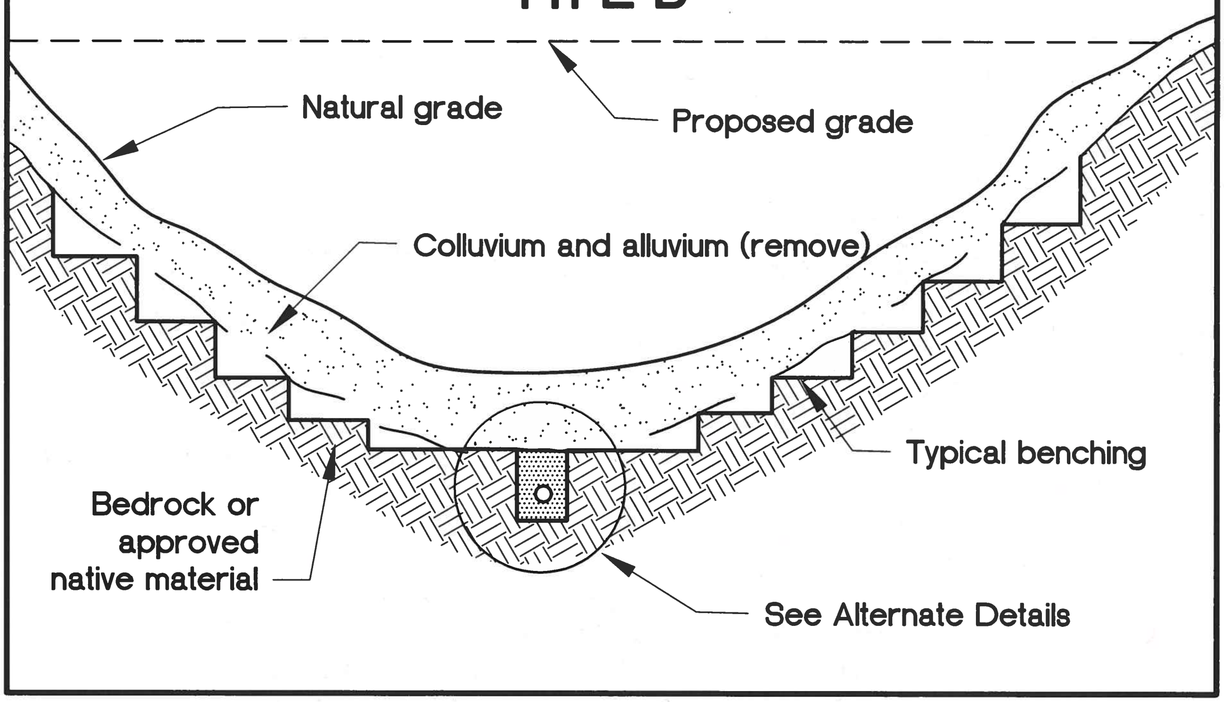
If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraw and notify his/her supervisor. The contractor's representative will be contacted in an effort to affect a solution. All backfill not tested due to safety concerns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

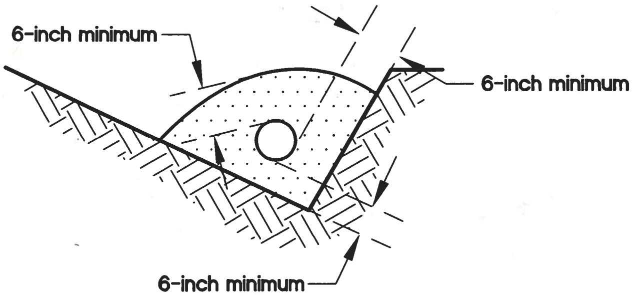


TYPE B

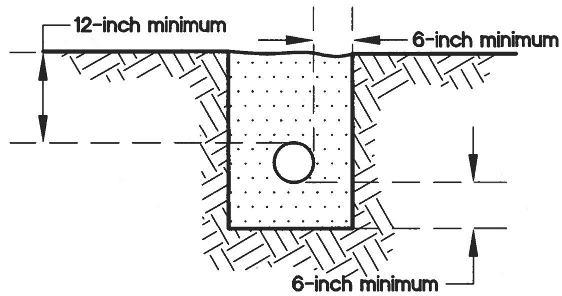


Selection of alternate subdrain details, location, and extent of subdrains should be evaluated by the geotechnical consultant during grading.

GeoSoils, Inc.	CANYON SUBDRAIN DETAIL	Plate G-1



A-1



B-1

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

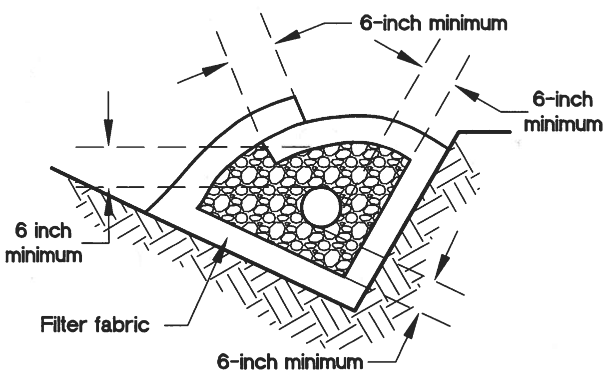
Perforated pipe: 6-inch-diameter ABS or PVC pipe or approved substitute with minimum 8 perforations ($\frac{1}{4}$-inch diameter) per lineal foot in bottom half of pipe (ASTM D-2751, SDR-35, or ASTM D-1527, Schd. 40).

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

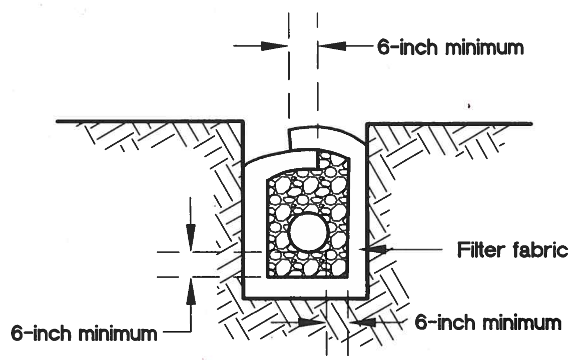
FILTER MATERIAL

Sieve Size	Percent Passing
1 inch	100
$\frac{3}{4}$ 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

ALTERNATE 1: PERFORATED PIPE AND FILTER MATERIAL



A-2

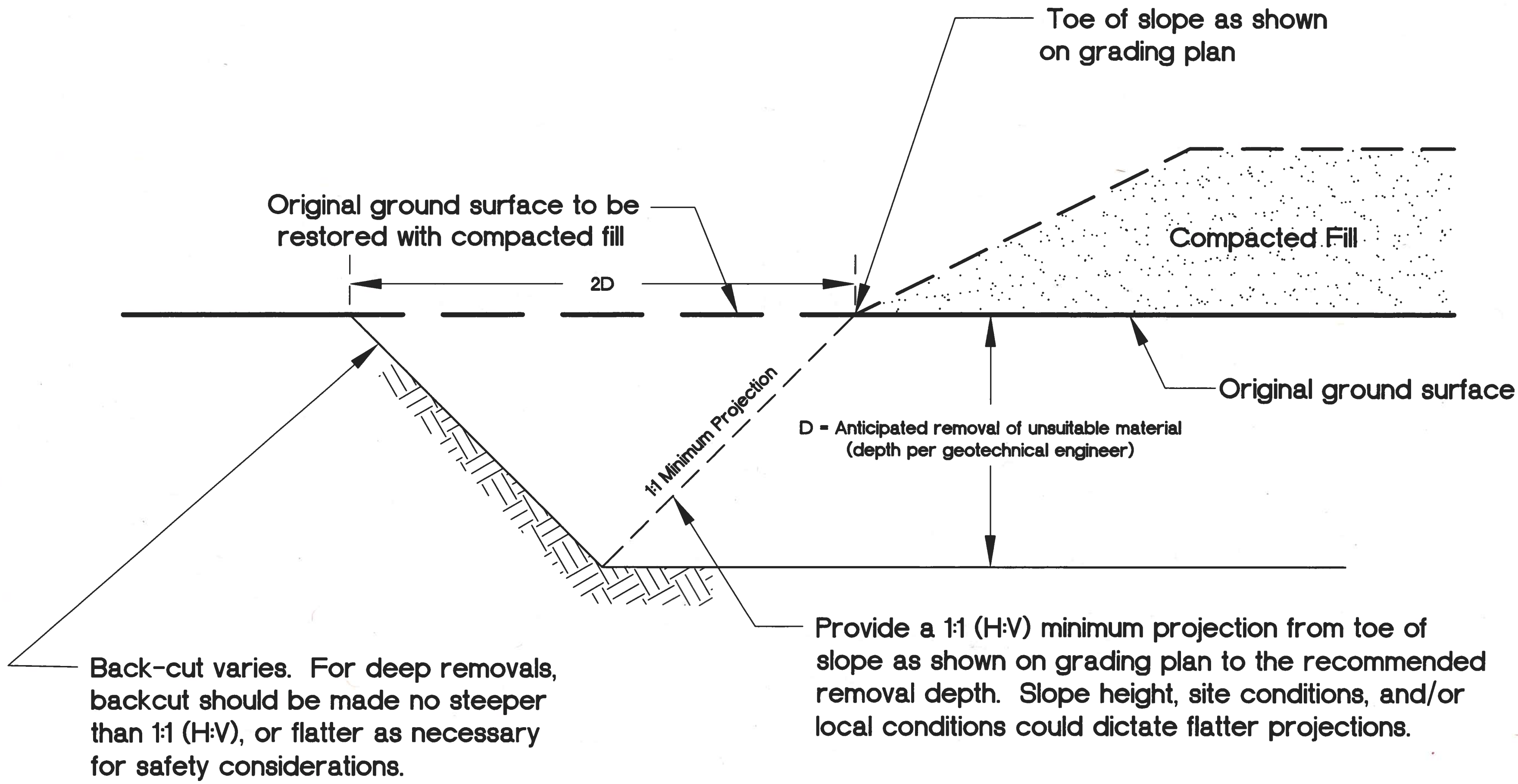


B-2

Gravel Material: 9 cubic feet per lineal foot.
Perforated Pipe: See Alternate 1
Gravel: Clean $\frac{3}{4}$-inch rock or approved substitute.
Filter Fabric: Mirafi 140 or approved substitute.

