GEOTECHNICAL EVALUATION
FOR THE VISTA AZUL RESIDENTIAL DEVELOPMENT
PARCEL 150-0003, TROY STREET
CITY OF LEMON GROVE, CALIFORNIA

FOR

VISTA AZUL, LLC
8109 SANTA LUZ VILLAGE GREEN SOUTH
SAN DIEGO, CALIFORNIA 92127

W.O. 6947-A-SC OCTOBER 2, 2015

GeoSoils, Inc.

Geotechnical • Geologic • Coastal • Environmental

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October 2, 2015

W.O. 6947-A-SC

Vista Azul, LLC
8109 Santaluz Village Green South
San Diego, California 92127

Attention: Mr. Chris Dahrling

Subject: Geotechnical Evaluation for the Vista Azul Residential Development, Parcel 150-0003, Troy Street, City of Lemon Grove, California

Dear Mr. Dahrling:

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 residential 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:

- The site consists of a previously graded pad with graded slopes below the east and north side of the pad. This phase of site grading appears to have coincided with the construction of the adjacent 125 freeway.

- Proposed development generally consists of preparing the site (i.e., existing building pad area) for the construction of 21 residential units, as well as associated underground and street improvements, typical exterior hardscape, and landscaping.

- The site appears to be underlain with Eocene-age sedimentary bedrock, consisting of interbedded sandstone and claystone belonging to the Mission Valley Formation. Regional and onsite mapping indicates bedding structure dipping gently to the southwest, on the order of 3 to 5 degrees.

- Due to the relatively compressible nature of undocumented fill, colluvium, and weathered bedrock, these materials are considered unsuitable for the support of settlement-sensitive improvements (i.e., residential foundations, concrete slab-on-grade floors, site walls, exterior hardscape, etc.) and/or engineered fill in their existing state. As such, it is recommended that these materials are removed, moisture conditioned and recompacted, prior to foundation and improvements construction. On a preliminary basis, localized zones of highly weathered formation may or may not remain in place, with some surficial processing, but this should be further evaluated during grading. Removal depths are estimated at 2 to 7 feet, with variation.

- Thin, surficial deposits of colluvium and undocumented fill overlie bedrock locally, within the south and southwestern portions of the site. All undocumented fill, colluvium and any weathered portions of the bedrock are typically considered unsuitable for support of settlement-sensitive improvements and planned fills (if any) in their existing state.

- Our evaluation did not encounter the regional groundwater table to the depths explore, or observed any evidence of a shallow water table (i.e., seeps, springs, phreatophytes, etc.). However, a perched water table may develop locally, along sandstone/claystone contacts, or the contact between fill and the underlying bedrock. Provided that the recommendations contained in our forthcoming geotechnical evaluation are followed, regional groundwater is not expected to be a major factor in development of the site.

- The 2013 California Building Code ([2013 CBC], California Building Standards Commission [CBSC], 2013) indicates that removals of unsuitable soils be performed across all areas to be graded, under the purview of the grading permit, not just within the influence of the residential structure. Relatively deep removals may also necessitate a special zone of consideration, on perimeter/confining areas. This zone would be approximately equal to the depth of removals, if removals cannot be performed onsite or offsite. Thus, any settlement-sensitive improvements (walls, curbs, flatwork, etc.), constructed within this zone may require deepened foundations, reinforcement, etc., or will retain some potential for settlement and associated distress. This will also require proper disclosure to any owners and all interested/affected parties should this condition exist at the conclusion of grading.

- Expansion Index (E.I.), and plasticity index (P.I.) testing performed on a representative sample of the onsite soil indicates E.I.s ranging from less than 20 (very low expansive) to 55 (medium expansive), and a Plasticity Index (P.I.) of up to 23. As such, some site soil (primarily undocumented fill, colluvium, highly weathered and claystone formation) meet the criteria of detrimentally expansive soils as defined in Section 1803.5.2 of the 2013 CBC. Soil expansivity should be re-evaluated at the conclusion of grading and provide updated data for final foundation design.

- Site soils are mildly alkaline (pH), corrosive to exposed buried metals when saturated, present negligible sulfate exposure to concrete and are slightly elevated with respect to chloride exposure. Corrosion testing at the completion of grading is recommended in order to obtain actual corrosion data specific to as-graded conditions.

- Foundation design and construction will need to consider the expansive soil conditions evaluated onsite, in accordance with minimum Code requirements for detrimentally expansive soils.

- Infiltration testing performed on representative site soils has yielded an infiltration rate of 0.047 in/hr for the most limiting soil layer (clay, or claystone) onsite. The Hydrologic Soil Group for this site is "D."

- Our evaluation indicates there are no known active faults crossing the site and the natural slope upon which the site is located has very low susceptibility to deep-seated landslides. Owing to the depth to groundwater and the dense nature of the underlying formational soils, the potential for the site to be adversely affected by liquefaction is considered very low. Site soils are considered erosive. Thus, properly designed site drainage is necessary in reducing erosion damage to the planned improvements.

- The seismic acceleration values and design parameters provided herein should be considered during the design of the proposed development. The adverse effects of seismic shaking on the structure(s) will likely be wall cracks, some foundation/slab distress, and some seismic settlement. However, it is anticipated that the structure will be repairable in the event of the design seismic event. This potential should be disclosed to any owners and all interested/affected parties.

- Additional adverse geologic features that would preclude project feasibility were not encountered, based on the available data.

- 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.

Robert G. Crisman
Engineering Geologist, CEG 1934

David W. Skelly
Civil Engineer, RCE 47857

RGC/JPF/DWS/jh

Distribution: (4) Addressee (2 wet signed)

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SCOPE OF SERVICES

The scope of our services has included the following:

1. Review of readily available published literature, 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 ten (10) exploratory test pit excavations with a rubber tire backhoe, and five (5) exploratory excavations with hand equipment, in order 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 representative soil samples collected during our geologic mapping and subsurface exploration program.

5. Analysis of field and laboratory data relative to the proposed development.

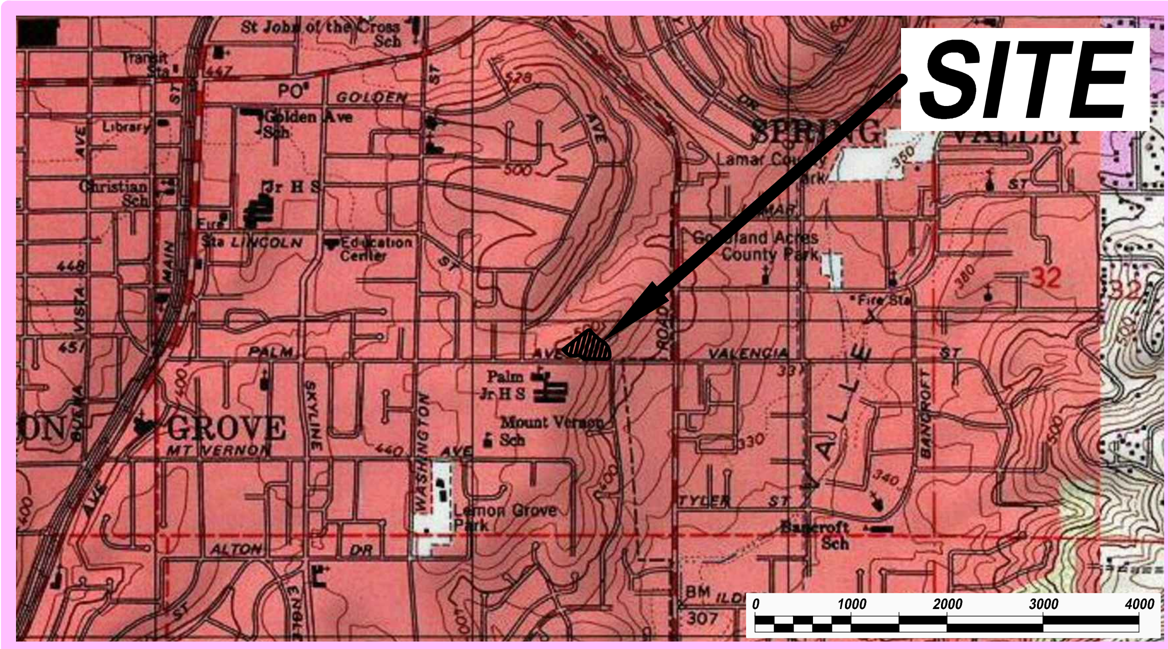
6. Slope stability evaluation (see Appendix D).

7. Completion of storm water infiltration testing and infiltration rate evaluation.

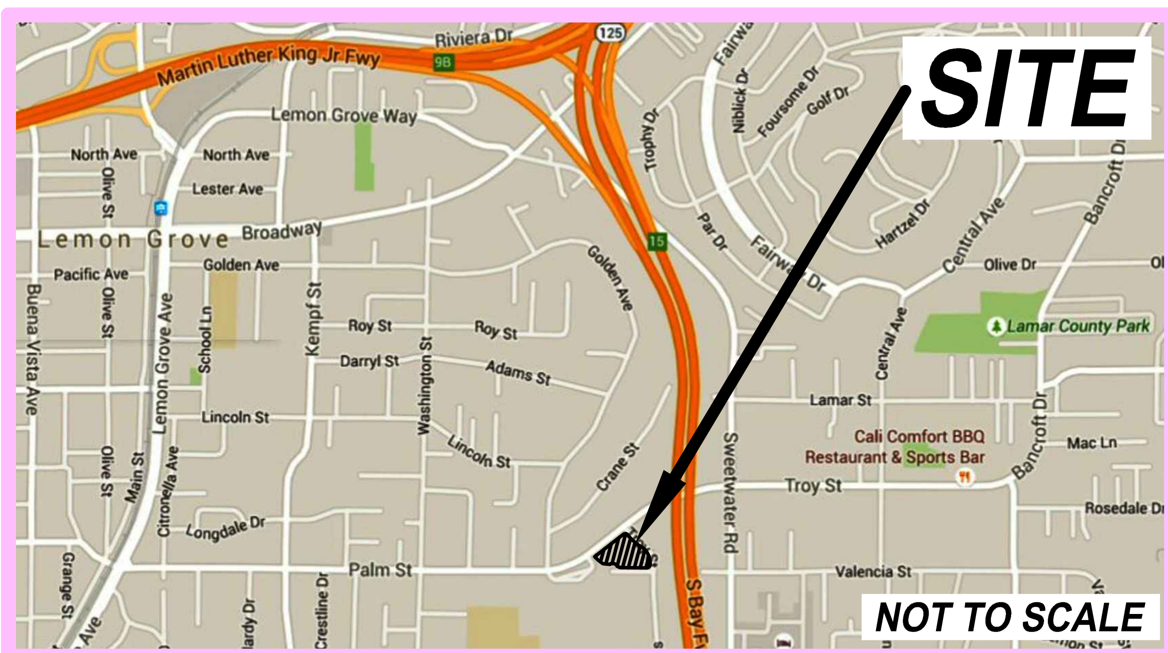
8. Appropriate engineering and geologic analyses of data collected, and the preparation of this geotechnical report and accompaniments.

SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The subject site consists of an irregular shaped, 2¼ acre property bounded by Troy Street to the northwest, Camino De Las Palmas to the northeast, and Palm Street on the south, in the City of Lemon Grove, California (see Figure 1, Site Location Map). Access to the site is from the Palm Street cul du sac. Topographically, the property consists of a relatively flay lying, central, or "pad" area, bounded almost entirely by descending slopes that vary in gradient from as steep as 2:1 (horizontal:vertical [h:v]) above Troy Street and Camino De Las Palmas, to gentler slopes at gradients on the order of 3:1 to 4:1 (h:v) above the southernmost frontage of Camino De Las Palmas, and along Palm Street. Existing slope heights generally vary up to about 30 feet, with the tallest slopes near the intersection of Troy Street and Camino De Las Palmas.



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



Base Map: Google Maps, Copyright 2015 Google, Map Data Copyright 2015 Google

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SITE LOCATION MAP

Figure 1

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Elevations across the site range from about 440 feet Mean Sea Level (MSL) near the southeastern corner of the site, to about 495 feet MSL near the Palm Street Cul Du Sac. Site drainage appears to be collected within a shallow depression located within the pad area, with sheet flow from slows directed offsite to the east and north. Vegetation onsite consists predominantly of grasses, weeds, and scattered shrubs. Existing site conditions are shown on the preliminary grading plan, prepared by Landmark Engineering (LE, 2015) This plan has been adapted for use as a geotechnical map for this report (Plate 1).

A review of Google Earth Imagery indicates that at least four (4) residential structures occupied the site as recent as 1996, including a former alignment of Palm Street. Between 1996 and 2002, it appears that the structures were removed, and the site graded to is current configuration. Based on other grading visible in the area, the removal of the structures and road realignment/removal onsite may have been associated with the construction of the adjacent 125 freeway. Of the previous development onsite, only two (2) power poles, likely located along the former alignment of Palm Street, remain (see Plate 1).

Existing improvements to the property consist of the aforementioned power poles, graded cut slopes above Troy Street and the north portion of Camino De Las Palmas, the existing pad area, and a fill embankment located adjacent to the southeast portion of the existing pad area.

It is our understanding that proposed development will consist of preparing the site for the construction of 21 residential structures, with associated improvements, such as underground utilities, retaining walls, landscaping, and driveway access to Palm Street. Cut and fill grading techniques are anticipated to be used to construct proposed grades. A review of LE (2015) indicates plan cuts and fills on the order of up to 16, and 12 feet, respectively. Graded slopes are planned up to about 28 feet in height, at gradients of 2:1 (h:v) or flatter. Plans also indicate that the existing cut slope above Troy Street will remain, albeit, lowered in height.

GSI anticipates that the proposed structures will be one- to two-stories and consist of wood frame and/or masonry block construction. GSI also anticipates that the proposed structures would utilize typical foundations with concrete slab-on-grade floors. Building loads are assumed to be typical for this type of relatively light residential/commercial construction. Sewage disposal is anticipated to be accommodated by tying into the regional municipal system. The need for import soils is currently unknown. LE (2015) indicates that a large detention structure will be located within the southeast corner of the site. Existing topography and planned construction is shown on Plate 1.

FIELD STUDIES

Site-specific field studies were conducted by GSI during June, 2015, and consisted of reconnaissance geologic mapping and the excavation of three (3) exploratory test pit excavations with a rubber tire backhoe, for an evaluation of near-surface soil and geologic conditions onsite. The test excavations were logged by a representative of this office who collected representative bulk and undisturbed soil samples for appropriate laboratory testing. The logs of the test excavations are presented in Appendix B. The approximate location of the test excavations are presented on the Geotechnical Map (see Plate 1).

REGIONAL GEOLOGY

The subject property lies within the coastal plain physiographic region of the Peninsular Ranges Geomorphic Province of southern California. This coastal 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 (1977, 2008) indicate the site is underlain by Tertiary (Eocene)-age sedimentary bedrock, belonging to the Mission Valley Formation.

SITE GEOLOGIC UNITS

General

The earth material units that were observed and/or encountered at the subject site consist of discontinuous surficial deposits of undocumented artificial fill, and colluvium, overlying Eocene-age sedimentary bedrock belonging to the Mission Valley Formation. A general description of each material type is presented as follows, from youngest to oldest. The general distribution of earth materials is shown on Plate 1 and in cross section on Plate 2.

Undocumented Fill (Map Symbol - afu)

An existing embankment of undocumented fill, up to approximately 8 feet in thickness where encountered, is located within the east central portion of the site, with undocumented fills, on the order of 1 foot, or less, in thickness, placed across existing cut slopes (for planting purposes?) above Troy Street and Camino De Las Palmas. Where encountered, undocumented fill generally consists of dark grayish brown to dark brown sandy clay and clayey sand. Undocumented fills were typically observed to be dry near the surface, becoming slightly moist with depth, loose (clayey sand)/firm (sandy clay), desiccated and burrowed near the surface. Existing undocumented fill 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.

Colluvium (Not Mapped)

Colluvial soils were encountered as a relatively thin, near surface, or surficial layer of sandy clay on the order of ½ to 2½ feet thick, and primarily located within the southern and southeastern portions of the site. Colluvium is typically brown to very dark brown, dry to slightly moist, firm to stiff, and desiccated. Existing deposits of colluvium are considered potentially compressible in its existing state. As such, colluvium should not be used for the support of settlement-sensitive improvements and/or any planned fill, unless adequately remediated.

Mission Valley Formation (Map Symbol - Tmv)

Eocene-age sedimentary bedrock occurs at the surface, and/or beneath existing undocumented fill and colluvium, at depths ranging from about 1 to 6 feet beneath surface grades. A zone of highly weathered bedrock, on the order of 1½ to 3½ feet thick, was observed locally, within the southern and southeastern portion of the site. This highly weathered bedrock zone consists of red brown, brownish gray, light gray/olive brown, dry and desiccated sandy claystone, claystone, and sandstone, characterized by either bioturbation (burrowing), an abundance of caliche (calcium carbonates), and/or dessication cracking. As observed, relatively unweathered "bedrock" deposits generally consist of interlayered olive brown, brown, and red brown claystone, and grayish brown to light gray sandstone. Claystones are typically moist and very stiff, while sandstones are typically dry to slightly moist and medium dense to dense. The general distribution of sandstone and claystone facies onsite are shown on Plate 1.

The zone of highly weathered bedrock 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. The underlying, less weathered sedimentary bedrock is considered suitable for the support of settlement-sensitive improvements and/or planned fills.

Onsite, and regional mapping (Kennedy and Tan, 1977, 2008) indicate gentle, southwesterly dipping bedding attitudes, on the order of 2 to 5 degrees, with local, northwesterly dipping cross bedding noted onsite. Fractures were noted to be relatively high angle. No significant adverse structures (within the formation) were observed onsite.

GROUNDWATER

GSI did not observe evidence of a regional groundwater table nor perched water within our subsurface explorations. Regional groundwater is anticipated to occur at depths greater than 50 feet below the site and is not anticipated to significantly affect proposed site development, provided that the recommendations contained in this report are properly incorporated into final design and construction. These observations reflect site conditions at the time of our investigation and do not preclude future changes in local groundwater conditions from excessive irrigation, precipitation, or that were not obvious, at the time of our investigation.

Seeps, springs, or other indications of subsurface water were not noted on the subject property during the time of our field investigation. However, perched water seepage may occur locally (as the result of heavy precipitation and/or irrigation, or damaged wet utilities) along zones of contrasting permeabilities/densities (fill/bedrock contacts, sandy/clayey fill lifts, etc.) or along geologic discontinuities (joints, fractures). This potential should be anticipated and disclosed to all interested/affected parties.

Due to the potential for post-development perched water to manifest near the surface, owing to as-graded permeability/density contrasts, more onerous slab design is necessary for any new slab-on-grade floor (State of California, 2015). 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.

GEOLOGIC HAZARDS EVALUATION

Mass Wasting/Landslide Susceptibility

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

According to regional landslide susceptibility mapping by Tan and Giffen (1995) the site is generally characterized as being "marginally susceptible" to landsliding. This characterization is likely due to the presence of formational materials that are traditionally associated with abundant expansive clay material, such as portions of the Mission Valley Formation. However, geomorphic expressions indicative of past mass wasting events (i.e., scarps and hummocky terrain) were not observed on the property during our field studies nor our review of regional geologic mapping. Further, no adverse geologic structures were encountered during our subsurface exploration, and regional geologic maps (Tan & Kennedy; 1977, 2008) do not indicate the presence of landslides on the property. Given the absence of adverse geologic structure, the dense nature of the underlying bedrock, and the lack of evidence with respect to existing slope instability, the potential for deep seated landslides to affect the proposed site development is considered low.

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, part of the Newport-Inglewood - Rose Canyon fault zone, is the closest known active fault to the site (located at a distance of approximately 8.8 miles [14.1 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 a few 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.

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 0.45g. 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 (mean plus 1 sigma) during the period 1800 through January 2015 was about 0.17g. 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.

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	I, II, III	Table 1604.5
Site Class	D	Section 1613.3.2/ASCE 7-10 (p. 203-205)
Spectral Response - (0.2 sec), S_s	0.878	Section 1613.3.1 Figure 1613.3.1(1)
Spectral Response - (1 sec), S_1	0.338	Section 1613.3.1 Figure 1613.3.1(2)
Site Coefficient, F_a	1.149	Table 1613.3.3(1)
Site Coefficient, F_v	1.724	Table 1613.3.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec), S_{MS}	1.008	Section 1613.3.3 (Eqn 16-37)

2013 CBC SEISMIC DESIGN PARAMETERS		
PARAMETER	**VALUE**	**2013 CBC/ASCE REFERENCE**
Maximum Considered Earthquake Spectral Response Acceleration (1 sec), S_{M1}	0.583	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	0.672	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.389	Section 1613.3.4 (Eqn 16-40)
Seismic Design Category	D	Section 1613.3.5/ASCE 7-10 (Table 11.6-1 or 11.6-2)
PGA_M	0.396 g	ASCE 7-10 (Eqn 11.8.1)

GENERAL SEISMIC PARAMETERS	
Distance to Seismic Source (Rose Canyon) “B” fault[1]	8.8 mi (14.1 km)[2]
Upper Bound Earthquake (Rose Canyon) “B” fault[1]	M_W = 7.2[1]
[1] - Cao, et al. (2003). [2] - From Blake (2000a)	

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

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:

- Liquefaction
- Lateral Spreading
- 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.

Expansion Index

Tests were performed on representative soil samples general accordance with ASTM D 4829. Test results and the soils expansion potential are presented in the following table.

SAMPLE LOCATION	DESCRIPTION	EXPANSION INDEX	EXPANSION POTENTIAL
TP-2 @ 2-4 feet	Sandy Clay	55	Medium
TP-2 @ 5-6 feet	Sand	<20	Very Low

Atterberg Limits

Testing of a representative soil sample to evaluate its liquid limit, plastic limit, and plasticity index (P.I.) was performed in general accordance with ASTM D 4318-4318. The test results are presented in following table:

SAMPLE LOCATION	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
TP-2 @ 2-4 feet	44	21	23
TP-2 @ 5-6 feet	--	--	non plastic

Direct Shear Test

Shear testing was performed on a relatively undisturbed sample of site soil in general accordance with ASTM test method D 3080 in a Direct Shear Machine of the strain control type. The shear test results are presented in the following table:

LOCATION AND DEPTH (FEET)	PRIMARY		RESIDUAL	
	COHESION (PSF)	FRICTION ANGLE (DEGREES)	COHESION (PSF)	FRICTION ANGLE (DEGREES)
TP-2 @ 5-6'	1638	50	289	36
TP-3 @ 2'	1154	30	293	36

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 the following table:

SAMPLE LOCATION	pH	SATURATED RESISTIVITY (ohm-cm)	SOLUBLE SULFATES (wt. %)	SOLUBLE CHLORIDE (ppm)
TP-1/TP-2 Composite	7.3	900	0.0035	189

Corrosion Summary

Laboratory testing indicates that tested samples of the onsite soils are mildly alkaline with respect to soil acidity/alkalinity, are corrosive to exposed, buried metals when saturated; present negligible (“not applicable” or “S0" per ACI 318-11) sulfate exposure to concrete; and, chloride levels are slightly elevated. Reinforced concrete mix design for foundations, slab-on-grade floors, and pavements should minimally conform to “Exposure Class C1” in Table 4.3.1 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.

SLOPE STABILITY

GSI performed slope stability analyses along Geologic Cross Sections A-A’, B-B’, in order to evaluate the stability of the existing cut slopes onsite. A third section was evaluated through a planned slope that includes a storm water detention basin. The locations of these cross section are presented in plan view on Plate 1. Geologic interpretations along Sections A-A’ and B-B’ are presented on Plate 2.

Slope stability analyses were performed with the aid of the two-dimensional slope stability computer program "GSTABL7 v.2" developed by Gregory (2003). For a complete discussion on the GSTABL7 program, please refer to Appendix D.

Soil shear strengths, used in the analyses, were obtained from laboratory testing performed in preparation of this report, and are presented in Appendix D. These analyses indicated a static FOS greater than 1.5, and a seismic FOS greater than 1.1.

Based on a review of LE (2015) and Plate 1, slopes with a combined height of up to approximately 32 feet are planned, at gradients of 2:1, or flatter. Assuming proper surface drainage, code-compliant routine and periodic maintenance, and normal rainfall, permanent graded slopes, constructed from the onsite materials, as recommended herein, are considered grossly and surficially stable in the absence of any unlined bioretention basins in close proximity to any slope.

LE (2015) and Plate 1 indicates a large detention basin located within a planned slope at the southeastern portion of the site. Water has been shown to weaken the inherent strength of all earth materials, and slope stability is significantly reduced by overly wet conditions. As such, the current location of bioretention basin shown on LE (2015) will generally increase the potential for slope instability, such as slumps, erosion, and concentrated offsite drainage, or increase the potential for distress to planned retaining walls located around the perimeter of the basin. As such, consideration should be given to redesigning storm water treatment systems to consist of a leak proof, lined system with subdrainage.

The onsite soils are considered erosive. Therefore, graded, and natural slopes comprised of these materials may be subject to rilling, gullying, sloughing, and surficial slope failures depending on rainfall severity and surface drainage. However, such risks can be minimized through properly designed and controlled surface drainage. Very thin “skin” fills placed across the existing cut slopes could become saturated during rain storm s and slump off slope faces. Temporary slopes for construction (i.e., trenching, etc.) are discussed in subsequent sections of our report.

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.
- Potential for perched water during and following site development.
- Slope stability in the presence of unlined basins.
- Temporary slope stability.
- 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.

1. Soil engineering, observation, and testing services should be provided during grading to aid the contractor in removing unsuitable soils and in his effort to compact the fill.

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

3. On a preliminary basis, existing undocumented fills, and colluvium are considered unsuitable for the support of the planned settlement-sensitive improvements (i.e., residential structure, walls, concrete slab-on-grade floors, and exterior pavements, etc.) or new planned fills. As indicated, there is a potential that highly weathered bedrock (formation) may remain in place, with some surficial processing, and this should be further evaluated during grading. Unsuitable soils within the influence of planned settlement-sensitive improvements and/or planned fill should be removed to expose suitable formation and then be reused as properly engineered fill. Based on the available data, remedial grading (removals) are anticipated to vary on the order of 2 to possibly 7 feet, with the deeper removals likely occurring in areas near the top of the existing fill slope along the north side of the existing building pad. Some localized variation should be anticipated (i.e., deeper or shallower removals).

4. Testing performed on representative samples of onsite soils indicates very low to medium expansive soil conditions. On a preliminary basis, specific foundation design to resist expansive soil effects appears to be necessary where detrimentally expansive soils are present (expansion index [E.I.] greater than 20), and the plasticity index (P.I.) is greater than 15. Expansion and plasticity testing should be further evaluated during grading.

5. Laboratory testing indicates that site soils are mildly alkaline (pH) and corrosive to exposed buried metals when saturated. Testing also indicates that site soils present negligible ("not applicable" per ACI 318-11) sulfate exposure to concrete and are slightly elevated for chloride exposure. 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. Additional testing at the completion of remedial grading is recommended in order to verify these assumptions.

6. Site soils are considered erosive. Surface drainage should be designed to eliminate the potential for concentrated flows. Positive surface drainage away from foundations and tops of slopes is recommended. Temporary erosion control measures should be implemented until vegetative covering is well established. The homeowner will need to maintain proper surface drainage over the life of the project.

7. No evidence of a high regional groundwater table nor perched water was observed during our subsurface exploration within the property. However, due to the nature of site earth materials, there is a potential for perched water to occur both during and following site development. This potential should be disclosed to all interested/affected parties. Should perched water conditions be encountered, this office could provide recommendations for mitigation. Typical mitigation includes subdrainage system, cut-off barriers, etc.

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

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

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

EARTHWORK CONSTRUCTION RECOMMENDATIONS

General

All earthwork should conform to the guidelines presented in the 2013 CBC (CBSC, 2013), the requirements of the County, and the General Earthwork and Grading Guidelines presented in Appendix E, 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 pools, 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, and colluvium. Based on our site work, removals depths on the order of approximately 2 to 7 feet should be anticipated locally, however, deeper removals cannot be precluded. 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.

2. On a preliminary basis, the near surface zone of highly weathered bedrock (formation) may be suitable for the support of the planned settlement-sensitive improvements new planned fills, and may likely remain, partially, or fully in-place, although complete removal may not be precluded. This should be further evaluated during grading.

3. Unsuitable soils within the influence of planned settlement-sensitive improvements and/or planned fill should be removed to expose suitable existing fill, or the underlying bedrock and then be reused as properly engineered fill.

4. In addition to removals within the building envelopes, overexcavation of the underlying formational/bedrock soil (if encountered) should be performed in order to provide for at least 3 feet of compacted fill below finish grade. Once removals and overexcavation is completed, the fill should be cleaned of deleterious materials, moisture conditioned, and recompacted to at least 90 percent relative compaction per ASTM D 1557.

