GEOTECHNICAL EVALUATION UPDATE
FOR OCEAN BREEZE RANCH, BONSALL
SAN DIEGO COUNTY, CALIFORNIA

FOR

OCEAN BREEZE RANCH
5820 WEST LILAC ROAD
BONSALL, CALIFORNIA 92003

W.O. 6960-A8-SC AUGUST 22, 2019

GeoSoils, Inc.
GEO SOILS, INC. 45th ANNIVERSARY 1974-2019
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August 22, 2019

W.O. 6960-A8-SC

Ocean Breeze Ranch
5820 West Lilac Road
Bonsall, California 92003

Attention: Mr. Jim Conrad

Subject: Geotechnical Evaluation Update for Ocean Breeze Ranch, Bonsall, San Diego County, California

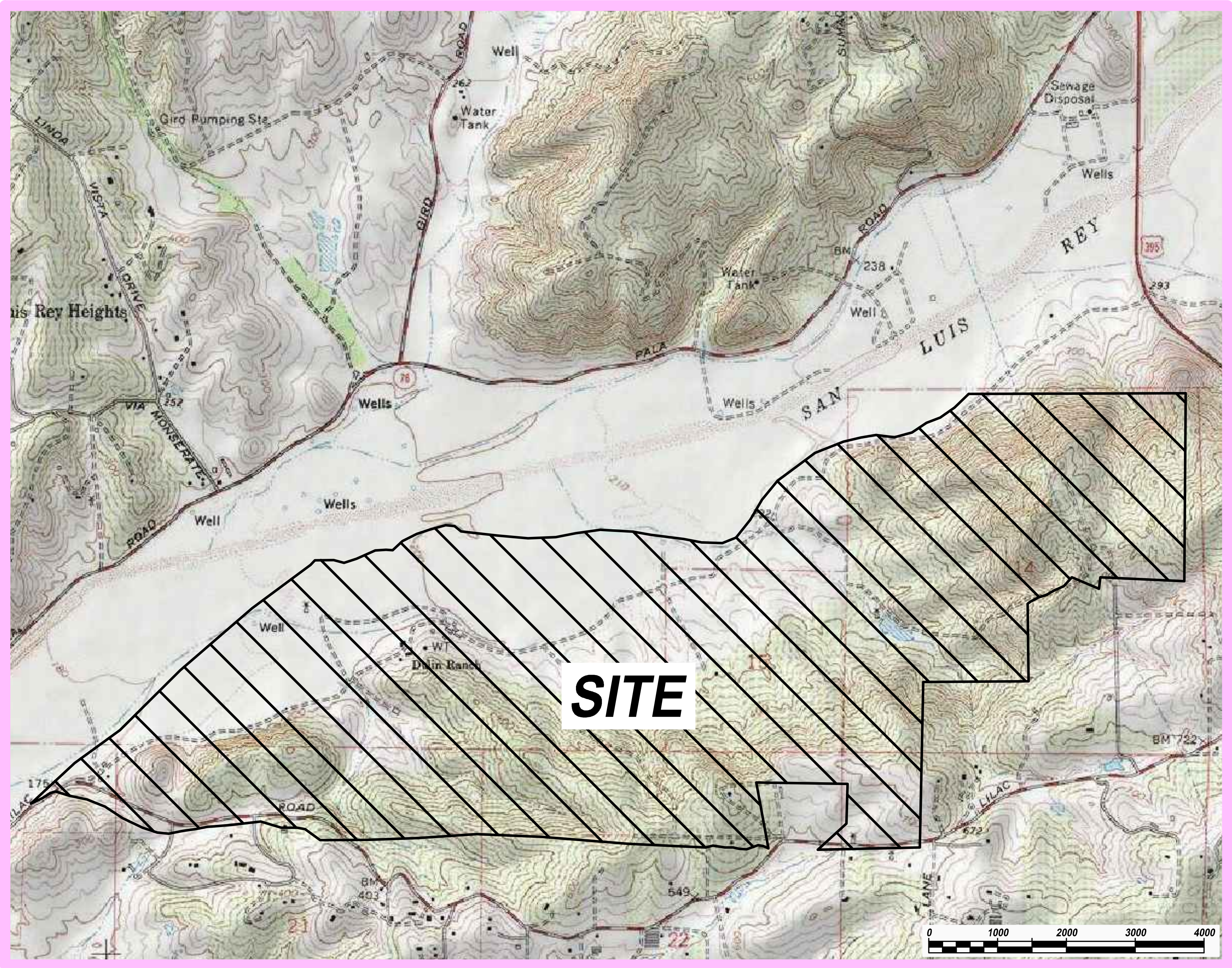
Dear Mr. Conrad:

In accordance with the request and authorization of Mr. Pete Fagrell, with Helios Property Solutions, LLC, this summary report updates the results of GeoSoils Inc.'s (GSI's) previous geotechnical evaluations for the Ocean Breeze Ranch property in the community of Bonsall, San Diego County, California. The purpose of this review was to update, as warranted, the previous conclusions and recommendations regarding the on-site geotechnical and geologic conditions and their impacts on proposed, residential use site development, from a geotechnical viewpoint.

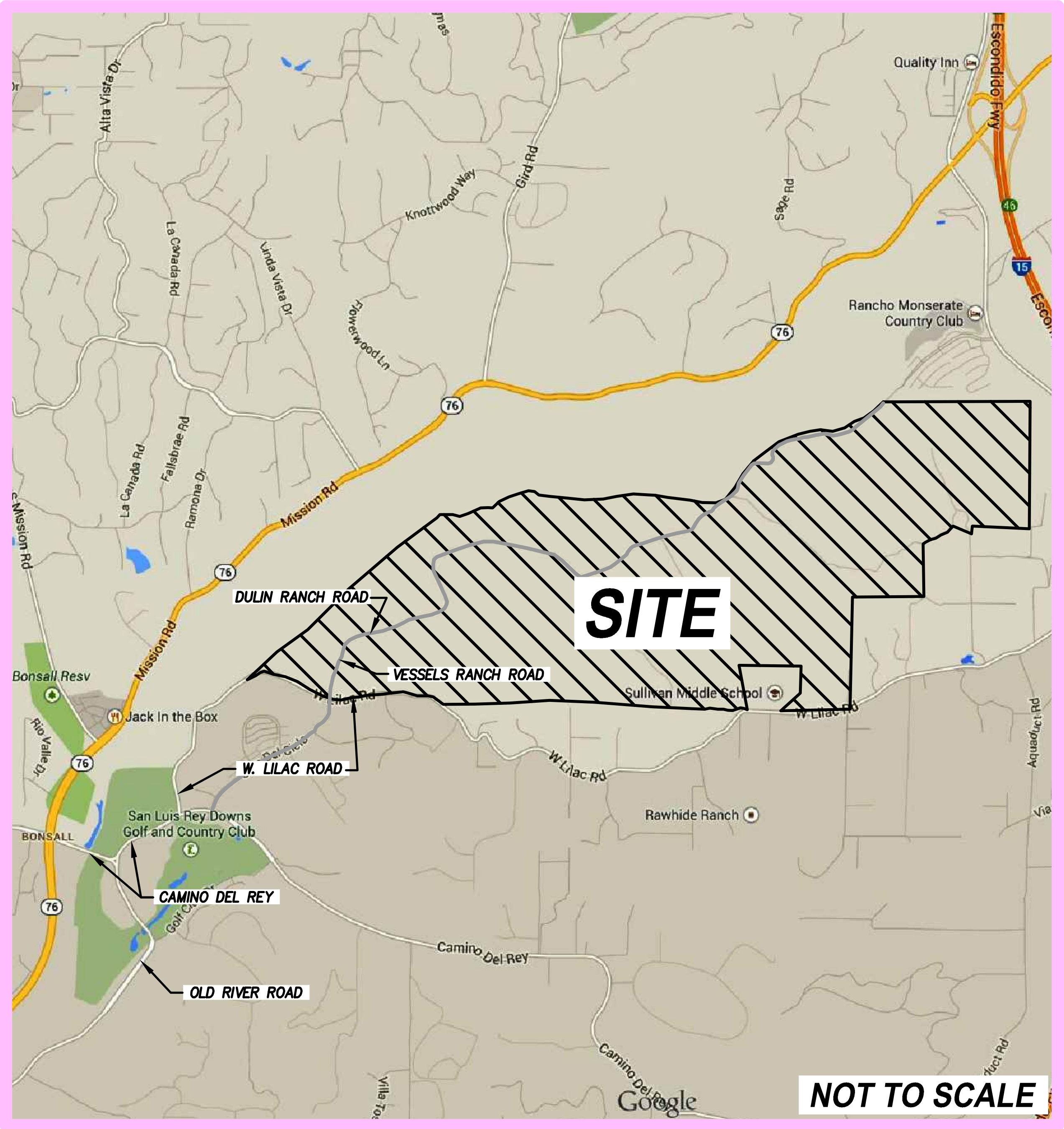
The field work, laboratory testing and analyses for this study were conducted previously; however, the proposed residential development has been reduced in magnitude, and the reduced residential footprint (Planning Areas [PAs]) and roadways, remain in areas previously evaluated. Accordingly, the geotechnical conclusions and recommendations contained in GSI (2015 and 2016 [see Appendix 1]), remain pertinent and valid. The proposed maximum height cut and fill slopes are analyzed in this current update (Appendix 2). The previous PA specific geotechnical conclusions and recommendations are modified herein, as appropriate, and are included in Appendix 3. All other conclusions and recommendations in GSI (2015 and 2016), should be appropriately implemented. The previous analyses are not repeated herein; however, for convenience, GSI (2015 and 2016) are included in Appendix 4, as PDFs (on CD only).

SITE DESCRIPTION

The irregularly-shaped property consists of about 1,402.52 acres (gross), located along the southern margin of the San Luis Rey River Valley, in the vicinity of Dulin Ranch Road, including hilly and more rugged terrain generally between Dulin Ranch Road and West Lilac Road, in the community of Bonsall, San Diego County, California (see Figure 1), south of Mission Road/Highway 76, and west of Interstate 15.



Base Map: TOPO!® ©2003 National Geographic, U.S.G.S. Bonsall Quadrangle, California -- San Diego Co., 7.5 Minute, dated 1975, current, 1975.



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

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GSI GeoSoils, Inc.

W.O.
6960-A-SC

SITE LOCATION MAP

Figure 1

Topographically, portions of the property (Planning Area PA-2) within the San Luis Rey Valley floor area are generally of a flat-lying/low gradient. South of the river valley (generally south of Dulin Ranch Road), the westernmost third of the property ascends from the valley floor to somewhat more rugged, inclined terrain, with slope gradients generally steeper than about 4:1 (horizontal to vertical [h:v]), that form a roughly east-west trending ridgeline across the southern portion of the site (Planning Area PA-1). Within the remaining, easternmost portion of the property, the relatively flat-lying river valley floor transitions to moderately sloping terrain, with north facing slopes at gradients generally on the order of 4:1 (h:v), or less (Planning Area PA-3). As with the western portion of the property, these low/moderate gradient slopes ascend to somewhat more rugged, craggy terrain along the southern portion of the property. Drainage is generally directed northward, from the crest of the east-west trending ridgeline, toward the San Luis Rey River, via tributary drainages incised into the north facing slope. On the backside, or south side of the ridge, drainage is generally directed offsite to the south.

The relatively flat-lying valley floor portion of the site has elevations ranging from about 175 to 225 feet Mean Sea Level (MSL), with the area of low gradient slopes, south of the valley floor, ranging from 175 to 225 feet MSL at the valley floor/margin, up to approximately 250 feet MSL. The somewhat rugged, steeper terrain that ascends to the south and southeast, range from about 250 to as much as 825 feet MSL. Thus, overall relief across the site is on the order of about 650 feet. Portions of the site (i.e., valley floor), generally within the low/flat-lying portions of the site, lie within a San Diego County 100-year flood plain.

The property is currently used for both equestrian and agricultural purposes. Existing improvements generally consist of an equestrian facility located within the low lying, northerly portions of the site (paddock areas currently located within of Planning Area PA-2), with an existing residence/ranch house, located within Planning Area PA-1, overlooking the equestrian facility. Scattered outbuildings were also noted throughout, and generally located in close proximity to the equestrian facility. Vegetation generally consists of some native trees, planted trees, grass pasture, areas of irrigated row crops, groves, and also areas with native grasses and brush.

PROPOSED DEVELOPMENT

The proposed development generally consists of three (3) planning areas (PA-1 through PA-3) distributed throughout the property, as shown in the following table:

OLD PLANNING AREA	NEW PLANNING AREA	APPROX. NUMBER OF LOTS	COMMENTS
PA-2	PA 1	144 Lots	5,000 sq ft Lots. Graded Pads Indicated*
PA-3	PA 2	237 Lots	4,500 and 5,000 sq ft Lots. Graded Pads Indicated*
PA-5	PA 3	14 Lots	5-Acre minimum Lot sizes. Raw Land, No Grading Indicated*
-	Hillside Estate Parcel	1 Lot	24.24 Acres. Raw Land, No Grading Indicated*
-	School Parcel	-	Raw Land, No Grading Indicated*
* - per PDC (2019b)			