ALTERNATE 2: PERFORATED PIPE, GRAVEL, AND FILTER FABRIC

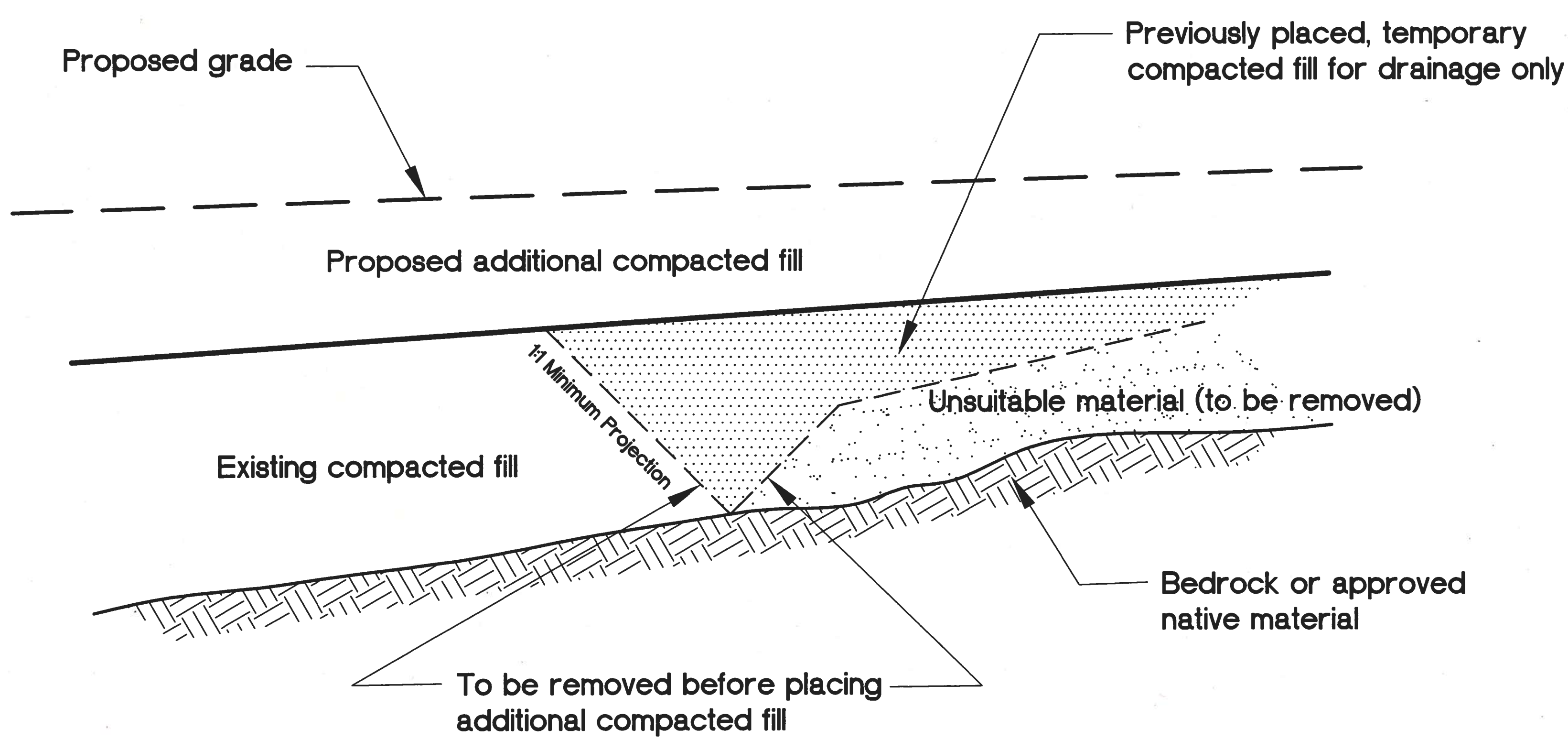
GeoSoils, Inc.	CANYON SUBDRAIN ALTERNATE DETAILS	Plate G-2



GeoSoils, Inc.

FILL SLOPE TOEING OUT ON FLAT ALLUVIATED CANYON DETAIL

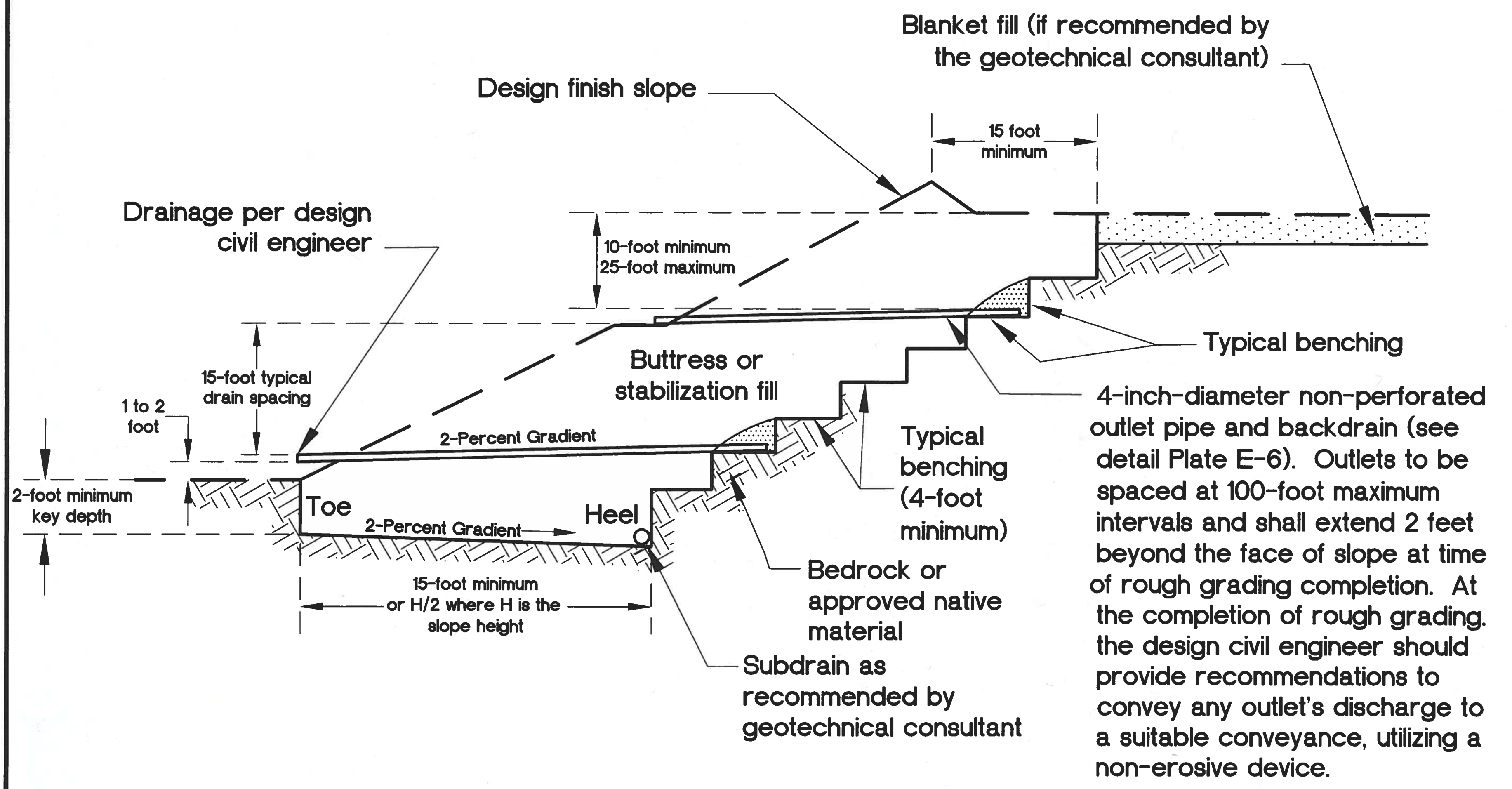
Plate G-3



GeoSoils, Inc.

REMOVAL ADJACENT TO EXISTING FILL ADJOINING CANYON FILL DETAIL

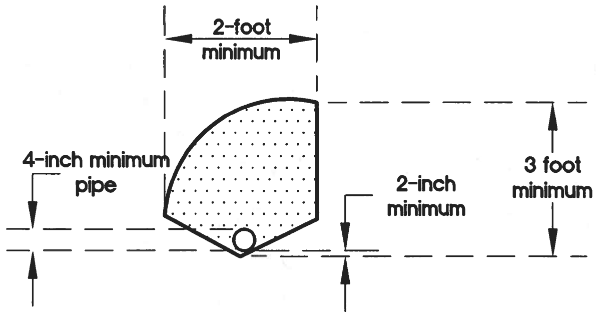
Plate G-4

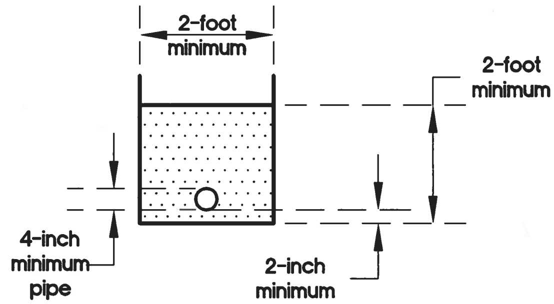


GeoSoils, Inc.

TYPICAL STABILIZATION / BUTTRESS FILL DETAIL

Plate G-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-spa 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.

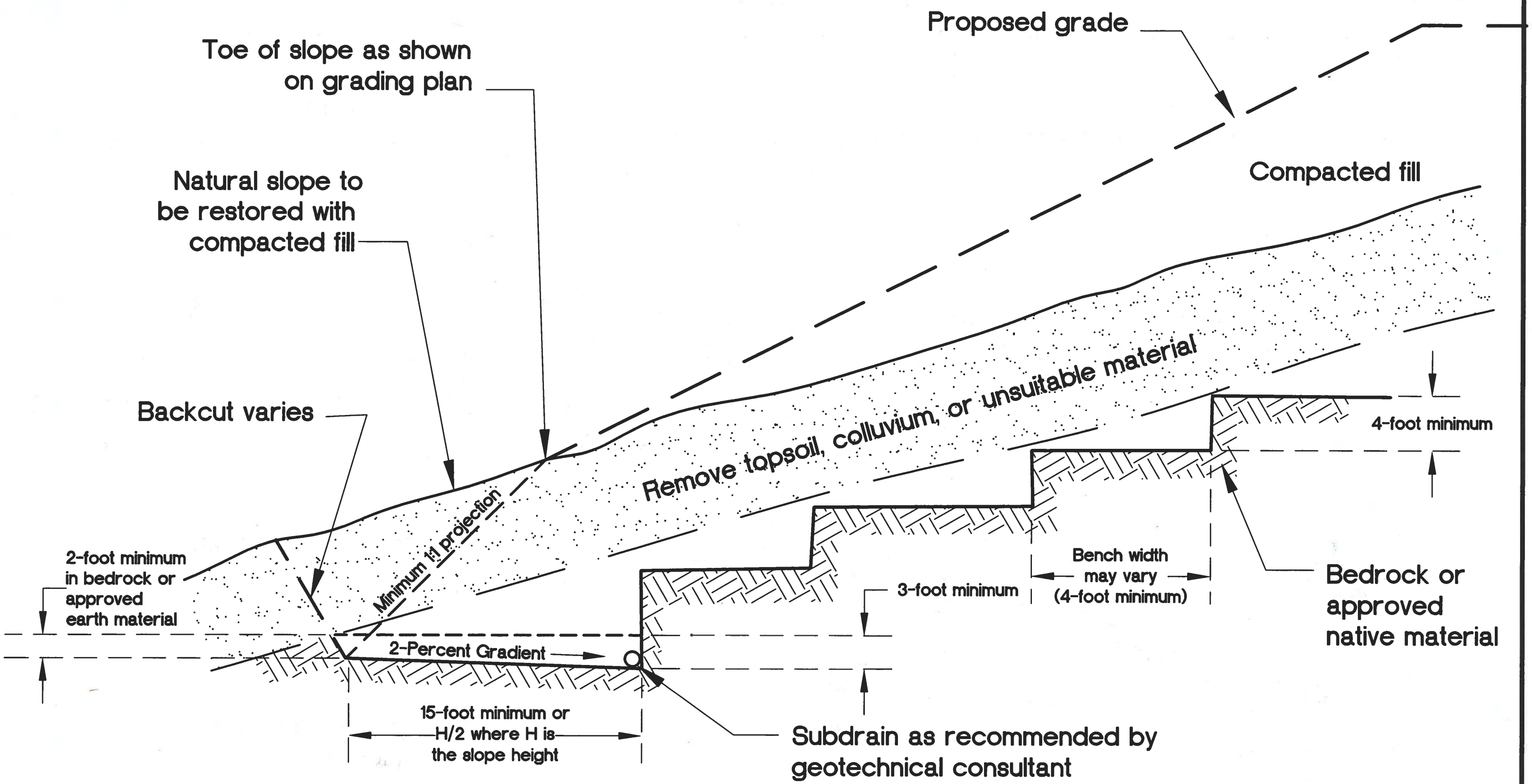
Filter Material shall be of the following specification or an approved equivalent.