5. Subsequent to the above removals/overexcavation, the exposed bottom should be scarified to a depth of at least 8 inches, brought to <u>at least</u> optimum moisture content, and recompacted to a minimum relative compaction of 90 percent of the laboratory standard, prior to any fill placement.

6. Localized deeper removals may be necessary due to buried drainage channel meanders or dry porous materials, septic systems, etc., or deeper sections of the former reservoir that may be present. The project soils engineer/geologist should observe all removal areas during the grading.

Mitigation Of Expansive Soils

Existing site soils range from a very low expansive sand, to highly expansive clay. If a conventional foundation design is desired, all expansive clays, if present within at least 7 feet of finish grade, should be removed and replaced with a very low expansive soil (Expansion Index [E.I.] < 20). Based on site conditions, sandstone bedrock and fill material derived from sandstone bedrock is generally considered suitable for the support of a conventional foundation. Existing undocumented fill, colluvium, highly weathered clayey bedrock (formation) and claystone may be removed and exported from the site and replaced with a select, very low expansive import, if a conventional foundation is desired; however, this may require removal deeper than 7 feet for the replacement fill.

Overexcavation

In order to mitigate the existing cut/fill transition within the building pad(s), and to provide uniform foundation support, existing undocumented fill, colluvium, and possibly weathered bedrock, or bedrock (formation) exposed within 36 inches from pad grade or 24 inches below the lowest foundation (whichever is greater) should be overexcavated and replaced with compacted fill. The maximum to minimum fill thickness beneath the planned improvements should not exceed a ratio of 3:1 (maximum:minimum). Mitigation of expansive soils may require deeper overexcavation and replacement with non-detrimentally expansive soils.

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. Existing fill soils range from a very low expansive silty sand, to a medium (and possibly highly) expansive clay. In order to apply a "conventional" foundation design to a given parcel/lot, very low expansive soils are recommended to be placed and/or occur within 7 feet of pad grade, and a minimum "hold down" depth of 7 feet is recommended for any expansive, clayey fill. The actual "hold down" depth would depend on the expansive potential of the materials left in-place, as evaluated during grading.

Fill Suitability

Existing earth materials onsite should generate relatively fine grained fill material. The suitability of expansive soil is addressed in a previous section.

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 very low expansive (i.e., E.I. less than 20 and P.I. <15).

Graded Slope Construction

Graded slopes should be constructed at gradients no steeper than 2:1 (h:v) to heights up to 20 feet, without further analysis. Fill slopes should be properly keyed and benched if constructed along surfaces steeper than 5:1 (h:v). All fill slopes should be compacted to at least 90 percent of the laboratory standard (ASTM D 1557) throughout, including the slope face. Keyways for any planned fill slope should be constructed in accordance with Appendix E.

Planned fill slopes generally located above Camino de las Palmas appear to be relatively thin "skin" fills, with a plan fill depths of less than 5 feet locally. In order to provide for adequate slope stability, sufficient benching will be necessary in order to provide for at least 15 feet of fill laterally from the face of slope to the back cut (see stability fill detail in Appendix E). In addition to the keyway at the toe of the slope adjacent to Camino de las Palmas, an additional key will be required behind (upslope from) the planned detention basin. These earthwork structures (keyways) will be necessary to provide for adequate support of the building pads and adequate slope stability.

All cut slopes should be mapped by a geologist during construction. Although not anticipated at this time, should intersecting planes of joints/fractures daylight the cut slope face, or should undocumented fill, colluvium, or highly weathered bedrock (expansive soils) be exposed in cut slopes, remedial grading including stabilization fills or inclining the cut slope to a gradient flatter than the adverse structure may be necessary. The type of remedial grading would be based on the conditions exposed during cut slope construction.

The existing graded "fill" slope, located below the existing building pad is composed of undocumented fill and, as such, should be reconstructed in accordance with recommendations presented in Appendix E.

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 E.

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. 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 anticipated 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 expansion index(E.I.) values ranging on the order of less than 20 to 55 (very low to medium [as possibly high]), with a plasticity index (P.I.) for medium expansive soils evaluated as 23. As such, some 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 PI $\geq$ 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.

Preliminary Foundation Design

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 of non-detrimentally expansive soils (i.e., E.I.<21 and P.I. <15). Should foundations be underlain by (detrimentally) 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. 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 18 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 bedrock 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 200 pcf, with a maximum earth pressure of 2,000 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.

9. Provided that the earthwork and foundation recommendations in this reported are adhered foundations bearing on engineered fill should be minimally designed to accommodate a differential settlement of 1 inch over a 40-foot horizontal span (angular distortion = 1/480).

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 12 or 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 one direction. If detrimentally expansive soils are present (per the 2013 CBC), grade beams should be tied in 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 4.5 inches is recommended. Recommendations for floor slab underlayment are presented in a later section of this report.

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

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

7. 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 4½ 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 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 (2008) 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.

Post-Tension Slab Foundations

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, 2015). 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 be increased in 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 4.3.1 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 suitable bedrock 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 do not include other superimposed loading conditions due to traffic, structures, seismic events or adverse geologic conditions. When wall configurations are finalized, the appropriate loading conditions for superimposed loads can be provided upon request.

For preliminary planning purposes, the structural consultant/wall designer should incorporate the surcharge of traffic on the back of retaining walls where vehicular traffic could occur within horizontal distance "H" from the back of the retaining wall (where "H" equals the wall height). The traffic surcharge may be taken as 100 psf/ft in the upper 5 feet of 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 $\geq$ 20, P.I. < 25, and $\leq$ 20% passing No. 200 sieve. (May not be sufficiently available onsite).
NOTE: The use of Clay as wall backfill is prohibited.

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:

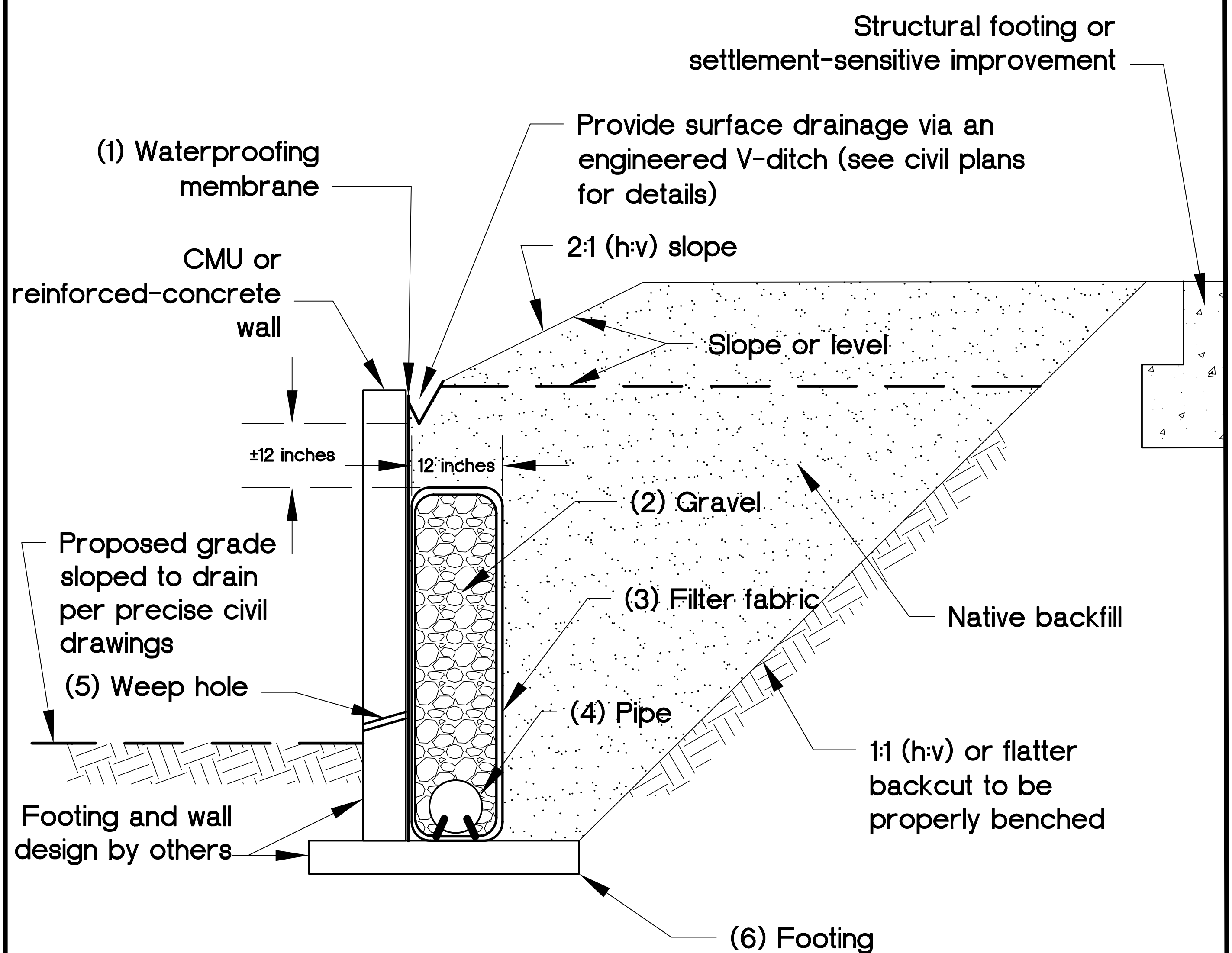
$$P_h = \tfrac{3}{8} \bullet a_h \bullet \gamma_t H$$

Where:	P_h	=	Seismic increment
	a_h	=	Probabilistic horizontal site acceleration with a percentage of "g"
	γ_t	=	total unit weight (120 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



(1) Waterproofing membrane.

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

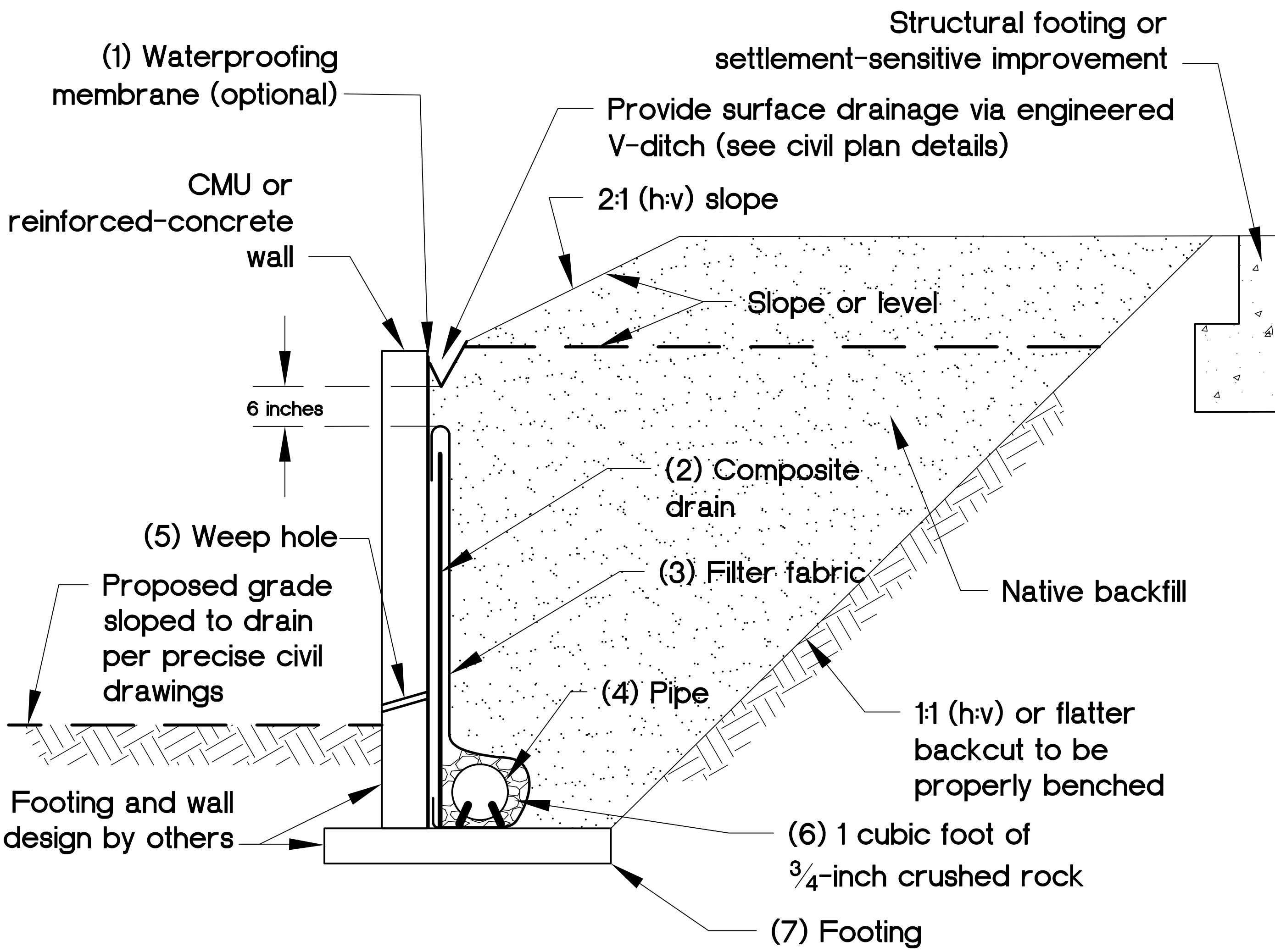
(3) Filter fabric: Mirafi 140N or approved equivalent.

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

(5) Weep 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.

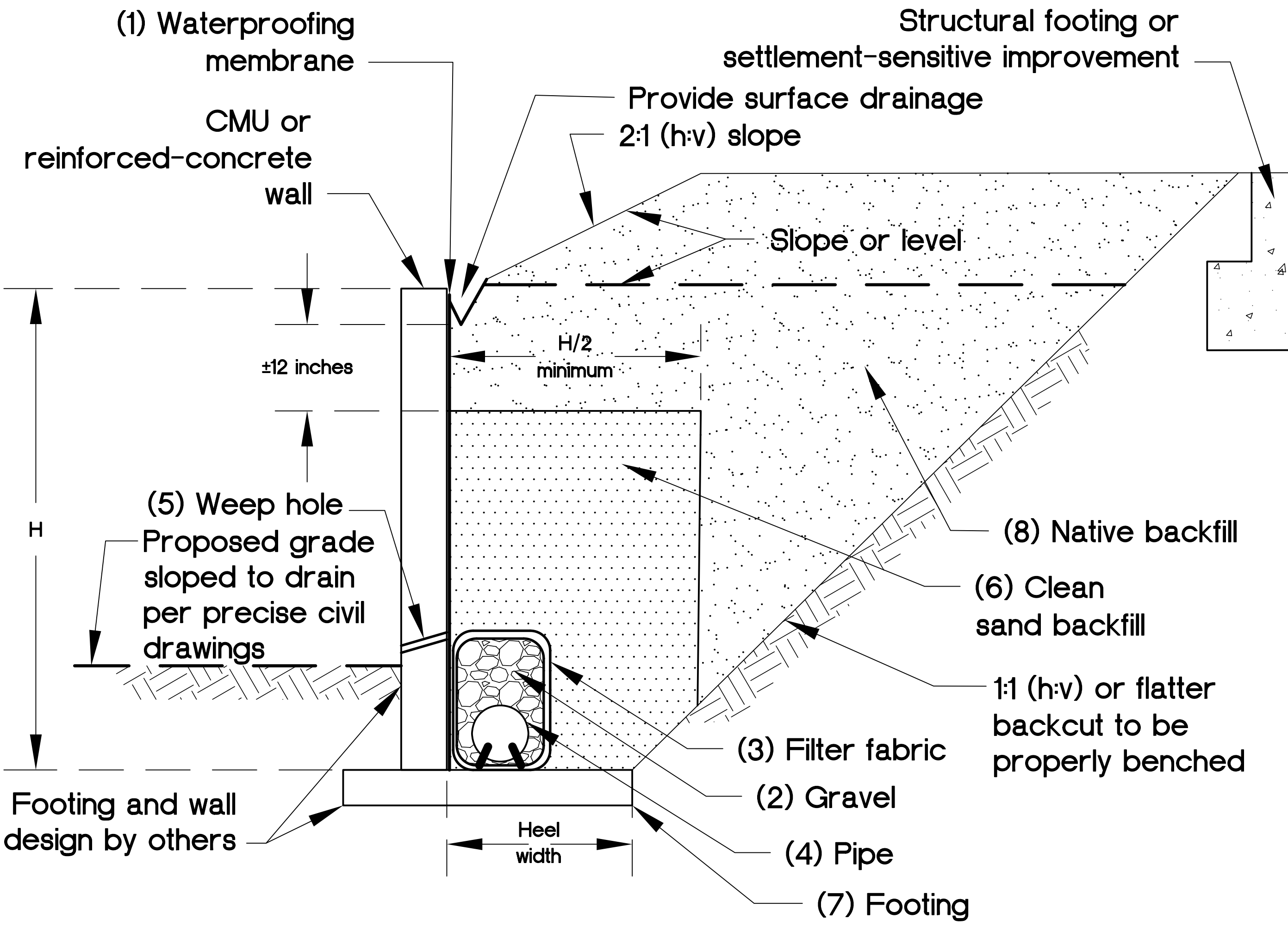
(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

(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

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. ≤50). Proper surface drainage should also be provided. For additional mitigation, consideration should be given to applying a water-proof membrane to the back of all retaining structures. The use of a waterstop should be considered for all concrete and masonry joints.

Wall/Retaining Wall Footing Transitions

Site walls are anticipated to be founded on footings designed in accordance with the recommendations in this report. 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.

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 120 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. $\leq$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.

PRELIMINARY PAVEMENT DESIGN RECOMMENDATIONS

General

The County may retain the authority to approve the final structural design sections after subgrade elevations and actual resistance values (R-values) have been obtained at the conclusion of earthwork. Based on an assumed R-value of 15, a review of County street design criteria, and for estimation and bidding purposes, the asphaltic concrete pavement section for the planned local and cul-de-sac streets, provided herein, should be considered for preliminary design. Typically, actual pavement sections will likely vary, therefore final pavement sections should be based on actual R-value testing performed following the backfill of underground utilities in the street right-of-way.

The preliminary pavement sections presented in the following table are based on the general Traffic Indices (T.I.), utilized by the County for a residential local and cul-de-sac streets, and the guidelines presented in the latest revision to the California Department of Transportation "Highway Design Manual" sixth edition. Based on an assumed R-value of 30 and respective T.I. values of 4.5 and 5.0 for cul-de-sacs and local streets, the following preliminary asphaltic concrete pavement designs are presented.

Asphaltic Concrete Pavement (ACP)

STREET CLASSIFICATION	TRAFFIC INDEX (T.I.)[(1)]	STANDARD PAVEMENT DESIGNS USING CLASS II AGGREGATE BASE (A.B.)		
		R-VALUE	A.C. (INCHES)	CLASS II A.B.[(2)] (INCHES)
Cul-De-Sac	4.5	15	3.0	7.0
Residential	5.0	15	3.0	9.0

[1] County of San Diego and to be confirmed by the project civil consultant.
[2] Assumed R-values for Class 2 aggregate base R=78 - Cal-Trans standard Class 2 Aggregate Base.

Portland Concrete Cement Pavement (PCCP)

Based on the anticipated subgrade soil conditions, the following Portland concrete cement pavement (PCCP) sections are provided.

PORTLAND CONCRETE CEMENT PAVEMENTS (PCCP)					
TRAFFIC AREAS	CONCRETE TYPE	PCCP THICKNESS (INCHES)	TRAFFIC AREAS	CONCRETE TYPE	PCCP THICKNESS (INCHES)
Light Vehicles	520-C-2500	7.0	Heavy Truck Traffic	520-C-2500	8.0
	560-C-3250	6.0		560-C-3250	7.0

NOTE: All PCCP is designed as un-reinforced and bearing directly on compacted subgrade. However, a 2-4 inch thick leveling course of compacted aggregate base, or crushed rock may be considered where pavement subgrade is uneven due to the presence of coarse rock. All PCCP should be properly detailed (jointing, etc.) per the industry standard. Pavements may be additionally reinforced with #4 reinforcing bars, placed 12 inches on center, each way, for improved performance. Trash truck loading pads shall be 8 inches per the County.

The preliminary pavement section provided above is intended as a minimum guideline. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. If the ADT (average daily traffic) or ADTT (average daily truck traffic) increases beyond that intended, as reflected by the T.I. used for design, increased maintenance and repair could be required for the pavement section. Consideration should be given to the increased potential for distress from overuse of paved street areas by heavy equipment and/or construction related heavy traffic (e.g., concrete trucks, loaded supply trucks, etc.), particularly when the final section is not in place (i.e., topcoat). Best management construction practices should be followed at all times, especially during inclement weather.

PAVEMENT GRADING RECOMMENDATIONS

General

All section changes should be properly transitioned. If adverse conditions are encountered during the preparation of subgrade materials, special construction methods may need to be employed. A GSI representative should be present for the preparation of subgrade, aggregate base, and asphaltic concrete.

Subgrade

Within street and parking areas, all surficial deposits of loose soil material should be removed and recompacted as recommended. After the loose soils are removed, the bottom is to be scarified to a depth of at least 6 inches, moisture conditioned as necessary and compacted to 95 percent of the maximum laboratory density, as determined by ASTM D 1557.

Deleterious material, excessively wet or dry pockets, concentrated zones of oversized rock fragments, and any other unsuitable materials encountered during grading should be removed. The compacted fill material should then be brought to the elevation of the proposed subgrade for the pavement. The subgrade should be proof-rolled in order to promote a uniform firm and unyielding surface. All grading and fill placement should be observed by the project geotechnical consultant.

Aggregate Base

Compaction tests are required for the recommended aggregate base section. Minimum relative compaction required will be 95 percent of the laboratory maximum density as determined by ASTM D 1557. Base aggregate should be in accordance to the “Greenbook” crushed aggregate base rock (minimum R-value=78).

Paving

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

1. The asphalt pavement layer is placed within two weeks of completion of aggregate base and/or subbase course.

2. Traffic is not routed over completed base before paving

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

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

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

Drainage

Positive drainage should be provided for all surface water to drain towards the area swale, curb and gutter, or to an approved drainage channel. Positive site drainage should be maintained at all times. Water should not be allowed to pond or seep into the ground, such as from behind unprotected curbs, both during and after grading. If planters or landscaping are adjacent to paved areas, measures should be taken to minimize the potential for water to enter the pavement section, such as thickened edges, enclosed planters, etc. Also, best management construction practices should be strictly adhered to at all times to minimize the potential for distress during construction and roadway improvements.

PCC Cross Gutters

PCC cross gutters should be designed in accordance with San Diego Regional Standard Drawing (SDRSD) G-12.

Additional Considerations

To mitigate perched groundwater, consideration should be given to installation of subgrade separators (cut-offs) between pavement subgrade and landscape areas, although this is not a requirement from a geotechnical standpoint. Cut-offs, if used, should be 6 inches wide and at least 12 inches below the pavement subgrade contact or 12 inches below the crushed aggregate base rock, if utilized.

ONSITE INFILTRATION-RUNOFF RETENTION SYSTEMS

General

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 bedrock.

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.

- An evaluation of the soils hydraulic conductivity, or (*K*) was performed in accordance with the Porchet, or inverse auger hole method (Van Hoorm, 1979; USBR, 1984). Based on the testing performed, a range of *K* values from 0.0047 inches/hour for claystone bedrock, 0.19 inches/hour for existing fill, and 0.24 inches/hour for sandstone bedrock, were evaluated. These values are generally below the recommended feasibility threshold (0.52 inches per hour) per the EPA (Clar, et al., 2004). Based on a review of USDA (2015) and the results of our onsite testing, site soils generally fall into Hydrologic subgroup "D."

- 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).

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

- 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.

- Impermeable liners and subdrains should be used along the bottom of bioretention swales/basins located within the influence of slopes. 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): 30 (lb/in-width, min.); and Tear Strength (ASTM D1004): 8 (lb/in, min); Seam Shear Strength (ASTM D882) 58.4 (lb/in, min); Seam Peel Strength (ASTM D882) 15 (lb/in, min).

- 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.

Proposed WQ & HMP Detention Basin

A proposed storm water detention basin is shown on LC (2015) as being constructed into a plan fill slope, descending from Units 1 through 3, to Camino De Las Palmas, within the southeastern portion of the site. Water has been shown to weaken the inherent strength of all earth materials, and slope stability is significantly reduced by overly wet conditions. As such, the current location of bioretention basin shown on LE (2015) will generally increase the potential for slope instability, such as slumps, erosion, and concentrated offsite drainage, and/or increase the potential for distress to planned retaining walls located around the perimeter of the basin. As such, consideration should be given to redesigning storm water treatment systems to consist of a leak proof, lined system with subdrainage.

Section E-E,' shown on LE (2015) indicates that the foundations for the retaining walls forming the detention facility would be bearing on both loose gravel and soil (either fill or bedrock) with an impermeable liner extending vertically below the footing, between the gravel and the adjacent earth material. This configuration results in dissimilar bearing materials, and potential for yielding bearing materials (loose gravel and saturated soil), increasing the potential for rotational failure of the walls. The configuration also appears to present constructability issues (i.e., footing excavation/construction over gravel, liner, soil). In order to mitigate this condition, the wall foundations should be deepened into suitable bearing soil below the bottom of the gravel "storage" layer.

The outlet structure for the detention basin will be subject to a high potential for internal erosion, such as by suffusion or piping. In order to mitigate this type of erosion, the penetration through the wall (shown on Section E-E of LC[2015]) should be waterproofed and sealed. Based on Section E-E of LC (2015) there is also a potential for piping erosion along the joint between the bottom of the wall footing and the impermeable liner, and along the base of the liner itself, as a pressure head within a potentially "full" basin forces water out of the basin along construction joints, etc. This potential is further increased by the very low permeability of site soils surrounding the basin. In order to mitigate this condition, the basin should be completely lined, and provided with subdrainage. Wall footing should also be deepened to be founded below the bottom of the gravel storage layer.

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, 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.

<u>**APPENDIX A**</u>

REFERENCES

APPENDIX A

REFERENCES

ACI Committee 318, 2011, Building code requirements for structural concrete (ACI 318-11) and commentary, dated January.

ACI Committee 302, 2004, Guide for concrete floor and slab construction, ACI 302.1R-04, dated June.

Allen, V., Connerton, A., and Carlson, C., 2011, Introduction to Infiltration Best Management Practices (BMP), Contech Construction Products, Inc., Professional Development Series, dated December.

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

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

American Society of Civil Engineers, 2010, Minimum design loads for buildings and other structures, ASCE Standard ASCE/SEI 7-10.

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

_____, 2000b, EQSEARCH, A computer program for the estimation of peak horizontal acceleration from California historical earthquake catalogs; Updated to December 2009, Windows 95/98 version.

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

B.P. Associates, 2015, Vista Azul, concept site study, dated June.

Bryant, W.A., and Hart, E.W., 2007, Fault-rupture hazard zones in California, Alquist-Priolo earthquake fault zoning act with index to earthquake fault zones maps; California Geological Survey, Special Publication 42, interim revision.

California Building Standards Commission, 2013, California Building Code, California Code of Regulations, Title 24, Part 2, Volume 2 of 2, Based on the 2012 International Building Code, 2013 California Historical Building Code, Title 24, Part 8; 2013 California Existing Building Code, Title 24, Part 10.

California Stormwater Quality Association (CASQA), 2003, Stormwater best management practice handbook, new development and redevelopment, dated January.

County of San Diego, Department of Planning and Land Use, 2007, Low impact development (LID) handbook, stormwater management strategies, dated December 31.

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

Clar, M.L., Barfield, B.J., O'Connor T.P., 2004, Stormwater best management practice design guide, volume 3, basin best management practices, US EPA/600/R-04/121B, dated September.

Google Earth, 2015, Google earth imagery data base.

Hydrologic Solutions, StormChamber™ installation brochure, pgs. 1 through 8, undated.

Jennings, C.W., 1994, Fault activity map of California and adjacent areas: California Division of Mines and Geology, Map Sheet No. 6, scale 1:750,000.

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

Kennedy, M.P. and Peterson, G.L., 1975, Geology of the San Diego, California, Bulletin 200.

Kennedy, M.P., and Tan, SS., 2008, Geologic map of the San Diego 30' by 60' quadrangle, California, Map no. 3, scale 1:100,000,California Geologic Survey and U.S. Geologic Survey.

_____, 1977, Geology of the National City, Imperial Beach, and Otay Mesa quadrangles, Southern San Diego metropolitan area, California, California Division of Mines and Geology, Map Sheet 29.

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

Post-Tensioning Institute, 2008, Addendum no. 2 to the 3rd edition of the design of post-tensioned slabs-on-ground, dated May.

_____, 2004, Design of post-tensioned slabs-on-ground, 3rd edition.

Romanoff, M., 1989, Underground corrosion, National Bureau of Standards Circular 579, Published by National Association of Corrosion Engineers, Houston, Texas, originally issued April 1, 1957.