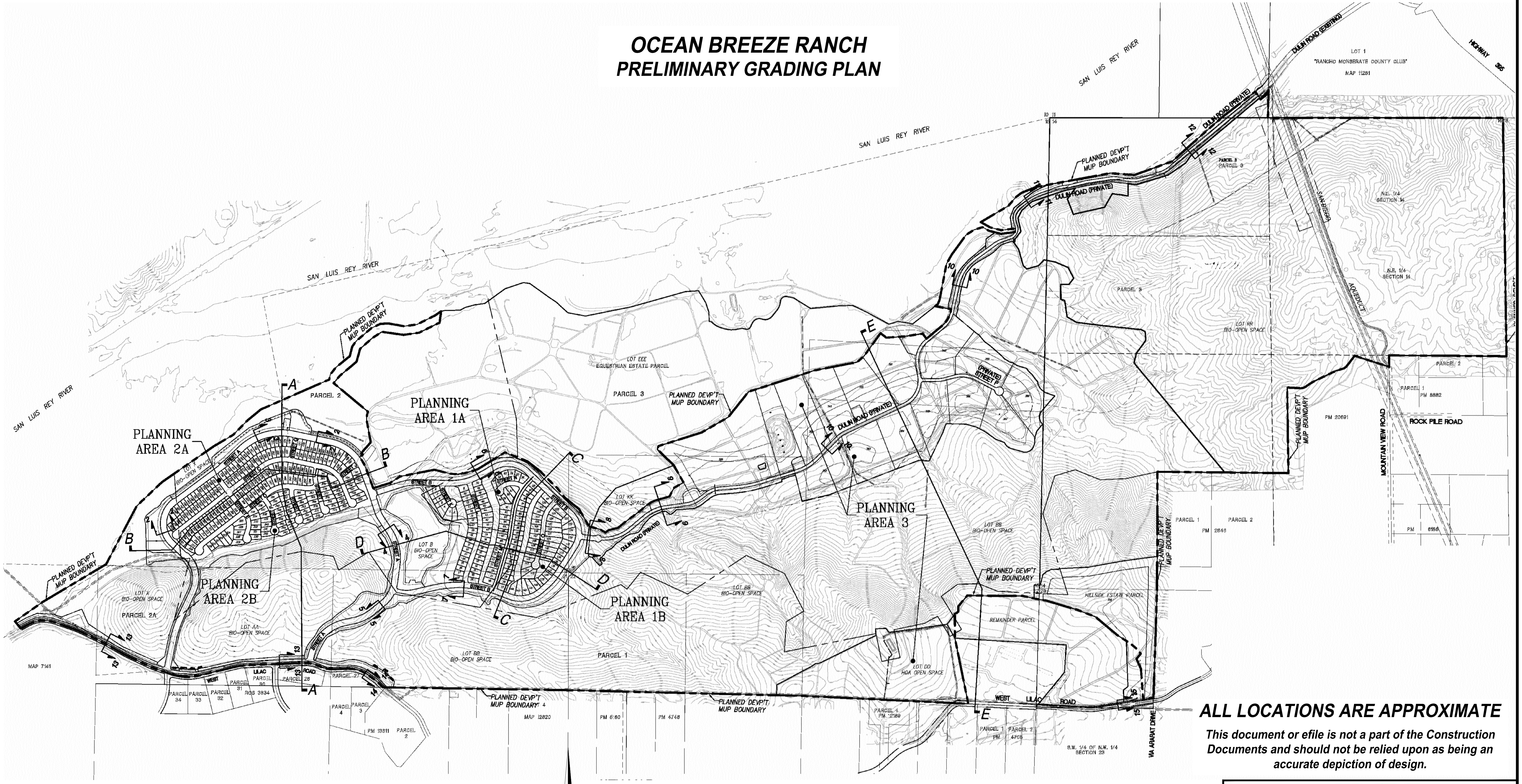
As indicated above, Planning Areas PA-1 and PA-2 include the construction of approximately 381 single-family residential structures, and associated improvements. Planning Area PA-3 consists of approximately (14) larger estate-size building lots, and associated improvements. Cut and fill grading techniques are anticipated to bring Planning Area PA-1 to the desired grades. Within Planning Area PA-1, maximum cut and fill thicknesses on the order of about 50 feet, and 45 feet, respectively, are anticipated, with graded slopes ranging from about 95 feet (cut [roadway]), and 50 feet (fill) in height, at gradients up to 1.5:1 (horizontal to vertical [h:v]), or flatter, for cut slopes, and 2:1 (h:v), or flatter, for fill slopes. Within Planning Area PA-2, maximum fill thicknesses on the order of 10 to 30 feet, are anticipated, with graded slopes ranging up to about 51 feet, or less, in height, at gradients of 2:1 (h:v), or flatter.

In Planning Areas 1 and 2, the project proposes streets which will be constructed to meet public standards. Street A, the primary backbone loop road, will connect to West Lilac Road at two locations. Street B, a secondary loop road, encircles Planning Area PA-1 and connects in two locations to Road A. Internal neighborhood roadways will connect all portions of Planning Areas 1 and 2, to either Road A or B.

Access to Planning Area PA-3 will be by private road. The primary backbone road through this planning area will be a segment of Dulin Road. This road segment will connect to Road B at the western end of Planning Area PA-3, and at the eastern end of the project it will connect to the existing segment of Dulin Road located within the Rancho Monserate mobile home community. This segment of Dulin Road will include private access gates constructed at the western and eastern ends.

We anticipate that structures will be one- or two-story buildings utilizing typical foundations on grade, with wood frame and/or masonry block construction. Building loads are assumed to be typical for this type of relatively light construction. Sewage disposal for is understood to be accommodated by tying into the regional sewage system. The need for import soils is unknown, based upon the data provided. The approximate limits of each planning area are shown on PDC (2019b), and are also indicated on Figure 2 included herein. PDC (2019b) is used as the base for the 400-scale Geotechnical Map, Plate 1.

OCEAN BREEZE RANCH
PRELIMINARY GRADING PLAN



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.

GRAPHIC SCALE

1000 0 500 1000 2000

1" = 1000'

N

MAP BASED ON:

PROJECT DESIGN CONSULTANTS
PRELIMINARY GRADING PLAN
OCEAN BREEZE RANCH TITLE SHEET

GeoSoils, Inc.

OBR PROPOSED LAYOUT

Figure 2

W.O. **6960-A8-SC**	DATE: **08/19**	SCALE: **1" = 1000'**

SLOPE STABILITY ANALYSES

General

GSI performed a slope stability evaluation utilizing the geologic conditions, observed in the subsurface explorations completed previously, for maximum planned cut and fill slopes within the project.

Analyses have been performed utilizing the two-dimensional slope stability computer program "GSTABL7 v.2.004." The program calculates the factor-of-safety (FOS) for specified surfaces or searches for the block, or irregular slip surface having the minimum FOS using the Janbu (non-circular block) method. Shear strength parameters used were obtained from prior laboratory testing of representative samples of site soils. Representative cross sections were prepared for analysis of maximum planned cut and fill slope stability (see Appendix 2).

Gross Stability

The stability of the planned maximum cut slopes and fill slopes was evaluated. These slopes vary up to approximately 96 and 51 feet in height and at slope gradients of 2:1 (h:v), for cut and fill slopes, respectively. Based on our current analysis, using the available soil parameters, the slopes appear to be stable, possessing an adequate FOS (greater than 1.5 static and 1.1 seismic). The results of our slope stability analysis have been included as Appendix 2.

All graded slope construction will require observation during grading in order to evaluate the findings and conclusions presented herein and in subsequent reports. Our analysis assumes that graded slopes are designed and constructed in accordance with guidelines provided by the County, the 2016 CBC (CBSC, 2016), the latest adopted version of the “Greenbook,” and recommendations provided by this office. These slopes are generally anticipated to be stable, assuming proper construction, routine and periodic maintenance, and normal climatic conditions.

Temporary backcuts for construction slopes and keyways, are anticipated to be 1:1 (h:v) or flatter, and are anticipated to have a static FOS of 1.2. Should perched groundwater or other unexpected conditions be exposed during excavation, the project geotechnical consultant should review the conditions and revise recommendations as needed.

Surficial Stability

Surficial stability was evaluated for graded slopes constructed of compacted fills and/or bedrock soil (see Appendix 2). Our analysis indicates that proposed slopes exhibit an adequate FOS (i.e., $\geq$1.5) against surficial failure, provided that the slopes are properly constructed and maintained, under normal rainfall.

Onsite soils are erosive. Planting and management of surficial drainage is imperative to the surficial performance of slopes. Foot traffic and other activities that exacerbate surficial erosion should not be allowed to occur on slope faces.

UPDATE SUMMARY

Based on our review of the available data (see Appendix 1), as well as previous field exploration, laboratory testing and geologic and engineering analysis, the proposed development of the property appears feasible from a geotechnical viewpoint, provided that mitigation measures presented in GSI (2016) and summarized in this report (Appendix 3), are properly incorporated into design and construction of the project. The most significant elements of this review are summarized below:

1. The site occupies the southern flank of a portion of the San Luis Rey River valley, consisting of a relatively flat-lying valley floor to the north, with bedrock highland to the south. Flat-lying ground in the vicinity of (primarily north of) Dulin Ranch Road, and generally within the 100-year flood plain, is underlain with Holocene alluvial sediments. Lower slopes descending to the valley floor, and flatter than about 4:1 (h:v) are developed on deposits of Quaternary (Pleistocene)-age older alluvium (stream terrace deposits). Steeper slopes and upland areas are primarily underlain by igneous granitic bedrock (tonalite), with minor outcroppings of granodiorite and metasedimentary rocks occurring in the northeastern margin of the site, outside of the proposed development.

2. In general and based upon the available data to date, regional groundwater is not expected to be a major factor in development of the more elevated portions of the site (i.e., areas underlain with deposits of older alluvium and/or granitic bedrock). Within lower-lying areas underlain with alluvium, groundwater was encountered at depths ranging from approximately 11½ to 18½ feet below existing grade within the San Luis Rey River drainage area, to slightly deeper, perched water tables within adjoining tributary drainages, and is anticipated to be a concern during development in these areas, including any deep utilities. This corresponds to fluctuating elevations ranging from about 169 to 189 feet above MSL within the San Luis Rey River drainage in the vicinity of Planning Area PA-2 (down gradient to the west), and this phreatic surface rises with the elevation of the drainage, to the east. Additionally, owing to the relatively cohesionless nature of near-surface soils, perched groundwater/sloughing should be anticipated during excavation. A shallow groundwater table will be encountered during removals/excavation within alluvium, primarily within Planning Area PA-2.

3. Proposed cut and fill slopes are anticipated to be grossly and surficially stable, under normal conditions of care, regular and periodic maintenance, and the prevailing climate. Site soils are erosive.

4. The presence of landslide deposits, slumps, or other significant forms of mass wasting were not observed nor encountered within the site.

5. GSI's review and field exploration indicates no known active faults are crossing the site, and the site is not located within an Alquist-Priolo Earthquake Fault Zone (California Geological Survey [CGS], 2018). However, strong shaking should be anticipated should an earthquake occur on one of the nearby regional active faults, and liquefaction effects within alluvial soils should be anticipated, if not mitigated.

6. The proposed structures and foundations, as well as other supporting infrastructure should be designed to resist seismic forces and deformation in accordance with the criteria contained in the 2016 California Building Code ([2016 CBC], California Building Standards Commission [CBSC], 2016). Based on our site-specific seismic hazard analysis, appropriate seismic design parameters are provided herein.

7. Based on our prior analysis, the potential for liquefaction to adversely affect those portions of the site underlain with older alluvium and/or granitic bedrock is considered low. Regardless, some seismic induced deformation should be anticipated due to densification, as discussed therein (GSI, 2016). Owing to the depth to groundwater, relatively low density, grain size, young age and lack of cementation, the potential for liquefaction and seismic densification to adversely affect those portions of the site underlain with younger alluvium is higher, when subjected to the design level earthquake, based on the available data, and will require mitigation by removals and/or engineering design.

8. Based upon our experience in this area, and the seismic refraction data previously obtained, assuming a D9L, or equivalent, bedrock within cut areas of the site appear to generally be rippable (i.e., seismic velocities of less than about 6,000 feet per second [fps]) at depths ranging up to ±30 feet from existing grade. Rock breakers and/or blasting should be considered during preliminary planning and budgeting for excavation depths (including foundations and utilities) greater than about ±30 feet from existing grade, on a preliminary basis.

9. Using the 3,800 fps cut-off for non-rippable trenching, assuming a CAT 235 hoe, or equivalent, it is likely that some areas will require blasting (e.g., "line-shooting") for trenching of utilities onsite. Seismic velocities near, or exceeding 3,800 fps generally occur at depths ranging from depths as shallow as ±9 feet, to as deep as about ±38 feet from existing grade. A conventional backhoe would likely encounter practical refusal at shallower depths.

10. Excavation within bedrock areas exhibiting a seismic velocity of ≥5,000 fps will generate appreciable quantities of oversize rock >12 inches in size, requiring specialized placement techniques during grading. In addition, hard rock requiring blasting, rock breakers, etc., may not be entirely precluded from occurring near the

surface, and may also generate oversize rock. Accordingly, oversize rock (<24 inches in size), may be placed in fills deeper than 10 feet from finish grade, subject to governing agency approval, or may be crushed to reduce their size for standard fill placement. Considering the thickness of proposed fills and the proximity of groundwater below existing grade, there are limited areas on the project that will accommodate the hold-down distance of 10 feet below finish grade, and that have significant volume for oversize material placement. Thus, onsite crushing of oversize materials to less than 12 inches may be necessary. This condition will need value engineering to evaluate the feasibility of either oversize rock placement and/or crushing oversize materials onsite.

11. Representative samples of near surface site soils were tested for expansion potential. The Expansion Index (E.I.) test was performed in general accordance with ASTM Standard D 4829. The laboratory test results indicate that the soil expansion potentials are generally very low (E.I. 0 to 20). However, this does not preclude the presence of higher expansive soils locally onsite.

12. Representative samples of site material has also been evaluated for corrosion, soluble sulfate, etc. Laboratory testing indicates that site soils generally have a negligible (not applicable) sulfate exposure to concrete, per Table 19.3.1.1 of ACI 318-14 (per the 2016 CBC [CBSC, 2016]), and the use of Type V cement is not required. Corrosion testing (pH/resistivity) indicates that the soils are slightly alkaline (pH of 6.45 to 6.99) with respect to soil acidity/alkalinity, and is mildly corrosive to ferrous metals when saturated (saturated resistivity of 1,800 to 3,400 ohm-cm [California Highway Design Manual, 2012]). Chloride content of the soil was measured as 122 to 192 ppm, which is slightly elevated. Alternative testing methods and additional comments should be obtained from a qualified corrosion engineer with regard to foundations, piping, etc. Additional corrosion testing should be performed at the completion of site grading to further evaluate geotechnical pad characteristics.

13. A settlement analysis was previously performed for three (3) general, as-built conditions anticipated onsite, in consideration of both static and dynamic settlement. Group 1 areas (i.e., lower elevations of Planning Area PA-3, and a portion of Planning Area PA-1) would consist of engineered fills placed over older alluvium, Group 2 areas (i.e., Planning Area PA-1, and the upper elevations of PA-3) would generally consist of engineered fills placed over granitic bedrock, and Group 3 areas (Planning Area PA-2) would be where portions of the site overly alluvium below the groundwater table. Group 3 areas may also display an increased potential to be affected by lateral spreading during a seismic event. A discussion of settlement potential for each general area is presented in the text of GSI (2016). Due to high estimated settlements within Planning Area PA-2, additional review and field investigation is recommended.

14. It should be kept in mind that drainage reversals could occur, when considering post-construction static and seismic settlement within Planning Area PA-2, if relatively flat yard drainage gradients are not periodically maintained in areas underlain by alluvium. Similarly, gravity flow utilities in areas underlain by alluvium are also subject to possible drainage reversals or deflections, considering the magnitude and angular distortions of settlement reported herein.

15. The treatment of existing ground prior to fill placement for specific areas of the site will vary according to each of the following two (2) general cases:

    Case I - Areas underlain with near surface, older alluvium, and/or granitic bedrock.

    Case II - Areas underlain with loose alluvium and a shallow groundwater table (i.e., alluvium left in place below the groundwater table).

    A discussion of specific recommendations for each case is included in the text of GSI (2016), and summarized in Appendix 3 of this report.

16. Given the potential for settlement, expansive soils and lateral movement due to the design basis earthquake, Planning Area PA-3 should be further evaluated using a truck mounted drill rig.