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

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

Sieve Size	Percent Passing
1½ inch	100
No. 4	50
No. 200	8

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

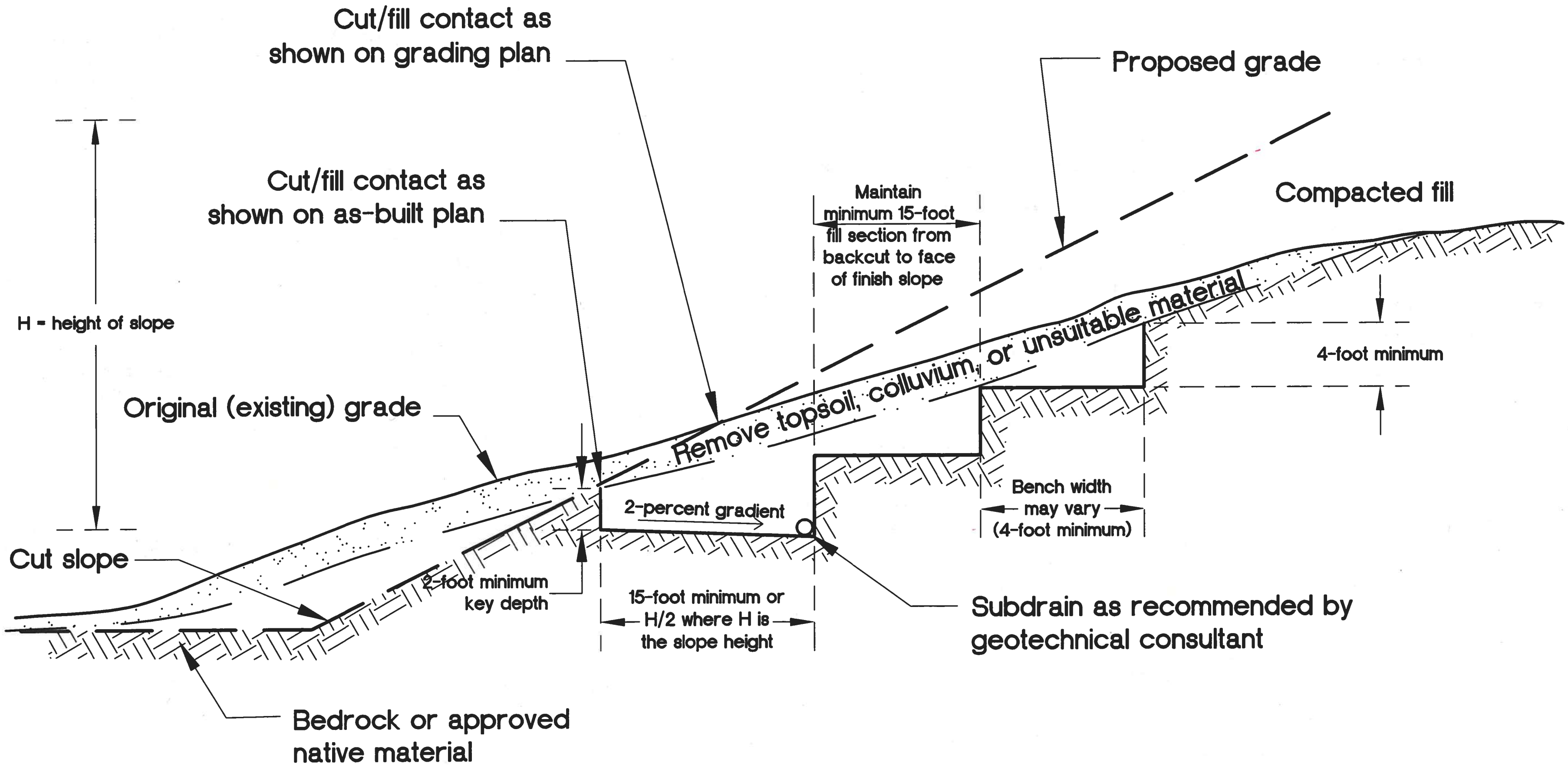


NOTES:

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 G-7

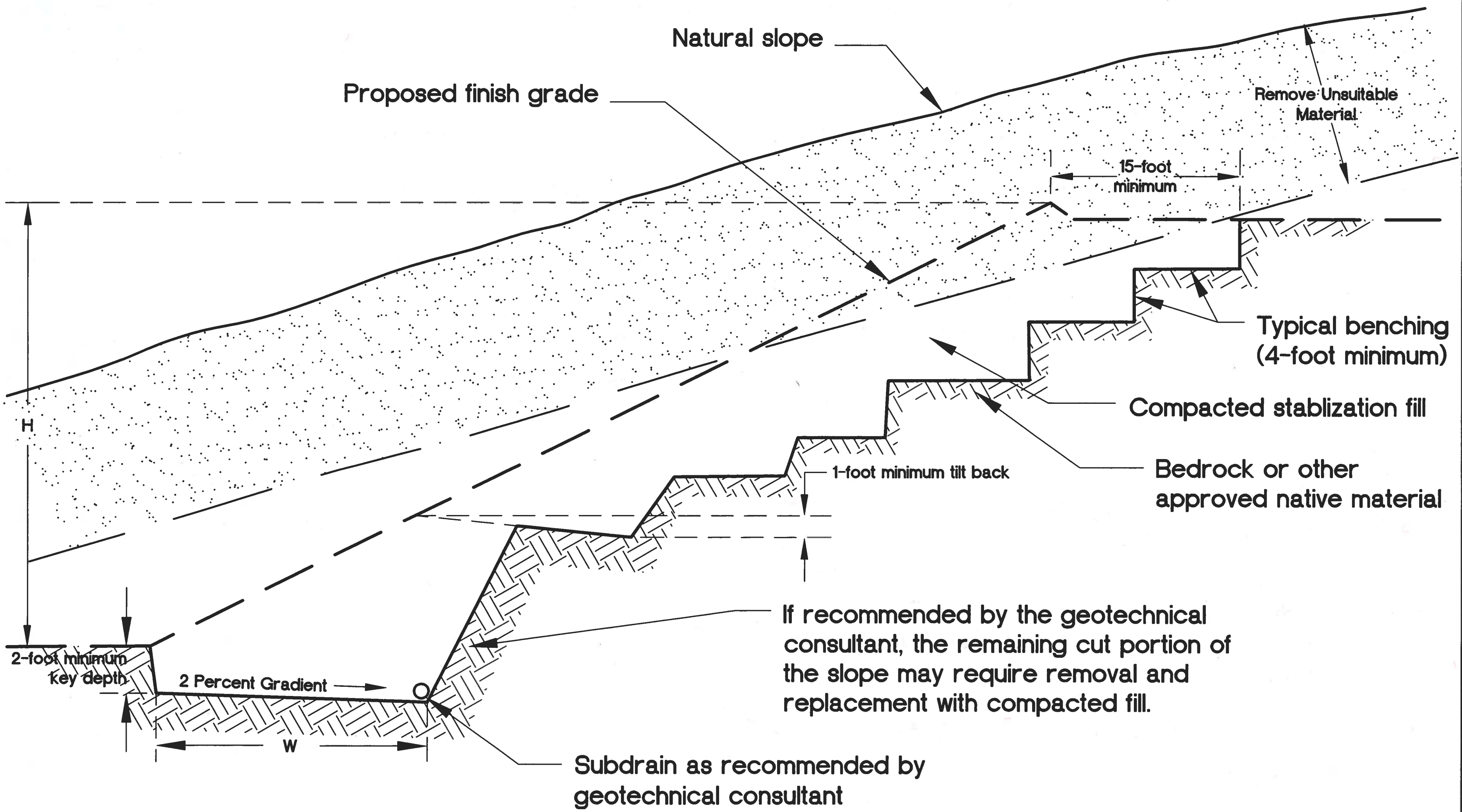


NOTE: The cut portion of the slope should be excavated and evaluated by the geotechnical consultant prior to construction of the fill portion.

GeoSoils, Inc.

FILL OVER CUT DETAIL

Plate G-8

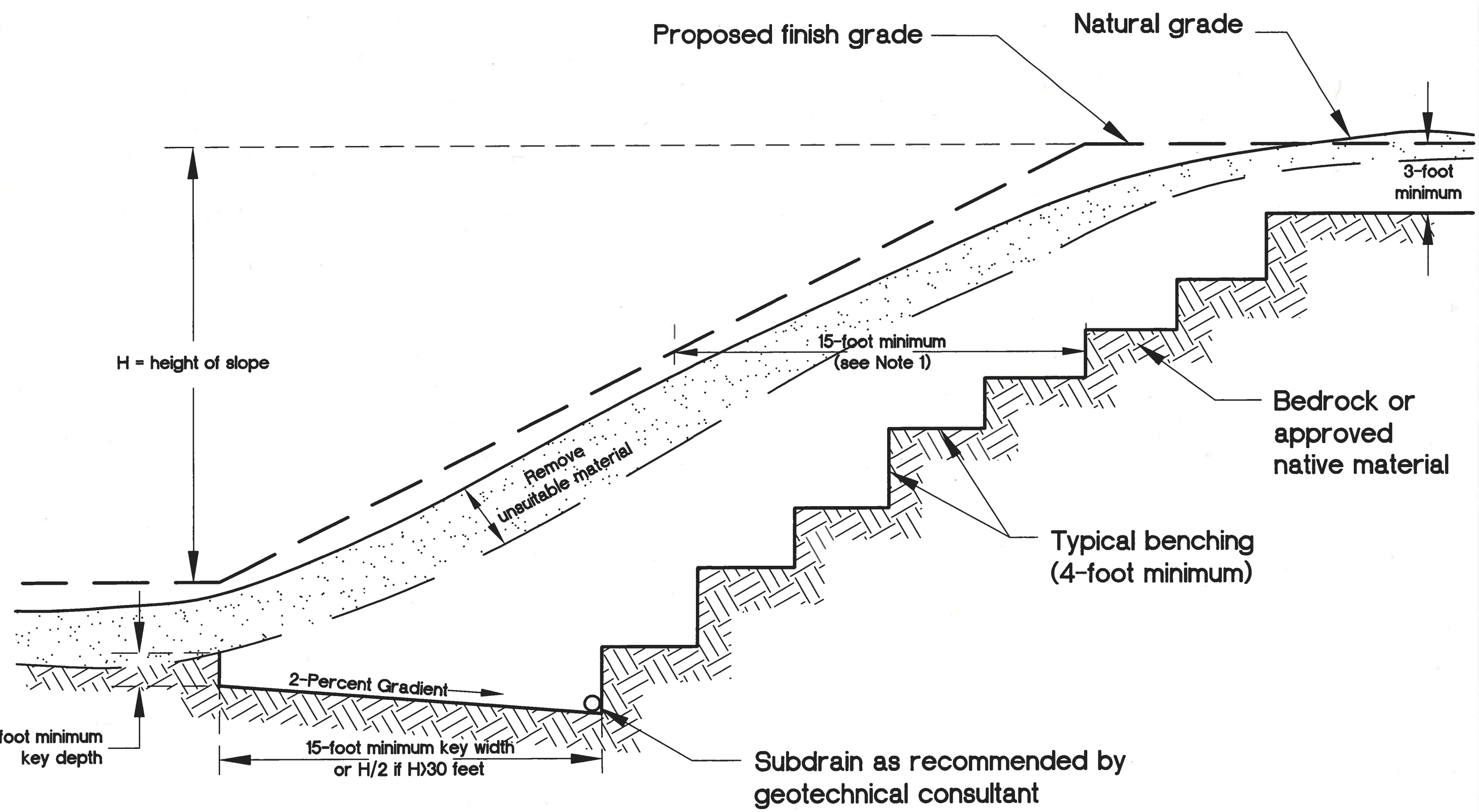


NOTES:

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 G-9



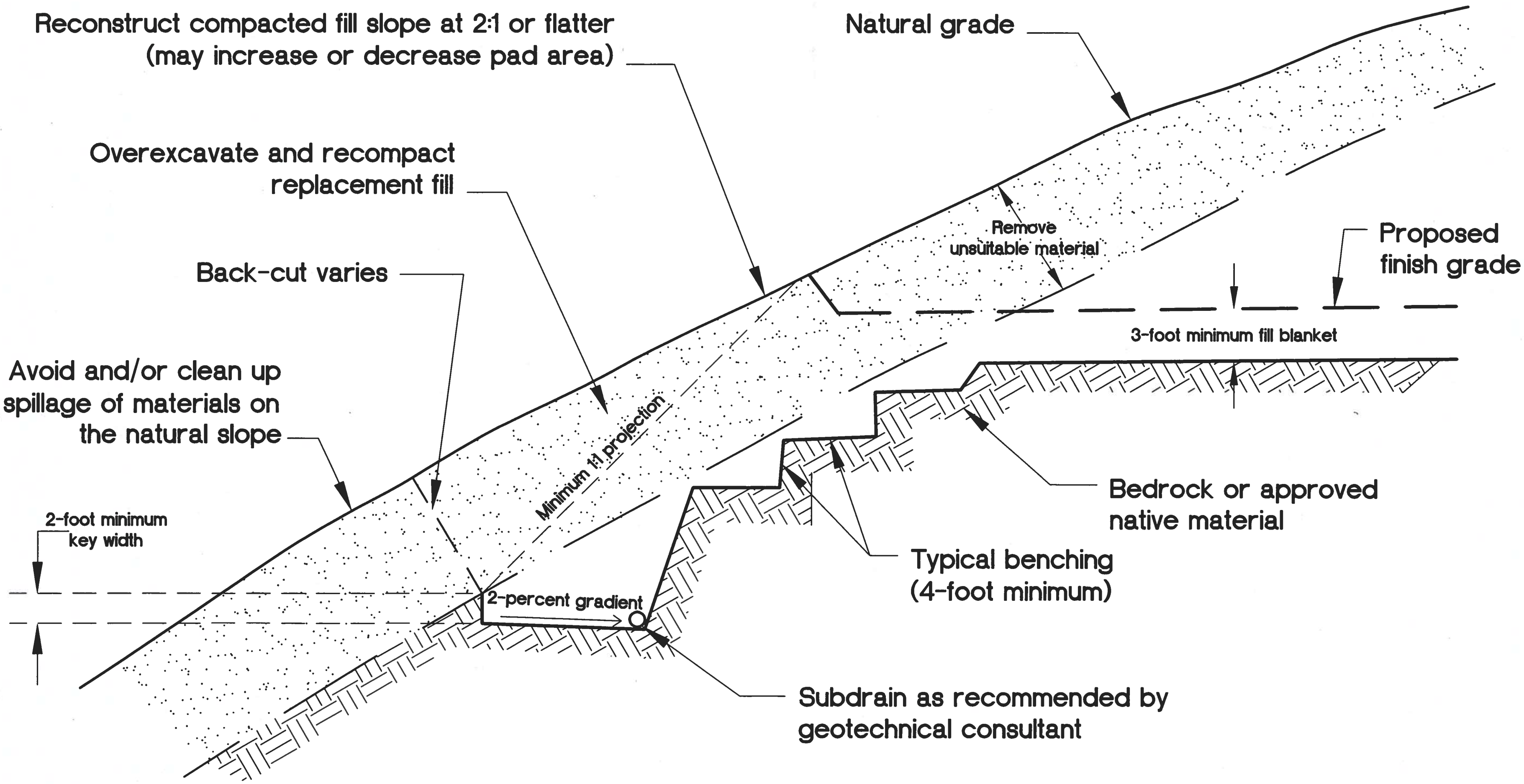
NOTES:

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 G-10

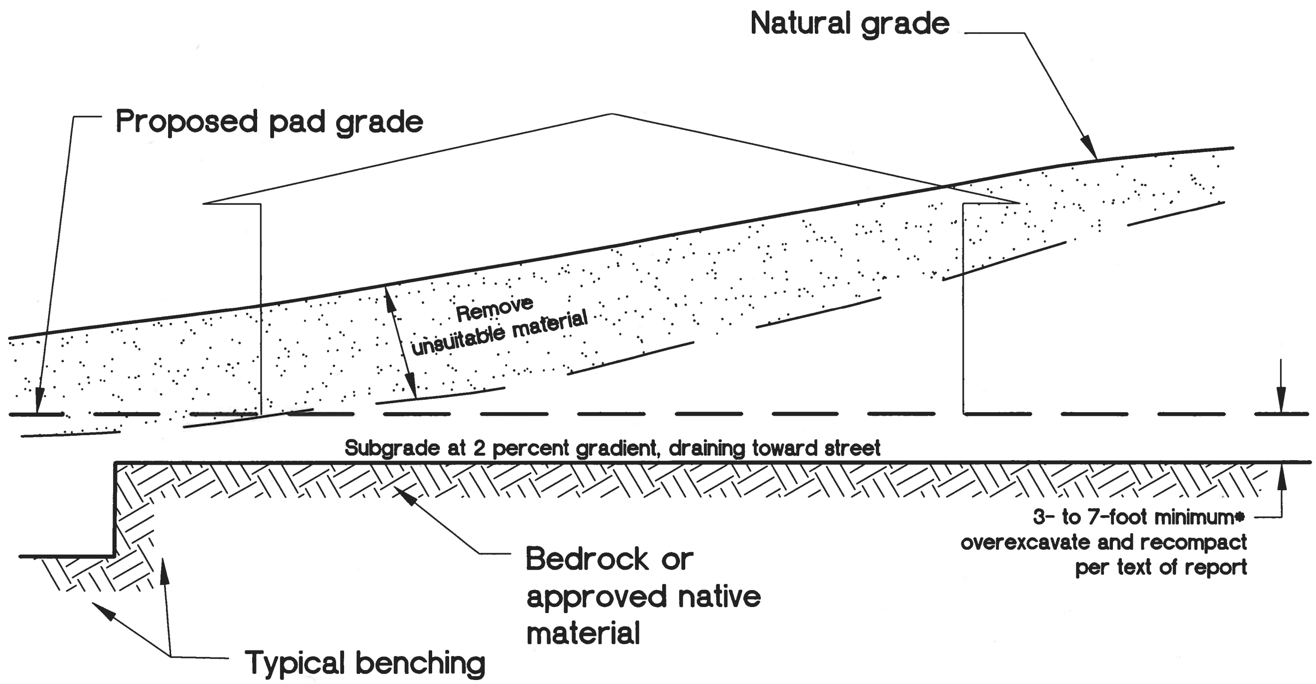


NOTES:

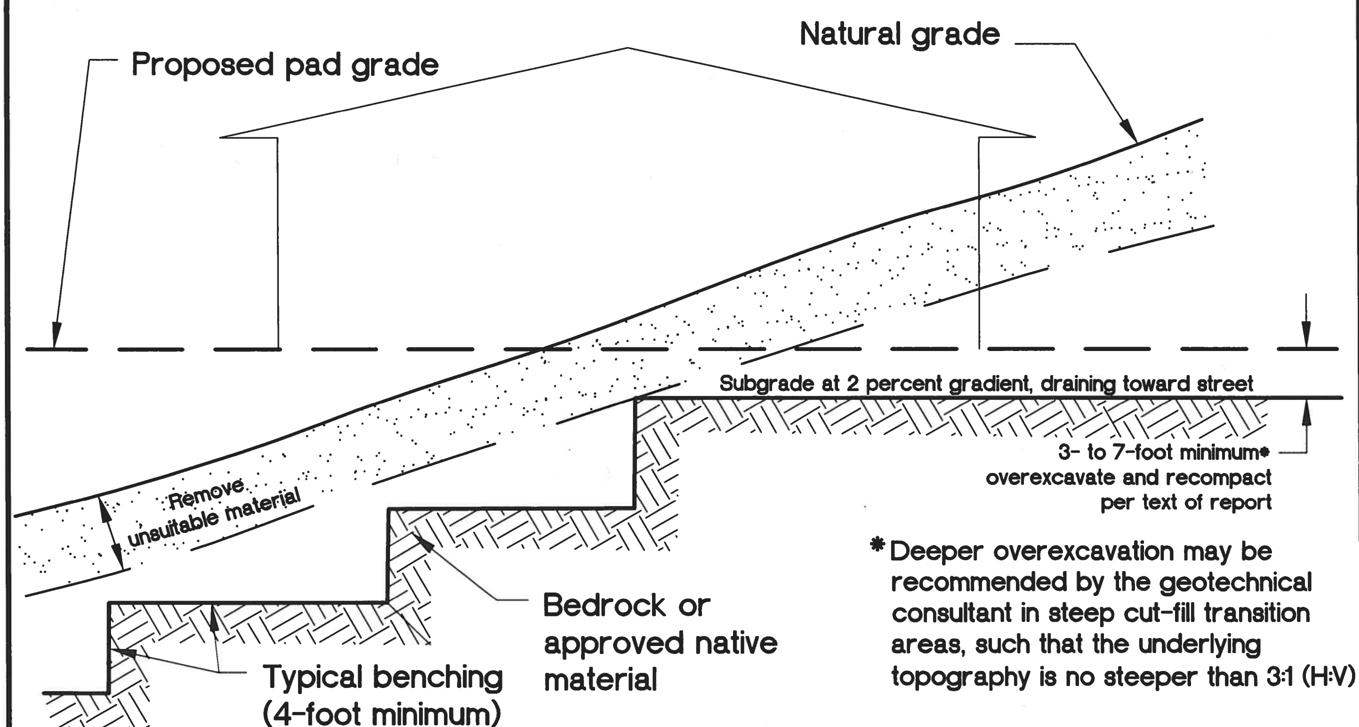
1. Subdrain and key width requirements will be evaluated based on exposed subsurface conditions and thickness of overburden.