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

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

State of California, 2015, Civil Code, Sections 895 et seq.

State of California Department of Transportation, Division of Engineering Services, Materials Engineering, and Testing Services, Corrosion Technology Branch, 2003, Corrosion Guidelines, Version 1.0, dated September.

Tan, S.S., 1987, Landslide hazards in the Rancho santa Fe quadrangle, San Diego County, California, Landslide hazard identification map #6, DMG Open File Report 86-15.

Tan, S.S., and Giffen, D.G., 1995, Landslide hazards in the southern part of the San Diego Metropolitan area, San Diego County, California, Landslide hazard identification map no. 33, Plate 33E, Department of Conservation, Division of Mines and Geology, DMG Open File Report 95-03.

United States Department of Agriculture, National Resources Conservation Service, 2015, Custom soils report for San Diego County area, Vista Azul site, dated September.

United States Department of the Interior, Bureau of Reclamation, 1984, Drainage manual, a water resources technical publication, second printing, Denver, U.S. Department of the Interior, Bureau of Reclamation, 286 pp.

United States Geological Survey, 2013, U.S. Seismic design maps, earthquake hazards program, http://geohazards.usgs.gov/designmaps/us/application.php. Version 3.1.0, dated July.

Van Hoorm, J.W., 1979, Determining hydraulic conductivity with the inversed auger hole and infiltrometer methods.

Wire Reinforcement Institute, 1996, Design of slab-on-ground foundations, an update, dated March.

APPENDIX B

TEST EXCAVATION LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM

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

CONSISTENCY OR RELATIVE DENSITY

CRITERIA

Standard Penetration Test

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

Standard Penetration Test

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

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

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

MOISTURE CONDITIONS

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

MATERIAL QUANTITY

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

OTHER SYMBOLS

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

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

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

GeoSoils, Inc.

W.O. 6947-A-SC
RINA, LLC
Vista Azul, Lemon Grove
Logged By: RGC
August 13, 2015

LOG OF EXPLORATORY TEST PITS AND HAND AUGER BORINGS

TEST PIT/ HAND AUGER NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-1 SE End	493	0-4	CL				**UNDOCUMENTED FILL:** SANDY CLAY, dark grayish brown, slightly moist, firm; desiccated.
		4-8	SC				CLAYEY SAND, dark brown, slightly moist, (density).
		8-12	CL				**MISSION VALLEY FORMATION:** CLAY STONE, olive brown, moist, very stiff.
							Total Depth = 12' No Groundwater/Caving Encountered Backfilled 8/13/2015
TP-1 NW End	493	0-½	CL				**UNDOCUMENTED FILL:** SANDY CLAY, dark grayish brown, dry, (density); desiccated, few roots.
		½-4	CL				**HIGHLY WEATHERED MISSION VALLEY FORMATION:** CLAYSTONE, light gray and light olive brown, dry, soft; highly fractured (random), abundant caliche, sub-horizontal basal contact.
		4-7	SM/SP				**MISSION VALLEY FORMATION:** SANDSTONE, grayish brown, dry, medium dense to dense; thickly bedded, sub-horizontal basal contact.
		7-12	CL				CLAYSTONE, olive brown, slightly moist, very stiff; fractured.
	Note: Contact between fill and bedrock steeply dipping, approximately 80 degrees to the southeast, becoming flatter with depth.						Total Depth = 12' No Groundwater/Caving Encountered Backfilled 8/13/2015

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TP-2	495	0-½	SC				**UNDOCUMENTED FILL:** CLAYEY SAND, dark grayish brown, dry, loose; few roots, heavily burrowed, desiccated.
		½'-2	SP/SM				**HIGHLY WEATHERED MISSION VALLEY FORMATION:** SANDSTONE, light gray, dry, loose to medium dense; thickly bedded, burrowed. Basal contact: N80°E, 4°NW; N70°W, 3°SW.
		2-4½	CL	@ 2-4' Bulk			**MISSION VALLEY FORMATION:** CLAY STONE, olive brown, slightly moist, very stiff; Fractured: N5°E, 88°NW; N20°W; 56°SW, Bedding: N80°E, 3°NW (internal), Basal contact: N60°W, 3°SW to N70°E, 2°SE (undulatory).
		4½-5	SP				SANDSTONE, grayish brown, slightly moist, dense; cemented.
		5-12	SP	@ 5-6' Bulk @ 5' Ring	5.9	122.4	SANDSTONE, brown, moist, dense; thickly bedded, cemented.
							Total Depth = 12' No Groundwater/Caving Encountered Backfilled 8/13/2015

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TEST PIT/ HAND AUGER NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-3	495	0-½	SP/SM				**WEATHERED MISSION VALLEY FORMATION:** SANDSTONE, light gray, dry, loose.
		½-1	SP				**MISSION VALLEY FORMATION:** SANDSTONE, light gray, dry, dense; cross bedded: <7° dip to SE and NW, basal contact: N60°E, 7°NW.
		1-4½	CL	2	20.2	92.2	CLAYSTONE, olive brown and light gray, slightly moist, very stiff; fractured, abundant caliche. Fracture: N60·W, 88°NE; N10°E, 89°SE.
		4½-6½	SP				SANDSTONE, brown, slightly moist, dense; thickly bedded, cemented.
							Total Depth = 6½' No Groundwater/Caving Encountered Backfilled 8/13/2015
TP-4	492.5	0-4	SC/CL	@ 1-4' Bulk			**UNDOCUMENTED FILL:** CLAYEY SAND to SANDY CLAY, dark grayish brown, dry, loose/soft; PVC pipe, brick, plastic debris, desiccated.
		4-6	SC				CLAYEY SAND, brown, slightly moist, loose; depth to basal contact varies from 5½' north to 6½' south.
		6-9	SP				**MISSION VALLEY FORMATION:** SANDSTONE, light brown, slightly moist, medium dense to dense; caliche, at 8' caliche generally absent.
							Total Depth = 9' No Groundwater/Caving Encountered Backfilled 8/13/2015

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LOG OF EXPLORATORY TEST PITS AND HAND AUGER BORINGS

TEST PIT/ HAND AUGER NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-5	466	0-2½	SC				**UNDOCUMENTED FILL:** CLAYEY SAND, grayish brown, dry, loose; filter fabric and concrete debris, few roots, desiccated.
		2½-6	CL				SANDY CLAY, mottled brown and red brown, moist, soft; fragments of claystone and caliche, some PVC pipe, asphalt, plastic.
		6-8	SC				CLAYEY SAND, brown, slightly moist, loose; concrete slab debris at 7'.
		8-12	SP				**MISSION VALLEY FORMATION:** SANDSTONE, light grayish brown, slightly moist, dense; slightly fractured.
							Total Depth = 12' No Groundwater/Caving Encountered Backfilled 8/13/2015
TP-6	451	0-1	CL				**COLLUVIUM:** SANDY CLAY, very dark brown, dry, stiff; desiccated.
		1-2½	CL				**HIGHLY WEATHERED MISSION VALLEY FORMATION:** SANDY CLAY, brownish gray, dry, stiff; highly fractured, abundant caliche.
		2½-3½	CL				**MISSION VALLEY FORMATION:** SANDY CLAYSTONE, brown, slightly moist, stiff.
		3½-6	CH				CLAYSTONE, red brown, moist, very stiff.
							Total Depth = 6' No Groundwater/Caving Encountered Backfilled 8/20/2015

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LOG OF EXPLORATORY TEST PITS AND HAND AUGER BORINGS

TEST PIT/ HAND AUGER NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-7	475	0-1	CL				**UNDOCUMENTED FILL:** SANDY CLAY, very dark grayish brown, dry, stiff; desiccated, burrowed.
		1-2	SP				**MISSION VALLEY FORMATION:** SANDSTONE, brown, slightly moist, dense; thickly bedded, cemented.
	Note: Fill occurs as surficial layer on slope face.						Total Depth = 2' No Groundwater/Caving Encountered Backfilled 8/13/2015
TP-8	468	0-1½	SC				**UNDOCUMENTED FILL:** CLAYEY SAND, dark gray brown, dry, loose; porous, desiccated, bioturbated.
		1½-2	CL				**COLLUVIUM:** SANDY CLAY, very dark brown, slightly moist, stiff; desiccated.
		2-5	SP				**HIGHLY WEATHERED MISSION VALLEY FORMATION:** SANDSTONE, light gray, dry, loose; caliche.
		6-9	SP				**MISSION VALLEY FORMATION:** SANDSTONE, brown, slightly moist, dense; thickly bedded.
							Total Depth = 9' No Groundwater/Caving Encountered Backfilled 8/13/2015

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August 13, 2015

LOG OF EXPLORATORY TEST PITS AND HAND AUGER BORINGS

TEST PIT/ HAND AUGER NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-9	493	0-1	SM				**HIGHLY WEATHERED MISSION VALLEY FORMATION:** SILTY SANDSTONE, grayish brown, dry, loose.
		1-6	SP	@ 2' Bulk			**MISSION VALLEY FORMATION:** SANDSTONE, brown, slightly moist, dense; thickly bedded.
							Total Depth = 6' No Groundwater/Caving Encountered Backfilled 8/13/2015

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LOG OF EXPLORATORY TEST PITS AND HAND AUGER BORINGS

TEST PIT/ HAND AUGER NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
HA-1	454	0-1	CL				**UNDOCUMENTED FILL:** SANDY CLAY, brown, dry, stiff; desiccated.
		1-3½	CL				**COLLUVIUM:** SANDY CLAY, brown, slightly moist, firm.
		3½-4½	CL				**HIGHLY WEATHERED MISSION VALLEY FORMATION:** CLAYSTONE, brownish gray, slightly moist, (density); highly fractured, abundant caliche.
							Total Depth = 4½' No Groundwater/Caving Encountered Backfilled 8/20/2015
HA-2	459	0-1½					**HIGHLY WEATHERED MISSION VALLEY FORMATION:** CLAYSTONE, red brown, dry, stiff; open desiccation, cracks.
		1½-2					**MISSION VALLEY FORMATION:** CLAYSTONE, red brown, moist, very stiff.
							Total Depth = 2' No Groundwater/Caving Encountered Backfilled 8/20/2015

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August 13, 2015

LOG OF EXPLORATORY TEST PITS AND HAND AUGER BORINGS

TEST PIT/ HAND AUGER NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
HA-3	490	0-1					**UNDOCUMENTED FILL:** SANDY CLAY, very dark grayish brown, dry, loose to medium dense; bioturbated.
		1-2					**MISSION VALLEY FORMATION:** SANDSTONE, brown, dry, dense.
							Total Depth = 2' No Groundwater/Caving Encountered Backfilled 8/20/2015
HA-4	482	0-1					**UNDOCUMENTED FILL:** SANDY CLAY, dark grayish brown, dry, stiff; bioturbated.
		1-1½					**MISSION VALLEY FORMATION:** SANDSTONE, brown, dry, dense.
							Total Depth = 1½' No Groundwater/Caving Encountered Backfilled 8/20/2015
HA-5	474	0-1					**UNDOCUMENTED FILL:** SANDY CLAY, dark grayish brown, dry, stiff; bioturbated.
		1-1½					**MISSION VALLEY FORMATION:** SANDSTONE, brown, dry, dense.
							Total Depth = 1½' No Groundwater/Caving Encountered Backfilled 8/20/2015