17. All existing structures, utilities, deleterious debris, and vegetation should be removed from the site and properly disposed, should settlement-sensitive improvements be proposed within their influence. It should be noted that the 2016 CBC (CBSC, 2016) indicates that for fill placed under the purview of the grading permit, removals of unsuitable soils be performed across all areas to be graded, not just within the influence of the 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 to mitigate site perimeter conditions or existing utilities. Thus, any settlement-sensitive improvements (walls, curbs, flatwork, etc.), constructed within this zone, may require deepened foundations, reinforcement, etc., or will retain some potential for settlement and associated distress. Current conditions indicate compressible colluvium, alluvium, weathered older alluvium, and bedrock, which should be included in remedial grading efforts.

18. In general, support of the new building(s) and structures may be provided entirely by engineered and compacted fill. As discussed herein, onsite soils appear to be very low, to possibly low expansive. However, the potential for higher expansive soils cannot be precluded locally.

19. Based on the underlying conditions supporting engineered fills onsite, the as-built conditions will likely result in at least three (3) different foundation design/construction scenarios. Refer to the foundation recommendations sections of GSI (2016) and Appendix 3 of this report. These foundations will require various ground treatments (recompaction, improvement, overexcavation) prior to placement, and discussed herein.

20. Retaining wall design and construction recommendations are provided in GSI (2016). Onsite soils are generally very low expansive, to possibly low expansive, and appear suitable for wall backfill, without select import, subject to verification testing.

21. Recommendations for concrete (PCC) and asphaltic concrete (AC) pavements are provided in GSI (2016). The majority of site soils anticipated at finish subgrade elevations are anticipated to be relatively sandy, and are considered to provide relatively good subgrade support for roadways. As such, County minimum pavement sections should be anticipated.

22. Storm water infiltration feasibility was evaluated for each of the three (3) dominant geologic units onsite (alluvium, older alluvium, and granitic substrates) in GSI (2016). Supplemental infiltration and percolation feasibility studies have subsequently been performed by GSI for the current layout, and are listed in Appendix 1.

23. Adverse geologic structures that would preclude project feasibility were not encountered. However, the potentially liquefiable and compressible deposits of alluvium will require more investigation in order to develop a program of ground mitigation and/or specialized foundation/infrastructure designs, as discussed herein.

24. The project design features summarized herein, and presented in GSI (2016), should be incorporated into the design and construction considerations of the project. If the design information and/or assumptions used as a basis for the geotechnical recommendations do not reflect current design information, GSI suggests a review of the current design(s) and modification of the geotechnical recommendations as needed.

ADDITIONAL STUDIES

Given the likelihood of significant seismic induced settlement on Planning Area PA-2, variable thickness fills and potential for steep buried contact(s) in Planning Area PA-3, GSI recommends that additional CPTs or hollow stem auger borings be performed in both areas, respectfully. Additional borings are recommended in Planning Area PA-2 to delineate: a) depth of alluvium (Qal); b) shape of buried formation/bedrock and alluvial contact; c) presence of fine grained soils or oversized earth materials; and d) groundwater.

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.

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.

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Certified Engineering Geologist

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RGC/ATG/DWS/JPF/jh

Attachments:	Attachment 1 - Selected References Attachment 2 - Slope Stability Calculations Attachment 3 - Update Recommendations Attachment 4 - PDFs of Previous Geotechnical Studies (GSI, 2015 and 2016) on CD Data Disc Plate 1 - Geotechnical Map
Distribution:	(3) Addressee (wet signed)

APPENDIX 1

SELECTED REFERENCES

APPENDIX 1

SELECTED REFERENCES

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APPENDIX 2

SLOPE STABILITY CALCULATIONS

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 analysis to compute the factor of safety (FOS) for a layered slope. This program can be used to search for the most critical surface or the FOS 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.

4. Landslides: Investigation and Mitigation, by A.K. Turner and R.J. Krizek (editors), Special Report 247, Transportation Research Board, National Research Board, 675 pages, ISBN 0 309 06208-X, 1996.

GSTABL7 v.2 Features

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

1. Allows user to calculate FOS for static stability and seismic stability evaluations.

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

3. Allows user to edit input for slope geometry and calculate corresponding FOS.

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

5. Allows user to automatically generate and analyze defined numbers 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, cohesion, and friction angle of earth materials and bedding planes.

2. Slope geometry and surcharge boundary loads.

3. Apparent dip of discontinuities can be modeled in an anisotropic angular range (i.e., from 0 to 90 degrees. For this analysis, GSI incorporated isotropic soil strengths for the bedrock. We used an anisotropic angular range between 5 and 55 degrees for this unit, owing to its nature.

4. Pseudo-static earthquake loading. A seismic coefficient (*k*) of 0.15 and a peak horizontal ground acceleration of 0.430 g were used in the analyses.

5. Static and seismic soil strength parameters used in the slope stability analyses are provided in Table 2-1.

TABLE 2-1 - SOIL STRENGTH PARAMETERS

SOIL MATERIALS	SOIL UNIT WEIGHT (pcf)		SHEAR STRENGTH PARAMETERS	
	Total	Saturated	C (psf)	Φ (degrees)
Artificial Fill	125.0	130.0	100.0	29.0
Bedrock	148.0	160.0	500.0	36.0
Bedrock (Along Discontinuity)	148.0	160.0	200.0	30.0

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 may be considered overly conservative; (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 earthquake loading.

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 = 0.1 to 0.15*i*.

The method developed by Krinitzsky, Gould, and Edinger (1993) which was in turn based on Taniguchi and Sasaki (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” California Department of Conservation, California Geological Survey ([CGS], 2008) indicates the State of California recommends using pseudo-static coefficient of 0.15*i* for design earthquakes of M 8.25 or greater and using 0.1 for earthquake parameter M 6.5. Therefore, for reasonable conservatism, a seismic coefficient of 0.15*i* was used in our analysis for a M7.2 event on the design fault. GSI also incorporated a peak horizontal ground acceleration (PGA_M) of 0.884 g into the seismic analysis.

Output Information

Output information includes:

1. All input data.
2. FOS for the 10 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 FOS.
4. Note, that in the analysis, ±1,000 trial surfaces were analyzed for each section for either static or pseudo-static analyses.

Results of Slope Stability Calculations

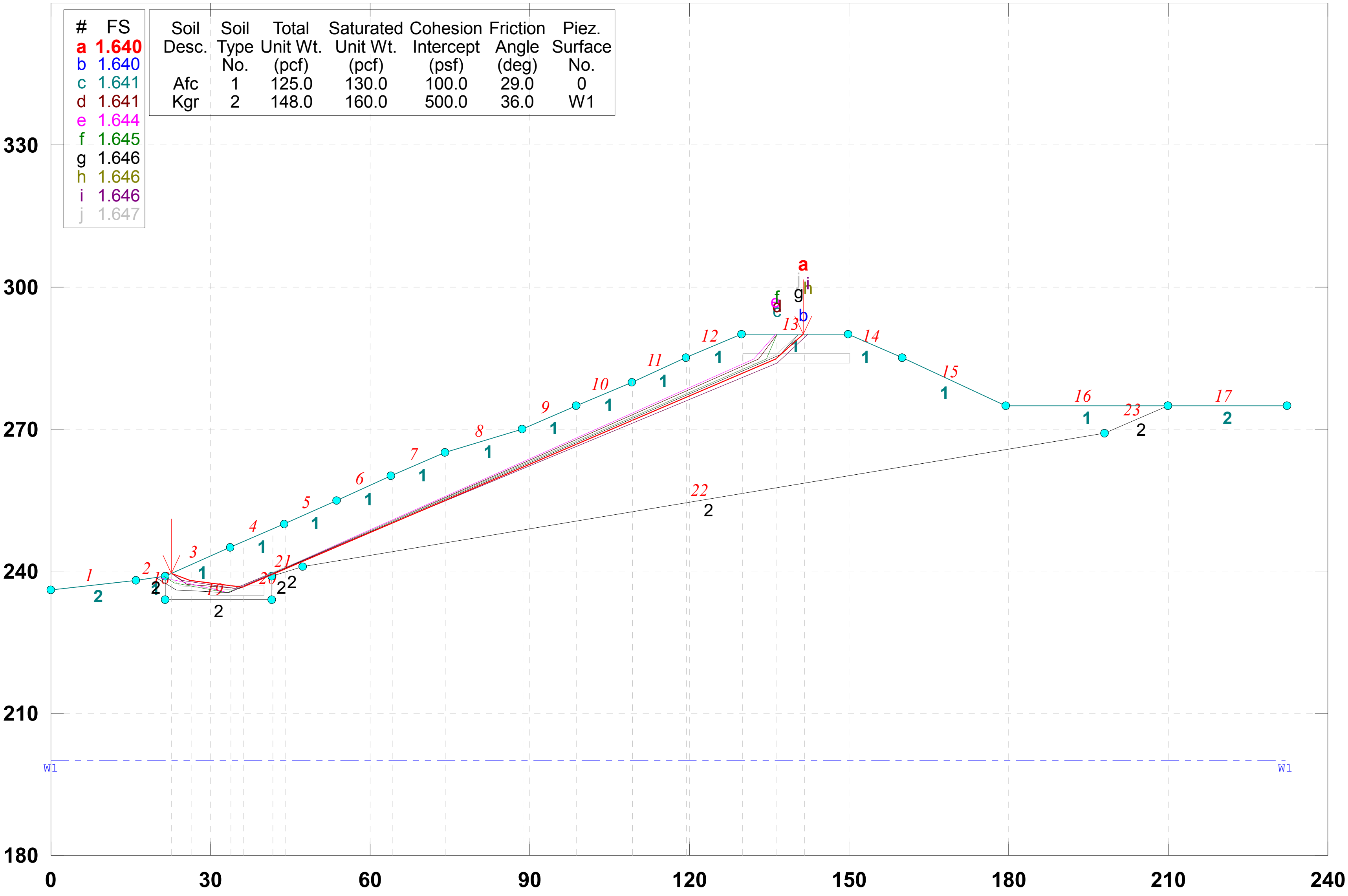
Table 2-2 provides a summary of the results of our stability analyses along Cross Sections A-A' and B-B' (see Plates 2-1, 2-2, 2-3, and 2-4). Computer printouts from the GSTABL7 program are also included herein.

TABLE 2-2 - SUMMARY OF SLOPE STABILITY ANALYSES

LOCATION	FACTOR-OF-SAFETY (FOS) EXISTING SLOPE CONDITION		METHOD	COMMENTS
	STATIC	SEISMIC		
Section A-A' Maximum 2:1 Fill Slope	1.64 (See Plate 2-1)	1.15 (See Plate 2-2)	Janbu	Adequate Static and Seismic FOS
Section B-B' Maximum 2:1 Fill Slope	1.68 (See Plate 2-3)	1.18 (See Plate 2-4)	Janbu	Adequate Static and Seismic FOS
Fill Slope Surficial Stability	1.54 (See Plate 2-14)	-		Adequate FOS
Cut Slope Surficial Stability	1.68 (See Plate 2-15)	-		Adequate FOS

WO 6960 - OCEAN BREEZE RANCH Section A'- A Static

x:\shared\word perfect data\carlsbad\6900\6960 ocean breeze (vessels)\slope stability\section a-a' janbu static.pl2   Run By: GeoSoils, Inc.   8/22/2019   01:44PM



#	FS
a	1.640
b	1.640
c	1.641
d	1.641
e	1.644
f	1.645
g	1.646
h	1.646
i	1.646
j	1.647

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Afc	1	125.0	130.0	100.0	29.0	0
Kgr	2	148.0	160.0	500.0	36.0	W1

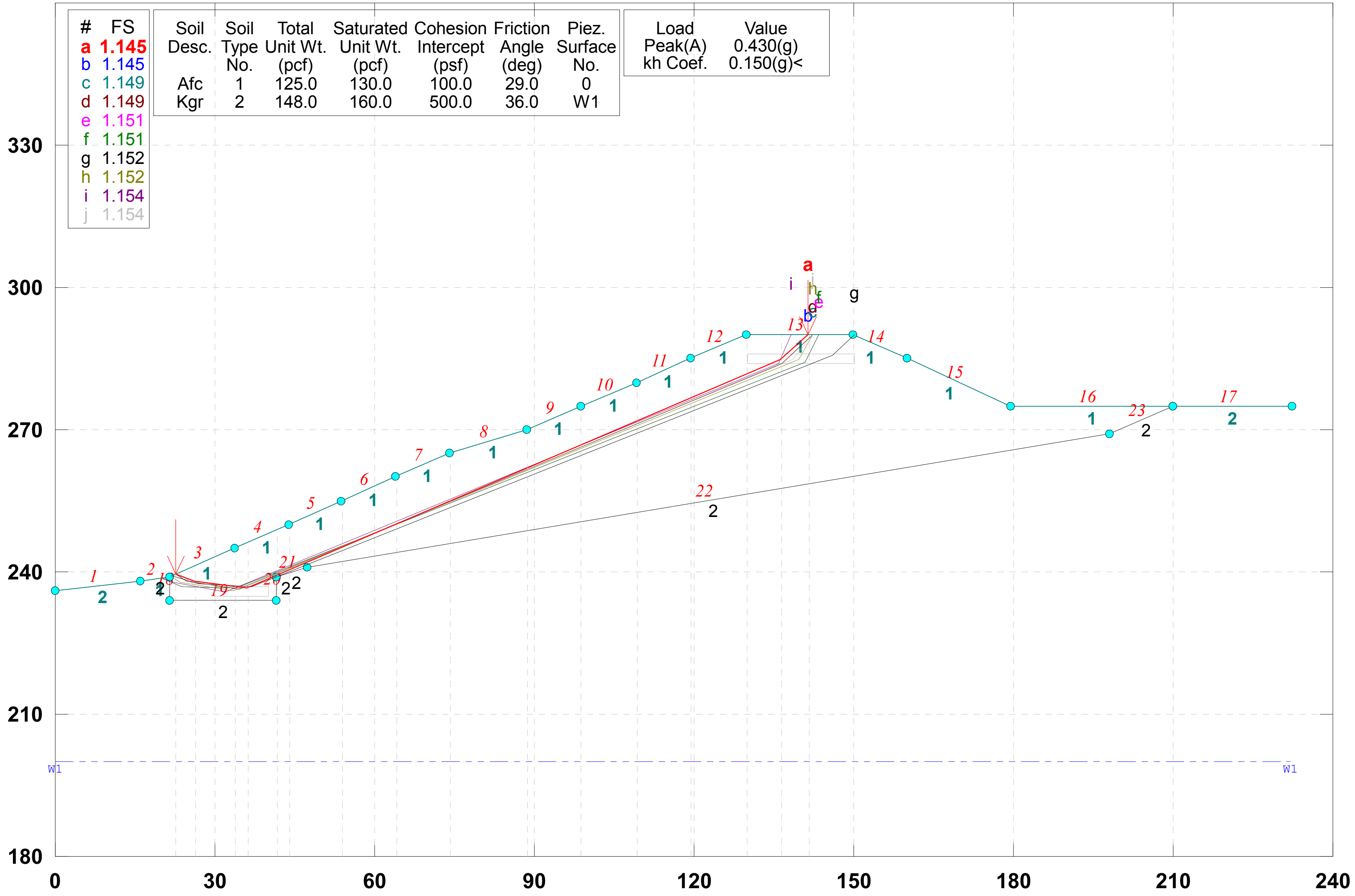
GSTABL7 v.2 FSmin=1.640

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

W.O. 6960-A-SC
PLATE 2-1

WO 6960 - OCEAN BREEZE RANCH Section A'- A Seismic

x:\shared\word perfect data\carlsbad\6900\6960 ocean breeze (vessels)\slope stability\section a-a' janbu seismic.pl2   Run By: GeoSoils, Inc.   8/22/2019   01:44PM