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

GeoSoils, Inc.	DAYLIGHT CUT LOT DETAIL	Plate G-11



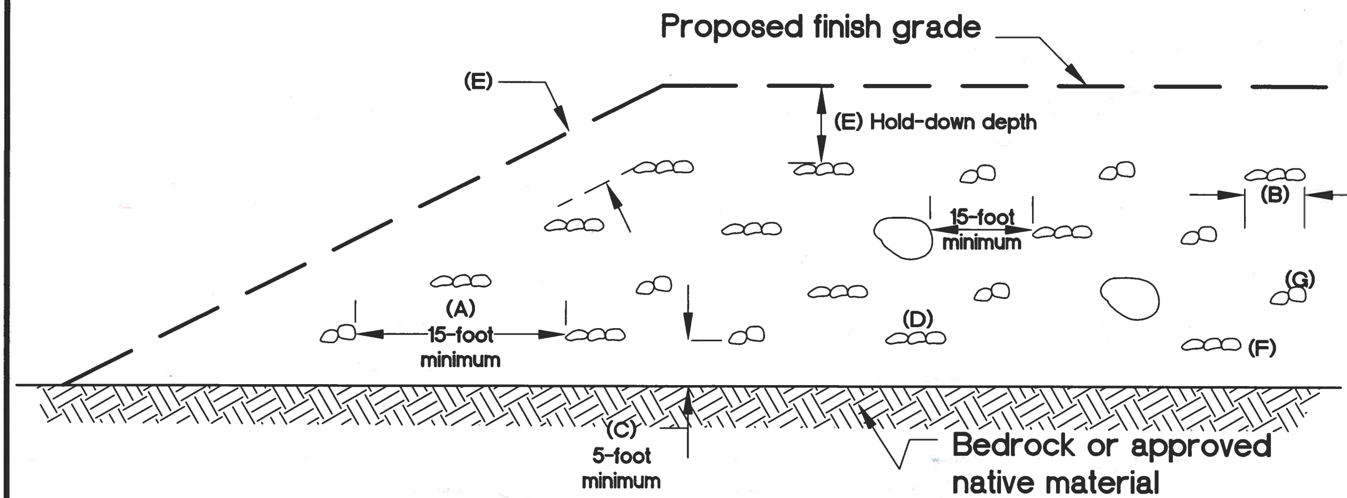
CUT LOT OR MATERIAL-TYPE TRANSITION



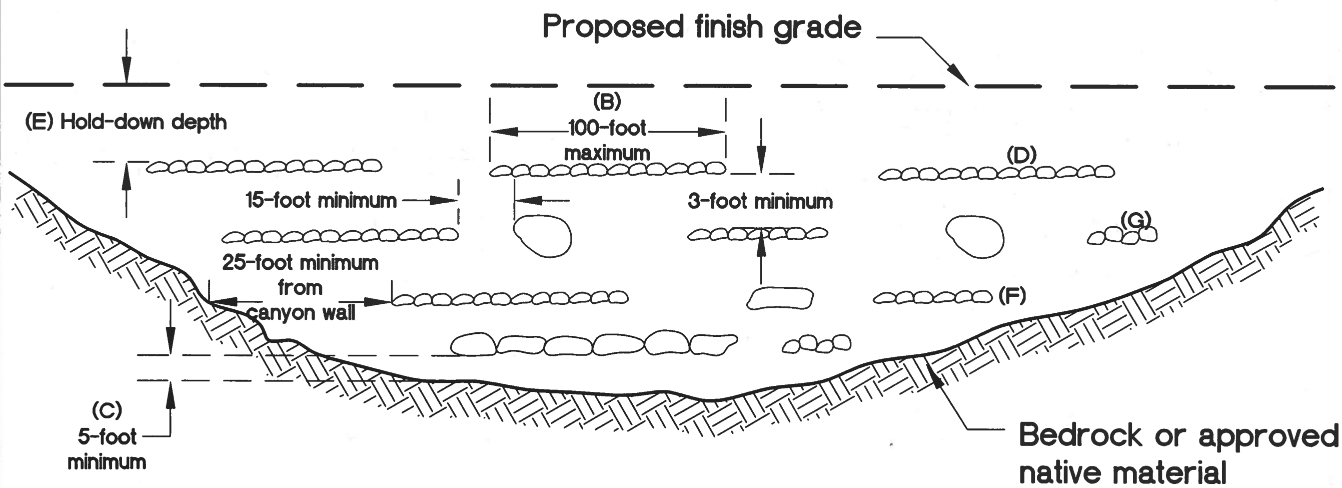
CUT-FILL LOT (DAYLIGHT TRANSITION)

GeoSoils, Inc.	TRANSITION LOT DETAILS	Plate G-12

VIEW NORMAL TO SLOPE FACE



VIEW PARALLEL TO SLOPE FACE



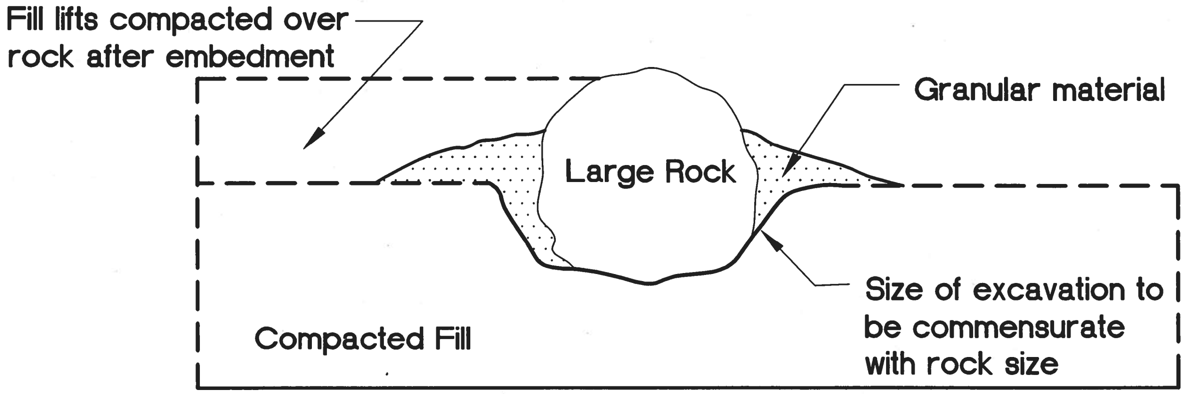
NOTES:

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 leas
ompaction 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.

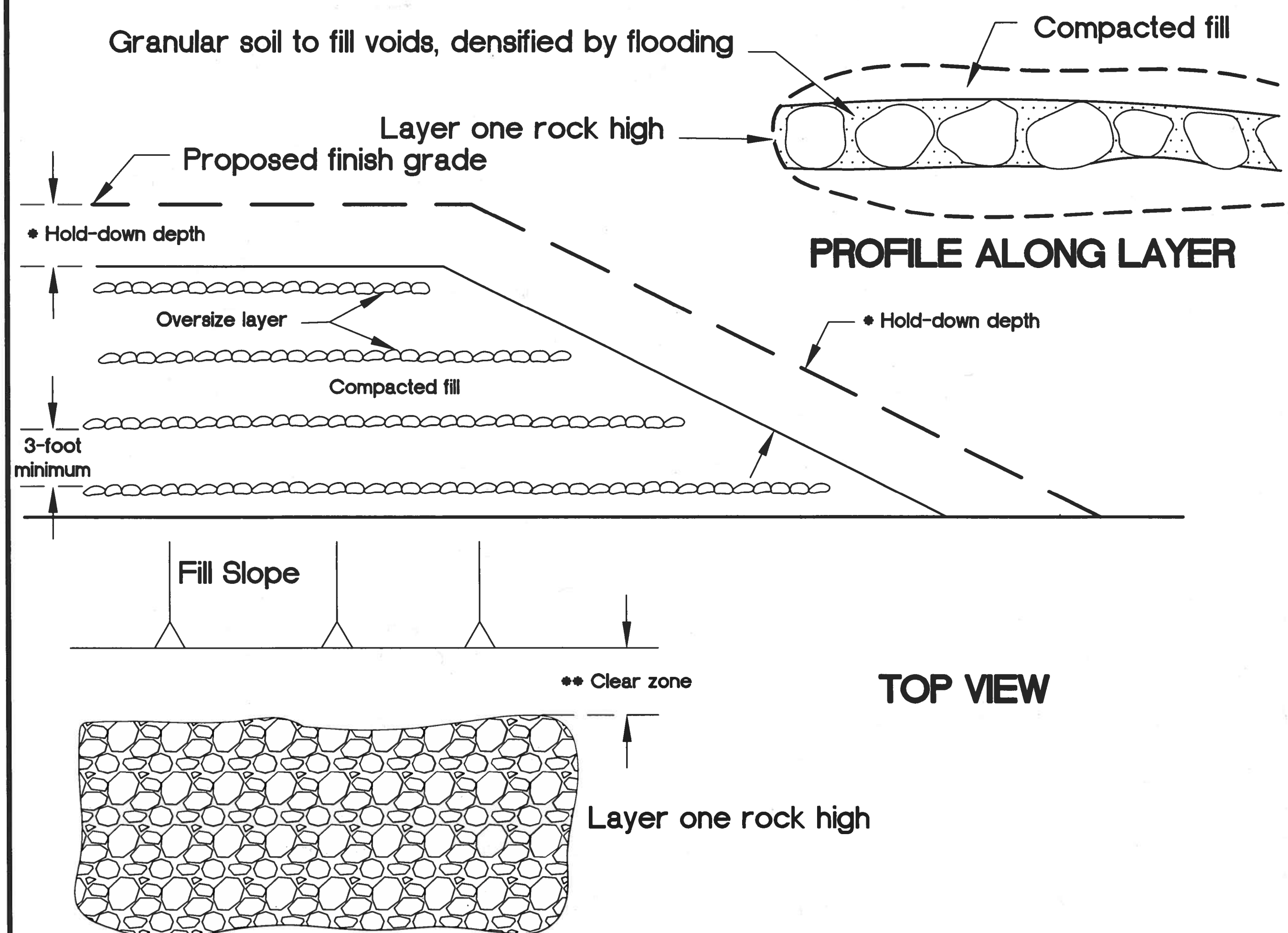
VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE
ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED

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

ROCK DISPOSAL PITS



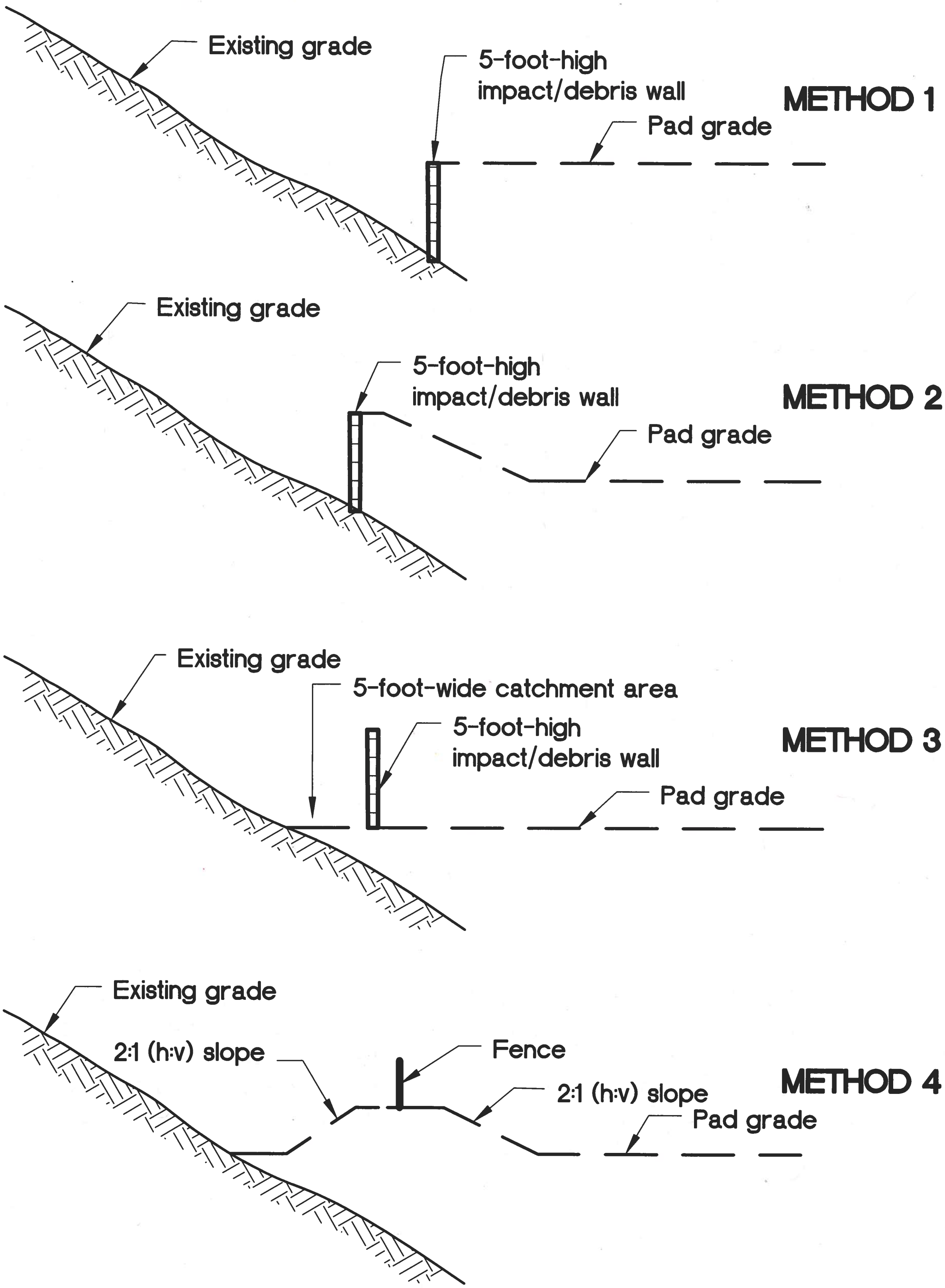
ROCK DISPOSAL LAYERS



* Hold-down depth or below lowest utility as specified in text of report, subject to governing agency approval.
** Clear zone for utility trenches, foundations, and swimming pools, as specified in text of report.

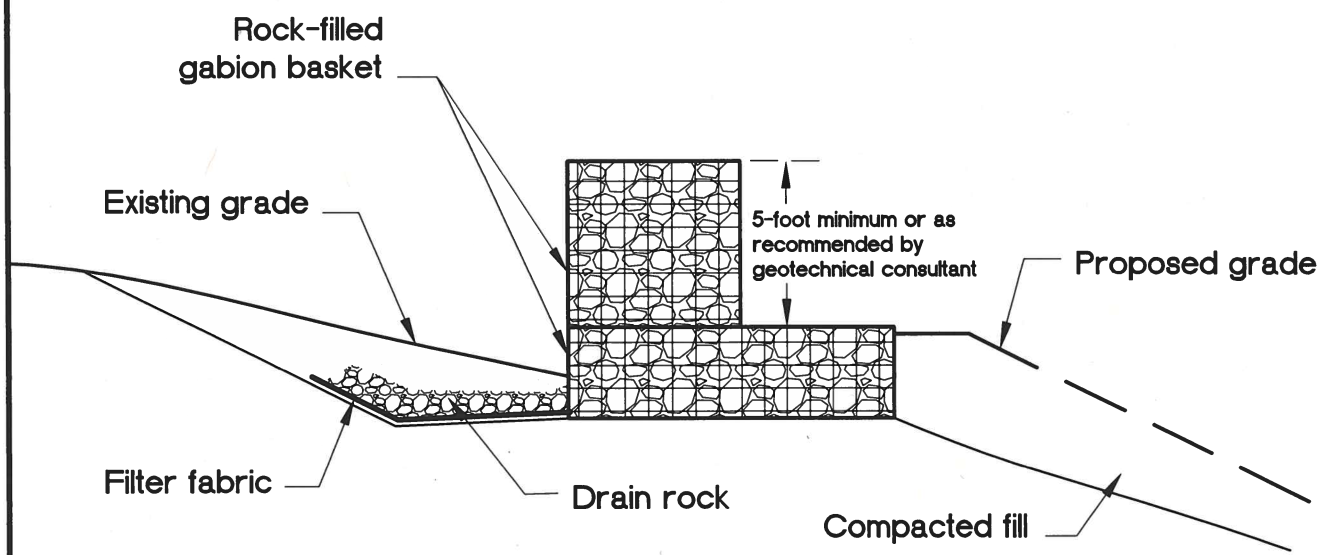
VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE
ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED IN

GeoSoils, Inc.	ROCK DISPOSAL DETAIL	Plate G-14



NOT TO SCALE

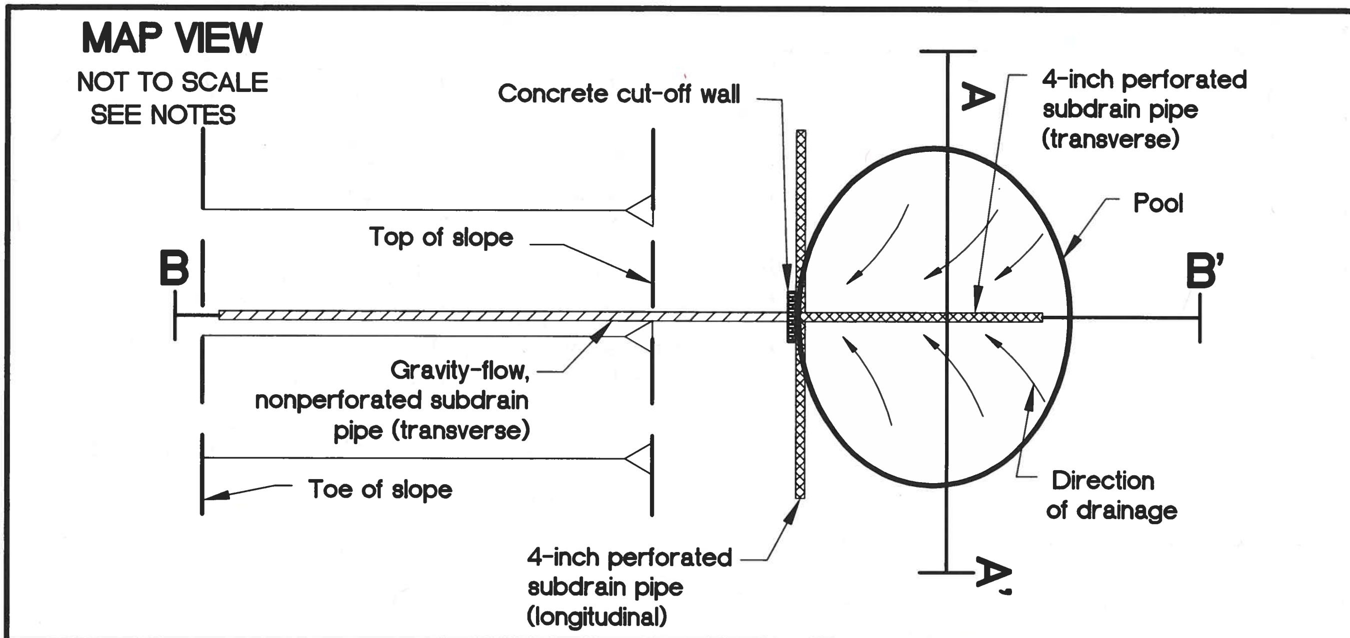
GeoSoils, Inc.	DEBRIS DEVICE CONTROL METHODS DETAIL	Plate G-15

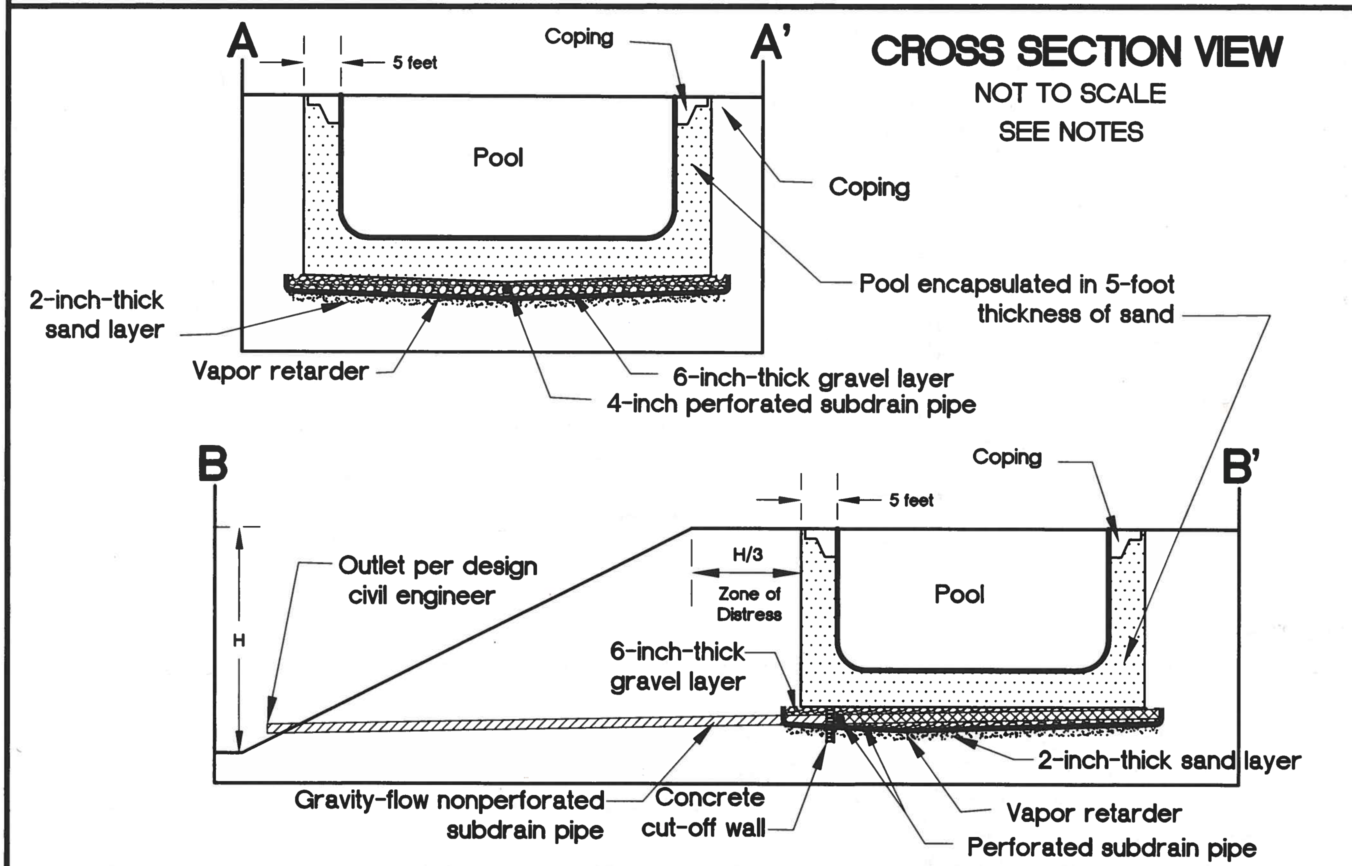


Gabion impact or diversion wall should be constructed at the base of the ascending slope subject to rock fall. Walls need to be constructed with high segments that sustain impact and mitigate potential for overtopping, and low segment that provides channelization of sediments and debris to desired depositional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