<u>APPENDIX C</u>

SEISMICITY

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***********************
*                     *
*   E Q F A U L T     *
*                     *
*   Version 3.00      *
*                     *
***********************
```

DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 6947

DATE: 09-23-2015

JOB NAME: Vista Azul

CALCULATION NAME: Test Run Analysis

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

SITE COORDINATES:
   SITE LATITUDE: 32.7351
   SITE LONGITUDE: 117.0176

SEARCH RADIUS: 62.4 mi

ATTENUATION RELATION: 12) Bozorgnia Campbell Niazi (1999) Hor.-Soft Rock-Cor.
   UNCERTAINTY (M=Median, S=Sigma): S   Number of Sigmas: 1.0
   DISTANCE MEASURE: cdist
   SCOND: 1
   Basement Depth: .01 km   Campbell SSR: 1   Campbell SHR: 0
   COMPUTE PEAK HORIZONTAL ACCELERATION

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

MINIMUM DEPTH VALUE (km): 3.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

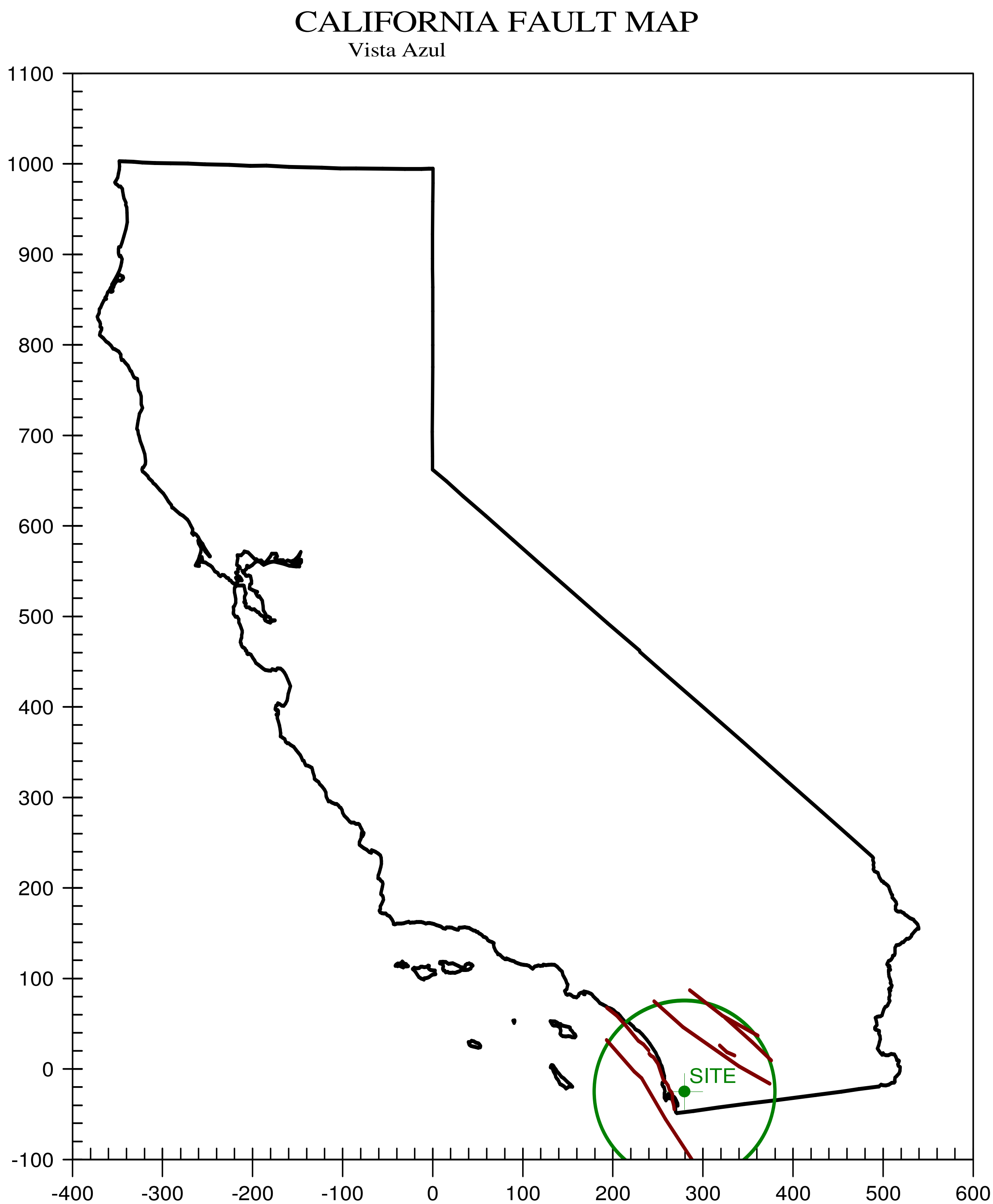
Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
ROSE CANYON	8.8(14.1)	7.2	0.448	X
CORONADO BANK	20.9(33.7)	7.6	0.271	IX
ELSINORE (JULIAN)	35.7(57.4)	7.1	0.113	VII
NEWPORT-INGLEWOOD (Offshore)	37.7(60.7)	7.1	0.107	VII
EARTHQUAKE VALLEY	39.8(64.1)	6.5	0.067	VI
ELSINORE (COYOTE MOUNTAIN)	41.3(66.5)	6.8	0.079	VII
ELSINORE (TEMECULA)	44.4(71.4)	6.8	0.073	VII
SAN JACINTO-COYOTE CREEK	56.4(90.8)	6.6	0.050	VI
SAN JACINTO - BORREGO	57.7(92.8)	6.6	0.049	VI
SAN JACINTO-ANZA	58.3(93.9)	7.2	0.073	VII

-END OF SEARCH- 10 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

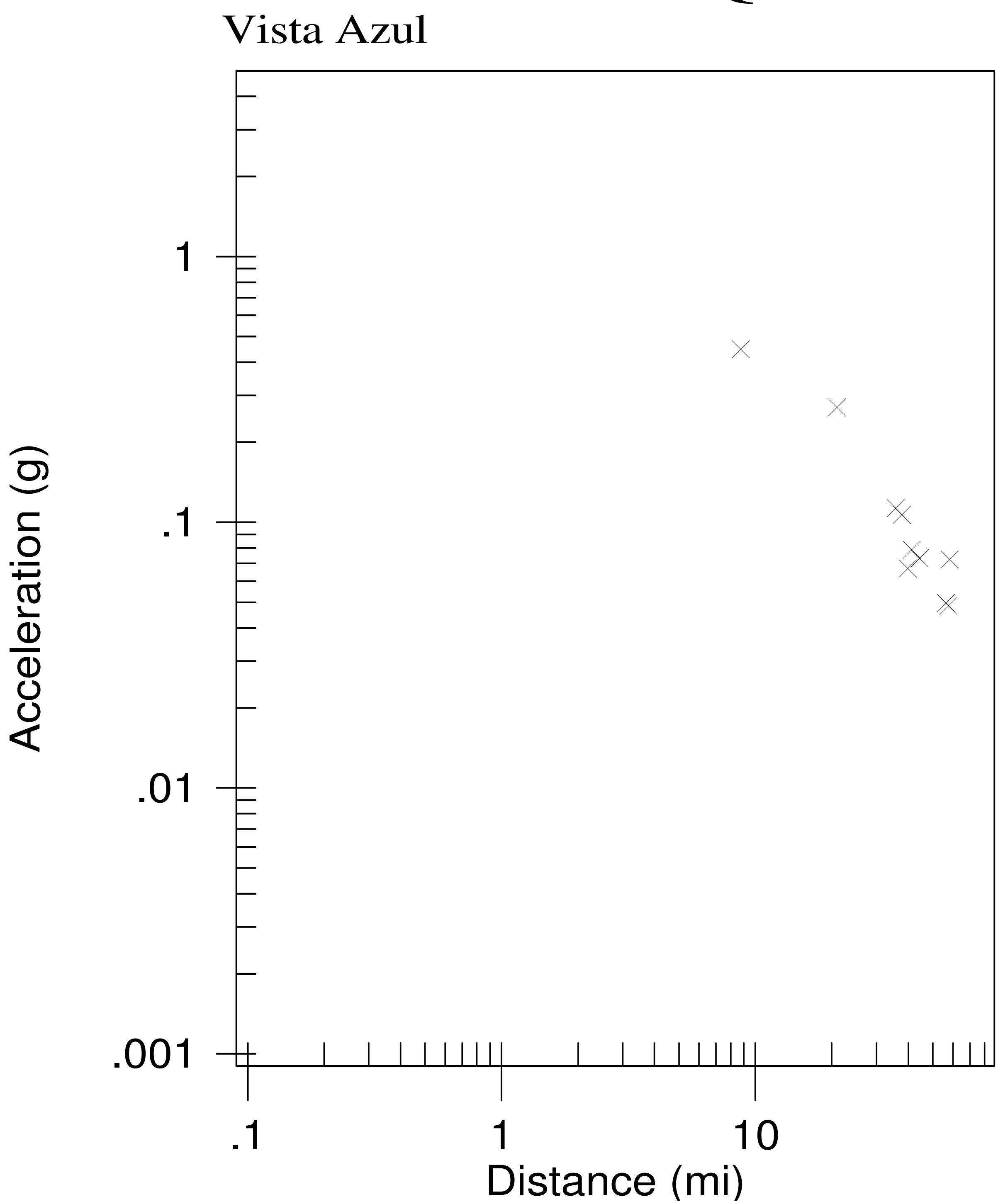
THE ROSE CANYON FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 8.8 MILES (14.1 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.4475 g



W.O. 6947-A-SC
PLATE C-3

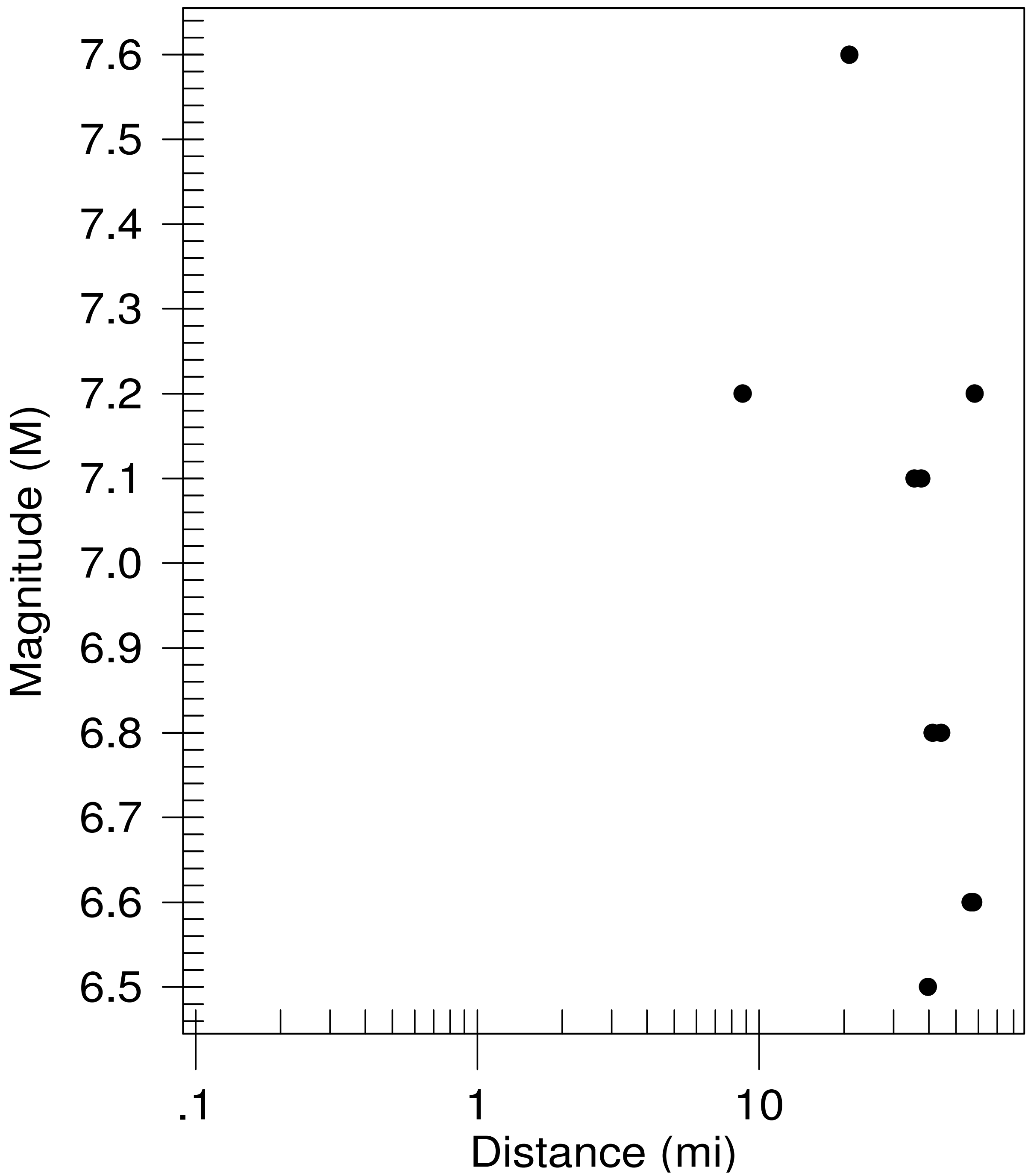
MAXIMUM EARTHQUAKES



W.O. 6947-A-SC
PLATE C-4

EARTHQUAKE MAGNITUDES & DISTANCES

Vista Azul



W.O. 6947-A-SC
PLATE C-5

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

JOB NUMBER: 6947

DATE: 09-23-2015

JOB NAME: Vista Azul

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

MAGNITUDE RANGE:
   MINIMUM MAGNITUDE: 5.00
   MAXIMUM MAGNITUDE: 9.00

SITE COORDINATES:
   SITE LATITUDE: 32.7351
   SITE LONGITUDE: 117.0176

SEARCH DATES:
   START DATE: 1800
   END DATE: 2015

SEARCH RADIUS:
   62.4 mi
   100.4 km

ATTENUATION RELATION: 12) Bozorgnia Campbell Niazi (1999) Hor.-Soft Rock-Cor.
   UNCERTAINTY (M=Median, S=Sigma): S   Number of Sigmas: 1.0
   ASSUMED SOURCE TYPE: SS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
   SCOND: 1 Depth Source: A
   Basement Depth: .01 km   Campbell SSR: 1   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]
MGI	32.8000	117.1000	05/25/1803	0 0 0.0	0.0	5.00	0.145	VIII	6.5(10.5)
T-A	32.6700	117.1700	05/24/1865	0 0 0.0	0.0	5.00	0.105	VII	9.9(16.0)
T-A	32.6700	117.1700	12/00/1856	0 0 0.0	0.0	5.00	0.105	VII	9.9(16.0)
T-A	32.6700	117.1700	10/21/1862	0 0 0.0	0.0	5.00	0.105	VII	9.9(16.0)
DMG	32.7000	117.2000	05/27/1862	20 0 0.0	0.0	5.90	0.172	VIII	10.9(17.5)
DMG	32.8000	116.8000	10/23/1894	23 3 0.0	0.0	5.70	0.123	VII	13.4(21.6)
MGI	33.0000	117.0000	09/21/1856	730 0.0	0.0	5.00	0.059	VI	18.3(29.5)
DMG	33.0000	117.3000	11/22/1800	2130 0.0	0.0	6.50	0.111	VII	24.5(39.5)
DMG	33.2000	116.7000	01/01/1920	235 0.0	0.0	5.00	0.029	V	37.0(59.5)
DMG	33.0000	116.4330	06/04/1940	1035 8.3	0.0	5.10	0.029	V	38.5(62.0)
MGI	33.2000	116.6000	10/12/1920	1748 0.0	0.0	5.30	0.031	V	40.2(64.7)
DMG	32.7000	116.3000	02/24/1892	720 0.0	0.0	6.70	0.073	VII	41.8(67.2)
T-A	32.2500	117.5000	01/13/1877	20 0 0.0	0.0	5.00	0.024	V	43.7(70.3)
DMG	32.2000	116.5500	11/05/1949	43524.0	0.0	5.10	0.024	V	45.9(73.9)
DMG	32.2000	116.5500	11/04/1949	204238.0	0.0	5.70	0.035	V	45.9(73.9)
DMG	32.0830	116.6670	11/25/1934	818 0.0	0.0	5.00	0.021	IV	49.4(79.6)
PAS	32.9710	117.8700	07/13/1986	1347 8.2	6.0	5.30	0.024	IV	52.1(83.8)
GSG	33.4200	116.4890	07/07/2010	235333.5	14.0	5.50	0.025	V	56.3(90.6)
DMG	33.3430	116.3460	04/28/1969	232042.9	20.0	5.80	0.029	V	57.2(92.1)
DMG	33.2000	116.2000	05/28/1892	1115 0.0	0.0	6.30	0.040	V	57.2(92.1)
DMG	32.0000	117.5000	05/01/1939	2353 0.0	0.0	5.00	0.018	IV	58.0(93.4)
DMG	32.0000	117.5000	06/24/1939	1627 0.0	0.0	5.00	0.018	IV	58.0(93.4)
GSP	32.3290	117.9170	06/15/2004	222848.2	10.0	5.30	0.021	IV	59.4(95.6)
DMG	33.1900	116.1290	04/09/1968	22859.1	11.1	6.40	0.040	V	60.3(97.0)
PAS	33.5010	116.5130	02/25/1980	104738.5	13.6	5.50	0.023	IV	60.4(97.2)
GSP	33.5290	116.5720	06/12/2005	154146.5	14.0	5.20	0.019	IV	60.6(97.5)
DMG	33.5000	116.5000	09/30/1916	211 0.0	0.0	5.00	0.017	IV	60.7(97.7)
GSP	33.5080	116.5140	10/31/2001	075616.6	15.0	5.10	0.018	IV	60.8(97.8)
DMG	33.2170	116.1330	08/15/1945	175624.0	0.0	5.70	0.026	V	61.1(98.3)
DMG	32.9670	116.0000	10/21/1942	162519.0	0.0	5.00	0.017	IV	61.2(98.4)
DMG	32.9670	116.0000	10/21/1942	162213.0	0.0	6.50	0.043	VI	61.2(98.4)
DMG	32.9670	116.0000	10/21/1942	162654.0	0.0	5.00	0.017	IV	61.2(98.4)
DMG	32.9670	116.0000	10/22/1942	181326.0	0.0	5.00	0.017	IV	61.2(98.4)
DMG	33.2830	116.1830	03/19/1954	102117.0	0.0	5.50	0.023	IV	61.4(98.8)
DMG	33.2830	116.1830	03/23/1954	41450.0	0.0	5.10	0.018	IV	61.4(98.8)
DMG	33.2830	116.1830	03/19/1954	95429.0	0.0	6.20	0.035	V	61.4(98.8)
DMG	33.2830	116.1830	03/19/1954	95556.0	0.0	5.00	0.017	IV	61.4(98.8)