#	FS
a	**1.145**
b	1.145
c	1.149
d	1.149
e	1.151
f	1.151
g	1.152
h	1.152
i	1.154
j	1.154

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Afc	1	125.0	130.0	100.0	29.0	0
Kgr	2	148.0	160.0	500.0	36.0	W1

Load	Value
Peak(A)	0.430(g)
kh Coef.	0.150(g)<

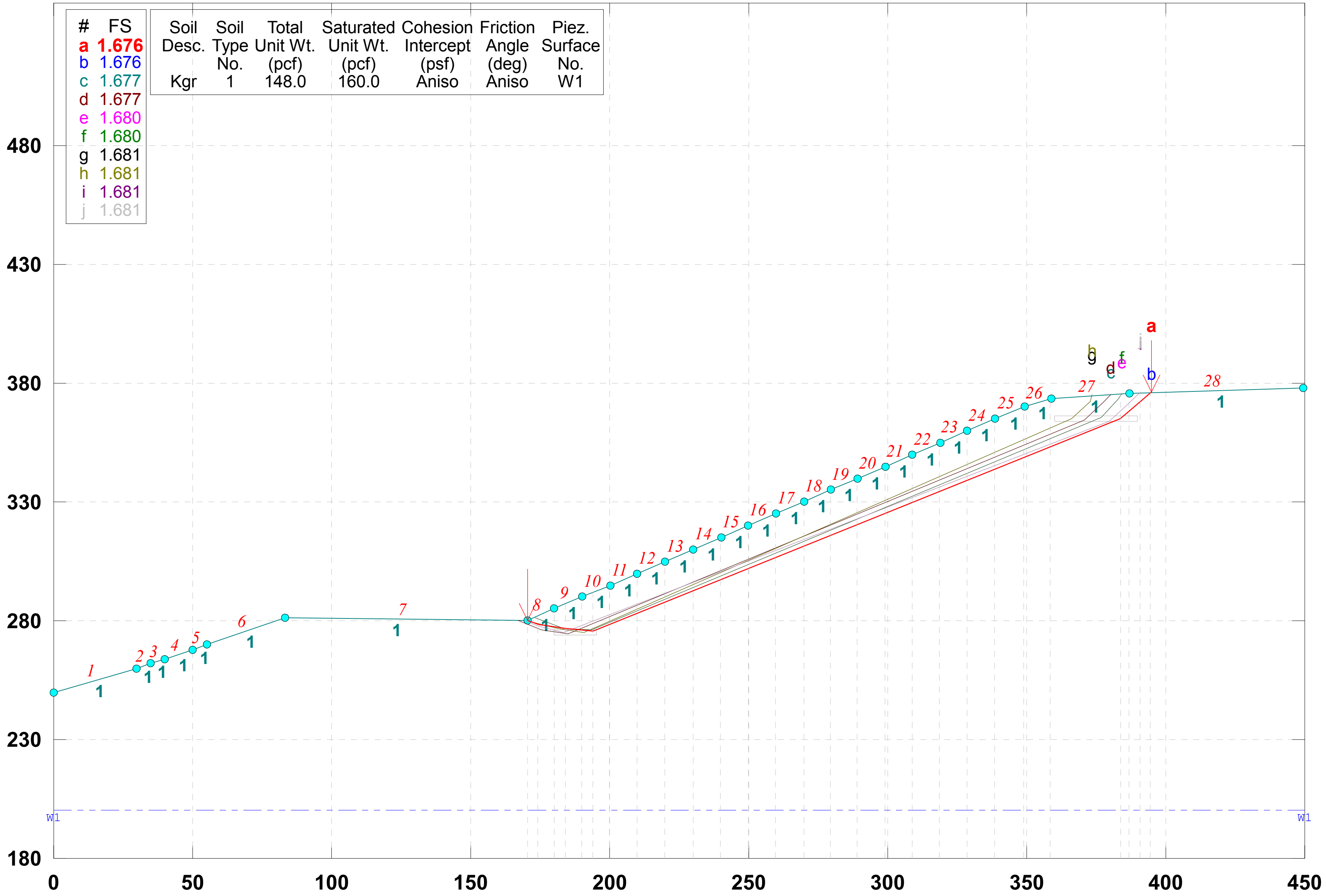
GSTABL7 v.2 FSmin=1.145

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

W.O. 6960-A-SC
PLATE 2-2

WO 6960 - OCEAN BREEZE RANCH Section B-B' Static

x:\shared\word perfect data\carlsbad\6900\6960 ocean breeze (vessels)\slope stability\section b-b' janbu static.pl2   Run By: GeoSoils, Inc.   8/22/2019   11:41AM



#	FS
a	1.676
b	1.676
c	1.677
d	1.677
e	1.680
f	1.680
g	1.681
h	1.681
i	1.681
j	1.681

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Kgr	1	148.0	160.0	Aniso	Aniso	W1

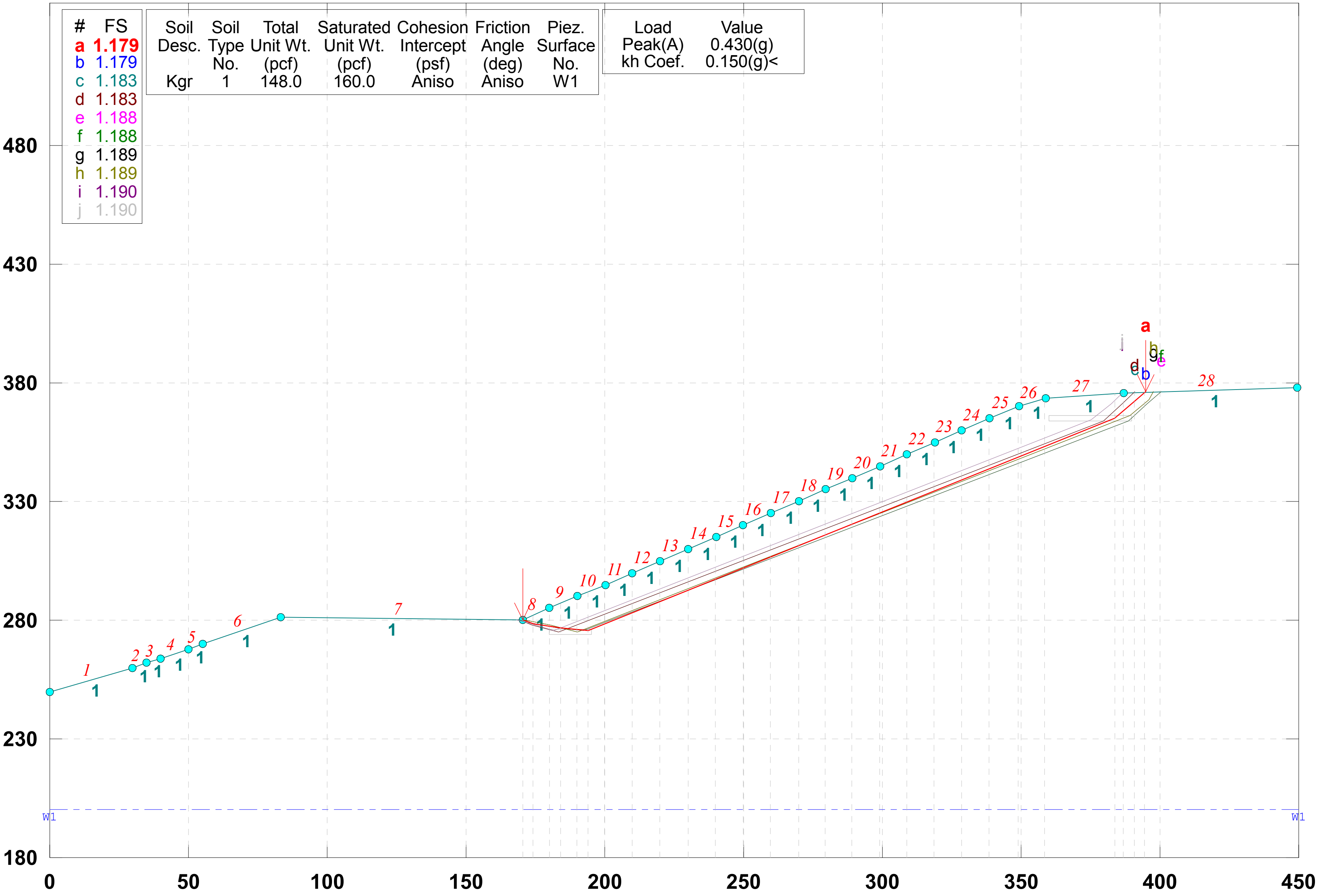
GSTABL7 v.2 FSmin=1.676

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

W.O. 6960-A-SC
PLATE 2-3

WO 6960 - OCEAN BREEZE RANCH Section B-B' Seismic

x:\shared\word perfect data\carlsbad\6900\6960 ocean breeze (vessels)\slope stability\section b-b' janbu seismic aniso.pl2   Run By: GeoSoils, Inc.   8/22/2019   11:57AM



#	FS
a	**1.179**
b	1.179
c	1.183
d	1.183
e	1.188
f	1.188
g	1.189
h	1.189
i	1.190
j	1.190

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Kgr	1	148.0	160.0	Aniso	Aniso	W1