From GSA, 1987

GeoSoils, Inc.	ROCK FALL MITIGATION DETAIL	Plate G-16





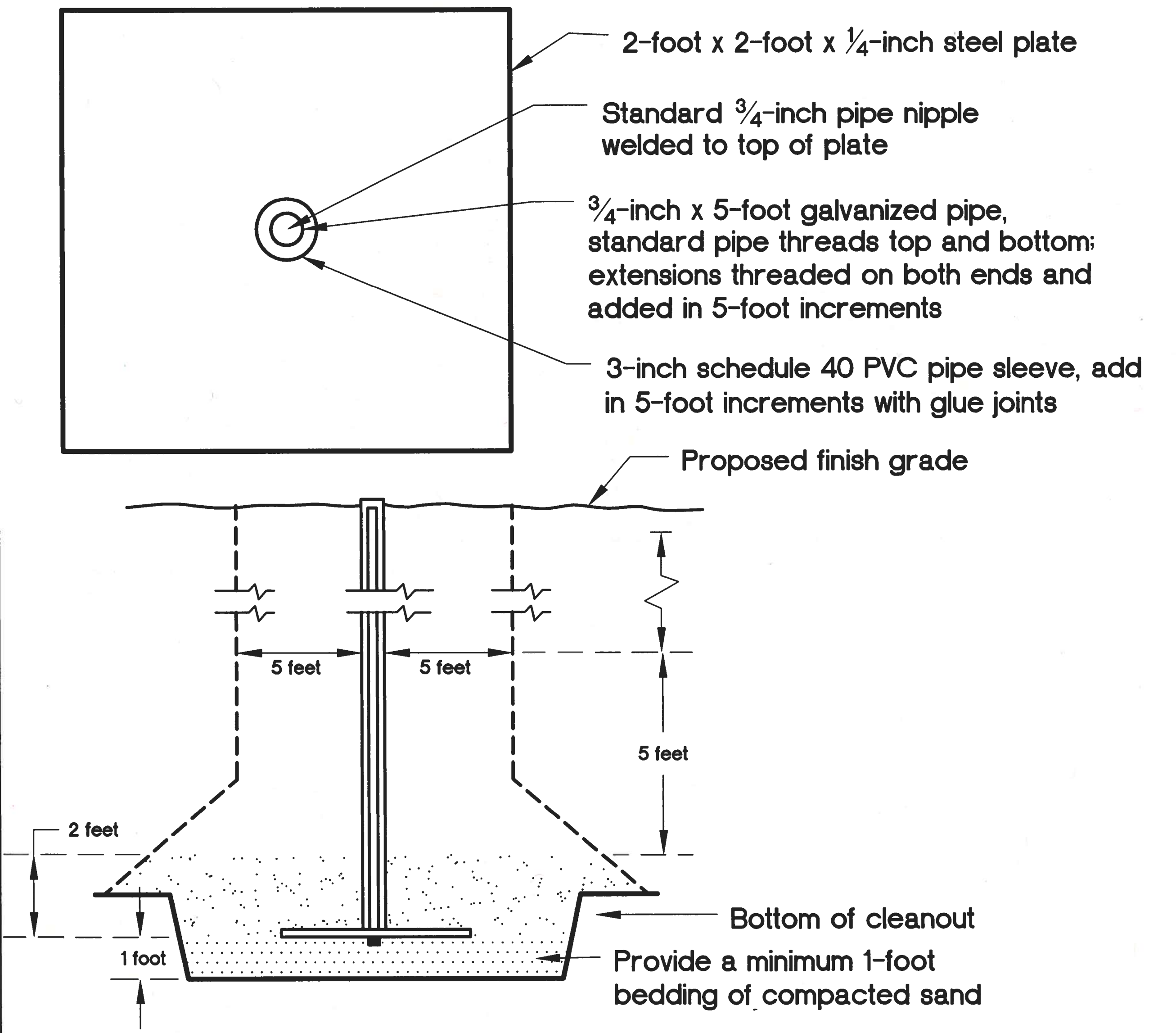
NOTES:

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

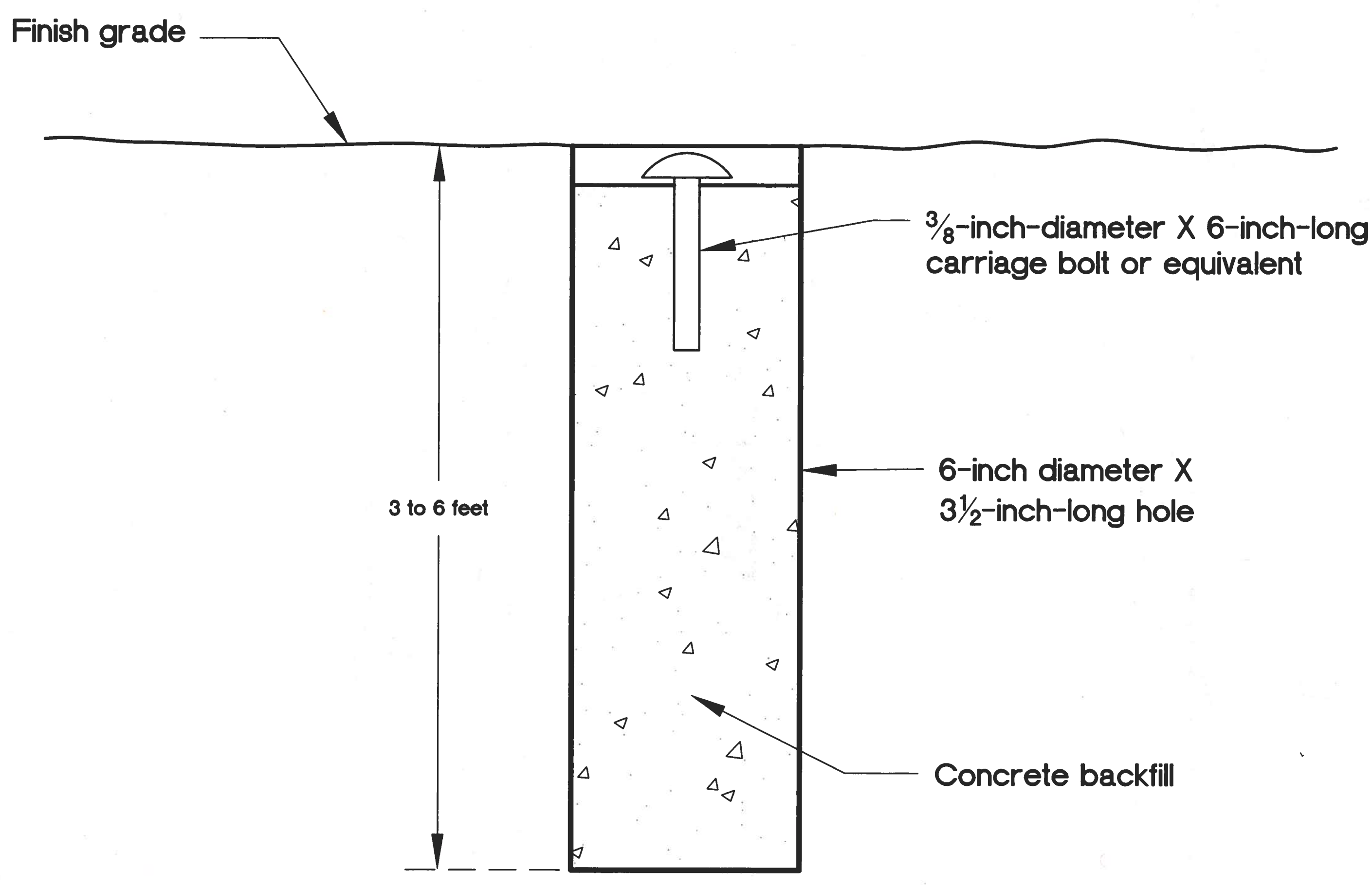
Plate G-17



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.	SETTLEMENT PLATE AND RISER DETAIL	Plate G-18

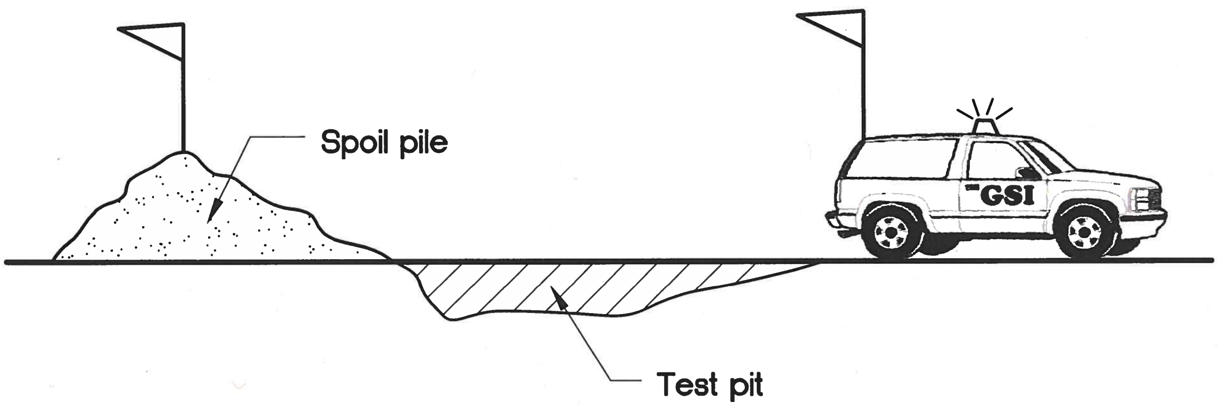


GeoSoils, Inc.