DMG	33.4000	116.3000	02/09/1890	12 6 0.0	0.0	6.30	0.037	V	61.9(99.6)
DMG	32.9830	115.9830	05/23/1942	154729.0	0.0	5.00	0.017	IV	62.4(100.4)

****

-END OF SEARCH- 39 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 2015

LENGTH OF SEARCH TIME: 216 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 6.5 MILES (10.5 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 6.7

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.172 g

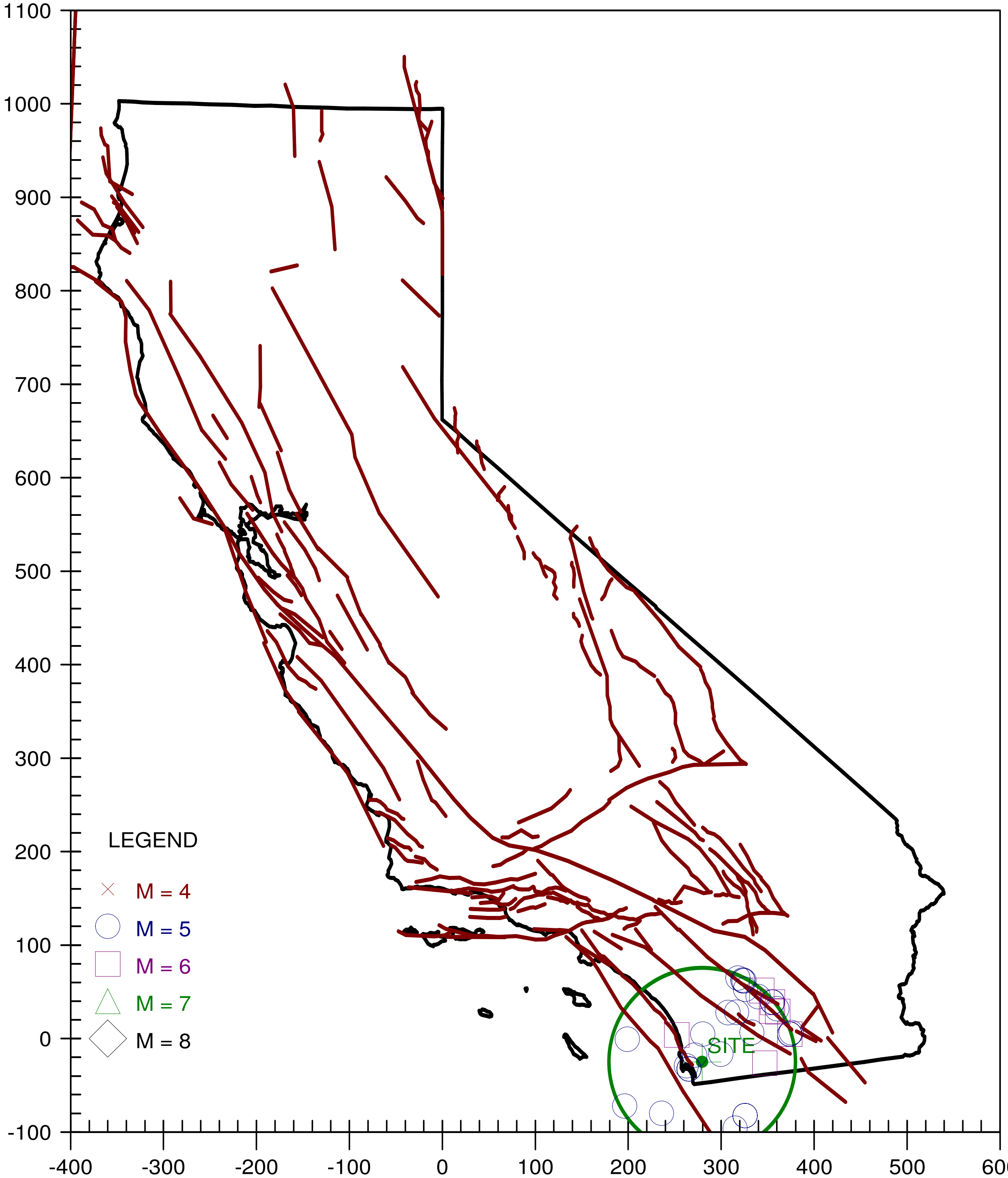
COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
a-value= 0.931
b-value= 0.381
beta-value= 0.878

TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	39	0.18056
4.5	39	0.18056
5.0	39	0.18056
5.5	15	0.06944
6.0	7	0.03241
6.5	3	0.01389

EARTHQUAKE EPICENTER MAP

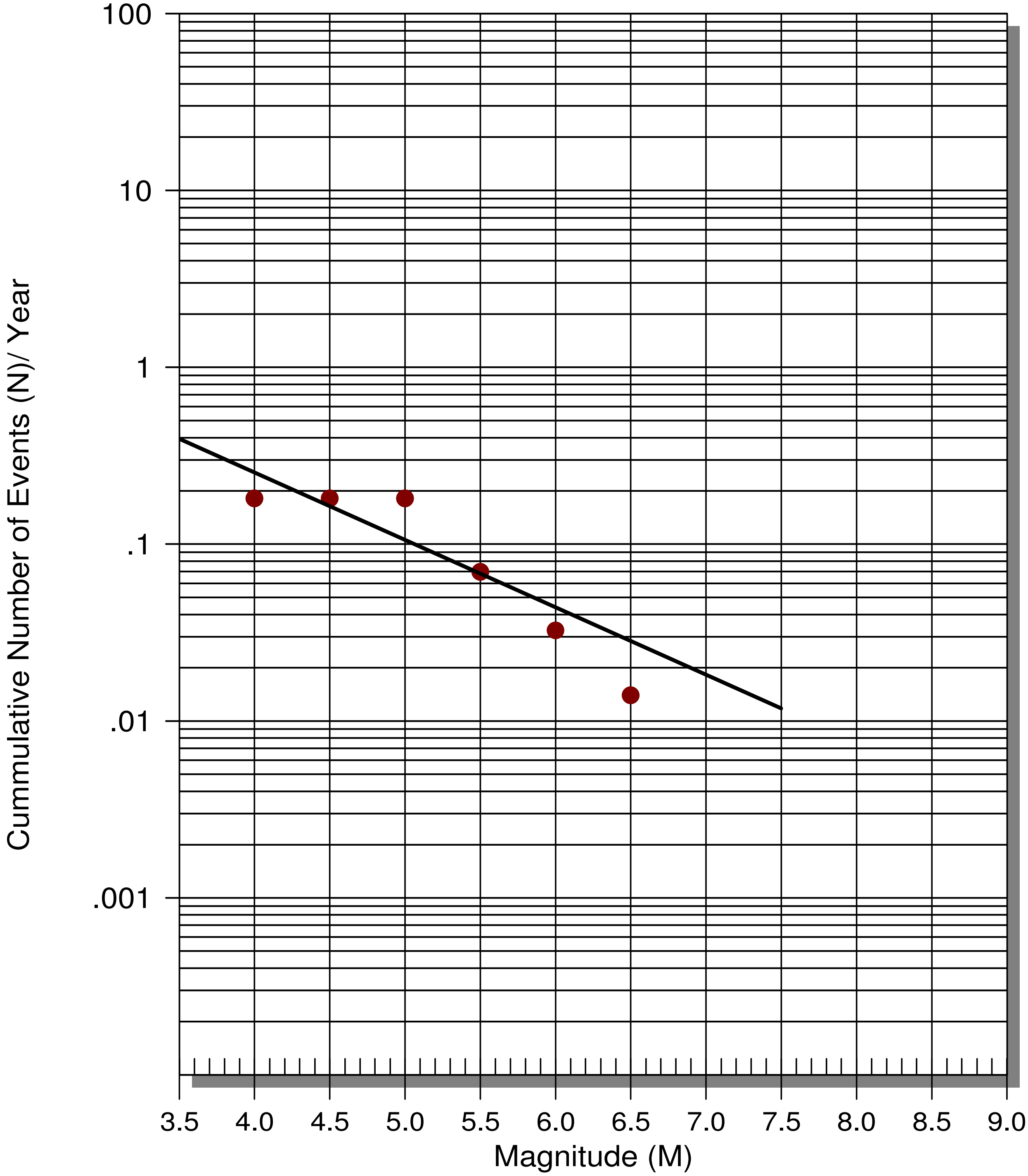
Vista Azul



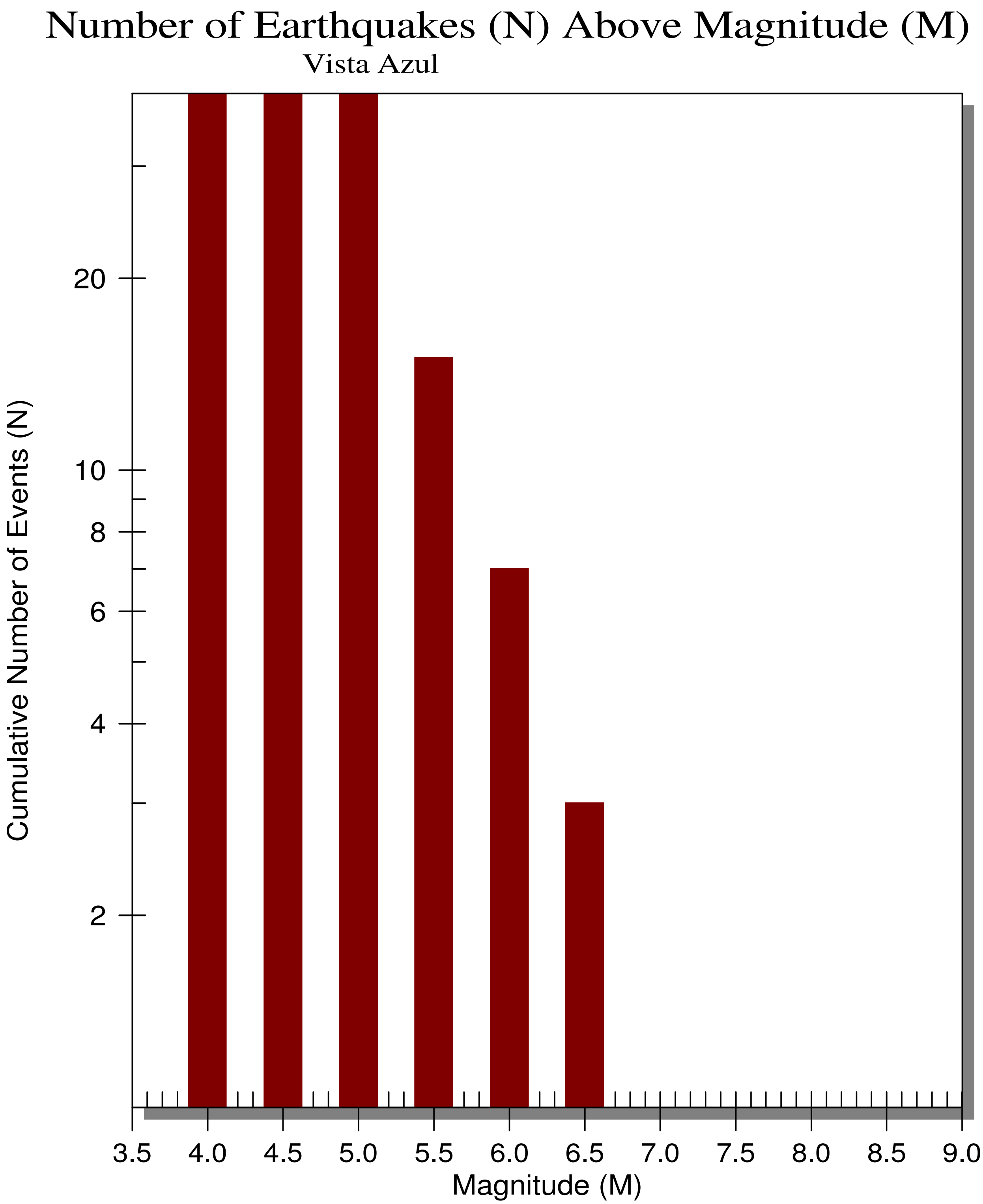
W.O. 6947-A-SC
PLATE C-9

EARTHQUAKE RECURRENCE CURVE

Vista Azul



W.O. 6947-A-SC
PLATE C-10



W.O. 6947-A-SC
PLATE C-11

APPENDIX D

SLOPE STABILITY ANALYSIS

APPENDIX D

SLOPE STABILITY ANALYSIS
INTRODUCTION OF GSTABL7 v.2 COMPUTER PROGRAM

Introduction

GSTABL7 v.2 is a fully integrated slope stability analysis program. It permits the engineer to develop the slope geometry interactively and perform slope analysis from within a single program. The slope analysis portion of GSTABL7 v.2 uses a modified version of the popular STABL program, originally developed at Purdue University.

GSTABL7 v.2 performs a two dimensional limit equilibrium analysis to compute the factor of safety for a layered slope using the simplified Bishop or Janbu methods. This program can be used to search for the most critical surface or the factor of safety may be determined for specific surfaces. GSTABL7, Version 2, is programmed to handle:

1. Heterogenous soil systems
2. Anisotropic soil strength properties
3. Reinforced slopes
4. Nonlinear Mohr-Coulomb strength envelope
5. Pore water pressures for effective stress analysis using:
    a. Phreatic and piezometric surfaces
    b. Pore pressure grid
    c. R factor
    d. Constant pore water pressure
6. Pseudo-static earthquake loading
7. Surcharge boundary loads
8. Automatic generation and analysis of an unlimited number of circular, noncircular and block-shaped failure surfaces
9. Analysis of right-facing slopes
10. Both SI and Imperial units

General Information

If the reviewer wishes to obtain more information concerning slope stability analysis, the following publications may be consulted initially:

1. The Stability of Slopes, by E.N. Bromhead, Surrey University Press, Chapman and Hall, N.Y., 411 pages, ISBN 412 01061 5, 1992.

2. Rock Slope Engineering, by E. Hoek and J.W. Bray, Inst. of Mining and Metallurgy, London, England, Third Edition, 358 pages, ISNB 0 900488 573, 1981.

3. Landslides: Analysis and Control, by R.L. Schuster and R.J. Krizek (editors), Special Report 176, Transportation Research Board, National Academy of Sciences, 234 pages, ISBN 0 309 02804 3, 1978.

GSTABL7 v.2 Features

The present version of GSTABL7 v.2 contains the following features:

1. Allows user to calculate factors of safety for static stability and dynamic stability situations.

2. Allows user to analyze stability situations with different failure modes.

3. Allows user to edit input for slope geometry and calculate corresponding factor of safety.

4. Allows user to readily review on-screen the input slope geometry.

5. Allows user to automatically generate and analyze unlimited number of circular, non-circular and block-shaped failure surfaces (i.e., bedding plane, slide plane, etc.).

Input Data

Input data includes the following items:

1. Unit weight, residual cohesion, residual friction angle, peak cohesion, and peak friction angle of fill material, bedding plane, and bedrock, respectively. Residual cohesion and friction angle is used for static stability analysis, where as peak cohesion and friction angle is for dynamic stability analysis.

2. Slope geometry and surcharge boundary loads.

3. Apparent dip of bedding plane can be specified in angular range (i.e., from 0 to 90 degrees.

4. Pseudo-static earthquake loading (an earthquake loading of 0.15 *i* was used in the analysis).

Seismic Discussion

Seismic stability analyses were approximated using a pseudo-static approach. The major difficulty in the pseudo-static approach arises from the appropriate selection of the seismic coefficient used in the analysis. The use of a static inertia force equal to this acceleration during an earthquake (rigid-body response) would be extremely conservative for several reasons including: (1) only low height, stiff/dense embankments or embankments in confined areas may respond essentially as rigid structures; (2) an earthquake's inertia force is enacted on a mass for a short time period. Therefore, replacing a transient force by a pseudo-static force representing the maximum acceleration is considered unrealistic; (3) assuming that total pseudo-static loading is applied evenly throughout the embankment for an extended period of time is an incorrect assumption, as the length of the failure surface analyzed is usually much greater than the wave length of seismic waves generated by earthquakes; and (4) the seismic waves would place portions of the mass in compression and some in tension, resulting in only a limited portion of the failure surface analyzed moving in a downslope direction, at any one instant of time.