Load	Value
Peak(A)	0.430(g)
kh Coef.	0.150(g)<

GSTABL7 v.2 FSmin=1.179

Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

W.O. 6960-A-SC
PLATE 2-4

```
                              ***  GSTABL7  ***
                ** GSTABL7 by Dr. Garry H. Gregory, Ph.D.,P.E.,D.GE **
      ** Original Version 1.0, January 1996; Current Ver. 2.005.3, Feb. 2013 **
                 (All Rights Reserved-Unauthorized Use Prohibited)
 *********************************************************************************
                       SLOPE STABILITY ANALYSIS SYSTEM
             Modified Bishop, Simplified Janbu, or GLE Method of Slices.
             (Includes Spencer & Morgenstern-Price Type Analysis)
             Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
             Nonlinear Undrained Shear Strength, Curved Phi Envelope,
             Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
             Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
 *********************************************************************************
 Analysis Run Date:          8/22/2019
 Time of Run:                11:41AM
 Run By:                     GeoSoils, Inc.
 Input Data Filename:        X:\shared\Word Perfect Data\CARLSBAD\6900\6960 Ocean Breeze
(Vessels)\Slope Stability\section b-b' janbu static.in
 Output Filename:            X:\shared\Word Perfect Data\CARLSBAD\6900\6960 Ocean Breeze
(Vessels)\Slope Stability\section b-b' janbu static.OUT
 Unit System:                English
 Plotted Output Filename:  X:\shared\Word Perfect Data\CARLSBAD\6900\6960 Ocean Breeze
(Vessels)\Slope Stability\section b-b' janbu static.PLT
 PROBLEM DESCRIPTION:  WO 6960 - OCEAN BREEZE RANCH
                       Section B-B'  Static
 BOUNDARY COORDINATES
    28 Top   Boundaries
    28 Total Boundaries
 Boundary     X-Left     Y-Left    X-Right    Y-Right    Soil Type
    No.        (ft)       (ft)       (ft)       (ft)     Below Bnd
     1          0.00     250.00      29.90     259.90        1
     2         29.90     259.90      34.90     262.15        1
     3         34.90     262.15      40.00     263.75        1
     4         40.00     263.75      50.00     267.75        1
     5         50.00     267.75      55.00     270.00        1
     6         55.00     270.00      83.00     281.00        1
     7         83.00     281.00     170.30     280.00        1
     8        170.30     280.00     180.20     285.00        1
     9        180.20     285.00     190.10     290.00        1
    10        190.10     290.00     200.10     295.00        1
    11        200.10     295.00     210.00     300.00        1
    12        210.00     300.00     220.00     305.00        1
    13        220.00     305.00     230.20     310.00        1
    14        230.20     310.00     240.00     315.00        1
    15        240.00     315.00     250.00     320.00        1
    16        250.00     320.00     259.80     325.00        1
    17        259.80     325.00     270.00     330.00        1
    18        270.00     330.00     279.60     335.00        1
    19        279.60     335.00     289.20     340.00        1
    20        289.20     340.00     299.20     345.00        1
    21        299.20     345.00     309.00     350.00        1
    22        309.00     350.00     318.80     355.00        1
    23        318.80     355.00     328.70     360.00        1
    24        328.70     360.00     338.60     365.00        1
    25        338.60     365.00     349.10     370.00        1
    26        349.10     370.00     358.60     373.70        1
    27        358.60     373.70     386.90     376.00        1
    28        386.90     376.00     449.30     378.00        1
 User Specified Y-Origin =       180.00(ft)
 Default X-Plus Value = 0.00(ft)
 Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  1 Type(s) of Soil
 Soil  Total  Saturated  Cohesion Friction   Pore    Pressure   Piez.
 Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
  No.  (pcf)    (pcf)     (psf)     (deg)    Param.   (psf)     No.
   1   148.0    160.0     500.0     36.0    0.00       0.0      1
ANISOTROPIC STRENGTH PARAMETERS
     1 soil type(s)
```

```
   Soil Type  1 Is Anisotropic
   Number Of Direction Ranges Specified =  3
   Direction    Counterclockwise     Cohesion     Friction
     Range       Direction Limit    Intercept       Angle
      No.            (deg)            (psf)        (deg)
       1               5.0            500.00         36.00
       2              55.0            200.00         30.00
       3              90.0            500.00         36.00
   ANISOTROPIC SOIL NOTES:
      (1) An input value of 0.01 for C and/or Phi will cause Aniso
          C and/or Phi to be ignored in that range.
      (2) An input value of 0.02 for Phi will set both Phi and
          C equal to zero, with no water weight in the tension crack.
      (3) An input value of 0.03 for Phi will set both Phi and
          C equal to zero, with water weight in the tension crack.
   1 PIEZOMETRIC SURFACE(S) SPECIFIED
   Unit Weight of Water =  62.40 (pcf)
   Piezometric Surface No.  1 Specified by  2 Coordinate Points
   Pore Pressure Inclination Factor =  0.50
     Point      X-Water     Y-Water
      No.         (ft)        (ft)
       1          0.00      200.00
       2        450.00      200.00
   Specified Peak Ground Acceleration Coefficient (A) =   0.430(g)
   Specified Horizontal Earthquake Coefficient (kh) =   0.150(g)
   Specified Vertical Earthquake Coefficient (kv) =   0.080(g)
   Specified Seismic Pore-Pressure Factor =   0.000
   EARTHQUAKE DATA HAS BEEN SUPPRESSED
   Janbus Empirical Coef is being used for the case of  c & phi both > 0
   A Critical Failure Surface Searching Method, Using A Random
   Technique For Generating Sliding Block Surfaces, Has Been
   Specified.
   1000 Trial Surfaces Have Been Generated.
   2 Boxes Specified For Generation Of Central Block Base
   Length Of Line Segments For Active And Passive Portions Of
   Sliding Block Is  10.0
   Box        X-Left     Y-Left    X-Right    Y-Right      Height
   No.         (ft)       (ft)       (ft)       (ft)         (ft)
    1         180.00     275.00     195.00     275.00        2.00
    2         360.00     365.00     390.00     365.00        2.00
   Following Are Displayed The Ten Most Critical Of The Trial
         Failure Surfaces Evaluated. They Are
         Ordered - Most Critical First.
         * * Safety Factors Are Calculated By The Simplified Janbu Method * *
         Total Number of Trial Surfaces Attempted =  1000
         Number of Trial Surfaces With Valid FS = 1000
         Statistical Data On All Valid FS Values:
            FS Max =   1.901   FS Min =   1.676   FS Ave =   1.762
            Standard Deviation =    0.046   Coefficient of Variation =    2.59 %
         Failure Surface Specified By  7 Coordinate Points
           Point      X-Surf      Y-Surf
            No.        (ft)        (ft)
             1       170.638     280.170
             2       174.327     278.500
             3       184.198     276.899
             4       194.135     275.778
             5       383.860     365.300
             6       390.893     372.409
             7       394.609     376.247
                Factor of Safety
               ***    1.676   ***
              Individual data on the    26  slices
                        Water  Water     Tie     Tie     Earthquake
                        Force  Force    Force   Force       Force   Surcharge
Slice  Width   Weight    Top    Bot     Norm     Tan     Hor     Ver    Load
 No.    (ft)    (lbs)   (lbs)  (lbs)    (lbs)   (lbs)   (lbs)   (lbs)   (lbs)
  1      3.7     964.8     0.0     0.0       0.      0.     0.0    77.2      0.0
  2      5.9    4775.0     0.0     0.0       0.      0.     0.0   382.0      0.0
  3      4.0    5198.9     0.0     0.0       0.      0.     0.0   415.9      0.0
  4      5.9   10433.0     0.0     0.0       0.      0.     0.0   834.6      0.0
```

```
 5      4.0    8959.3     0.0     0.0       0.      0.     0.0   716.7      0.0
 6      6.0   14411.3     0.0     0.0       0.      0.     0.0  1152.9      0.0
 7      9.9   24281.2     0.0     0.0       0.      0.     0.0  1942.5      0.0
 8     10.0   24978.0     0.0     0.0       0.      0.     0.0  1998.2      0.0
 9     10.2   25831.2     0.0     0.0       0.      0.     0.0  2066.5      0.0
10      9.8   25226.5     0.0     0.0       0.      0.     0.0  2018.1      0.0
11     10.0   26227.7     0.0     0.0       0.      0.     0.0  2098.2      0.0
12      9.8   26179.8     0.0     0.0       0.      0.     0.0  2094.4      0.0
13     10.2   27673.4     0.0     0.0       0.      0.     0.0  2213.9      0.0
14      9.6   26512.5     0.0     0.0       0.      0.     0.0  2121.0      0.0
15      9.6   27180.6     0.0     0.0       0.      0.     0.0  2174.4      0.0
16     10.0   28869.3     0.0     0.0       0.      0.     0.0  2309.5      0.0
17      9.8   28768.6     0.0     0.0       0.      0.     0.0  2301.5      0.0
18      9.8   29313.8     0.0     0.0       0.      0.     0.0  2345.1      0.0
19      9.9   30129.1     0.0     0.0       0.      0.     0.0  2410.3      0.0
20      9.9   30610.6     0.0     0.0       0.      0.     0.0  2448.8      0.0
21     10.5   32756.5     0.0     0.0       0.      0.     0.0  2620.5      0.0
22      9.5   29118.7     0.0     0.0       0.      0.     0.0  2329.5      0.0
23     25.3   57521.2     0.0     0.0       0.      0.     0.0  4601.7      0.0
24      3.0    4066.7     0.0     0.0       0.      0.     0.0   325.3      0.0
25      4.0    3353.0     0.0     0.0       0.      0.     0.0   268.2      0.0
26      3.7    1022.6     0.0     0.0       0.      0.     0.0    81.8      0.0
        Failure Surface Specified By  7 Coordinate Points
          Point      X-Surf      Y-Surf
           No.        (ft)        (ft)
            1        170.638      280.170
            2        174.327      278.500
            3        184.198      276.899
            4        194.135      275.778
            5        383.860      365.300
            6        390.893      372.409
            7        394.609      376.247
               Factor of Safety
              ***    1.676   ***
        Failure Surface Specified By  6 Coordinate Points
          Point      X-Surf      Y-Surf
           No.        (ft)        (ft)
            1        167.300      280.034
            2        175.402      276.201
            3        185.287      274.692
            4        370.853      364.277
            5        377.385      371.849
            6        380.245      375.459
               Factor of Safety
              ***    1.677   ***
        Failure Surface Specified By  6 Coordinate Points
          Point      X-Surf      Y-Surf
           No.        (ft)        (ft)
            1        167.300      280.034
            2        175.402      276.201
            3        185.287      274.692
            4        370.853      364.277
            5        377.385      371.849
            6        380.245      375.459
               Factor of Safety
              ***    1.677   ***
        Failure Surface Specified By  7 Coordinate Points
          Point      X-Surf      Y-Surf
           No.        (ft)        (ft)
            1        173.249      281.490
            2        174.354      280.822
            3        183.412      276.585
            4        193.391      275.942
            5        376.807      365.766
            6        382.870      373.718
            7        384.072      375.770
               Factor of Safety
              ***    1.680   ***
        Failure Surface Specified By  7 Coordinate Points
          Point      X-Surf      Y-Surf
```

```
   No.        (ft)        (ft)
    1       173.249      281.490
    2       174.354      280.822
    3       183.412      276.585
    4       193.391      275.942
    5       376.807      365.766
    6       382.870      373.718
    7       384.072      375.770
       Factor of Safety
      ***    1.680   ***
Failure Surface Specified By  7 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1       170.522      280.112
    2       171.315      279.430
    3       180.966      276.813
    4       190.773      274.857
    5       366.004      364.981
    6       372.997      372.129
    7       373.525      374.913
       Factor of Safety
      ***    1.681   ***
Failure Surface Specified By  7 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1       170.522      280.112
    2       171.315      279.430
    3       180.966      276.813
    4       190.773      274.857
    5       366.004      364.981
    6       372.997      372.129
    7       373.525      374.913
       Factor of Safety
      ***    1.681   ***
Failure Surface Specified By  6 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1       169.275      280.012
    2       173.938      277.646
    3       183.548      274.880
    4       379.620      364.203
    5       386.566      371.396
    6       391.179      376.137
       Factor of Safety
      ***    1.681   ***
Failure Surface Specified By  6 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1       169.275      280.012
    2       173.938      277.646
    3       183.548      274.880
    4       379.620      364.203
    5       386.566      371.396
    6       391.179      376.137
       Factor of Safety
      ***    1.681   ***
          **** END OF GSTABL7 OUTPUT ****
```

```
                              ***  GSTABL7  ***
               ** GSTABL7 by Dr. Garry H. Gregory, Ph.D.,P.E.,D.GE **
        ** Original Version 1.0, January 1996; Current Ver. 2.005.3, Feb. 2013 **
                  (All Rights Reserved-Unauthorized Use Prohibited)
    *********************************************************************************
                        SLOPE STABILITY ANALYSIS SYSTEM
           Modified Bishop, Simplified Janbu, or GLE Method of Slices.
           (Includes Spencer & Morgenstern-Price Type Analysis)
           Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
           Nonlinear Undrained Shear Strength, Curved Phi Envelope,
           Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
           Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
    *********************************************************************************
    Analysis Run Date:        8/22/2019
    Time of Run:              11:57AM
    Run By:                   GeoSoils, Inc.
    Input Data Filename:      X:\shared\Word Perfect Data\CARLSBAD\6900\6960 Ocean Breeze
(Vessels)\Slope Stability\section b-b' janbu seismic Aniso.in
    Output Filename:          X:\shared\Word Perfect Data\CARLSBAD\6900\6960 Ocean Breeze
(Vessels)\Slope Stability\section b-b' janbu seismic Aniso.OUT
    Unit System:              English
    Plotted Output Filename:  X:\shared\Word Perfect Data\CARLSBAD\6900\6960 Ocean Breeze
(Vessels)\Slope Stability\section b-b' janbu seismic Aniso.PLT
    PROBLEM DESCRIPTION:  WO 6960 - OCEAN BREEZE RANCH
                          Section B-B'  Seismic
    BOUNDARY COORDINATES
       28 Top   Boundaries
       28 Total Boundaries
    Boundary     X-Left     Y-Left    X-Right    Y-Right    Soil Type
       No.        (ft)       (ft)       (ft)       (ft)     Below Bnd
        1          0.00     250.00      29.90     259.90        1
        2         29.90     259.90      34.90     262.15        1
        3         34.90     262.15      40.00     263.75        1
        4         40.00     263.75      50.00     267.75        1
        5         50.00     267.75      55.00     270.00        1
        6         55.00     270.00      83.00     281.00        1
        7         83.00     281.00     170.30     280.00        1
        8        170.30     280.00     180.20     285.00        1
        9        180.20     285.00     190.10     290.00        1
       10        190.10     290.00     200.10     295.00        1
       11        200.10     295.00     210.00     300.00        1
       12        210.00     300.00     220.00     305.00        1
       13        220.00     305.00     230.20     310.00        1
       14        230.20     310.00     240.00     315.00        1
       15        240.00     315.00     250.00     320.00        1
       16        250.00     320.00     259.80     325.00        1
       17        259.80     325.00     270.00     330.00        1
       18        270.00     330.00     279.60     335.00        1
       19        279.60     335.00     289.20     340.00        1
       20        289.20     340.00     299.20     345.00        1
       21        299.20     345.00     309.00     350.00        1
       22        309.00     350.00     318.80     355.00        1