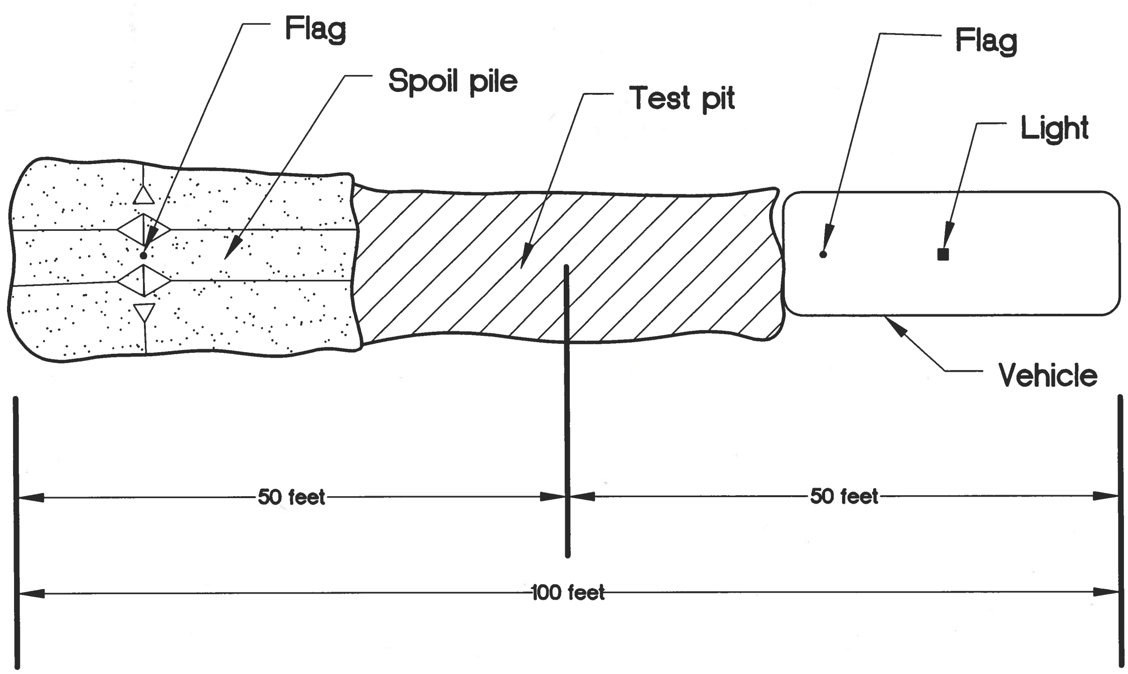
TYPICAL SURFACE SETTLEMENT MONUMENT

Plate G-19

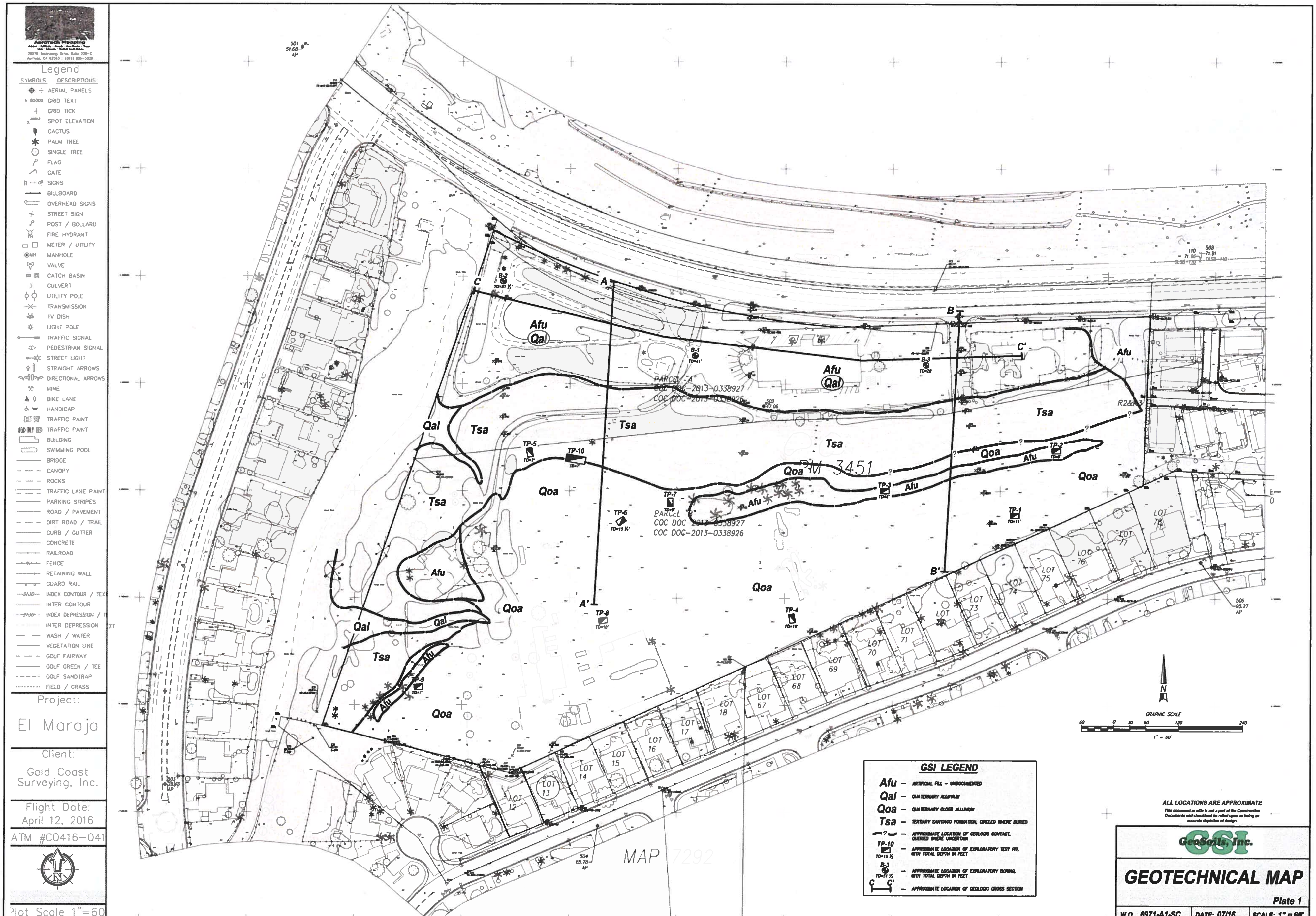
SIDE VIEW



TOP VIEW



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


GEOTECHNICAL MAP
Plate 1
W.O. 6971-A1-SC
DATE: 07/16
SCALE: 1" = 60'
GeoSoils, Inc.
ALL LOCATIONS ARE APPROXIMATE
This document or efile is not a part of the Construction Documents and should not be relied upon as being an accurate depiction of design.
GSI LEGEND
Afu – ARTIFICIAL FILL - UNDOCUMENTED
Qal – QUATERNARY ALLUVIUM
Qoa – QUATERNARY OLDER ALLUVIUM
Tsa – TERTIARY SANTIAGO FORMATION, CIRCLED WHERE BURIED
APPROXIMATE LOCATION OF GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN
TP-10 TD=15 ½ – APPROXIMATE LOCATION OF EXPLORATORY TEST PIT, WITH TOTAL DEPTH IN FEET
B-3 TD=31 ½ – APPROXIMATE LOCATION OF EXPLORATORY BORING, WITH TOTAL DEPTH IN FEET
C C' – APPROXIMATE LOCATION OF GEOLOGIC CROSS SECTION
GRAPHIC SCALE
1" = 60'
AeroTech Mapping
Legend
SYMBOLS DESCRIPTIONS
AERIAL PANELS
GRID TEXT
GRID TICK
SPOT ELEVATION
CACTUS
PALM TREE
SINGLE TREE
FLAG
GATE
SIGNS
BILLBOARD
OVERHEAD SIGNS
STREET SIGN
POST / BOLLARD
FIRE HYDRANT
METER / UTILITY
MANHOLE
VALVE
CATCH BASIN
CULVERT
UTILITY POLE
TRANSMISSION
TV DISH
LIGHT POLE
TRAFFIC SIGNAL
PEDESTRIAN SIGNAL
STREET LIGHT
STRAIGHT ARROWS
DIRECTIONAL ARROWS
MINE
BIKE LANE
HANDICAP
TRAFFIC PAINT
TRAFFIC PAINT
BUILDING
SWIMMING POOL
BRIDGE
CANOPY
ROCKS
TRAFFIC LANE PAINT
PARKING STRIPES
ROAD / PAVEMENT
DIRT ROAD / TRAIL
CURB / GUTTER
CONCRETE
RAILROAD
FENCE
RETAINING WALL
GUARD RAIL
INDEX CONTOUR / TEXT
INTER CONTOUR
INDEX DEPRESSION / TEXT
INTER DEPRESSION
WASH / WATER
VEGETATION LINE
GOLF FAIRWAY
GOLF GREEN / TEE
GOLF SANDTRAP
FIELD / GRASS
Project:
El Maraja
Client:
Gold Coast Surveying, Inc.
Flight Date:
April 12, 2016
ATM #C0416-041
Plot Scale 1"=60
PM 3451
MAP 7292
PARCEL
COC DOC-2013-0338927
COC DOC-2013-0338926
A
A'
B
B'
C
C'
TP-1
TP-2
TP-3
TP-4
TP-5
TP-6
TP-7
TP-8
TP-9
TP-10
B-1
B-2
B-3
LOT 12
LOT 13
LOT 14
LOT 15
LOT 16
LOT 17
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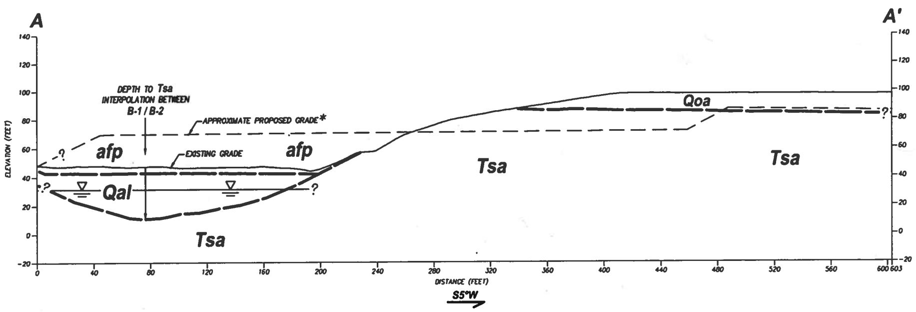
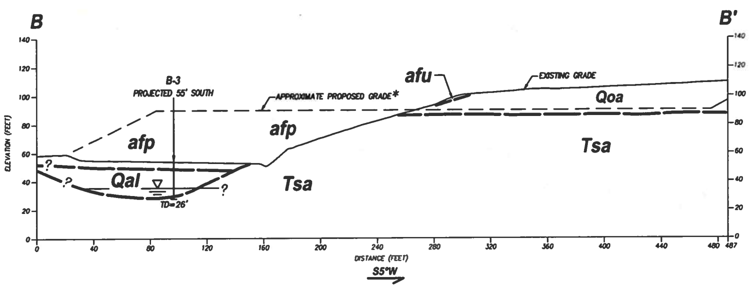
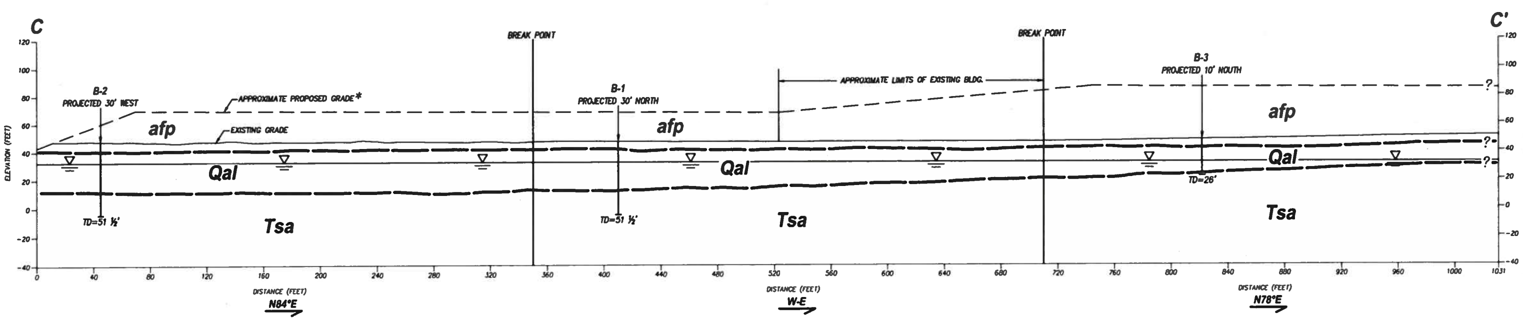


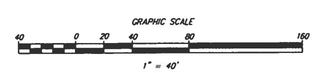




* NOTE: APPROXIMATE PROPOSED GRADE BASED ON INTERPRETATION OF CONCEPTUAL SITE PLAN v.5 BY SWA(2015)

GSI LEGEND

- **afp** – ARTIFICIAL FILL – PROPOSED *
- **afu** – ARTIFICIAL FILL – UNDOCUMENTED
- **Qal** – QUATERNARY ALLUVIUM
- **Qoa** – QUATERNARY OLDER ALLUVIUM
- **Tsa** – TERTIARY SANTIAGO FORMATION
- **—?—** – APPROXIMATE LOCATION OF GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN
- **▽** – GROUNDWATER SURFACE



ALL LOCATIONS ARE APPROXIMATE

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GSI GeoSoils, Inc.

GEOLOGIC CROSS SECTIONS A-A', B-B', C-C'

Plate 2

W.O. 6971-A1-SC	DATE: 07/16	SCALE: 1" = 40'