The coefficients usually suggested by regulating agencies, counties and municipalities are in the range of 0.05g to 0.25g. For example, past regulatory guidelines within the city and county of Los Angeles indicated that the slope stability pseudostatic coefficient = 0.15 *i*.

The method developed by Krinitzsky, Gould, and Edinger (1993) which was in turn based on Taniguchi and Sasaki, 1986, (T&S, 1986), was referenced. This method is based on empirical data and the performance of existing earth embankments during seismic loading. Our review of "Guidelines for Evaluating and Mitigating Seismic Hazards in California (Davis, 1997) indicates the State of California recommends using pseudo-static coefficient of 0.15 for design earthquakes of M 8.25 or greater and using 0.1 for earthquake parameter M 6.5. Therefore, for conservatism a seismic coefficient of 0.12 *i* was used in our analysis.

<u>Output Information</u>

Output information includes:

1. All input data.

2. Factors of safety for the ten most critical surfaces for static and pseudo-static stability situation.

3. High quality plots can be generated. The plots include the slope geometry, the critical surfaces and the factor of safety.

4. Note, that in the analysis, a minimum of 100 trial surfaces were analyzed for each section for either static or pseudo-static analyses.

<u>Results of Slope Stability Calculation</u>

Table D-1shows parameters used in slope stability calculations. Summaries of the slope stability analysis are presented in Table D-2. Surficial slope stability calculations are presented as Figure D-2. Detailed output information is presented in Figures D-3 and D-4. The Geologic Cross-Sections are presented on Plate 2. The locations of the geologic cross-sections are presented on Plate 1.

TABLE D-1

SOIL PARAMETERS USED

SOIL MATERIALS	PEAK VALUES	
	C (psf)	Φ (degrees)
Compacted Fill	300	28
Sandy Claystone Bedrock	115	30
Sandstone Bedrock	280	34

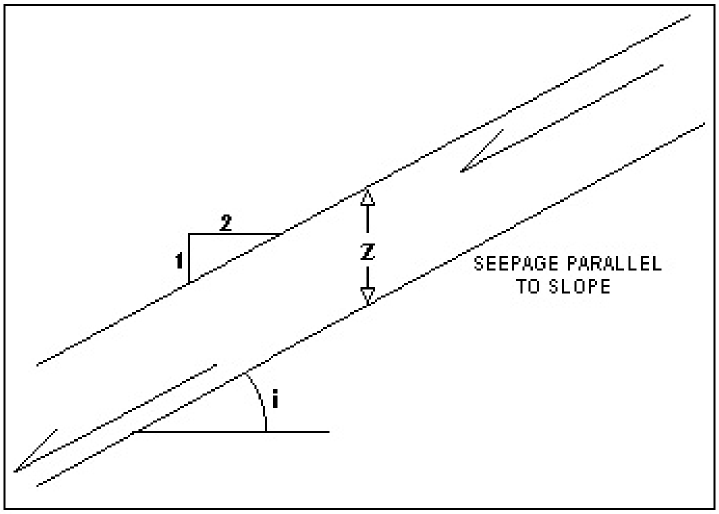
TABLE D-2

SUMMARY OF SLOPE ANALYSIS

STABILITY	SLOPE CONFIGURATION	SLOPE GRADIENT	FACTORS OF SAFETY		REMARKS
			STATIC	SEISMIC	
Gross A - A’	Existing Cut Slope	2:1	2.27	1.64	Simplified Janbu
Gross B - B’	Existing Cut Slope	2:1	1.88	1.36	Simplified Janbu

Figure D-1

SURFICIAL SLOPE STABILITY ANALYSIS



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W.O 6947-A-SC

	COMPACTED FILL SLOPE	CUT SLOPE
Depth of Saturation (z) =	4 ft	4 ft
Slope Angle (i) (2:1 slopes)	26.6˚	26.6˚
Unit Weight of Water (γ_W)	62.4 pcf	62.4 pcf
Saturated Unit of Soil (γ_{SAT})	132 pcf	129 pcf
Apparent Angle of Internal Friction (ϕ)	28˚	30˚
Apparent Cohesion (C) =	300 psf	280 psf

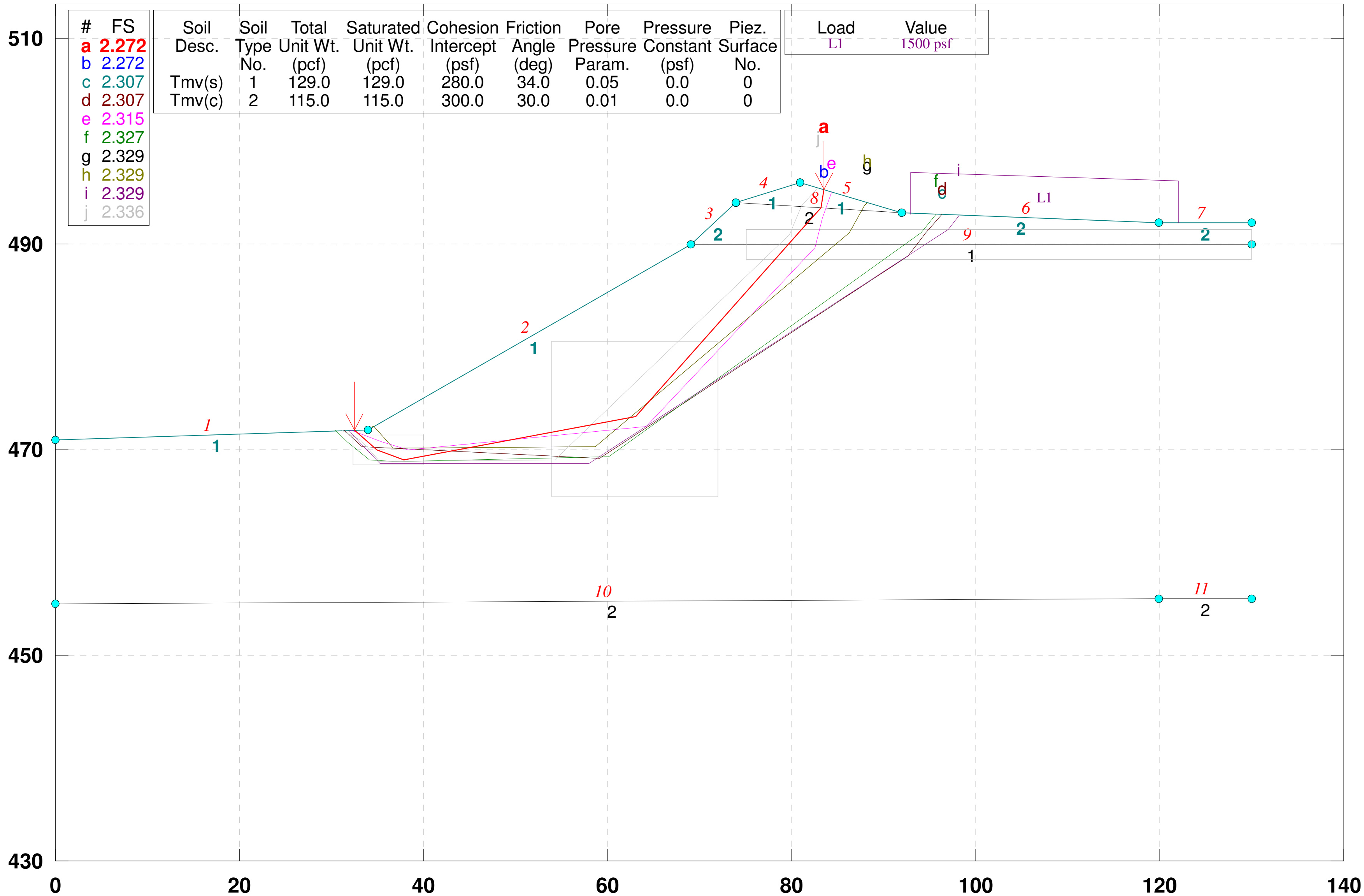
Fs, Static Safety Factor = $\dfrac{z\,(\gamma_{SAT}-\gamma_w)\,\mathrm{Cos}^2(i)\,\mathrm{Tan}\,(\phi) + C}{z\,(\gamma_{SAT})\,\mathrm{Sin}\,(i)\,\mathrm{Cos}\,(i)}$

DEPTH OF SATURATION	STATIC F.S.	STATIC F.S.
	FILL	CUT
4 FEET	1.52	1.58

Figure D-2

6947-A-SC, Vista Azul Cross-Section A-A' (North End)

z:\shared\word perfect data\carlsbad\6900\6947 rina-la mesa\slope stability\6947-a-sc-static-b.pl2 Run By: GeoSoils, Inc. 9/10/2015 02:22PM



GSTABL7 v.2 FSmin=2.272

Safety Factors Are Calculated By The Simplified Janbu Method

W.O. 6947-A-SC
PLATE D-3

6947-A-SC, Vista Azul Cross-Section B-B' (East End)

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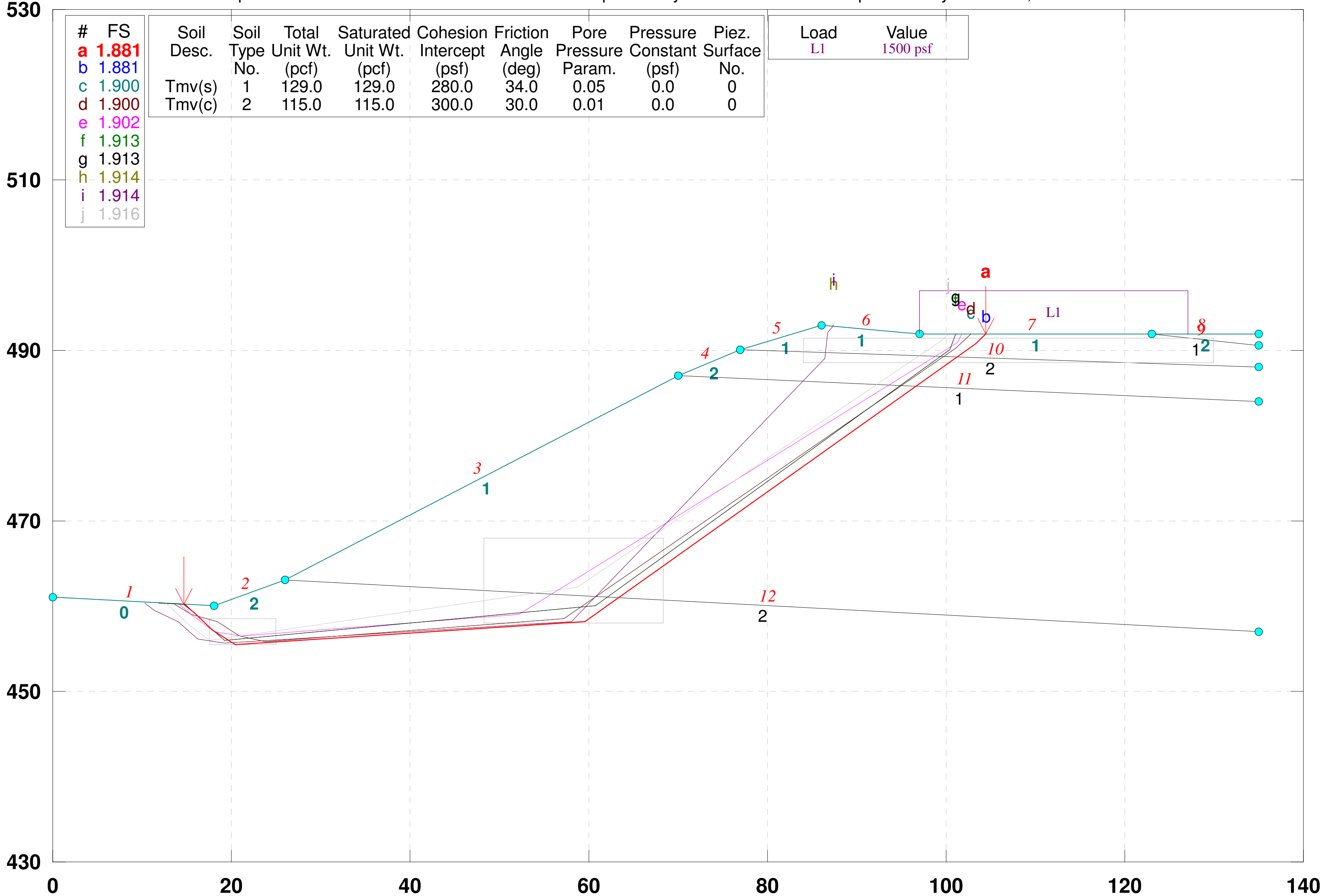
#	FS
a	1.881
b	1.881
c	1.900
d	1.900
e	1.902
f	1.913
g	1.913
h	1.914
i	1.914
j	1.916

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Tmv(s)	1	129.0	129.0	280.0	34.0	0.05	0.0	0
Tmv(c)	2	115.0	115.0	300.0	30.0	0.01	0.0	0

Load	Value
L1	1500 psf



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Safety Factors Are Calculated By The Simplified Janbu Method

W.O. 6947-A-SC
PLATE D-4

6947-A-SC, Vista Azul Cross-Section A-A' (North End) Seismic

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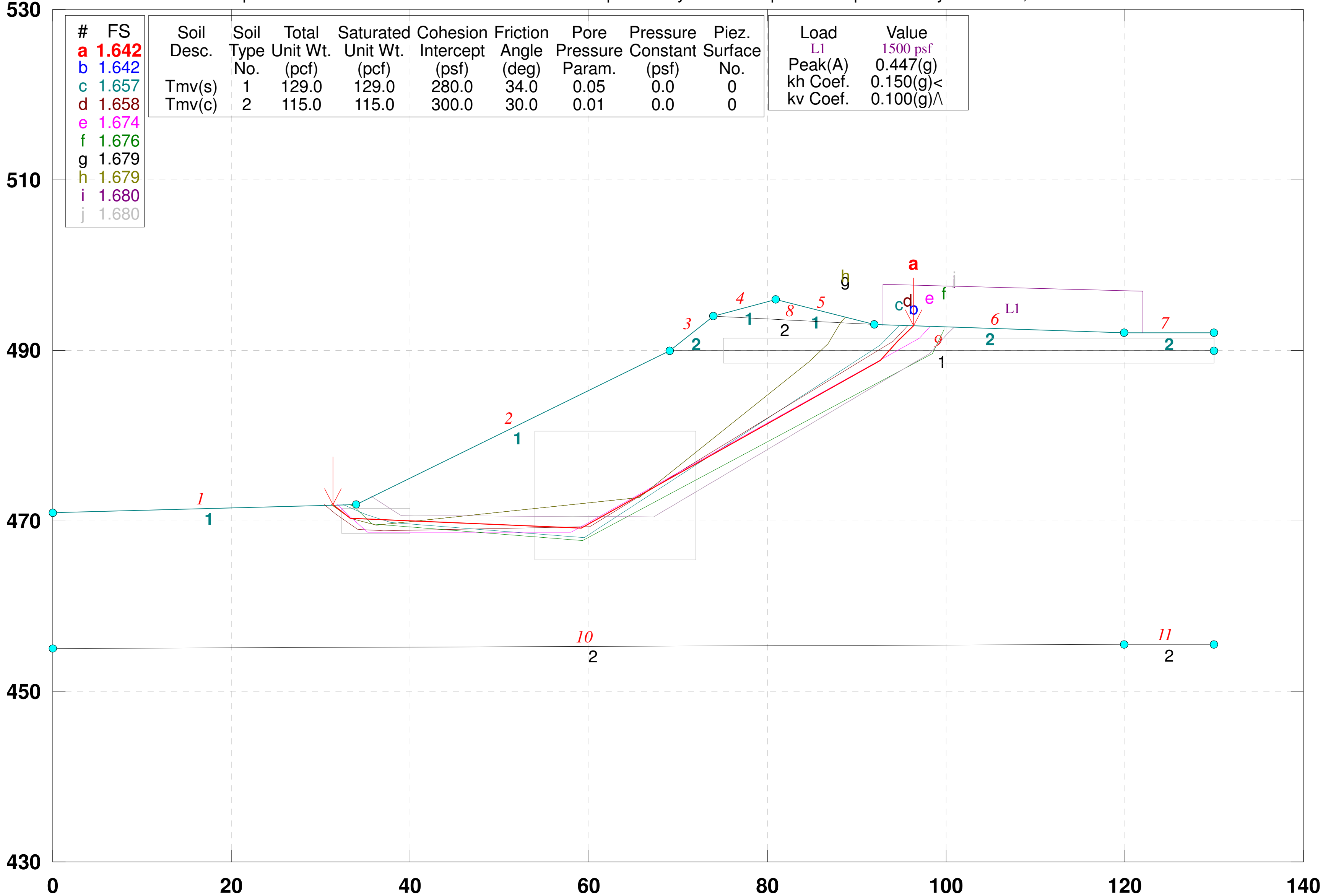


GSTABL7 v.2 FSmin=1.642

Safety Factors Are Calculated By The Simplified Janbu Method

W.O. 6947-A-SC
PLATE D-5

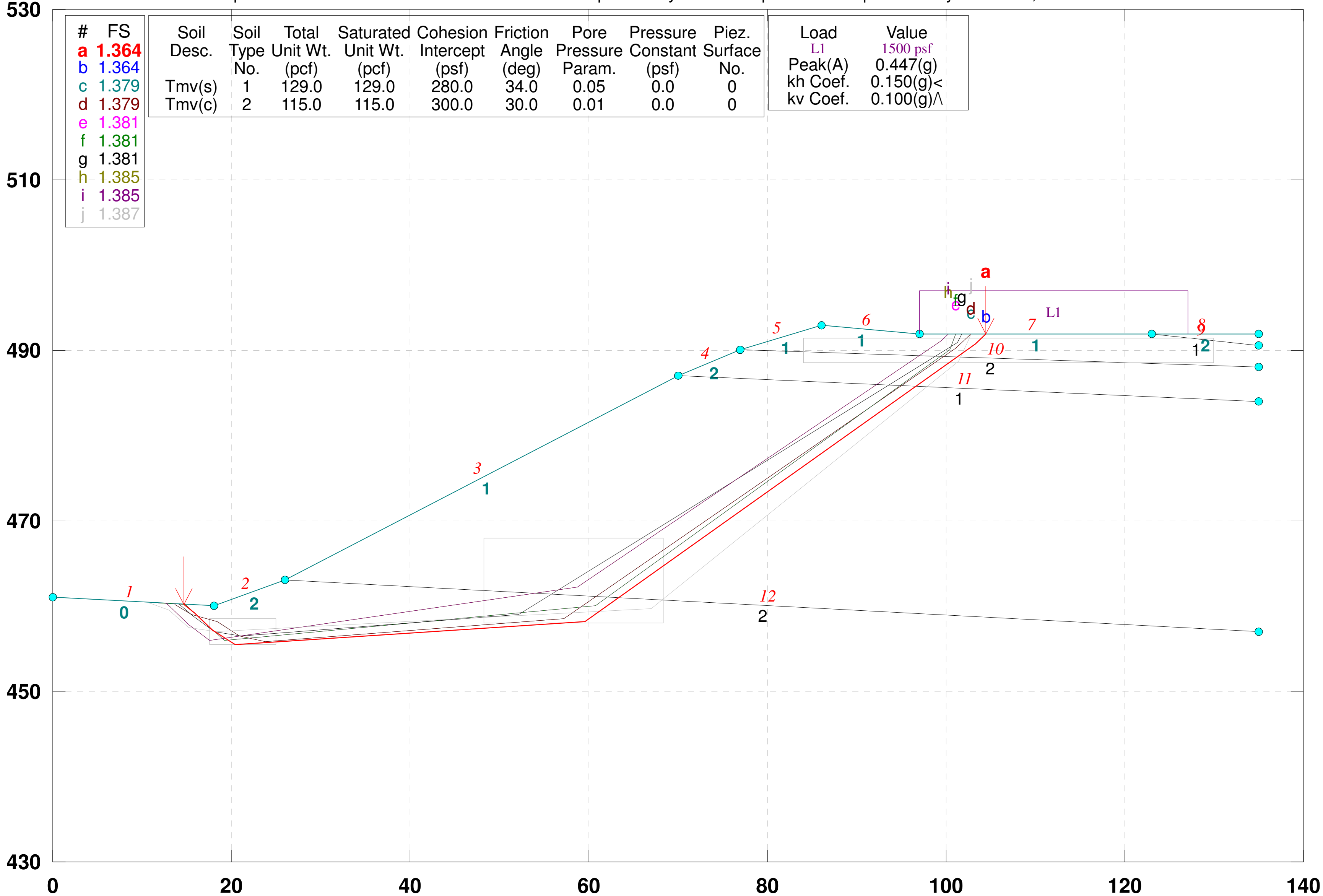
6947-A-SC, Vista Azul Cross-Section B-B' (East End) Seismic

z:\shared\word perfect data\carlsbad\6900\6947 rina-la mesa\slope stability\6947-a-sc-pstatic-b-b'-b.pl2 Run By: GeoSoils, Inc. 9/30/2015 05:42PM

#	FS
a	1.364
b	1.364
c	1.379
d	1.379
e	1.381
f	1.381
g	1.381
h	1.385
i	1.385
j	1.387

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Tmv(s)	1	129.0	129.0	280.0	34.0	0.05	0.0	0
Tmv(c)	2	115.0	115.0	300.0	30.0	0.01	0.0	0

Load	Value
L1	1500 psf
Peak(A)	0.447(g)
kh Coef.	0.150(g)<
kv Coef.	0.100(g)/\



GSTABL7 v.2 FSmin=1.364

Safety Factors Are Calculated By The Simplified Janbu Method

W.O. 6947-A-SC
PLATE D-6

APPENDIX E

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), will experience some movement or tilting. While the pool/spa shell or coping may not necessarily crack, the levelness of the pool/spa will likely tilt toward the slope, and may not be 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.

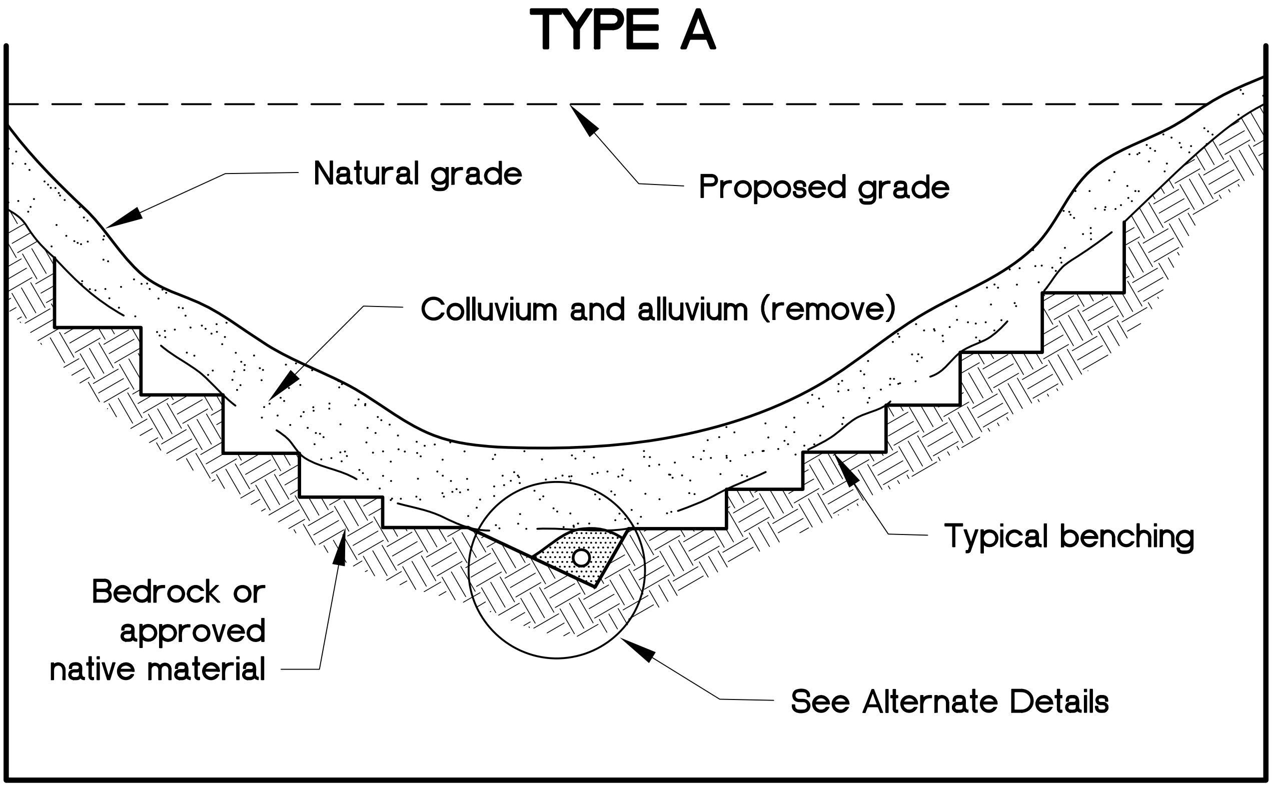
Trench and Vertical Excavation