       23        318.80     355.00     328.70     360.00        1
       24        328.70     360.00     338.60     365.00        1
       25        338.60     365.00     349.10     370.00        1
       26        349.10     370.00     358.60     373.70        1
       27        358.60     373.70     386.90     376.00        1
       28        386.90     376.00     449.30     378.00        1
    User Specified Y-Origin =      180.00(ft)
    Default X-Plus Value = 0.00(ft)
    Default Y-Plus Value = 0.00(ft)
   ISOTROPIC SOIL PARAMETERS
     1 Type(s) of Soil
    Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
    Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
     No.  (pcf)    (pcf)     (psf)     (deg)    Param.   (psf)     No.
      1   148.0    160.0     500.0     36.0    0.00      0.0      1
   ANISOTROPIC STRENGTH PARAMETERS
        1 soil type(s)
```

```
   Soil Type  1 Is Anisotropic
   Number Of Direction Ranges Specified =  3
   Direction    Counterclockwise     Cohesion     Friction
     Range       Direction Limit    Intercept       Angle
       No.            (deg)            (psf)         (deg)
        1               5.0            500.00         36.00
        2              55.0            200.00         30.00
        3              90.0            500.00         36.00
   ANISOTROPIC SOIL NOTES:
      (1) An input value of 0.01 for C and/or Phi will cause Aniso
          C and/or Phi to be ignored in that range.
      (2) An input value of 0.02 for Phi will set both Phi and
          C equal to zero, with no water weight in the tension crack.
      (3) An input value of 0.03 for Phi will set both Phi and
          C equal to zero, with water weight in the tension crack.
   1 PIEZOMETRIC SURFACE(S) SPECIFIED
   Unit Weight of Water =  62.40 (pcf)
   Piezometric Surface No.  1 Specified by  2 Coordinate Points
   Pore Pressure Inclination Factor =  0.50
     Point      X-Water     Y-Water
       No.         (ft)        (ft)
        1          0.00      200.00
        2        450.00      200.00
   Specified Peak Ground Acceleration Coefficient (A) =   0.430(g)
   Specified Horizontal Earthquake Coefficient (kh) =   0.150(g)
   Specified Vertical Earthquake Coefficient (kv) =   0.000(g)
   Specified Seismic Pore-Pressure Factor =   0.000
   Janbus Empirical Coef is being used for the case of  c & phi both > 0
   A Critical Failure Surface Searching Method, Using A Random
   Technique For Generating Sliding Block Surfaces, Has Been
   Specified.
   1000 Trial Surfaces Have Been Generated.
   2 Boxes Specified For Generation Of Central Block Base
   Length Of Line Segments For Active And Passive Portions Of
   Sliding Block Is  10.0
   Box        X-Left     Y-Left    X-Right    Y-Right      Height
   No.         (ft)       (ft)       (ft)       (ft)        (ft)
    1         180.00     275.00     195.00     275.00        2.00
    2         360.00     365.00     390.00     365.00        2.00
   Following Are Displayed The Ten Most Critical Of The Trial
         Failure Surfaces Evaluated. They Are
         Ordered - Most Critical First.
         * * Safety Factors Are Calculated By The Simplified Janbu Method * *
         Total Number of Trial Surfaces Attempted =  1000
         Number of Trial Surfaces With Valid FS = 1000
         Statistical Data On All Valid FS Values:
            FS Max =   1.421   FS Min =   1.179   FS Ave =   1.269
            Standard Deviation =    0.049   Coefficient of Variation =    3.87 %
         Failure Surface Specified By  7 Coordinate Points
           Point      X-Surf      Y-Surf
             No.        (ft)        (ft)
              1       170.638     280.170
              2       174.327     278.500
              3       184.198     276.899
              4       194.135     275.778
              5       383.860     365.300
              6       390.893     372.409
              7       394.609     376.247
                  Factor of Safety
                 ***    1.179   ***
                Individual data on the    26  slices
                          Water  Water     Tie     Tie     Earthquake
                          Force  Force    Force   Force       Force   Surcharge
Slice  Width   Weight     Top    Bot      Norm     Tan     Hor     Ver    Load
 No.    (ft)    (lbs)    (lbs)  (lbs)     (lbs)   (lbs)   (lbs)   (lbs)   (lbs)
  1      3.7     964.8     0.0     0.0       0.      0.   144.7     0.0      0.0
  2      5.9    4775.0     0.0     0.0       0.      0.   716.3     0.0      0.0
  3      4.0    5198.9     0.0     0.0       0.      0.   779.8     0.0      0.0
  4      5.9   10433.0     0.0     0.0       0.      0.  1565.0     0.0      0.0
  5      4.0    8959.3     0.0     0.0       0.      0.  1343.9     0.0      0.0
```

```
  6      6.0   14411.3     0.0     0.0       0.      0.  2161.7     0.0      0.0
  7      9.9   24281.2     0.0     0.0       0.      0.  3642.2     0.0      0.0
  8     10.0   24978.0     0.0     0.0       0.      0.  3746.7     0.0      0.0
  9     10.2   25831.2     0.0     0.0       0.      0.  3874.7     0.0      0.0
 10      9.8   25226.5     0.0     0.0       0.      0.  3784.0     0.0      0.0
 11     10.0   26227.7     0.0     0.0       0.      0.  3934.2     0.0      0.0
 12      9.8   26179.8     0.0     0.0       0.      0.  3927.0     0.0      0.0
 13     10.2   27673.4     0.0     0.0       0.      0.  4151.0     0.0      0.0
 14      9.6   26512.5     0.0     0.0       0.      0.  3976.9     0.0      0.0
 15      9.6   27180.6     0.0     0.0       0.      0.  4077.1     0.0      0.0
 16     10.0   28869.3     0.0     0.0       0.      0.  4330.4     0.0      0.0
 17      9.8   28768.6     0.0     0.0       0.      0.  4315.3     0.0      0.0
 18      9.8   29313.8     0.0     0.0       0.      0.  4397.1     0.0      0.0
 19      9.9   30129.1     0.0     0.0       0.      0.  4519.4     0.0      0.0
 20      9.9   30610.6     0.0     0.0       0.      0.  4591.6     0.0      0.0
 21     10.5   32756.5     0.0     0.0       0.      0.  4913.5     0.0      0.0
 22      9.5   29118.7     0.0     0.0       0.      0.  4367.8     0.0      0.0
 23     25.3   57521.2     0.0     0.0       0.      0.  8628.2     0.0      0.0
 24      3.0    4066.7     0.0     0.0       0.      0.   610.0     0.0      0.0
 25      4.0    3353.0     0.0     0.0       0.      0.   503.0     0.0      0.0
 26      3.7    1022.6     0.0     0.0       0.      0.   153.4     0.0      0.0
        Failure Surface Specified By  7 Coordinate Points
          Point      X-Surf      Y-Surf
           No.        (ft)        (ft)
            1        170.638      280.170
            2        174.327      278.500
            3        184.198      276.899
            4        194.135      275.778
            5        383.860      365.300
            6        390.893      372.409
            7        394.609      376.247
               Factor of Safety
              ***    1.179   ***
        Failure Surface Specified By  6 Coordinate Points
          Point      X-Surf      Y-Surf
           No.        (ft)        (ft)
            1        169.275      280.012
            2        173.938      277.646
            3        183.548      274.880
            4        379.620      364.203
            5        386.566      371.396
            6        391.179      376.137
               Factor of Safety
              ***    1.183   ***
        Failure Surface Specified By  6 Coordinate Points
          Point      X-Surf      Y-Surf
           No.        (ft)        (ft)
            1        169.275      280.012
            2        173.938      277.646
            3        183.548      274.880
            4        379.620      364.203
            5        386.566      371.396
            6        391.179      376.137
               Factor of Safety
              ***    1.183   ***
        Failure Surface Specified By  7 Coordinate Points
          Point      X-Surf      Y-Surf
           No.        (ft)        (ft)
            1        171.041      280.374
            2        172.786      278.768
            3        182.541      276.568
            4        192.483      275.493
            5        388.585      364.098
            6        395.342      371.469
            7        400.260      376.428
               Factor of Safety
              ***    1.188   ***
        Failure Surface Specified By  7 Coordinate Points
          Point      X-Surf      Y-Surf
           No.        (ft)        (ft)
```

```
    1        171.041      280.374
    2        172.786      278.768
    3        182.541      276.568
    4        192.483      275.493
    5        388.585      364.098
    6        395.342      371.469
    7        400.260      376.428
       Factor of Safety
      ***    1.188   ***
Failure Surface Specified By  6 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1        171.031      280.369
    2        180.505      277.708
    3        190.183      275.194
    4        389.376      365.971
    5        396.270      373.214
    6        397.773      376.348
       Factor of Safety
      ***    1.189   ***
Failure Surface Specified By  6 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1        171.031      280.369
    2        180.505      277.708
    3        190.183      275.194
    4        389.376      365.971
    5        396.270      373.214
    6        397.773      376.348
       Factor of Safety
      ***    1.189   ***
Failure Surface Specified By  6 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1        168.052      280.026
    2        172.093      278.797
    3        181.519      275.457
    4        375.445      364.429
    5        382.510      371.506
    6        386.375      375.957
       Factor of Safety
      ***    1.190   ***
Failure Surface Specified By  6 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1        168.052      280.026
    2        172.093      278.797
    3        181.519      275.457
    4        375.445      364.429
    5        382.510      371.506
    6        386.375      375.957
       Factor of Safety
      ***    1.190   ***
          **** END OF GSTABL7 OUTPUT ****
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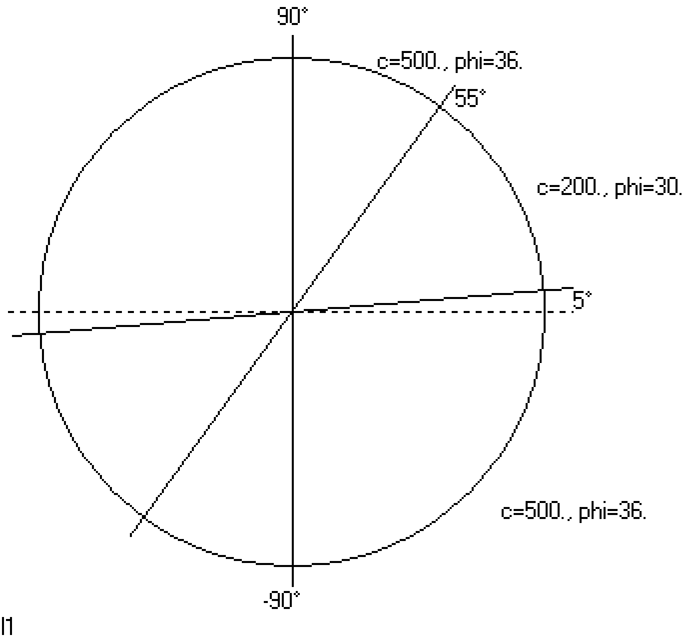
Anisotropic Soil Definition

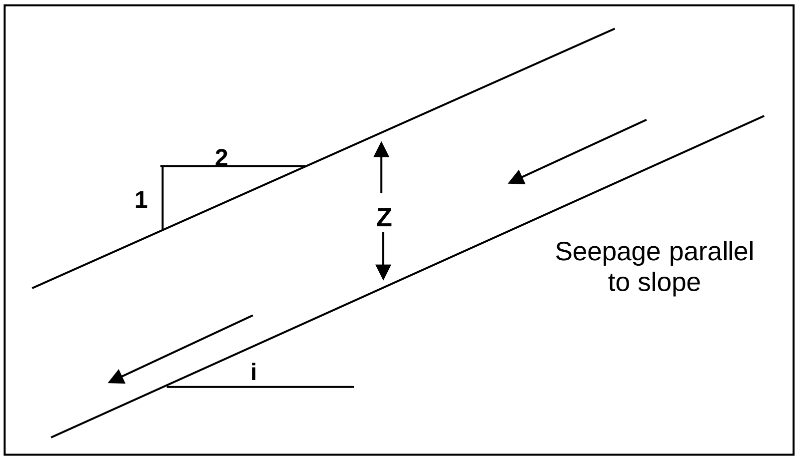


Soil1

WO 6960 - OCEAN BREEZE RANCH

W.O. 6960-A-SC
PLATE 2-13

SURFICIAL SLOPE STABILITY ANALYSIS



Tract/Project:	Ocean Breeze Ranch
Material Type:	Engineered Fill
	Silty SAND

Depth of Saturation (z)	4	feet
Slope Angle (i) (for 2:1 slopes)	26.6	degrees
Unit Weight of Water (γ_w)	62.4	lb/ft^3
Saturated Unit Weight of Soil (γ_{sat})	130	lb/ft^3
Apparent Angle of Internal Friction (ϕ)	29	degrees
Apparent Cohesion (C)	200	lb/ft^2

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

DEPTH OF SATURATION	SLOPE	FACTOR OF SAFETY
4 FEET	2:1	1.54

GeoSoils, Inc.

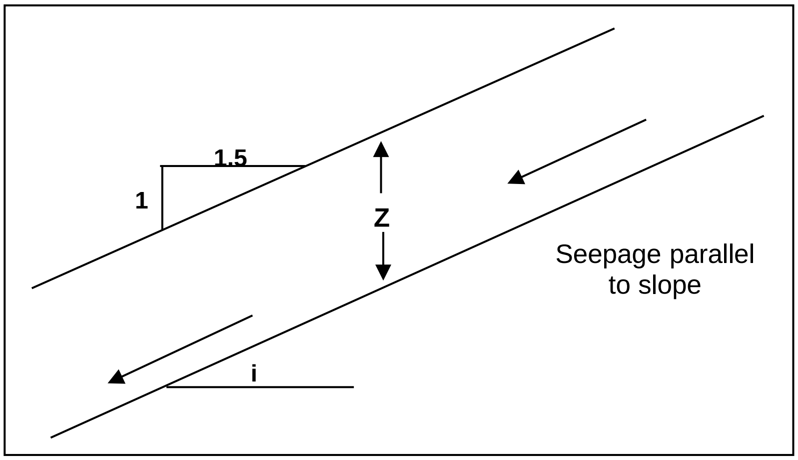
W.O. 6960-A8-SC

SURFICIAL SLOPE STABILITY

2: 1 SLOPE

Plate 2-14

SURFICIAL SLOPE STABILITY ANALYSIS



Tract/Project:	Ocean Breeze Ranch
Material Type:	Bedrock

Depth of Saturation (z)	4	feet
Slope Angle (i) (for 1.5:1 slopes)	33.7	degrees
Unit Weight of Water (γ_w)	62.4	lb/ft^3
Saturated Unit Weight of Soil (γ_{sat})	160	lb/ft^3
Apparent Angle of Internal Friction (ϕ)	36	degrees
Apparent Cohesion (C)	300	lb/ft^2

Fs = Static Safety Factor =

$$\frac{z\,(\gamma_{sat}-\gamma_w)\,\mathrm{Cos}^2(i)\,\mathrm{Tan}\,(\phi) + C}{z\,(\gamma_{sat})\,\mathrm{Sin}\,(i)\,\mathrm{Cos}\,(i)}$$

DEPTH OF SATURATION	SLOPE	FACTOR OF SAFETY
4 FEET	1.5:1	1.68

GeoSoils, Inc.

W.O. 6960-A8-SC

SURFICIAL SLOPE STABILITY

1½: 1 SLOPE

Plate 2-15

APPENDIX 3

UPDATE RECOMMENDATIONS

APPENDIX 3 - TABLE OF CONTENTS

SEISMIC SHAKING PARAMETERS

Based on the site conditions, the following table summarizes the updated site-specific design criteria obtained from the 2016 CBC (CBSC, 2016), Chapter 16 Structural Design, Section 1613, Earthquake Loads. The computer program "U.S. Seismic Design Maps, provided by the Structural Engineers Association of California/California's Office of Statewide Health Planning and Development was utilized for design (https://seismicmaps.org/).

2016 CBC SEISMIC DESIGN PARAMETERS			
PARAMETER	**ALLUVIUM/ OLDER ALLUVIUM**	**GRANITIC BEDROCK**	**2016 CBC AND/OR REFERENCE**
Site Class	D	C (>10' of fill)	Section 1613.3.2/ASCE 7-10 (Chapter 20)
Spectral Response - (0.2 sec), S_s	1.137 g	1.137g	Figure 1613.3.1(1)
Spectral Response - (1 sec), S_1	0.443g	0.443g	Figure 1613.3.1(2)
Site Coefficient, F_a	1.045	1.00	Table 1613.3.3(1)
Site Coefficient, F_v	1.557	1.357	Table1613.3.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec), S_{MS}	1.188g	1.137g	Section 1613.3.3 (Eqn 16-37)
Maximum Considered Earthquake Spectral Response Acceleration (1 sec), S_{M1}	0.689g	0.601g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	0.792g	0.758g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.460g	0.401g	Section 1613.3.4 (Eqn 16-40)
PGA_M	0.46g	0.43g	ASCE 7-10 (Eqn 11.8.1)
Seismic Design Category	D	D	Section 1613.3.5/ASCE 7-10 (Table 11.6-1 or 11.6-2)

ROCK HARDNESS EVALUATION

SEISMIC LINE NO.	GENERAL RIPPABILITY (ASSUMING A D9L DOZER OR CAT 235 HOE, OR EQUIVALENT)
ST-1 (PA-3)	Rippable and trenchable to depths explored of 30 feet. Difficult trenching below depths of 2 to 4 feet. Localized blasting and/or rock breaking may not be precluded below depths of 10 feet.
ST-2 (open space between PA-1 and former PA-4)	Rippable and trenchable to depths explored of 30 feet. Difficult trenching below depths of 2½ to 3 feet. Localized blasting and/or rock breaking may not be precluded below depths of 10 feet.

SEISMIC LINE NO.	GENERAL RIPPABILITY (ASSUMING A D9L DOZER OR CAT 235 HOE, OR EQUIVALENT)
ST-3 (former PA-4)	Rippable and trenchable to depths explored of 30 feet. Moderate to difficult trenching below depths of 3½ feet. Localized blasting and/or rock breaking may not be precluded below depths of 30 feet.
ST-4 (PA-1)	Rippable to depths explored of 30 feet. Not trenchable below depths of 3 to 4 feet. Localized blasting and/or rock breaking may not be precluded below depths of 10 feet. Oversize material is significant.
ST-101 (PA-1)	Rippable and trenchable to depths explored of ±30 feet. Difficult trenching below depths of 2½ to 5½ feet. Localized blasting and/or rock breaking may not be precluded below depths of 30 feet.
ST-102 (PA-1)	Rippable and trenchable to depths explored of ±30 feet. Difficult trenching below a depth of 2½ feet. Localized blasting and/or rock breaking may not be precluded below depths of 30 feet.
ST-103 (PA-1)	Rippable and trenchable to depths explored of ±38 feet. Difficult trenching below depths of 4½ to 7 feet. Localized blasting and/or rock breaking may not be precluded below depths of 30 feet.

Rock Hardness Summary

In general, utilizing the seismic data, it appears that the site area in the vicinity of our seismic lines may be characterized as being underlain by a surficial soils (fill, colluvium, weathered rock) to depths ranging from about ±1 to about ±7 feet in thickness, with less weather bedrock below those depths. At depths inferred to be approximately 30 feet or more, relatively fresh and very dense granitic bedrock likely exists. Based on all of the above, the need for overexcavation, blasting and/or line shooting would be anticipated on the site, should proposed cut grades exceed the depths indicated herein, in areas underlain with granitic bedrock (see Plate 1), and may be required near the surface. It should be noted that a conventional rubber-tired backhoe will experience non-productive trenching at seismic velocities much less than ±2,000 to 2,500 fps. The seismic refraction data presented herein should be further reviewed in conjunction with final grading plans (when available). It should be noted that due to the variability of bedrock weathering, and the potential for local boulders, or less weathered bedrock, very difficult ripping, rock breaking, and/or blasting cannot be entirely precluded at shallower depths, even at or near the surface.

EARTHWORK CONSTRUCTION RECOMMENDATIONS

General

Remedial earthwork will likely be necessary for the support of the proposed settlement-sensitive improvements. Remedial grading should conform to the guidelines presented in Appendix J of the 2016 CBC (CBSC, 2016), the requirements of the County, and the General Earthwork, Grading Guidelines, and Preliminary Criteria presented in Appendix H of GSI (2016), except where specifically superceded in the text of this report. In case of conflict, the more onerous code or recommendations should govern. Prior to grading, a GSI representative should be present at the pre-construction meeting to provide additional grading guidelines, if needed, and review the earthwork schedule.

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

Demolition/Grubbing

1. Vegetation, and any miscellaneous deleterious debris generated from the demolition of existing site improvements should be removed from the areas of proposed grading/earthwork.

2. Cavities or loose soils remaining after demolition and site clearance should be cleaned out and observed by the geotechnical consultant. 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.

3. Any septic systems encountered should be removed and disposed of per County guidelines.

Treatment of Existing Ground

The treatment of existing ground will vary by area/geologic conditions onsite, and may be subdivided into at least three (3) general cases, as follows:

Case I - Areas underlain with near surface, older alluvium and/or granitic bedrock.

Case II - Areas underlain with alluvium below a shallow groundwater table.

A discussion of existing ground treatment is presented for each case as follows:

Case I, Areas Underlain With Near Surface, Older Alluvium and/or Granitic Bedrock (Settlement Group Areas 1 and 2)

1. Areas underlain with near surface, older alluvium and/or granitic rock generally occur in the vicinity of Planning Areas PA-1, and PA-3.

2. Where not removed by the planned excavations, all undocumented fill, colluvium, alluvium, and weathered older alluvium/bedrock should be removed to competent older alluvium/bedrock, cleaned of deleterious materials, moisture conditioned, and recompacted within areas proposed for settlement-sensitive improvements. In general, the remedial removal excavations are anticipated to be on the order of 1½ to 5½ feet, to depths potentially as much as 17 to 18 feet locally (lower elevations of Planning Areas PA-1 and PA-3), where observed in our subsurface explorations. However, local deeper removal excavations elsewhere cannot be precluded and should be anticipated. Actual depths of removals will be evaluated in the field during grading by the soil engineer. This recommended earthwork does not include in-place ground improvement/treatment.

3. Subsequent to the above removals, the upper 8 inches of the exposed subsoils/bedrock should be scarified, brought to <u>at least</u> optimum moisture content, and recompacted to a minimum relative compaction of 90 percent of the laboratory standard (ASTM D 1557), prior to any fill placement.

4. Localized deeper removals may be necessary due to buried drainage channel meanders or dry porous materials. The project soils engineer/geologist should observe all removal areas during the grading.

Case II, Areas Underlain with Loose Alluvium and a Shallow Groundwater Table (Settlement Group Area 3):

1. Areas underlain with loose, surficial deposits of alluvium and a shallow groundwater table, generally occur in the vicinity of Planning Area PA-2.

2. Alluvium should be removed to near the existing groundwater table, cleaned of deleterious materials, moisture conditioned, and recompacted within areas proposed for settlement-sensitive improvements. In general, the remedial removal excavations are anticipated to near the groundwater table, at depths on the order of 10 to 17 feet below existing grades, and be completed to at least 15 feet outside the improvement. Excavations may generate wet materials that will require "drying back" to a workable moisture content prior to placement as compacted fill. In order to reduce damaging effects of liquefaction to tolerable levels an additional 5 to 15 feet below the groundwater may also be modified (in-place ground improvement) or using previously discussed grading techniques.

3. Yielding subgrades near the groundwater table may require bottom stabilization with stone prior to fill placement. In this case, stones consisting of gravel to cobble size material should be worked into the soil until a relatively firm bottom is achieved. The use of crushed rock and Mirafi HP 570 should be considered to stabilize removal bottoms.

4. For Planning Area PA-2, deep foundation would potentially mitigate residential foundation, but not reduce static/seismic pad settlement.

5. In order to mitigate the potential for adverse settlement/lateral spreading due to earthquake shaking, ground treatment options for alluvial soils are presented in the following table.

GROUND TREATMENT	DESCRIPTION	COMPATIBLE FOUNDATION TYPES	QUALITY AND COST
Partial Removal/ Recompaction (R&R)	R&R completed to near the groundwater table.	Structural mat*	Treats surficial, unsaturated soils. Foundation design must accommodate potential settlements due to differential settlement and liquefaction. Structural mats could potentially require re-leveling after event or after significant time.
Partial Removal/ Recompaction (R&R) with geotextile reinforcement	R&R completed to near the groundwater table. Placement of geotextile fabrics (Mirafi HP 570, or equivalent) along removal bottom. The use of geotextiles in slope construction potentially mitigates lateral spreading.	Structural mat Post-tension slab	Treats surficial, unsaturated soils. Geotextile reinforces fill embankment, further minimizing differential settlements. Foundation design must accommodate potential settlements due to differential settlement and Liquefaction. Potential for foundation re-leveling after event.
Complete R&R	Complete R&R to suitable formation. Dewatering and perimeter shoring required	Structural mat Post-tension slab	Treats loose, near surface unsaturated and saturated soils below the groundwater table. Dewatering and shoring may be cost, or time prohibitive.
R&R with stone columns	R&R completed to near the groundwater table. Stone columns are vibrated stone columns, which are continuous vertical columns of dense interlocking aggregate, free of non-granular inclusions.	Structural mat Post-tension slab	Treats loose, near surface unsaturated soil. Stone columns reinforce cohesive soils and densify granular soils in order to increase bearing capacity, decrease total and differential settlement, provide vertical drainage pathways to increase the time-rate of consolidation settlement, and reduce the potential for liquefaction. A Cost/benefit evaluation vs. other methods will be needed.
R&R with Deep Soil Mixing	R&R completed to near the groundwater table. Deep soil mixing, or DSM is a process of mechanically blending the in situ soil with cementitious materials that are referred to as binders using a hollow stem auger and paddle arrangement. The intent of the soil mixing method is to achieve improved soil properties.	Structural mat Post-tension slab	Treats loose, near surface unsaturated soil. Deep soil mixing provides similar benefits as stone columns. A Cost/benefit evaluation vs. other methods will be needed.
R&R with compaction grouting.	R&R completed to near the groundwater table. Compaction grouting is a method of ground treatment that involves injecting a very stiff homogeneous grout mix in order to displace and compact soils. The injected grout pushes the soils to the side as it forms a grout column or bulb.	Structural mat Post-tension slab	Treats loose, near surface unsaturated soil. Compaction grouting provides similar benefits as stone columns. A Cost/benefit evaluation vs. other methods will be needed.
Vibro Compaction	The Vibro compaction technique is used in granular soils with limited fines content. It uses sustained vibrations to rearrange the soil particles of non-cohesive soils into a denser state. The action of the vibrator reduces the inter-granular forces between the soil particles, allowing them to move into a more compact configuration.	Structural mat Post-tension slab	This process is used in fully saturated and very weak soils. Water jetting removes soft materials, stabilizes the hole and allows the sand backfill to reach the bottom of the vibrator. This is then compacted and interlocked with the surrounding soil. A Cost/benefit evaluation vs. other methods will be needed.

GROUND TREATMENT	DESCRIPTION	COMPATIBLE FOUNDATION TYPES	QUALITY AND COST
Dynamic Deep Compaction	The process involves of dropping a heavy weight repeatedly on the ground at regularly spaced intervals. The weight and the height determine the amount of compaction that would occur. The weight that is used, depends on the degree of compaction desired and is between 8 ton to 36 tons. The height varies from about 3 to 90 feet.	Structural mat Post-tension slab	Most soil types can be improved with dynamic compaction. Soils that are below the water table have to be treated carefully to permit emission of the excess pore water pressure that is created when the weight is dropped onto the surface. A Cost/benefit evaluation vs. other methods will be needed.

* Deep foundations may be considered, but will not mitigate pad settlement in this condition.

Rock Crushing and/or Placement Guidelines

Crushing/Rock Disposal

GSI anticipates that some of the onsite soils to be utilized as fill material for the subject project may contain some rock, especially during grading operations in the vicinity of Planning Areas PA-1 and the upper elevations of PA-3. Appropriately, the need for rock crushing and/or disposal may be necessary during grading operations on the site. The option for crushing rocks or oversize disposal should be value engineered. 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 rock 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 in occurring in cut or natural areas, would need to be disclosed to all interested/affected parties. Once approved by the governing agency, the hold-down depth for oversized rock (i.e., greater than 12 inches) in fills on this project is provided as 10 feet. The re-use of oversized materials around pools (next to or below) is not recommended.

RECOMMENDATIONS - FOUNDATIONS

Typical foundation design for very low to low expansive soil conditions is anticipated where support is provided by engineered fill overlying older alluvium or bedrock. Building areas underlain with alluvial deposits and shallow groundwater will require relatively more onerous foundation design, in addition to mitigative earthwork such as, but not necessarily limited to fill surcharging, and/or other ground improvement.

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 structure are made, the conclusions and recommendations contained in this report are for the subject site only and 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 considered minimums and are not meant to supercede design(s) by the project structural engineer or civil engineer specializing in structural design. Upon request, GSI could provide additional consultation regarding soil parameters, as related to foundation design. They are considered preliminary recommendations for proposed construction, in consideration of our field investigation, laboratory testing, and engineering analysis. We anticipate that the wall loads of 1.5 to 3.0 kips/foot, and column loads of 5 to 50 kips will be utilized.

As previously indicated, foundation systems will be supported by engineered fill bearing on older alluvium and/or granitic bedrock, left in-place alluvium below the groundwater table, or left in place alluvium that has been improved by methods such as stone columns, grouting, deep mixing, etc. Based on the as-built conditions, including area geology, soil expansion, treatment of existing ground, and/or ground improvement, etc., GSI recommends foundation design in accordance with the following categories:

> **Category I** - Conventional slabs. Limited to very low to low expansive soil conditions. Best suited for settlement Group Areas 1 and 2 (Planning Areas PA-1 and PA-3), excluding deep fill areas.
>
> **Category II** - Post-tension [PT] slab foundations. May be used for all expansive soil conditions onsite, and may be used for settlement Group Areas 1, and 2, including deep fill areas. May be used for structures within settlement Group Area 3, dependant upon method or extent of ground improvement.
>
> **Category III** - Structural mat slabs and/or stiffened slabs per WRI (1981, 1996). May be used for all expansive soil conditions onsite. May be used for settlement Group Areas 1 and 2. May be used for Group 3 areas, dependant upon method or extent of ground improvement.

Ancillary structures (benches, light poles, utility boxes) may use either these types, or conventional spread footings for support.

Settlement Summary

For preliminary design purposes, a summary of potential foundation settlement is presented in the following table.

SETTLEMENT SUMMARY ESTIMATES*			
SETTLEMENT GROUP AREA	**STATIC***	**SEISMIC**	**STATIC PLUS SEISMIC DIFFERENTIAL SETTLEMENT**
Group 1 - Fill over older alluvium (PA-1, PA-3)	1¼-inch total, ⅝-inch differential in 40 feet for fills up to 25 feet 1½-inch total, ¾-inch differential in 40 feet for fills up to 30 feet 2¼-inch total, 1⅛-inch differential in 40 feet for fills between 30 to 50 feet	Less than ¾-inch total, less than ⅜-inch differential in 40 feet for fills up to 25 feet ¾-inch total, ⅜-inch differential in 40 feet for fills up to 30 feet 1¼-inch total, ⅝ -inch differential in 40 feet for fills between 30 to 50 feet.	¾ inch in 40 feet for fills up to 25 feet thick. 1⅛ inches in 40 feet for fills between 25 to 30 feet thick. May be reduced to less than 1 inch in 40 feet when minimum relative compaction for fill is 95 percent. 1¾-inch differential in 40 feet for fills between 30 to 50 feet. May be reduced to 1 inch in 40 feet when minimum relative compaction for fill is 95 percent.
Group 2 - Fill over Granitic bedrock (PA-1, PA-3)	1¼-inch total, less than ¾-inch differential in 40 feet for fills up to 25 feet 1½-inch total, ¾-inch differential in 40 feet for fills up to 30 feet. 2¼-inch total, 1⅛-inch differential in 40 feet for fills between 30 to 50 feet.	Less than ¾-inch total, less than ⅜-inch differential in 40 feet for fills up to 25 feet ¾-inch total, ⅜-inch differential in 40 feet for fills up to 30 feet 1¼-inch total, ⅝ -inch differential in 40 feet for fills between 30 to 50 feet	¾ inch in 40 feet for fills up to 25 feet thick. 1⅛ inches in 40 feet for fills between 25 to 30 feet thick. May be reduced to less than 1 inch in 40 feet when minimum relative compaction for fill is 95 percent. 1¾-inch differential in 40 feet for fills between 30 to 50 feet. May be reduced to 1 inch in 40 feet when minimum relative compaction for fill is 95 percent.
Group 3, fill over alluvium and shallow groundwater table. (PA-2)	Angular distortions of greater than 1/480. With wait periods on the order of at least 180 days, angular distortions could be reduced to 1/480 with ground improvements.	Up to ±6 inch total, and up to 2 to 4 inches differential over 40 feet. Seismic settlement reduced with increased fill surcharge (i.e., fill placed above existing grade) and ground improvement.	Reduce to 2 inches in 40 feet (with ground improvement)

* Does not include foundation settlement due to applied footing loads.

It should also be kept in mind that drainage reversals could occur in areas underlain with alluvium left in place below the groundwater table (Group 3 areas), when considering post-construction static and seismic settlement, if relatively flat yard drainage gradients are not periodically maintained by the maintenance department, owners, and/or other interested/affected parties. Similarly, gravity flow utilities in areas underlain by alluvium are also subject to possible drainage reversals or deflections, considering the magnitude and angular distortions of settlement reported herein.

Foundation Category I (i.e., Very Low Expansive Soils, Settlement Group Areas 1 and 2)

Conventional Slabs

The following foundation construction recommendations are presented as a minimum criteria from a soils engineering viewpoint for very low expansive soils consisting of engineered fill over older alluvium, or granitic bedrock only. Recommendations by the project's design/structural engineer or architect, which may exceed the soils engineer's recommendations, should take precedence over the following minimum requirements. These are for conventional foundations of ancillary structures (other than buildings) that need not comply with criteria for foundations on expansive soils per Code.

1. Continuous footings should be founded at a minimum depth of 12 and 18 inches below the lowest adjacent ground surface bearing properly compacted fill, for one- or two-story floor loads, respectively. All footings should be reinforced with a minimum of two No. 4 reinforcing bars at the top and two No. 4 reinforcing bars at the bottom (four bars total). Reinforcement of Isolated footings should be provided by the structural engineer. The depth of embedment is measured from the lowest adjacent grade, and does not include slab underlayment or the landscape zone.