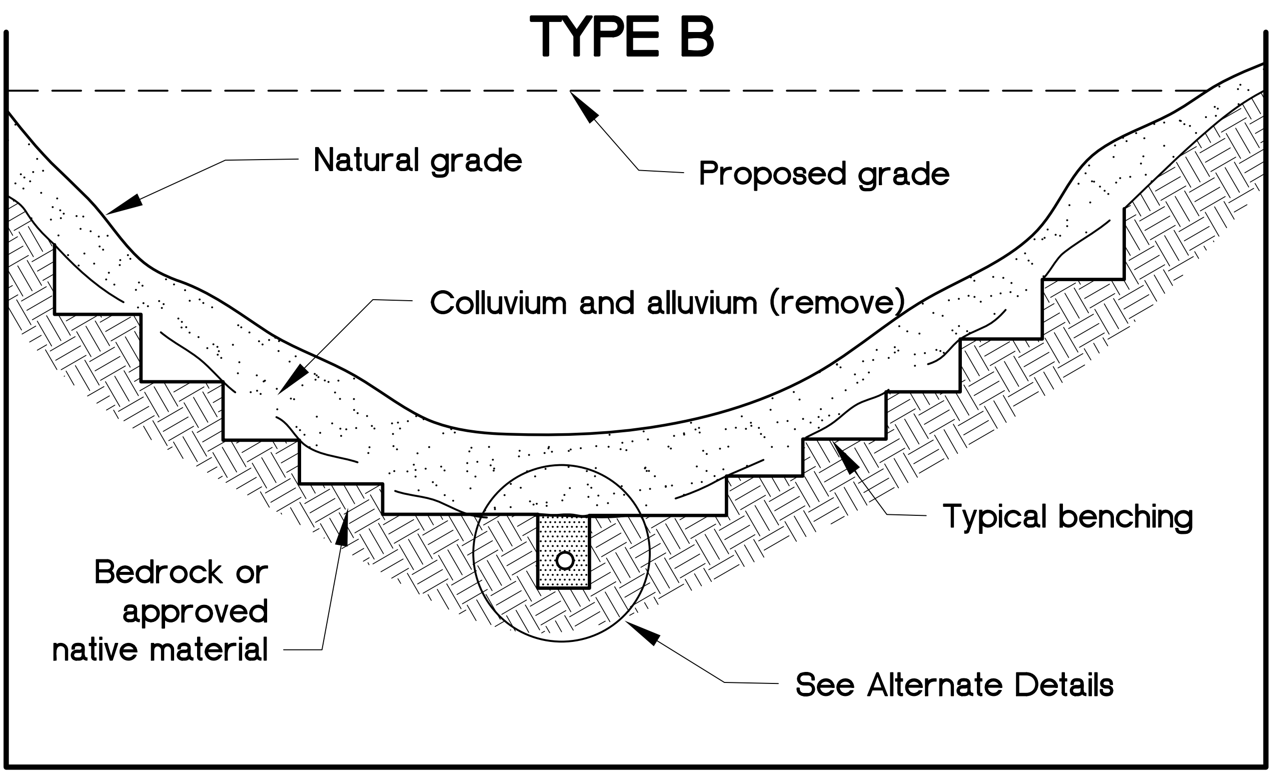
It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Our personnel are directed not to enter any excavation or vertical cut which: 1) is 5 feet or deeper unless shored or laid back; 2) displays any evidence of instability, has any loose rock or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



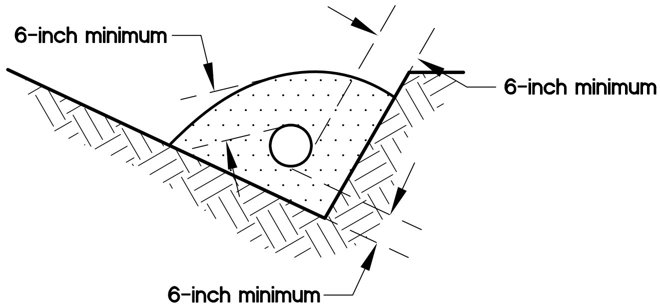


Selection of alternate subdrain details, location, and extent of subdrains should be evaluated by the geotechnical consultant during grading.

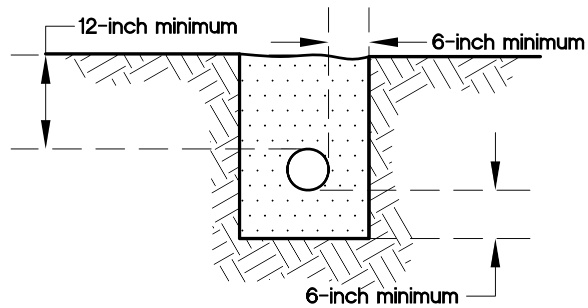
GeoSoils, Inc.

CANYON SUBDRAIN DETAIL

Plate E-1



A-1



B-1

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

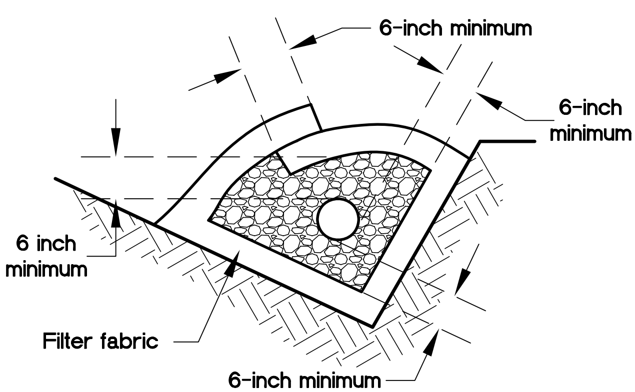
Perforated pipe: 6-inch-diameter ABS or PVC pipe or approved substitute with minimum 8 perforations ($\frac{1}{4}$-inch diameter) per lineal foot in bottom half of pipe (ASTM D-2751, SDR-35, or ASTM D-1527, Schd. 40).

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

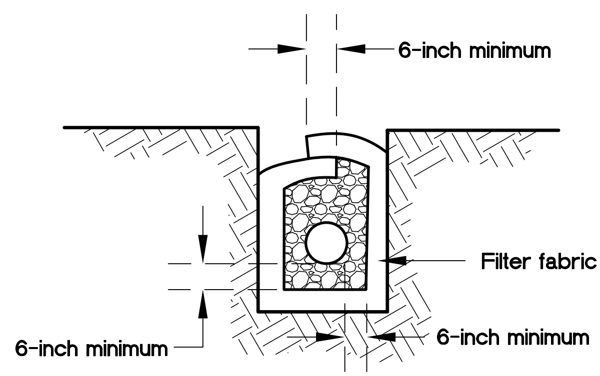
FILTER MATERIAL

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



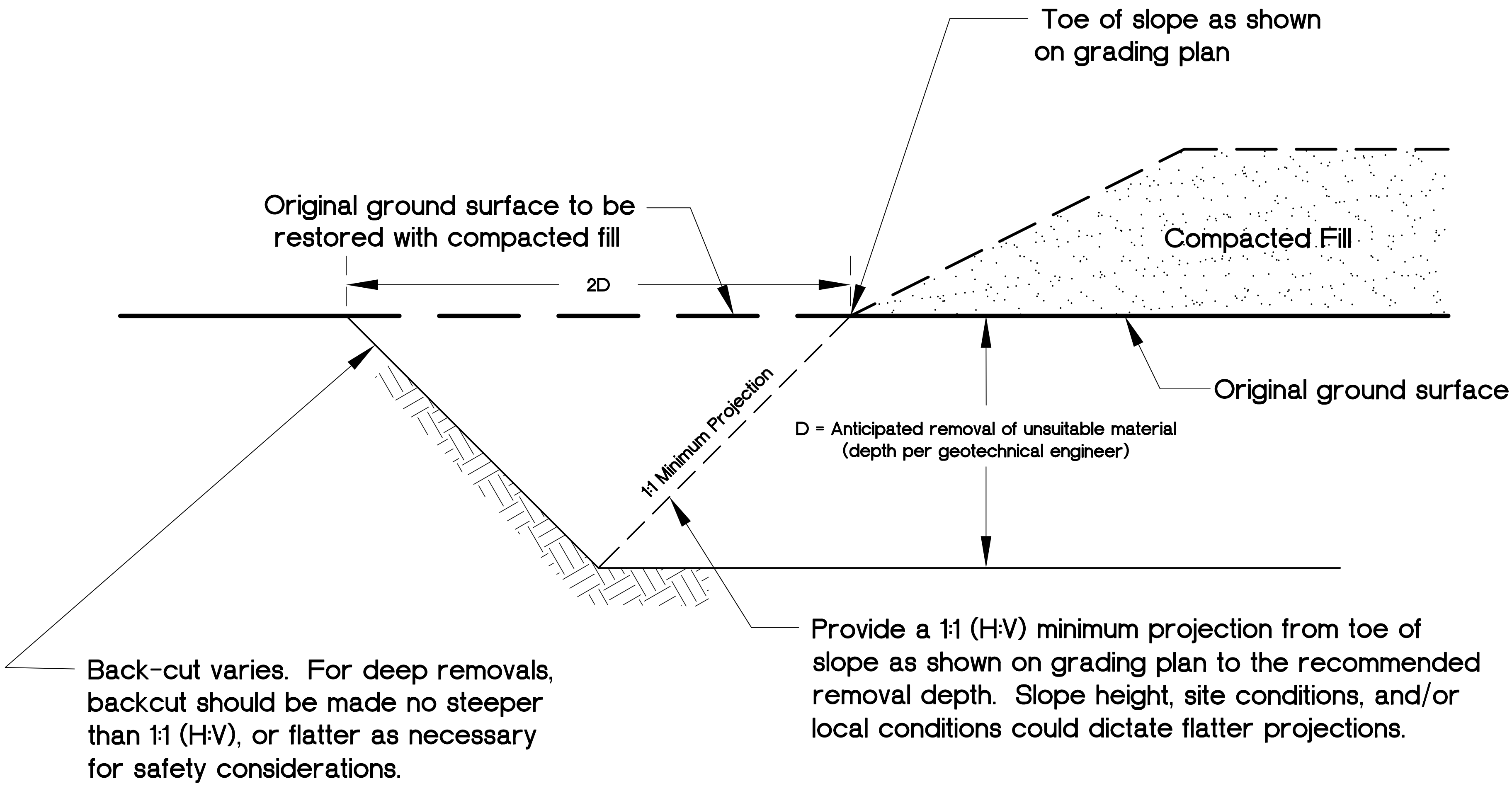
A-2



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.

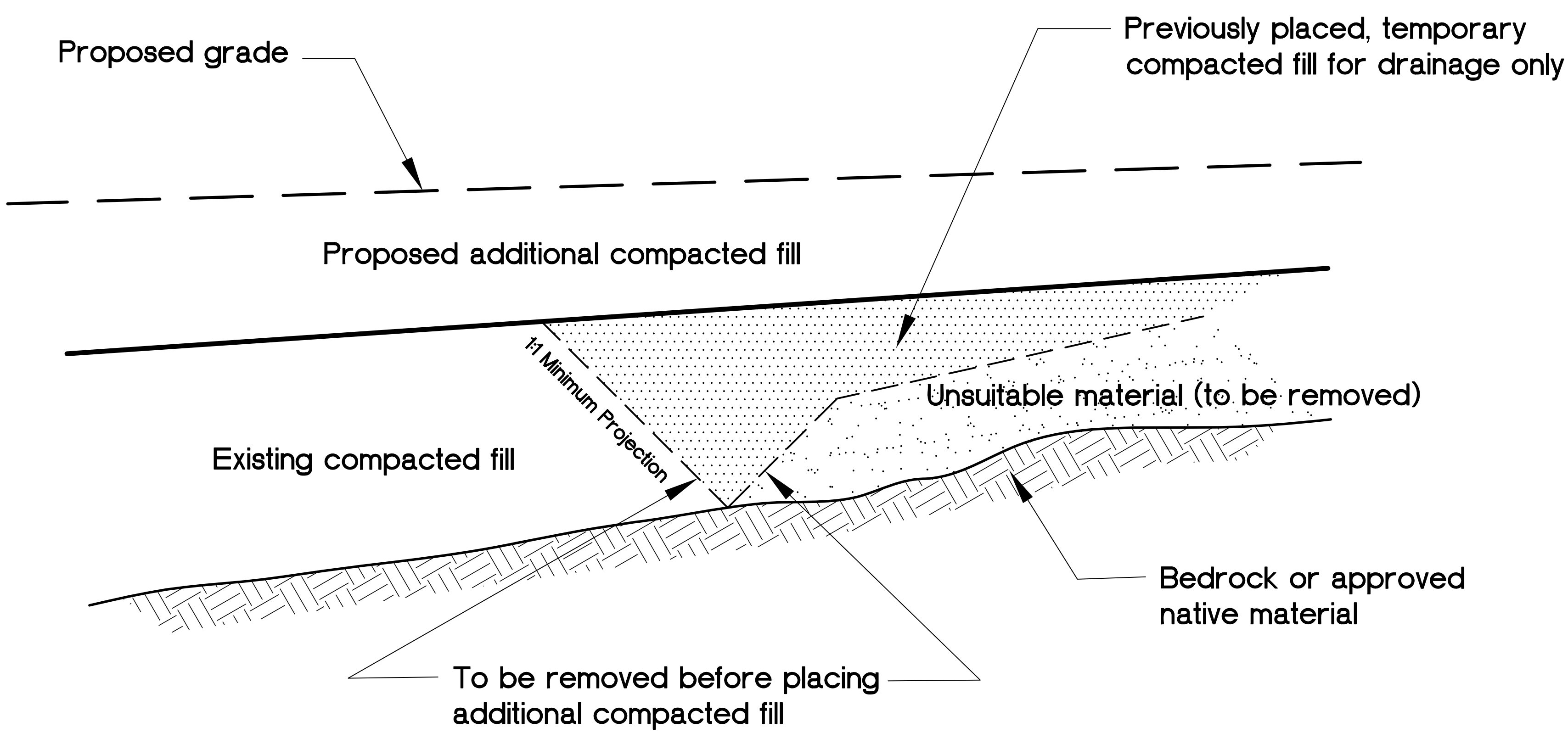
ALTERNATE 2: PERFORATED PIPE, GRAVEL, AND FILTER FABRIC



GeoSoils, Inc.

FILL SLOPE TOEING OUT ON FLAT ALLUVIATED CANYON DETAIL

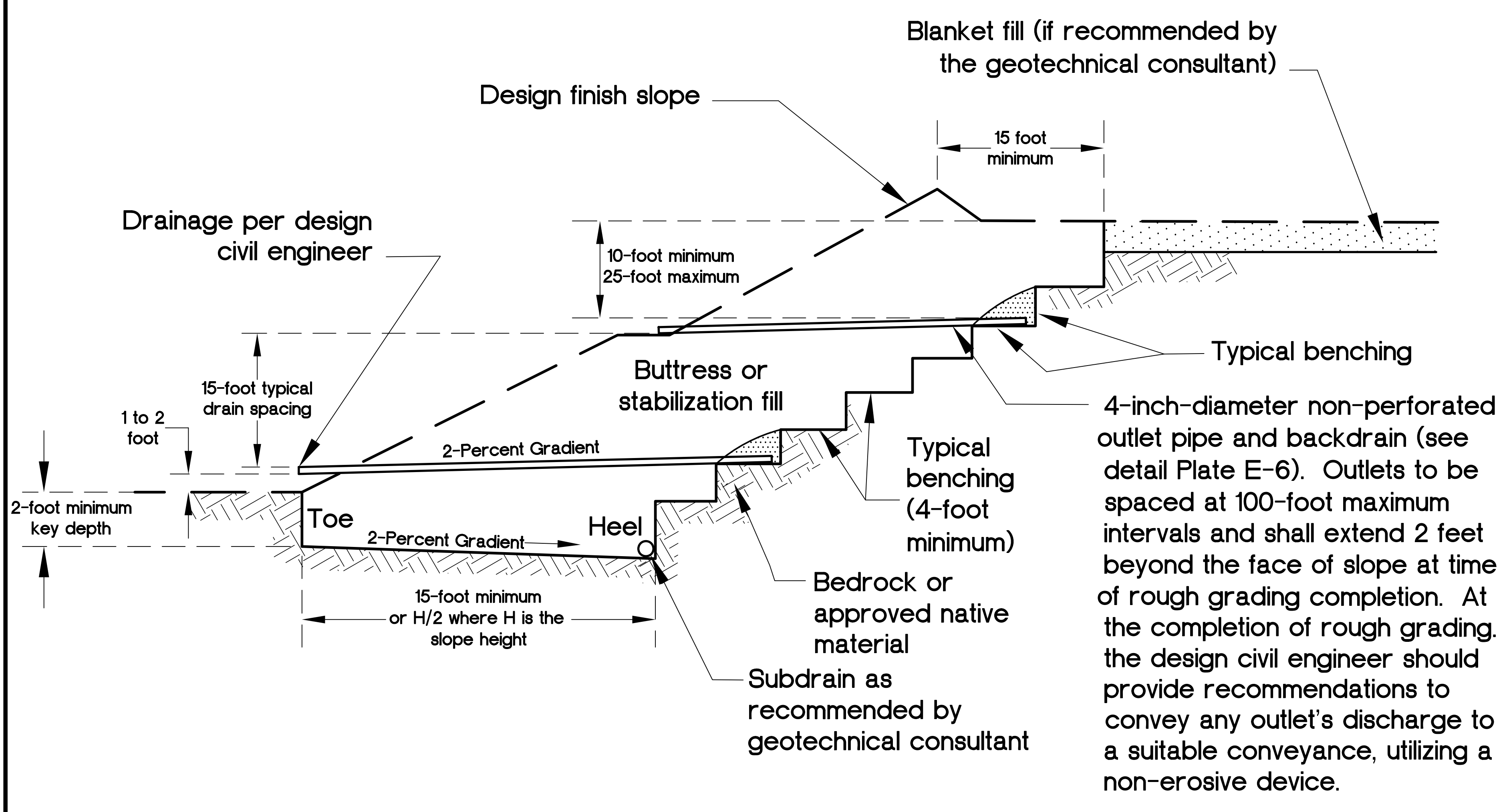
Plate E-3



GeoSoils, Inc.

REMOVAL ADJACENT TO EXISTING FILL ADJOINING CANYON FILL DETAIL

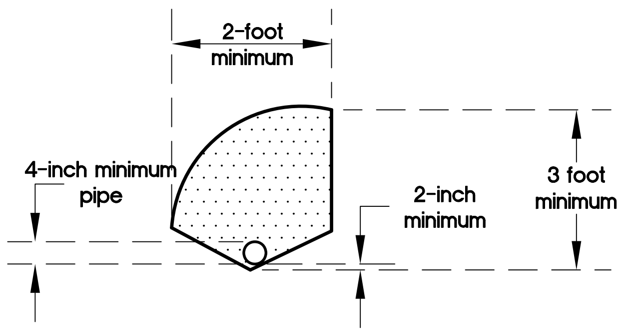
Plate E-4

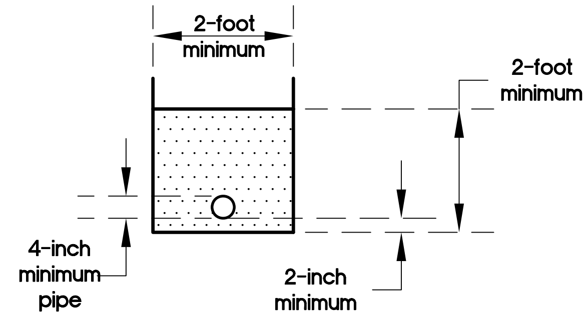


GeoSoils, Inc.

TYPICAL STABILIZATION / BUTTRESS FILL DETAIL

Plate E-5





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

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

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

Notes:

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

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

Sieve Size	Percent Passing
1 inch	100
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

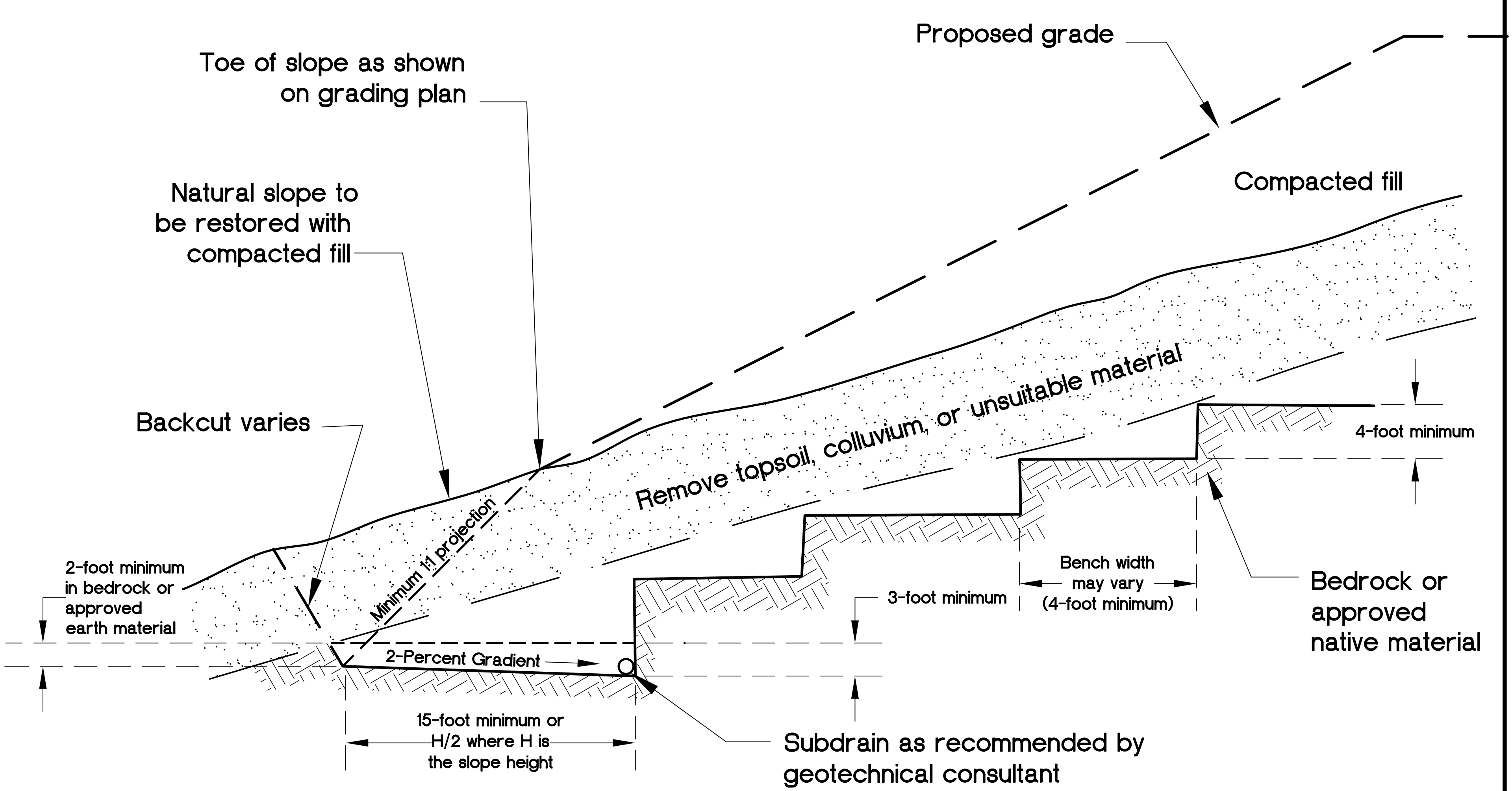
Gravel shall be of the following specification or an approved equivalent.

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

GeoSoils, Inc.

TYPICAL BUTTRESS SUBDRAIN DETAIL

Plate E-6



NOTES:

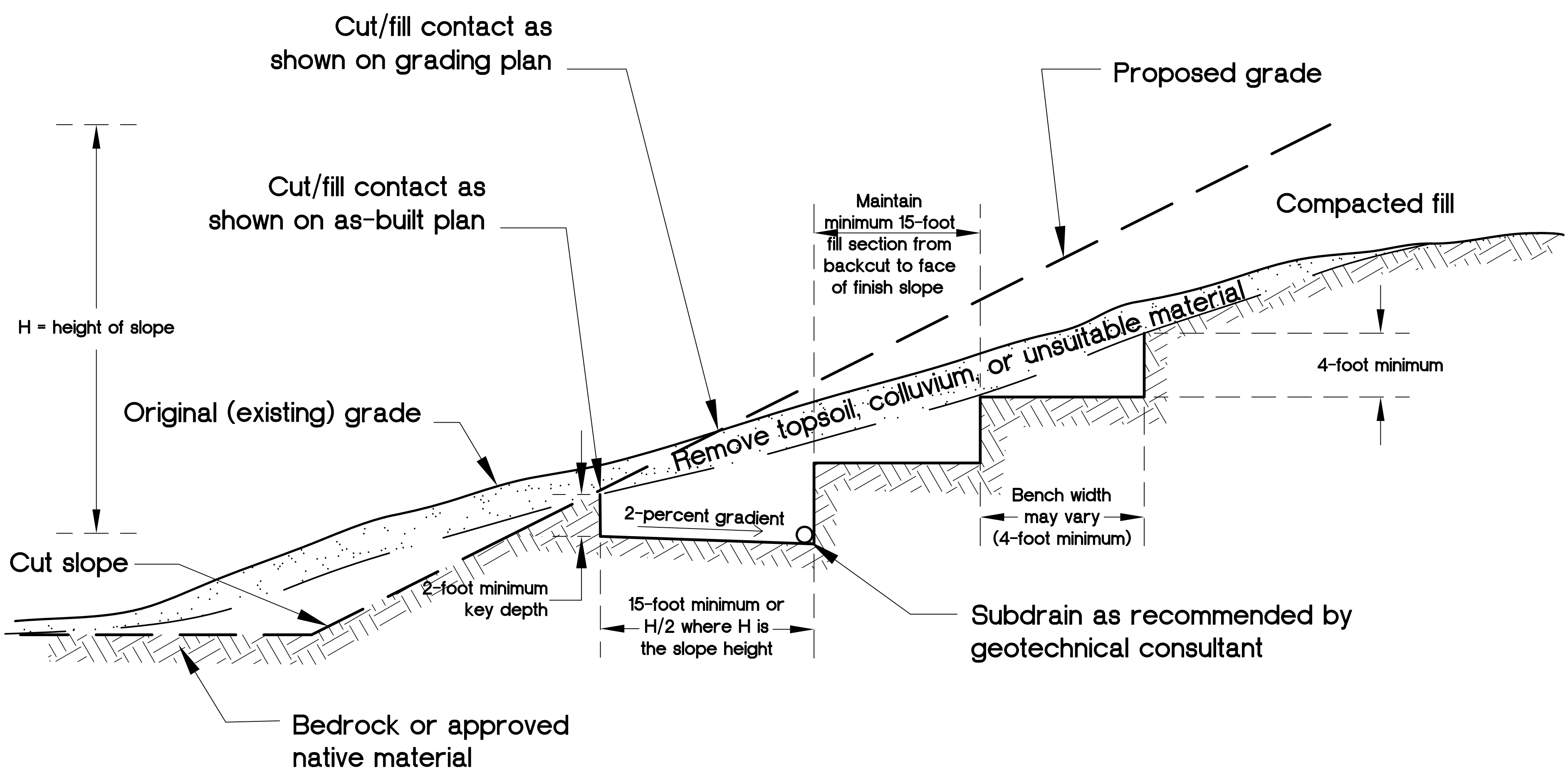
1. Where the natural slope approaches or exceeds the design slope ratio, special recommendations would be provided by the geotechnical consultant.

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 E-7

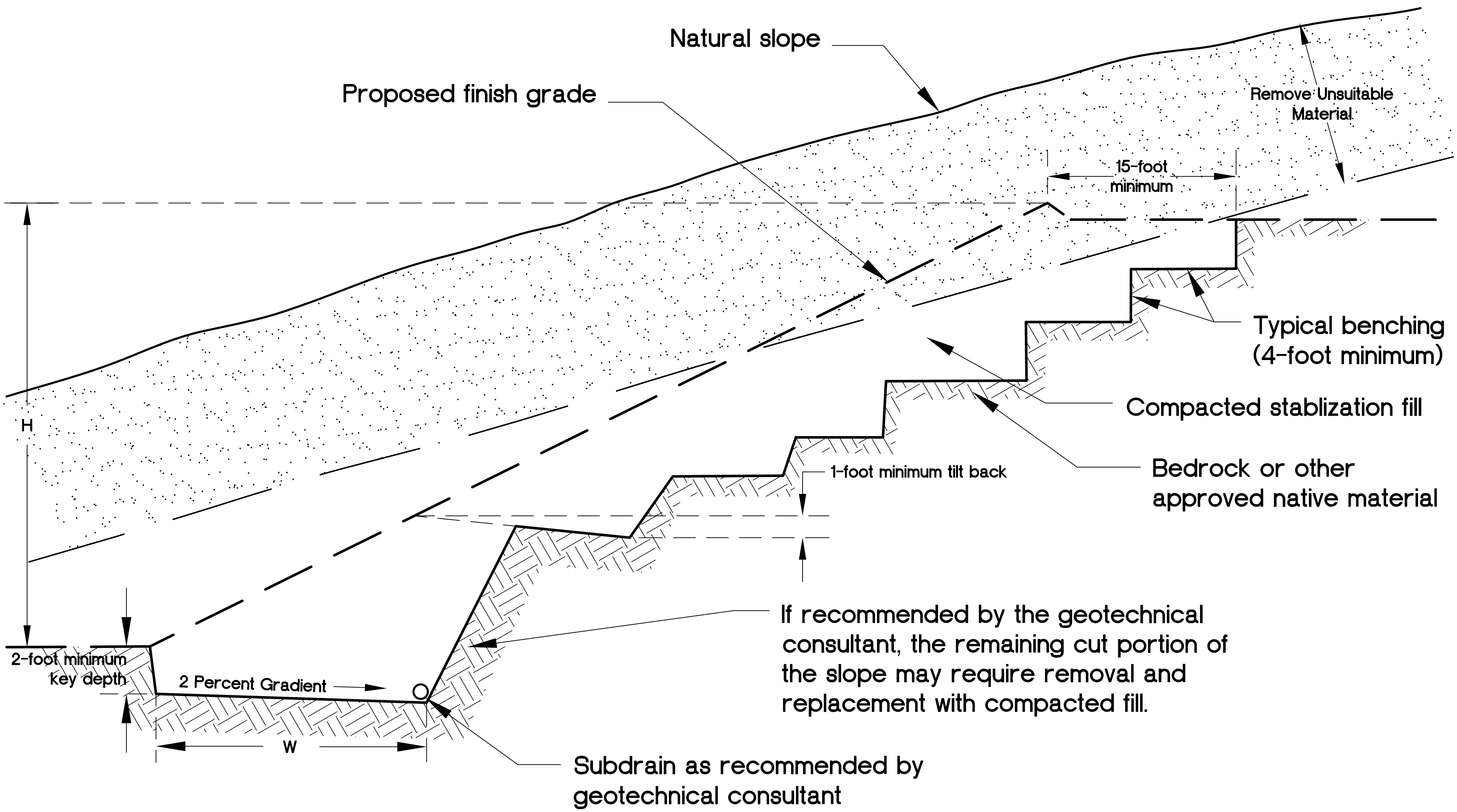


NOTE: The cut portion of the slope should be excavated and evaluated by the geotechnical consultant prior to construction of the fill portion.

GeoSoils, Inc.

FILL OVER CUT DETAIL

Plate E-8

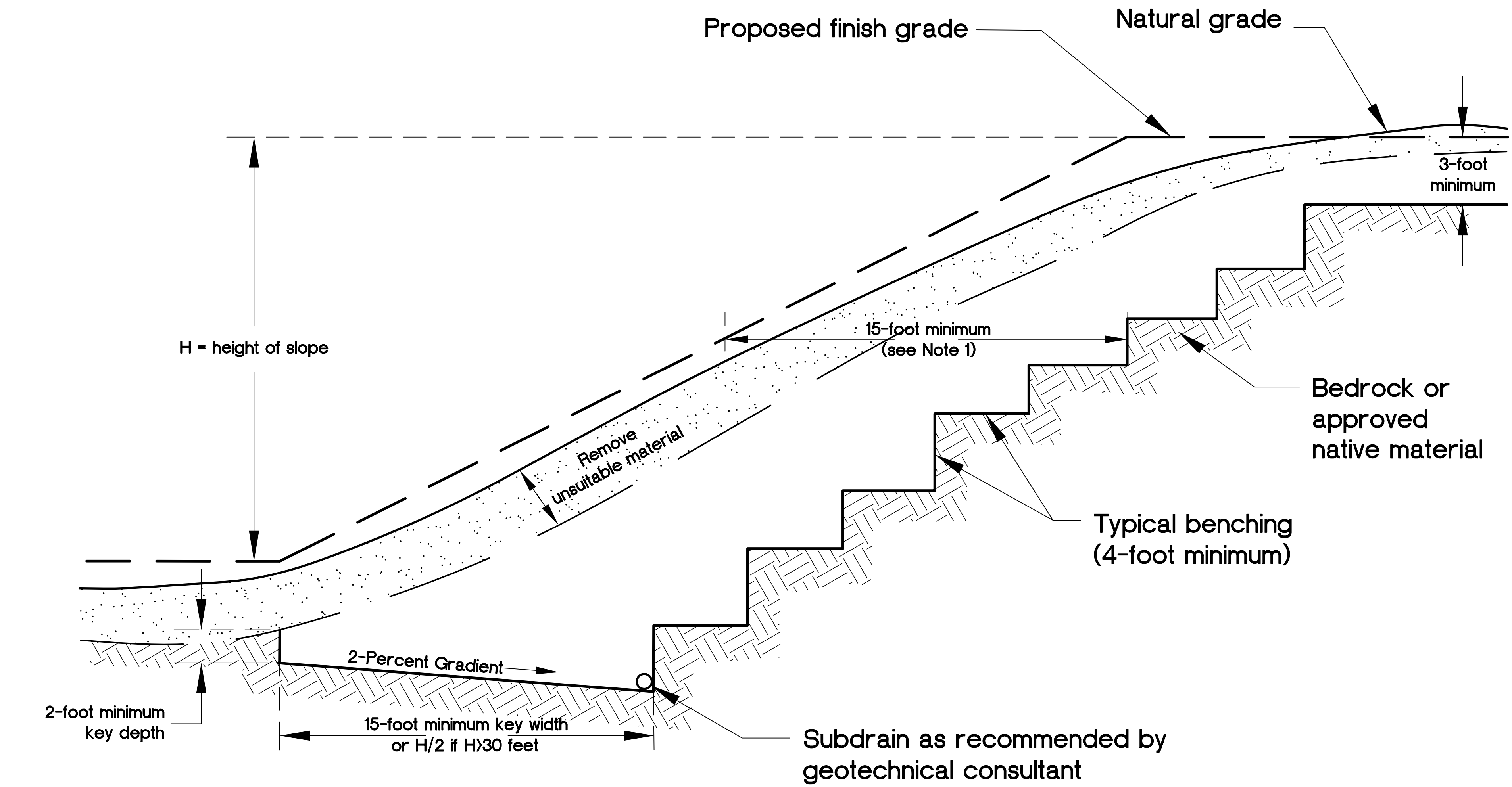


NOTES:

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 E-9



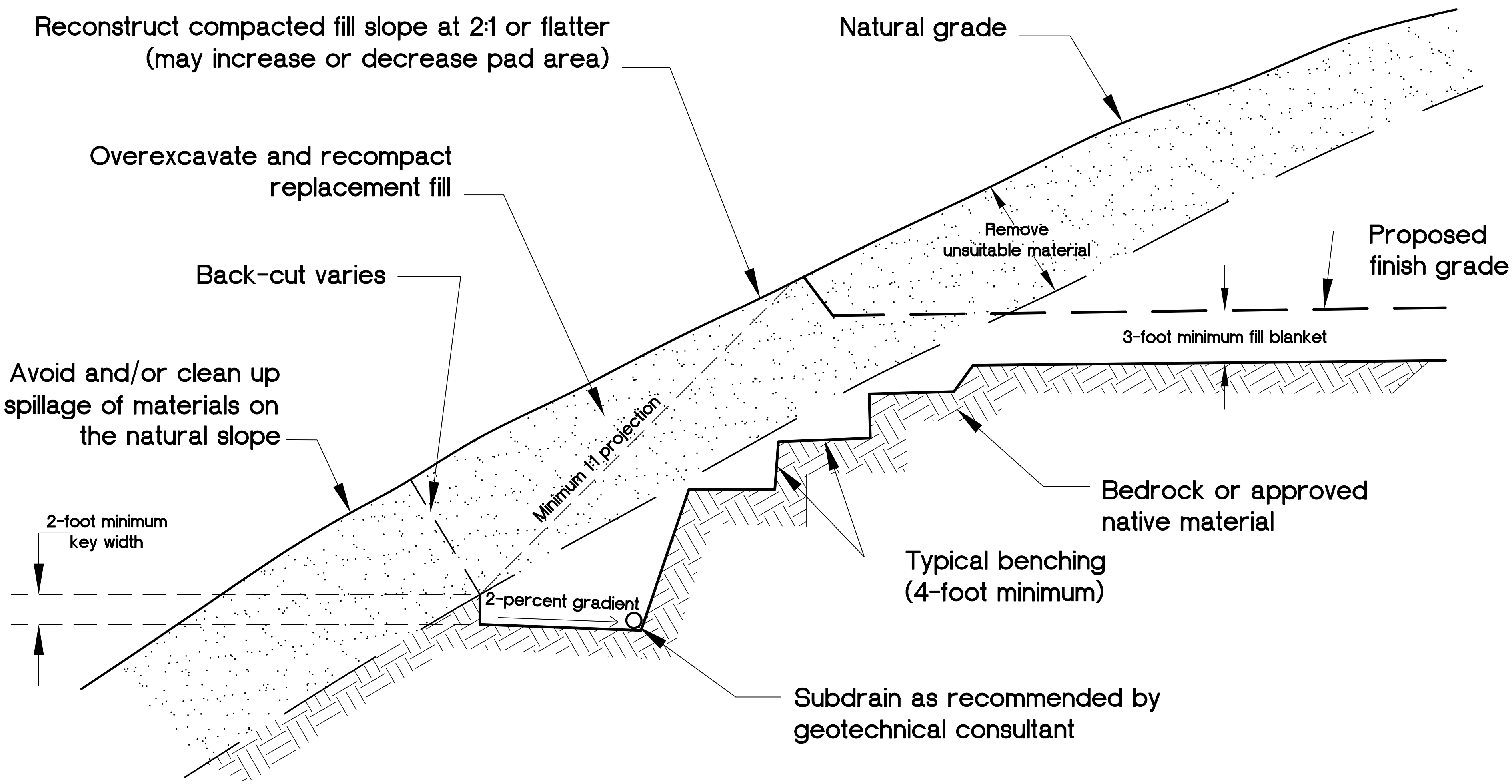
NOTES:

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 E-10



NOTES:

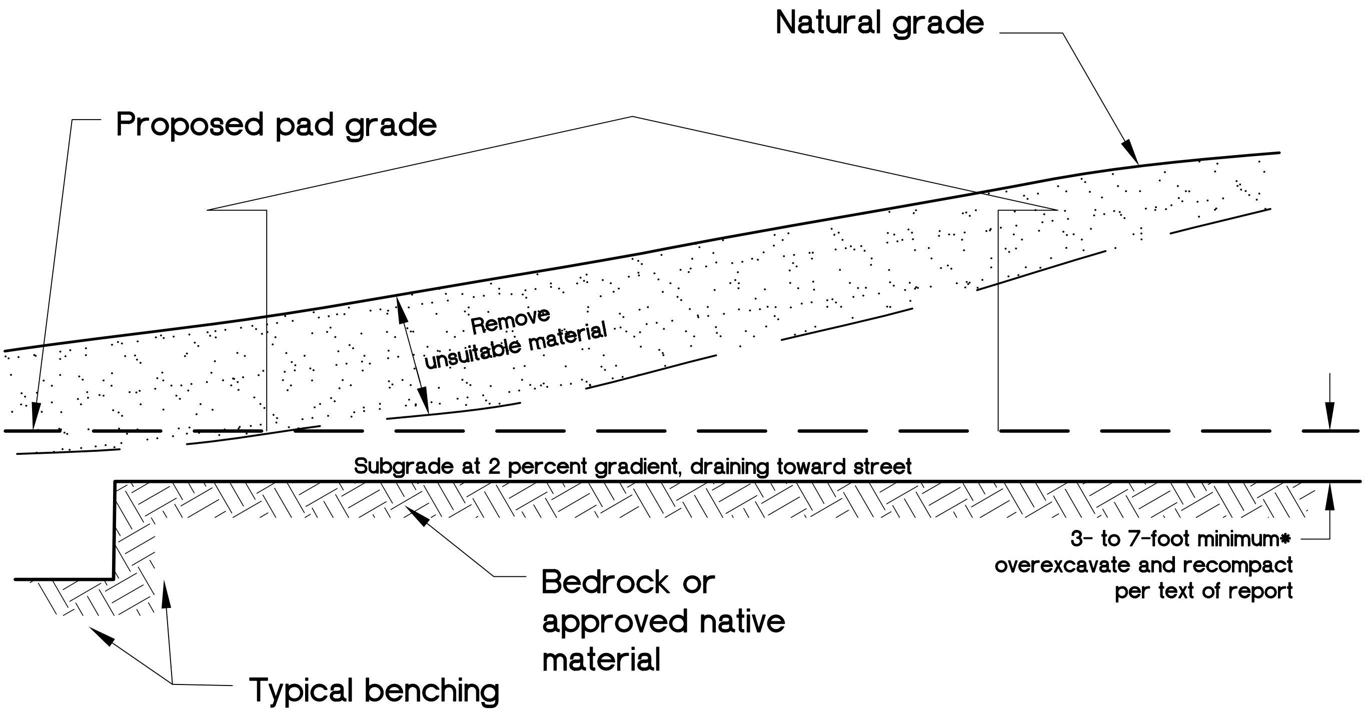
1. Subdrain and key width requirements will be evaluated based on exposed subsurface conditions and thickness of overburden.

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

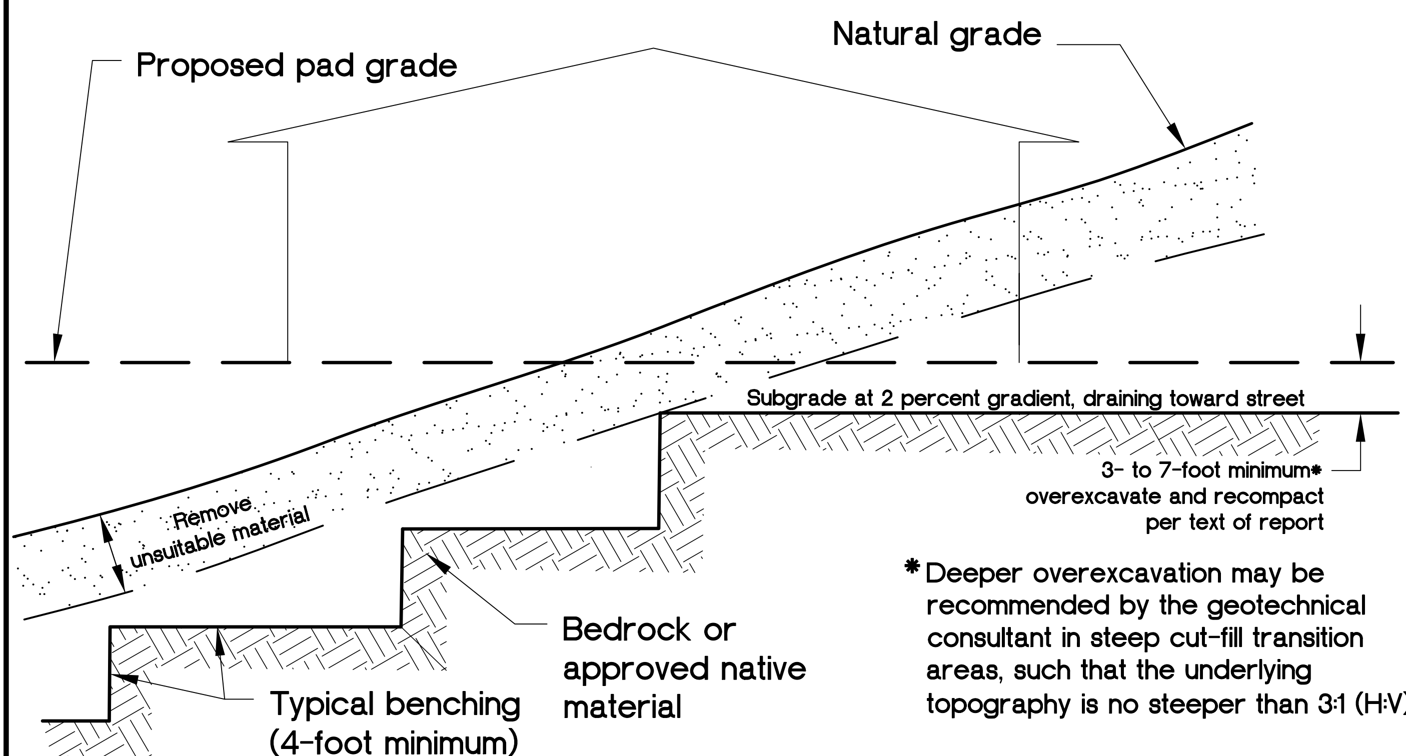
GeoSoils, Inc.

DAYLIGHT CUT LOT DETAIL

Plate E-11



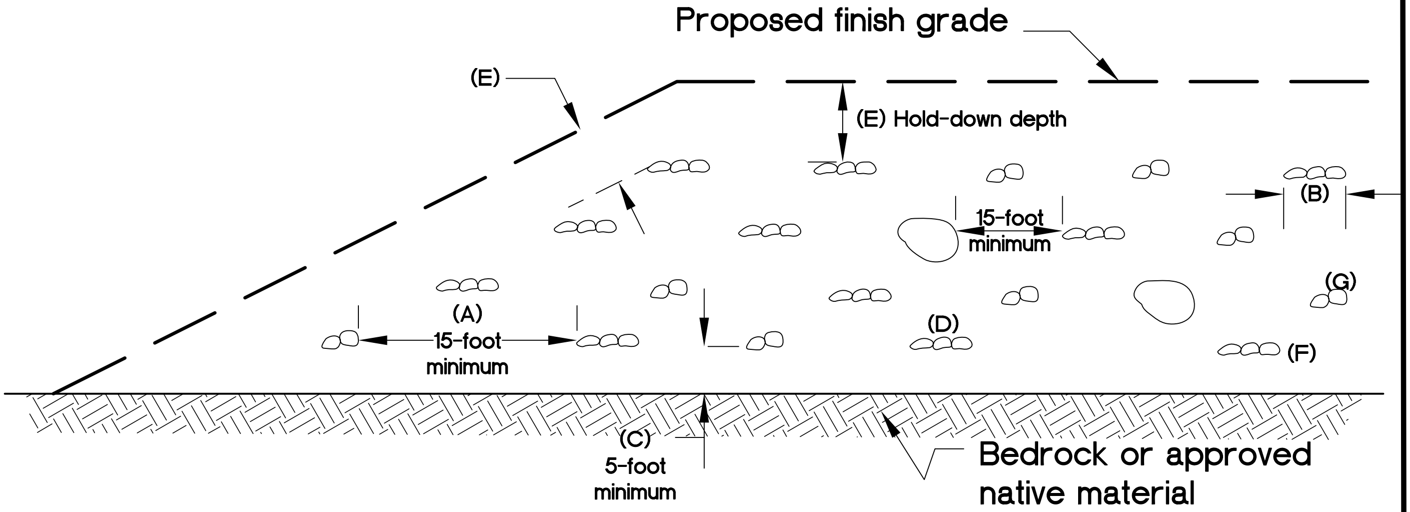
CUT LOT OR MATERIAL-TYPE TRANSITION



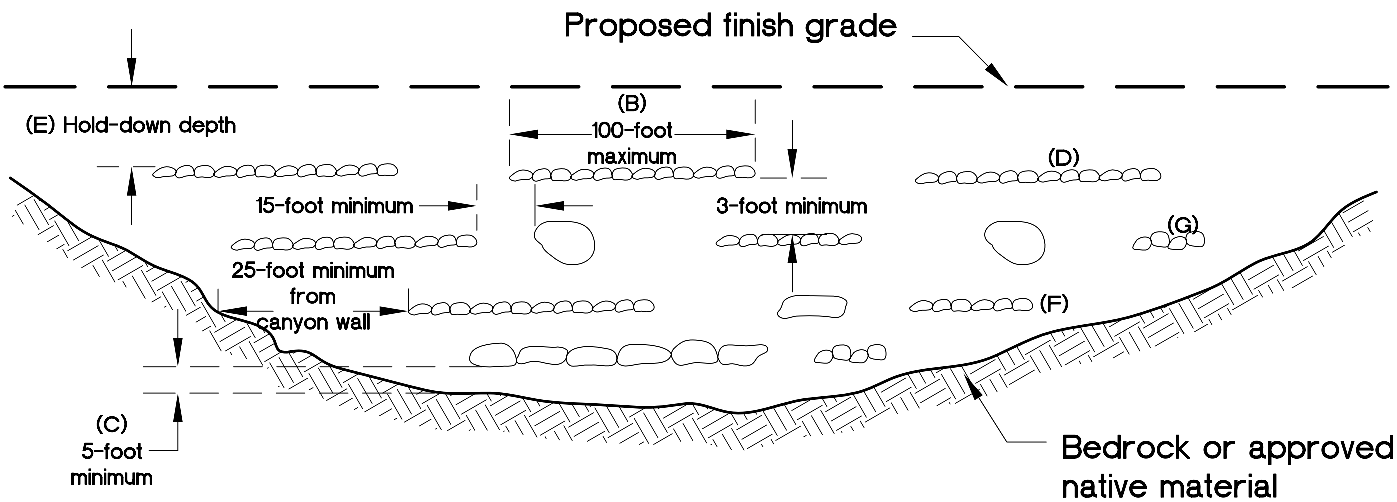
CUT-FILL LOT (DAYLIGHT TRANSITION)

GeoSoils, Inc.	TRANSITION LOT DETAILS	Plate E-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 slze and type of equipment. Length of windrow shall be no greater than 100 feet.

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

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

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

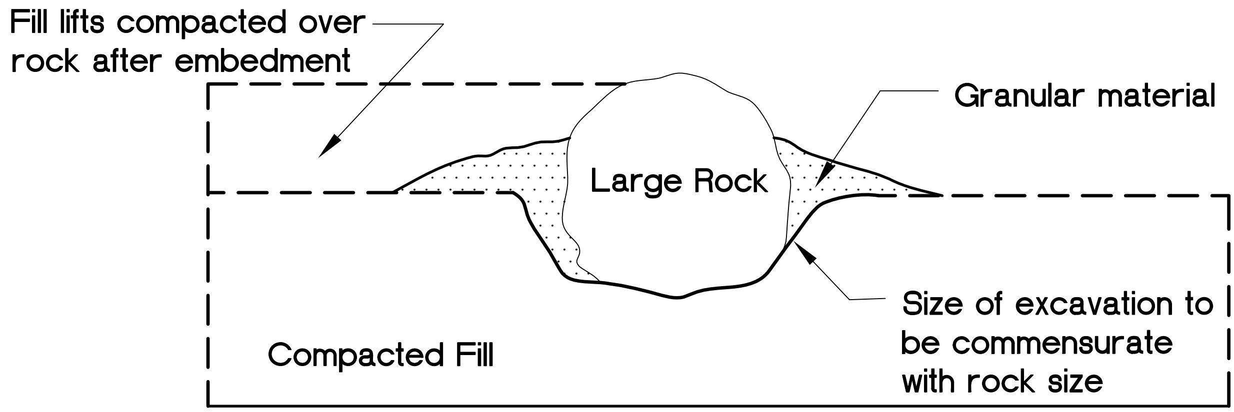
F. All fill over and around rock windrow shall be compacted to at least 90 percent relative compaction or as recommended.

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

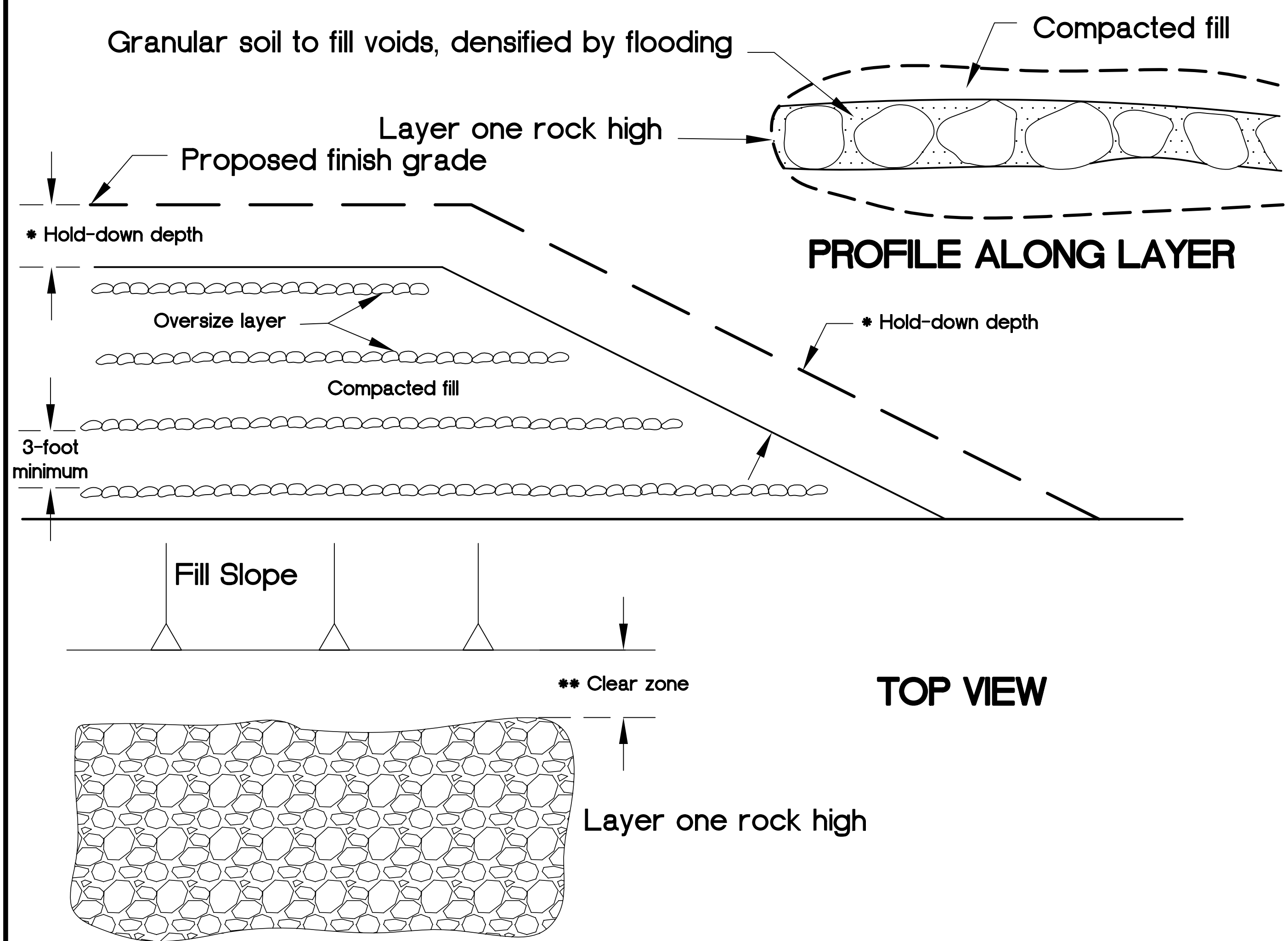
VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE
ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED

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

ROCK DISPOSAL PITS

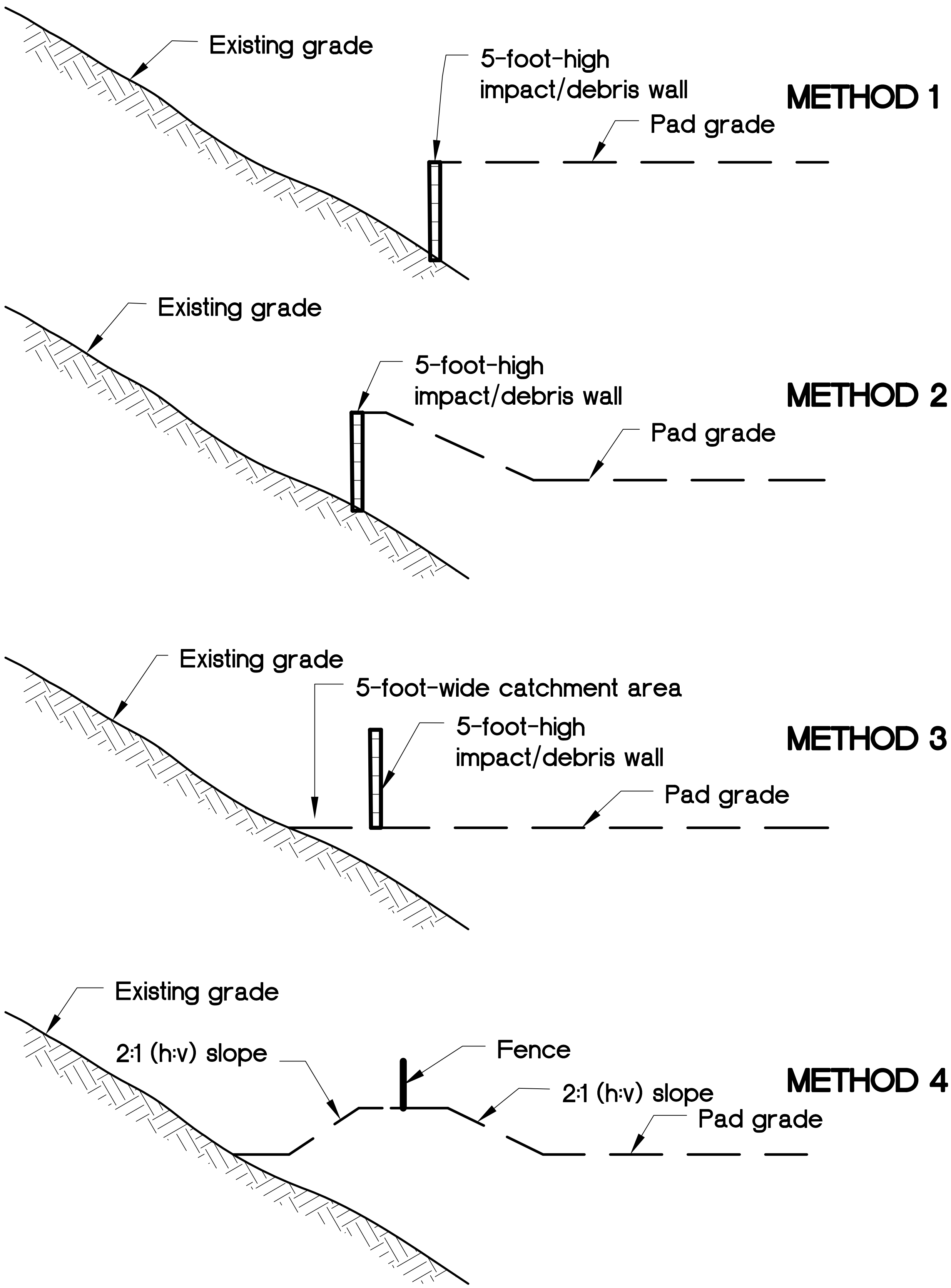


ROCK DISPOSAL LAYERS



* 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 E-14

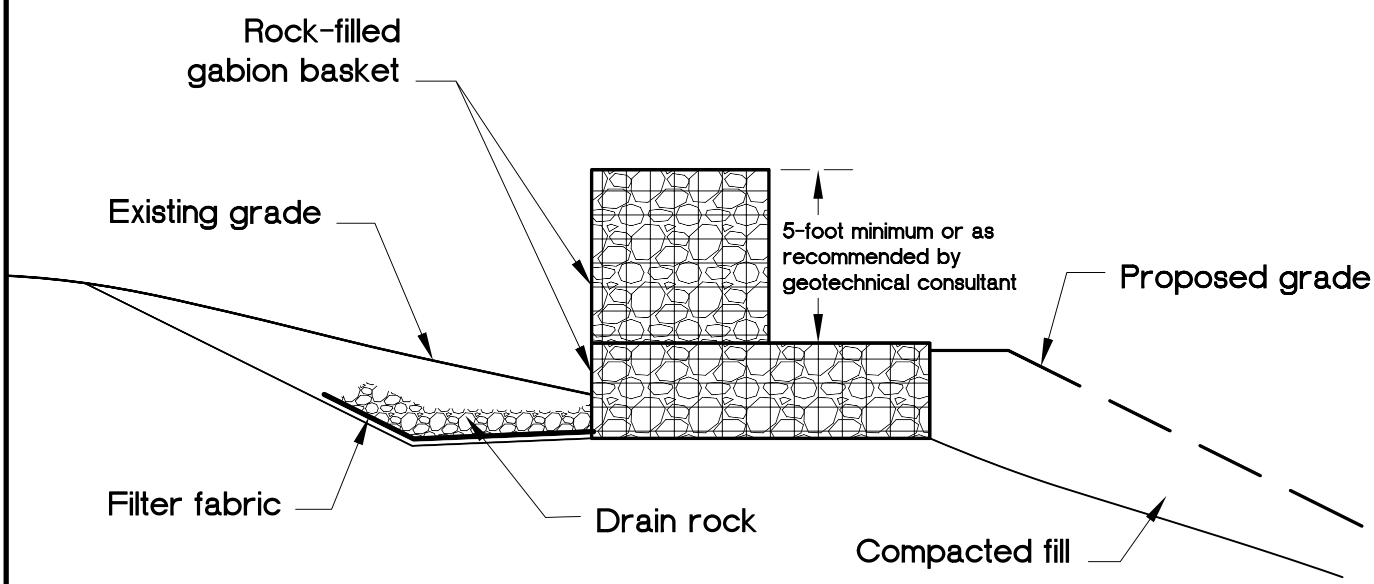


NOT TO SCALE

GeoSoils, Inc.

DEBRIS DEVICE CONTROL METHODS DETAIL

Plate E-15



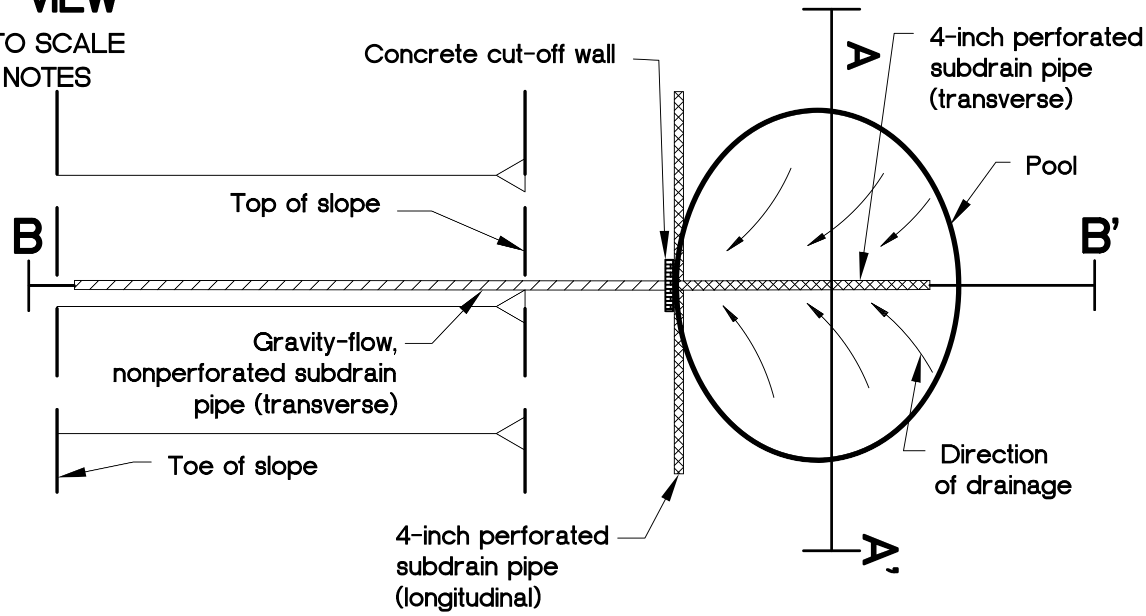
Gabion impact or diversion wall should be constructed at the base of the ascending slope subject to rock fall. Walls need to be constructed with high segments that sustain impact and mitigate potential for overtopping, and low segment that provides channelization of sediments and debris to desired depositional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

From GSA, 1987

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

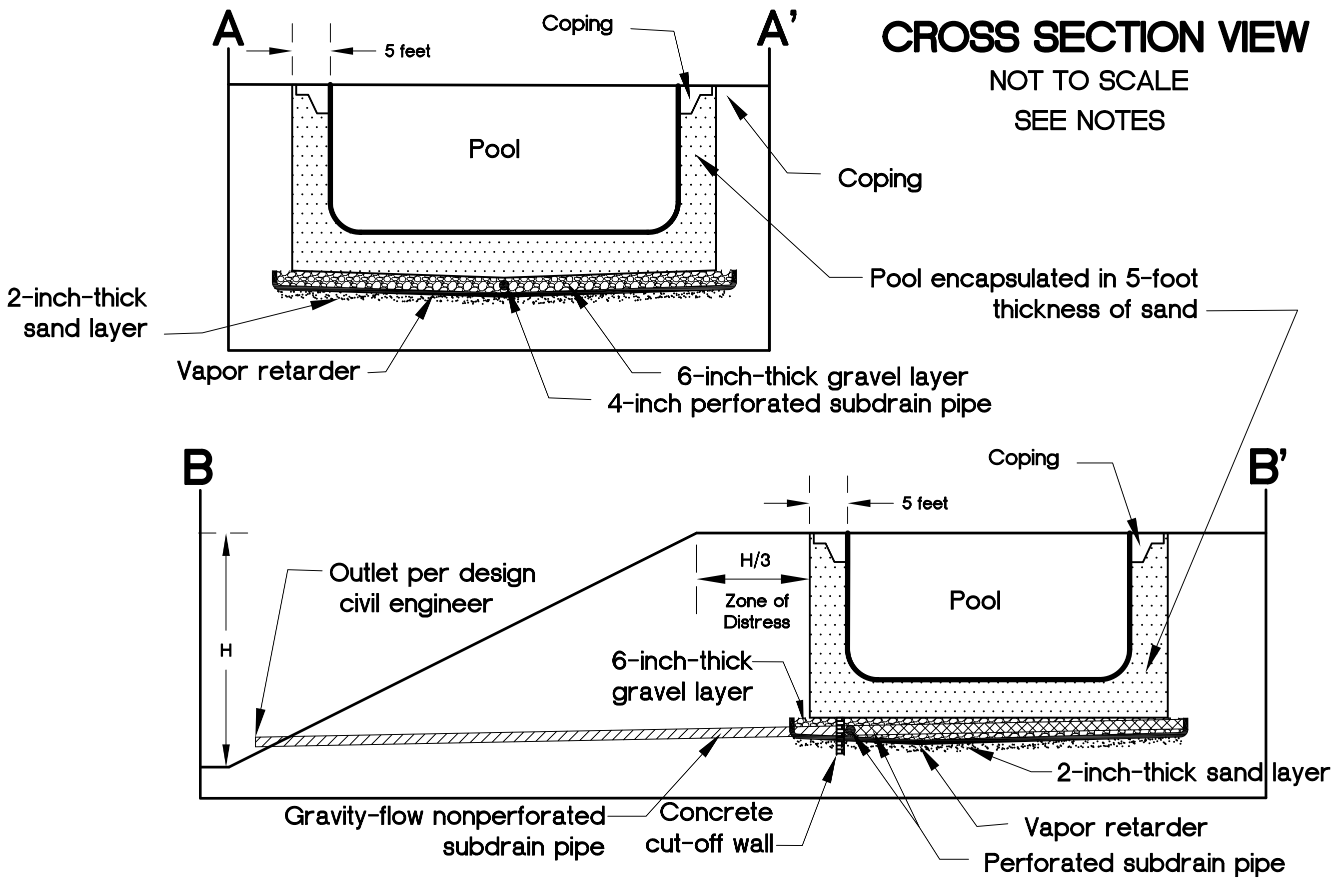
MAP VIEW

NOT TO SCALE
SEE NOTES



CROSS SECTION VIEW

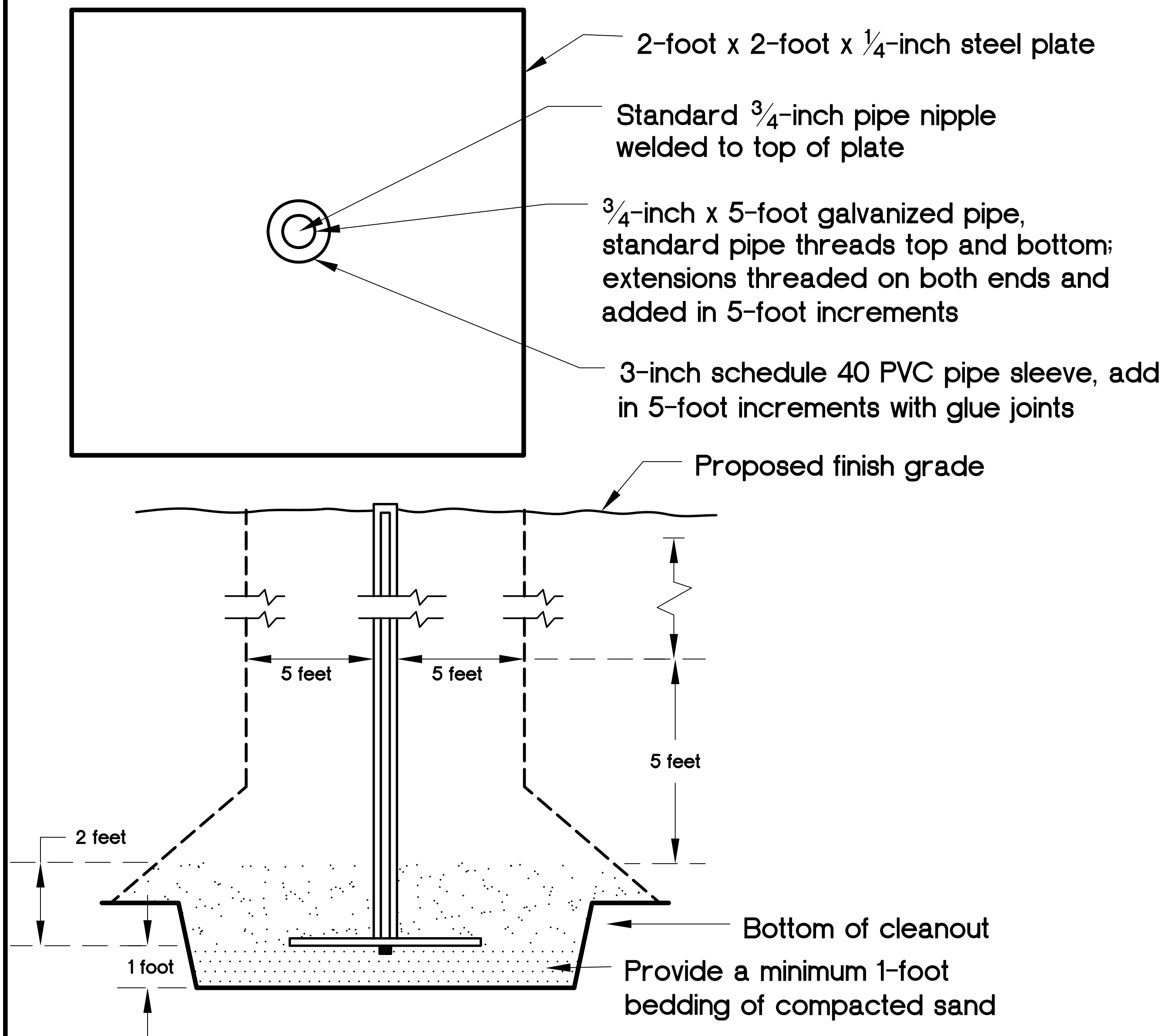
NOT TO SCALE
SEE NOTES



NOTES:

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

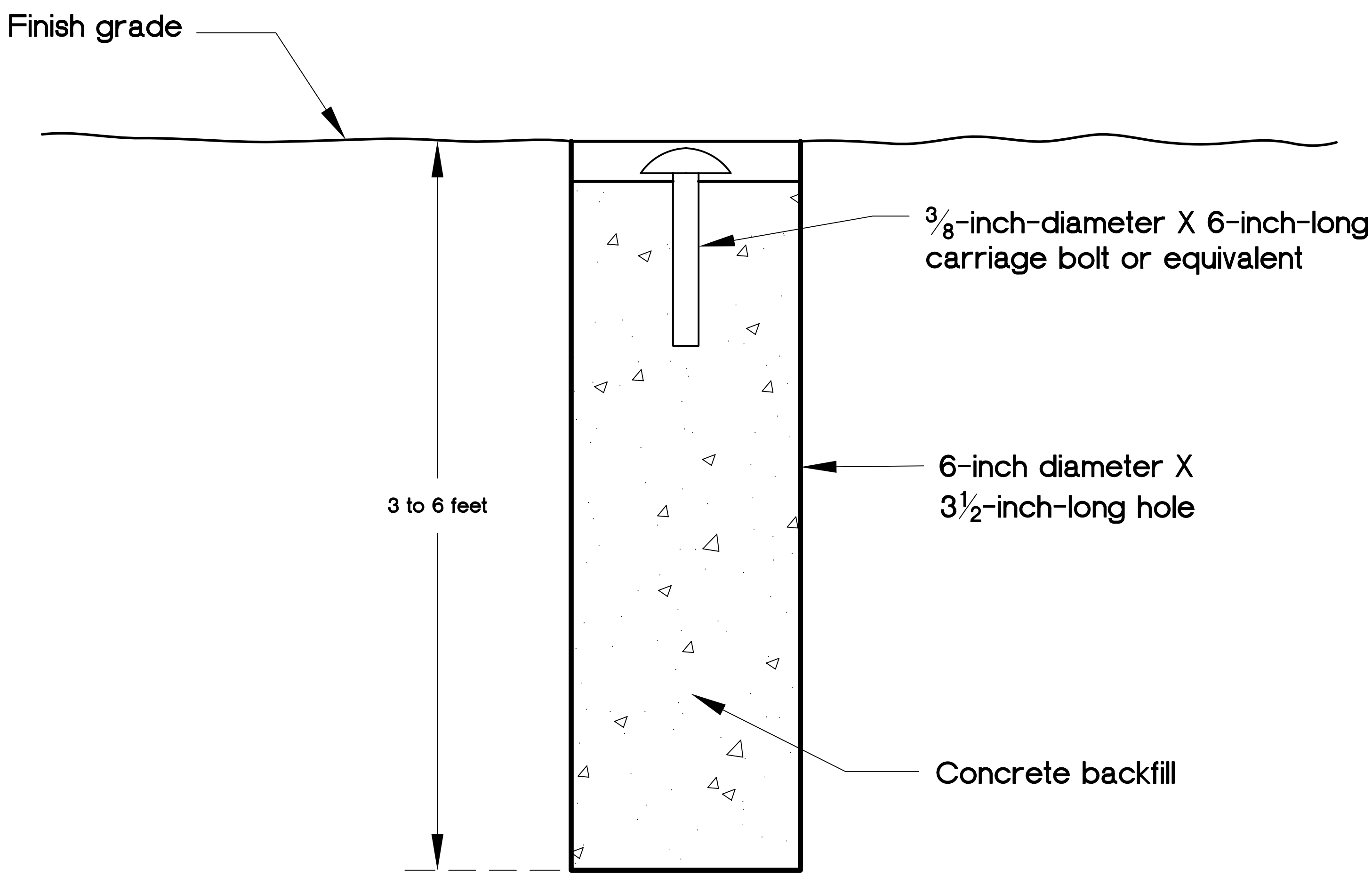
GeoSoils, Inc.	TYPICAL POOL/SPA DETAIL	Plate E-17



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.

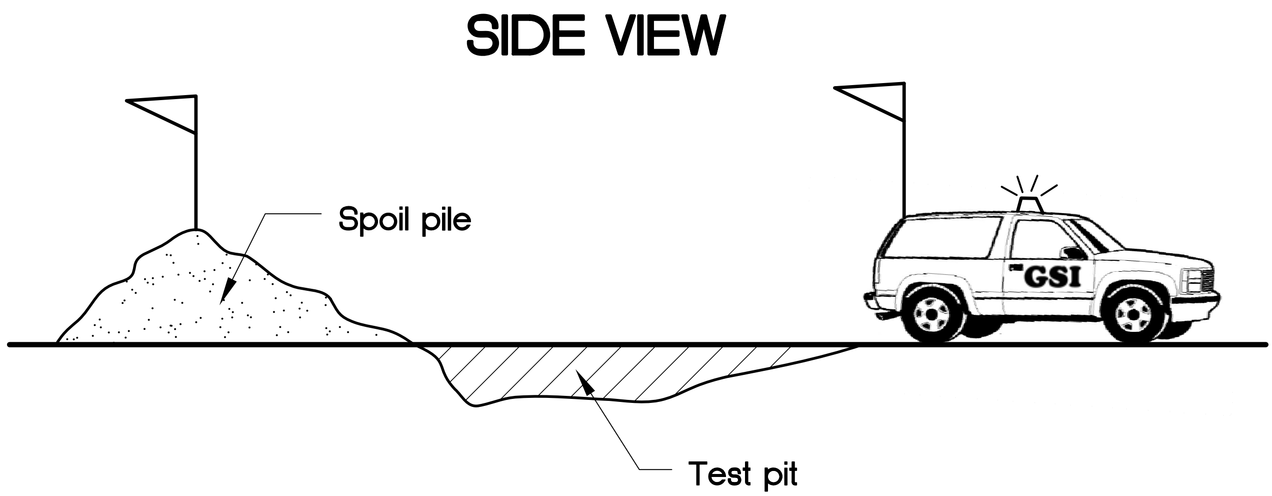
GeoSoils, Inc.	SETTLEMENT PLATE AND RISER DETAIL	Plate E-18

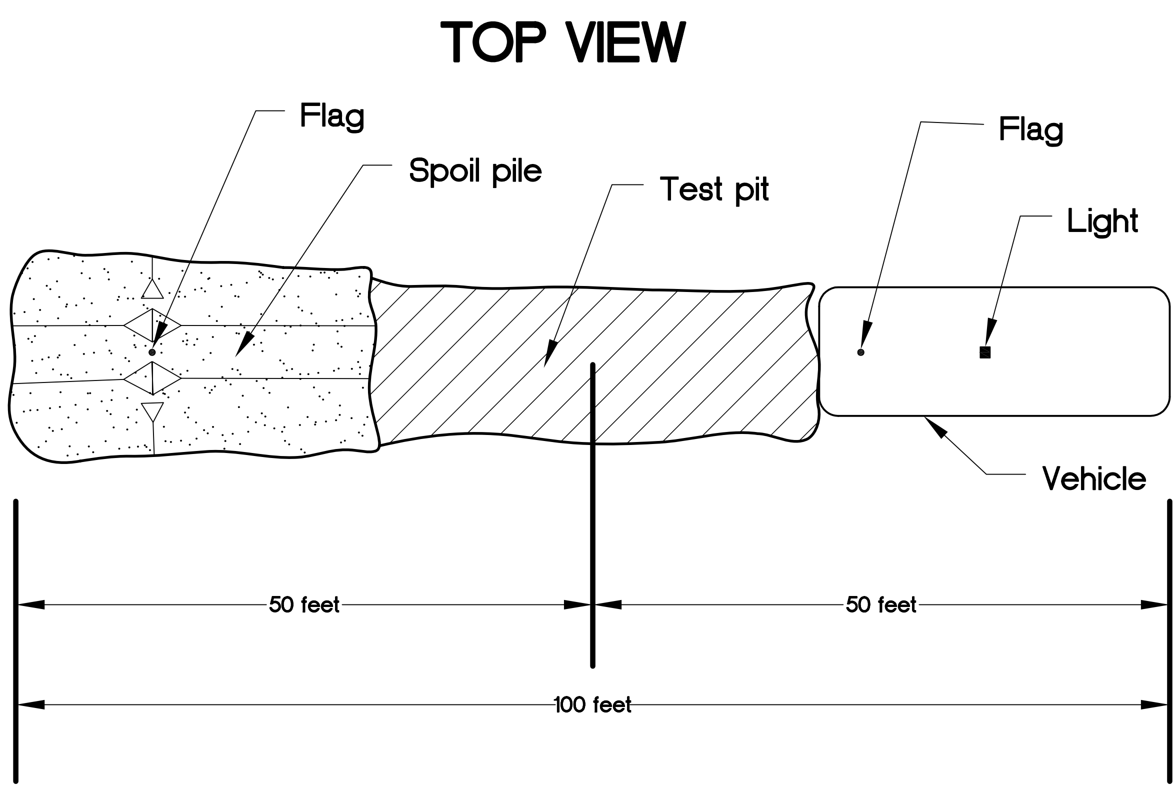


GeoSoils, Inc.

TYPICAL SURFACE SETTLEMENT MONUMENT

Plate E-19



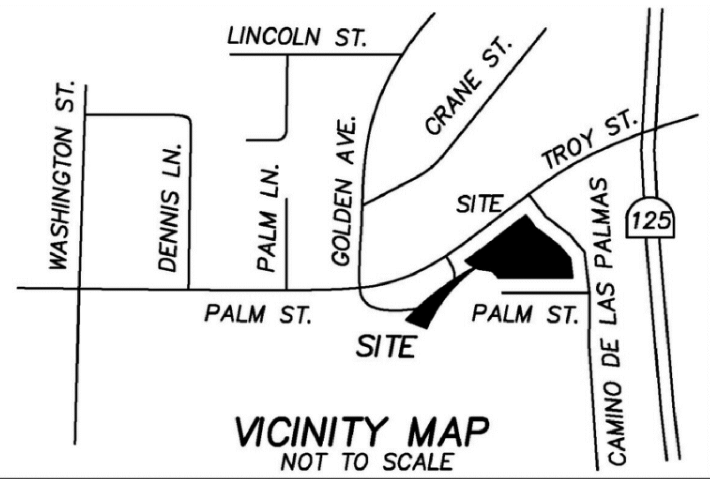


GeoSoils, Inc.

TEST PIT SAFETY DIAGRAM

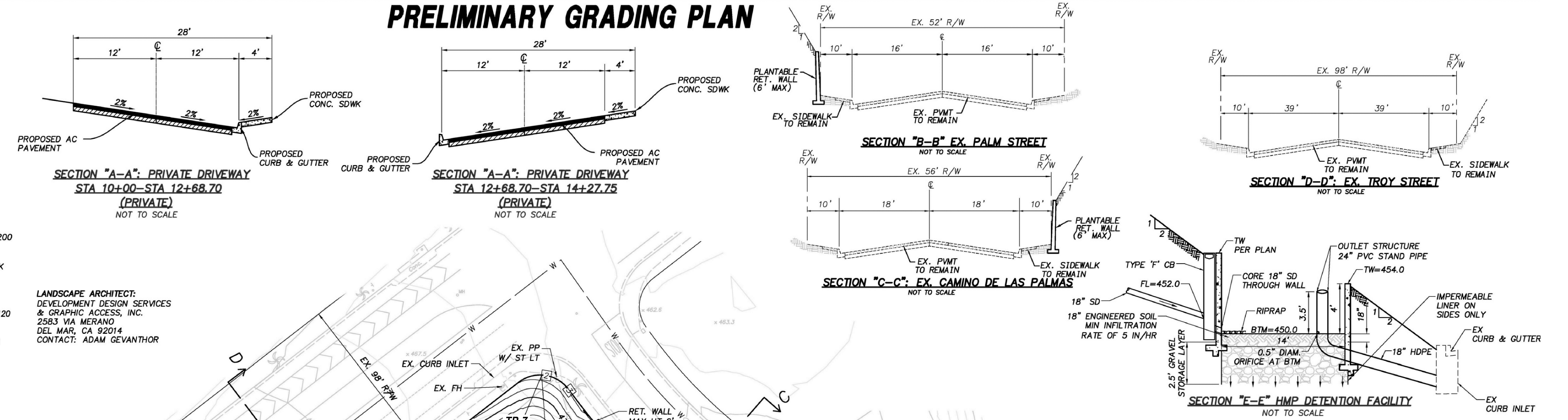
Plate E-20

PRELIMINARY GRADING PLAN



VICINITY MAP
NOT TO SCALE

THOMAS GUIDE
PAGE 1270, GRID J7



OWNER:
RINA. LLC
8109 SANTALUZ VILLAGE GREEN SOUTH
SAN DIEGO, CA 92127
(619) 559-0372
CONTACT: CHRIS DAHRLING

CIVIL ENGINEER:
LANDMARK CONSULTING
9555 GENESEE AVE., SUITE 200
SAN DIEGO, CA 92121
(858) 587-8070
CONTACT: MARK A. BRENCICK

GEOTECHNICAL:
GEOSOILS, INC.
5741 PALMER WAY
CARLSBAD, CA 92010
(760) 439-3155
CONTACT: ROBERT BOEHMER

ARCHITECT:
ARCHITECTS BP ASSOCIATES
11858 BERNARDO PLAZA CT # 120
SAN DIEGO, CA 92128
(858) 592-4710
CONTACT: ROGER BASSINGER

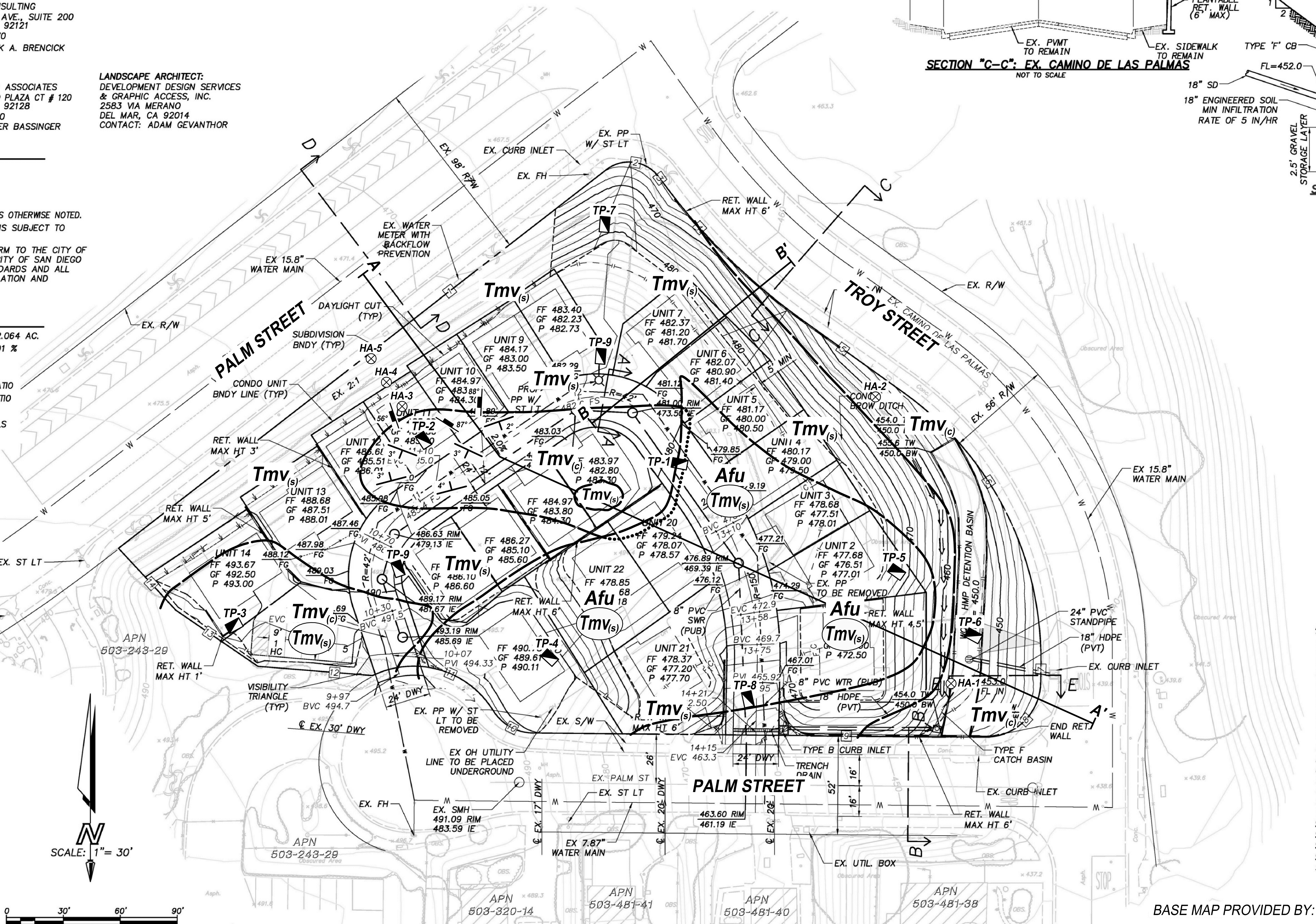
LANDSCAPE ARCHITECT:
DEVELOPMENT DESIGN SERVICES
& GRAPHIC ACCESS, INC.
2583 VIA MERANO
DEL MAR, CA 92014
CONTACT: ADAM GEVANTHOR

GENERAL NOTES:

1. TOPOGRAPHIC CONTOUR INTERVAL: 1 FOOT
   DATUM: U.S.C. & G.S.
   SOURCE: INLAND AERIAL SURVEYS, INC.
   DATE: 5-28-2015
2. ALL PROPOSED SLOPES ARE 2:1 OR FLATTER UNLESS OTHERWISE NOTED.
3. GRADING SHOWN HEREON IS PRELIMINARY AND IS SUBJECT TO MODIFICATION IN FINAL DESIGN.
4. ALL LANDSCAPE AND IRRIGATION SHALL CONFORM TO THE CITY OF LEMON GROVE LANDSCAPE REGULATIONS AND CITY OF SAN DIEGO LAND DEVELOPMENT MANUAL LANDSCAPE STANDARDS AND ALL REGIONAL STANDARDS FOR LANDSCAPE INSTALLATION AND MAINTENANCE.

GRADING:

1. TOTAL AMOUNT OF SITE TO BE GRADED: 2.064 AC.
2. PERCENT OF TOTAL SITE GRADED: 91 %
3. AMOUNT OF FILL: 3850 CY
4. AMOUNT OF CUT: 13620 CY
6. MAX. HT. OF FILL SLOPE(S): 13' 2:1 SLOPE RATIO
7. MAX. HT. OF CUT SLOPE(S): -17' 2:1 SLOPE RATIO
8. AMOUNT OF EXPORT SOIL: 9770 CY
9. MASONRY/PLANTABLE WALLS: HOW MANY: 8 WALLS; MAX. LENGTH: 150 L.F.; MAX. HEIGHT: 9 FT.



BOUNDARY DATA

L#/C#	DELTA/BRG	RADIUS	LENGTH
1	S52°13'06"W		325.10'
2	Δ=91°47'33"	19.68'	31.54'
3	N35°59'21"W		1.33'
4	Δ=18°23'58"	326.56'	104.87'
5	N54°23'19"W		66.91'
6	Δ=54°59'44"	172.13'	165.22'
7	N00°36'25"E		10.28'
8	Δ=89°55'40"	29.21'	45.85'
9	S89°27'55"E		238.05'
10	Δ=57°46'35"	25.00'	25.21'
11	Δ=84°24'05"	49.21'	72.49'
12	S85°19'43"E		60.61'
13	S55°41'12"E		52.33'
14	S42°56'24"E		44.41'

SCALE: 1" = 30'

BENCHMARK:

BRASS PLUG – CITY OF SAN DIEGO B.M. ON SW CORNER OF PALM ST. AND GOLDEN AVE.

ELEVATION = 482.131 USGS DATUM

ASSESSOR PARCEL NO.

503-252-42-00

CA COORDINATE SYSTEM:

CCS 27 206-1761
CCS 83 1846-6322

Note:
Cross Sections A-A' and B-B' are based on the existing site conditions topographic map.

LEGEND:

ITEM	SYMBOL
SUBDIVISION BOUNDARY	
EXISTING CONTOUR LINES	455
EX. SPOT ELEVATION	(70.4) FS
EASEMENT	
CONC. CURB	
CURB AND GUTTER	
CONC. SIDEWALK/PRIVATE SIDEWALK	
8" PVC SEWER MAIN	
8" PVC WATER MAIN	W
STORM DRAIN (PVT.)	
MASONRY RETAINING WALL	
PARKING SPACE NUMBER	5
CURB INLET	
CONC. BROW DITCH	
TYPE 'F' CATCH BASIN	
EX. SEWER MAIN	S
EX. WATER MAIN	W
PROPOSED CUT/FILL SLOPE (ALL SLOPES 2:1 MAXIMUM)	
PROP. STREET LIGHT	
TRENCH DRAIN	
VISIBILITY TRIANGLE	
EX OVERHEAD UTILITY LINE	

ABBREVIA

AC ASPHAL
C&G CURB &
DI DROP IN
EOP EDGE OF
EG EXISTING
FF FINISH F
FH FIRE HY
FL FLOW LI
FS FINISH S
GFF GARAGE
IE INVERT
PRKG PARKING
PL PROPERT
PP POWER
R RADIUS
R/W RIGHT-O
WTR WATER
SD STORM D
SMH SEWER M
ST LT STREET LIGHT
SWR SEWER
SDWK SIDEWALK

GSI LEGEND

- Afu — ARTIFICIAL FILL – UNDOCUMENTED
- $Tmv_{(s)}$ — TERTIARY AGE – MISSION VALLEY FORMATION (SANDSTONE MEMBER), CIRCLED WHERE BURIED
- $Tmv_{(c)}$ — TERTIARY AGE – MISSION VALLEY FORMATION (CLAYSTONE MEMBER)
- 4° — BEDDING ATTITUDE WITH DIP IN DEGREES
- 88° — JOINT ATTITUDE WITH DIP IN DEGREES
- ? — APPROXIMATE LOCATION OF GEOLOGIC CONTACT, DOTTED WHERE BURIED, QUERIED WHERE UNCERTAIN
- TP-10 — APPROXIMATE LOCATION OF EXPLORATORY TEST PIT
- HA-5 TD=?' — APPROXIMATE LOCATION OF HAND-AUGER BORING
- B B' — APPROXIMATE LOCATION OF GEOLOGIC CROSS SECTION

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.

BASE MAP PROVIDED BY:

NAME: LANDMARK CONSULTING, MARK A. BRENCICK, PE, PLS
ADDRESS: 9555 GENESEE AVE., STE. 200, SAN DIEGO, CA. 92121
PHONE NO. (858) 587-8070
FAX NO. (858) 587-8750
PROJECT ADDRESS: TROY STREET, LEMON GROVE, CA. 91945
PROJECT NAME: VISTA AZUL
SHEET TITLE: PRELIMINARY GRADING PLAN

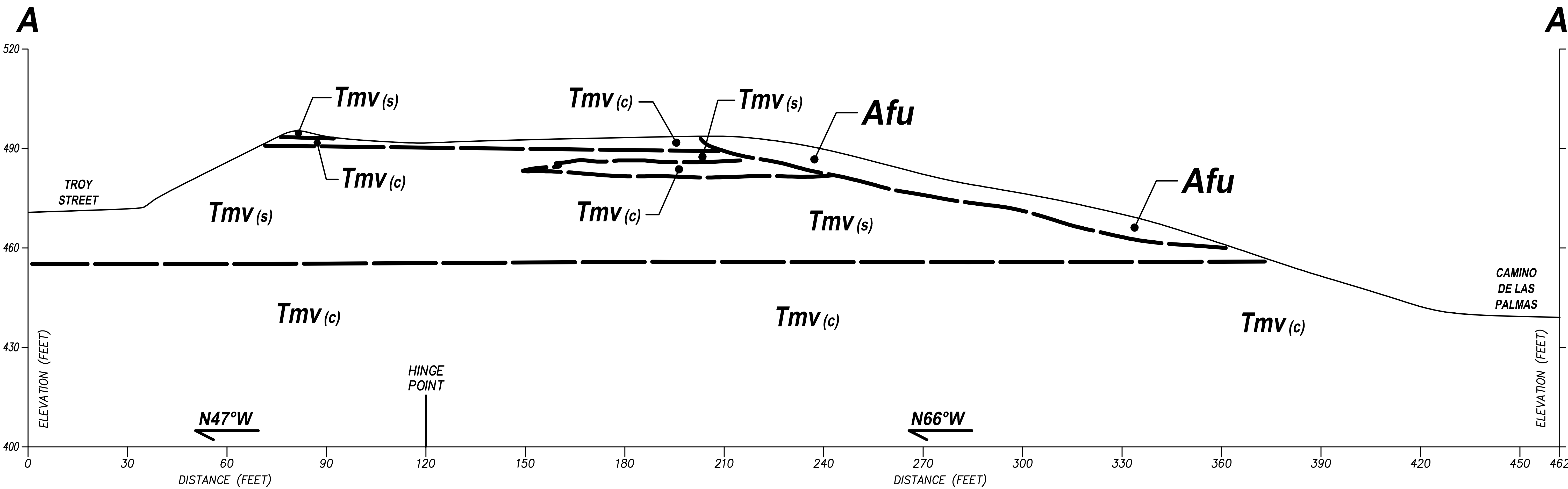
LANDMARK CONSULTING
Planning Engineering Surveying
9555 Genesee Avenue, Suite 200
San Diego, CA 92121, (858) 587-8070

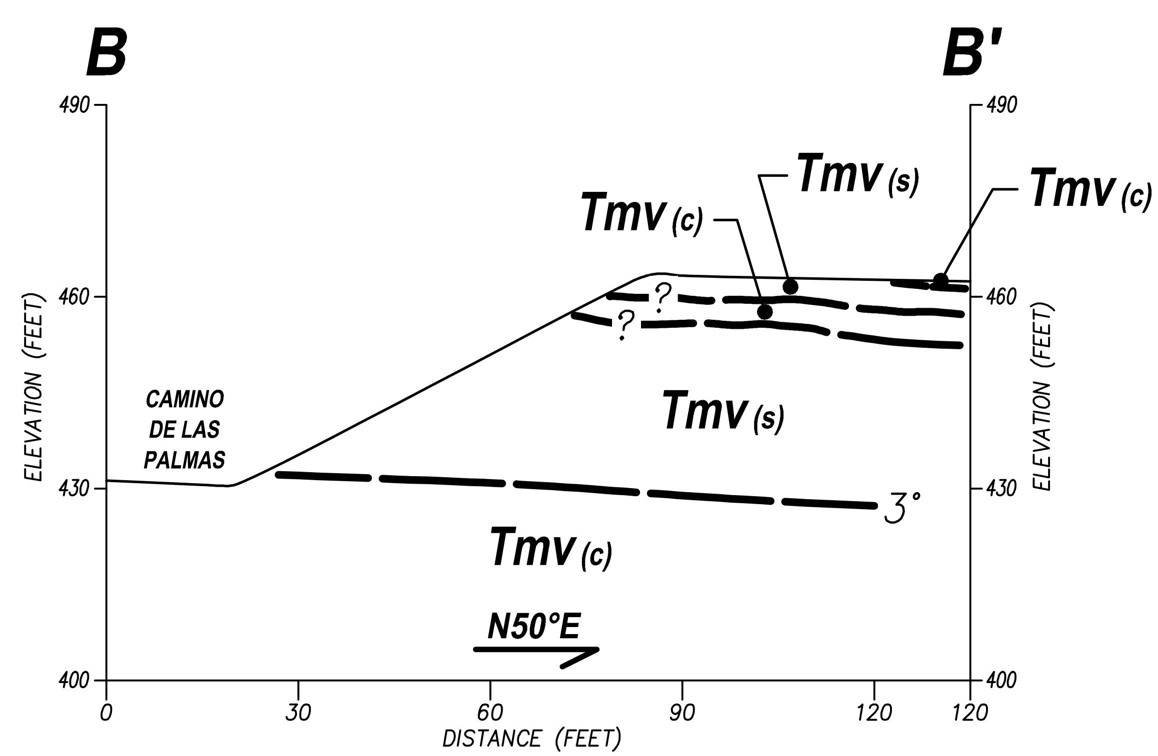
GeoSoils, Inc.

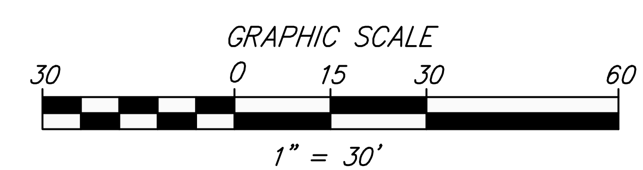
GEOTECHNICAL MAP

Plate 1

W.O. 6947-A-SC	DATE: 10/15	SCALE: 1" = 30'







GSI LEGEND

- **Afu** — ARTIFICIAL FILL – UNDOCUMENTED
- **Tmv (s)** — MISSION VALLEY FORMATION (SANDSTONE MEMBER)
- **Tmv (c)** — MISSION VALLEY FORMATION (CLAYSTONE MEMBER)
- ? — APPROXIMATE LOCATION OF GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN

ALL LOCATIONS ARE APPROXIMATE

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GeoSoils, Inc.

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

Plate 2

W.O. 6947-A-SC	DATE: 10/15	SCALE: 1" = 30'