2. A grade beam, reinforced as above, and at least 12 inches square, should be provided across any large entrance (garage, etc.). The base of the reinforced grade beam should be at the same elevation as the adjoining footings.

3. Concrete slabs should be a minimum of 5 inches. Recommendations for floor slab construction and the mitigation of moisture vapor transmission are presented in a later section of this report.

4. Concrete slabs, including large building entrance areas, should be minimally reinforced with No. 3 reinforcement bars placed on 18-inch centers, in two horizontally perpendicular directions (i.e., long axis and short axis). 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.

5. The slab and footing subgrade should be free of loose and uncompacted material prior to placing concrete.

6. Soils generated from footing excavations to be used onsite should be compacted to a minimum relative compaction 90 percent of the laboratory standard (ASTM D 1557), whether it is 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.

7. Footings should maintain a horizontal distance, X, between any adjacent descending slope face and the bottom outer edge of the footing. The horizontal distance, X, may be calculated by using X = H/3, where "H" is the height of the slope. X should not be less than 7 feet, nor need not be greater than 40 feet. X may be maintained by deepening the footings. Setbacks should minimally conform to Section 1808.7.2, and 1808.7.3 of the 2016 CBC (CBSC, 2016) guidelines as applicable, unless specifically superceded herein.

Stiffened Slabs

All foundations supported by expansive soils (as defined per Section 1803.5.3 of the 2016 CBC), shall be in compliance with Section 1808.6 of the 2016 CBC (CBSC, 2016), and the findings of this report, including the above recommendations for conventional slabs.

For a typical slab designed with interior ribs, or stiffeners, the slab should minimally be at least 5 inches thick. The ribs should be provided in both transverse and longitudinal directions. The interior rib spacing and depth should be provided by the project structural engineer. The perimeter beams, however, should be embedded as specified in the post-tension slabs section of this report, 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. Please note that stiffener beams will tend to make water vapor retarder installation more complex.

Foundation Category II - Post-tension Slab Foundations, Settlement Group Areas 1 and 2. Group 3 with Ground Improvement

Post-tension (PT) slab foundation may also be used to support the structure. PT slab foundations should be designed in accordance with 2016 CBC (CBSC, 2016), 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 2016 CBC and 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 2016 CBC and the PTI Method (Latest Edition).

TABLE - POST-TENSION FOUNDATION DESIGN(3)	
DESIGN PARAMETER(3)	VERY LOW TO LOW EXPANSION POTENTIAL
e_m center lift	9.0 feet
e_m edge lift	5.2 feet
y_m center lift	0.3 inches

TABLE - POST-TENSION FOUNDATION DESIGN$^{(3)}$	
DESIGN PARAMETER$^{(3)}$	**VERY LOW TO LOW EXPANSION POTENTIAL**
y_m edge lift	0.7 inch
Bearing Value (1)	1,000 psf
Lateral Pressure	250 psf
Subgrade Modulus (k)	100 pci/inch
Minimum Perimeter Footing Embedment (2)	12 inches

(1) Internal bearing values within the perimeter of the post-tension slab may be increased to 1,500 psf for a minimum embedment of 18 inches, then by 20 percent for each additional foot of embedment to a maximum of 2,000 psf.
(2) As measured below the lowest adjacent compacted subgrade surface.
(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 maintenance staff, owners, affected/interested parties. The values tabulated above may not be appropriate to account for possible differential settlement of the slab due to other factors, such as excessive settlements. If a stiffer slab is desired, alternative Post-Tensioning Institute ([PTI] latest edition) parameters may be recommended.

Foundation Category III - Structural Mat Foundations, Settlement Group Areas 1 and 2. Group 3 with Ground Improvement

As previously, indicated soils within the influence of the proposed structures are generally considered to be very low to possibly low expansive. However, settlement potentials due to the presence of left in place alluvium in settlement area 3 (Planning Area PA-2) generally exceed the tolerance of a typical slab on grade foundation system. As such, a mat slab foundation may be considered in these areas.

A structural mat-type foundation slabs should be a minimum of 10 inches in thickness, and in accordance with the structural engineer, and also be reinforced with a double mat of rebars at the spacing recommended by the structural engineer. Footings should be embedded as indicated herein, below the lowest adjacent grade into properly compacted fill, unless expansive soil conditions dictate deeper embedments as discussed in a following section. The need and arrangement of grade beams will be in accordance with the structural consultant's recommendations. Alternative uniform thickness mat slabs may be used in the design if the structural consultant can demonstrate that the alternative is equivalent to the recommended mat slab/footing. All mat-type designs should resist differential settlement and expansive soil conditions as explained herein.

Recommended design parameters used in the design of WRI foundations (WRI, 1996) and slabs-on-grade are provided in the following table.

WRI DESIGN PARAMETERS	
Effective Plasticity Index*	20
Unconfined Compressive Strength*	1,000 psf (0.5 tsf)
Modulus of Subgrade Reaction	100 pci
Settlement Potential	see Text
Resistance Value (R-value)*	38
Minimum Slab Thickness	6 inches
Minimum Steel Reinforcement per Structural Engineer	Double Mat of Steel Reinforcement Bars per Structural Consultant

* To be re-evaluated upon completion of grading.

For this method, either a uniform thickness foundation (UTF) or mat may be used. Alternatively, the slab (in plan view) may be divided up into at least quarters and grade beams should be used to enhance the strength of the slab to resist the expansive soil forces. The foundation bearing capacity and other geotechnical parameters previously provided in this report are still applicable.

Perimeter cut-off walls may be incorporated into the UTF design and should be 18 inches deep for the medium to highly expansive soil conditions evaluated onsite. The cut-off walls may be integrated into the slab design or independent of the slab. The cut-off walls should be a minimum of 6 inches thick. The bottom of the perimeter cut-off wall should be designed to resist tension, using reinforcement per the structural engineer.

Slab Subgrade Pre-Soaking

Pre-moistening of the slab subgrade soil is recommended for these soil conditions. The moisture content of the subgrade soils should be equal to or greater than optimum moisture to a depth equivalent to the exterior footing depth in the slab areas (typically 12 inches for very low to low expansive soils). Pre-moistening and/or pre-soaking should be evaluated by the soils engineer 72 hours prior to vapor retarder placement. In summary:

EXPANSION INDEX	PAD SOIL MOISTURE	CONSTRUCTION METHOD	SOIL MOISTURE RETENTION
Very Low (0-20)	Upper 12 inches of pad at or above soil optimum moisture	Wetting and/or reprocessing	Periodically wet or cover with plastic after trenching. Evaluation 72 hours prior to placement of concrete.
Low (21-50)	Upper 12 inches of pad soil moisture 2 percent over optimum	Wetting and/or reprocessing	Periodically wet or cover with plastic after trenching. Evaluation 72 hours prior to placement of concrete.

SOIL MOISTURE CONSIDERATIONS

GSI has evaluated the potential for vapor or water transmission through the slabs, in light of typical floor coverings and improvements. Generally, slab moisture emission rates range from about 2 to 27 lbs./1,000 square feet from a typical slab (Kanare, 2005), while most floor covering manufacturers recommend about 3 lbs./24 hours as an upper limit. Thus, the client will need to evaluate the following in light of a cost versus benefit analysis (tenant complaints and repairs/replacement), along with disclosure to owners.

Considering the proximity of groundwater, potential for perched groundwater to occur, E.I. test results, anticipated typical water vapor transmission rates, and floor coverings and improvements (to be chosen by the client) that can tolerate those rates without distress, the following alternatives are provided:

- Concrete slabs should be thicker than the minimum specified herein.

- Concrete slab underlayment should consist of a 15-mil vapor retarder, or equivalent, with all laps sealed per the 2016 CBC (CBSC, 2016) 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 D 1643.

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

- The vapor retarder should be underlain with 2 inches of washed sand, and should be overlain by a 2-inch thick layer of washed sand (SE>30).

- Concrete should have a maximum water/cement ratio of 0.50. This does not supercede the 2016 CBC (CBSC, 2016) 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 above, 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.

- Owner(s) should be specifically advised which areas are suitable for tile flooring, wood 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, and areas potentially using moisture sensitive floor coverings and/or moisture sensitive storage, should be identified 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.

Corrosion and Concrete Mix

Upon completion of grading, laboratory testing should be performed of site materials for corrosion to concrete and corrosion to steel. Soils with negligible to moderate levels of sulfate content are present near the surface. As such, the use of Type V concrete is not required per 2016 CBC, as well as ACI 318-14, on a preliminary basis. Additional comments may be obtained from a qualified corrosion engineer.

APPENDIX 4

PDFs OF PREVIOUS OBR GEOTECHNICAL STUDIES (GSI; 2015 AND 2016)

GEOTECHNICAL EVALUATION FOR OCEAN BREEZE RANCH BONSALL, SAN DIEGO COUNTY, CALIFORNIA

GeoSoils, Inc.

FOR

OCEAN BREEZE RANCH
5820 WEST LILAC ROAD
BONSALL, CALIFORNIA 92003

W.O. 6960-A-SC OCTOBER 6, 2016

GeoSoils, Inc.

Geotechnical • Geologic • Coastal • Environmental

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October 6, 2016

W.O. 6960-A-SC

Ocean Breeze Ranch
5820 West Lilac Road
Bonsall, California 92003

Attention: Mr. Jim Conrad

Subject: Geotechnical Evaluation for Ocean Breeze Ranch, Bonsall, San Diego County, California

Dear Mr. Conrad:

In accordance with your request and authorization, this report presents the results of GeoSoils Inc.'s (GSI's) geotechnical evaluation for the Ocean Breeze Ranch property in the community of Bonsall, San Diego County, California. The purpose of the study was to evaluate the on-site geotechnical and geologic conditions and their impacts on proposed, residential use site development, from a geotechnical viewpoint.

EXECUTIVE SUMMARY

Based on our review of the available data (see Appendix A), as well as field exploration (see Appendix B), seismicity analysis (see Appendix C), and geologic and engineering analysis, the proposed development of the property appears to be feasible from a geotechnical viewpoint, provided that mitigation measures presented in the text of this report are properly incorporated into design and construction of the project. The most significant elements of this study are summarized below:

- The planned development generally consists of five (5) planning areas (PA-1, through PA-5) distributed throughout the property. Planning Areas PA-1, PA-2, and PA-3, include the construction of approximately 360 single-family residential structures, and associated improvements. Planning Areas PA-4 and PA-5 generally consist of approximately thirty (30) larger estate-size building lots, and associated improvements.

- The site occupies the southern flank of a portion of the San Luis Rey River valley, consisting of a relatively flat-lying valley floor to the north, with bedrock highland to the south. Flat-lying ground in the vicinity of (primarily north of) Dulin Ranch Road, and generally within the 100-year flood plain, is underlain with Holocene alluvial sediments. Lower slopes descending to the valley floor, and flatter than about 4:1 (horizontal:vertical [h:v]) are developed on deposits of

Quaternary (Pleistocene)-age older alluvium (stream terrace deposits). Steeper slopes and upland areas are underlain with granitic bedrock.

- In general and based upon the available data to date, regional groundwater is not expected to be a major factor in development of the more elevated portions of the site (i.e., areas underlain with deposits of older alluvium and/or granitic bedrock). Within lower-lying areas underlain with alluvium, groundwater was encountered at depths ranging from approximately 11½ to 18½ feet below existing grade within the San Luis Rey River drainage area, to slightly deeper, perched water tables within adjoining tributary drainages, and is anticipated to be a concern during development in these areas, including any deep utilities. This corresponds to elevations ranging from about 178 to 179 feet above Mean Sea Level (MSL) within the San Luis Rey River drainage in the vicinity of Planning Area PA-3. Additionally, owing to the relatively cohesionless nature of near-surface soils, perched groundwater/sloughing should be anticipated during excavation.

- The presence of landslide deposits, slumps, or other significant forms of mass wasting were not observed within the site.

- GSI's review and field exploration indicates no known active faults are crossing the site, and the site is not located within an Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). However, strong shaking should be anticipated should an earthquake occur on one of the nearby regional active faults, and liquefaction effects within alluvial soils should be anticipated, if not mitigated.

- The proposed structures and foundations, as well as other supporting infrastructure should be designed to resist seismic forces and deformation in accordance with the criteria contained in the 2013 California Building Code ([2013 CBC], California Building Standards Commission [CBSC], 2013). Based on our site-specific seismic hazard analysis, appropriate seismic design parameters are provided herein.

- Based on our analysis, the potential for liquefaction to adversely affect those portions of the site underlain with older alluvium and/or granitic bedrock is considered low. Regardless, some seismic induced deformation should be anticipated due to densification, and will be discussed herein. Owing to the depth to groundwater, relatively low density, grain size, young age and lack of cementation, the potential for liquefaction and seismic densification to adversely affect those portions of the site underlain with younger alluvium is higher, when subjected to the design level earthquake, based on the available data.

- Based upon our experience in this area, and the seismic refraction data obtained, assuming a D9L, or equivalent, bedrock within cut areas of the site appear to be rippable (i.e., seismic velocities of less than about 6,000 feet per second [fps]) at depths ranging up to ±30 feet from existing grade. Rock breakers and/or blasting should be considered during preliminary planning and budgeting for excavation depths (including foundations and utilities) greater than about ±30 feet from existing grade, on a preliminary basis.

- Using the 3,800 fps cut-off for non-rippable trenching, assuming a CAT 235 hoe, or equivalent, it is likely that some areas will require blasting (e.g., “line-shooting”) for trenching of utilities onsite. Seismic velocities near, or exceeding 3,800 fps generally occur at depths ranging from depths as shallow as ±9 feet, to as deep as about ±38 feet from existing grade. A conventional backhoe would likely encounter practical refusal at shallower depths.

- Excavation within bedrock areas exhibiting a seismic velocity of ≥5,000 fps will generate appreciable quantities of oversize rock >12 inches in size, requiring specialized placement techniques during grading. In addition, hard rock requiring blasting, rock breakers, etc., may not be entirely precluded from occurring near the surface, and may also generate oversize rock. Accordingly, oversize rock (<24 inches in size), may be placed in fills deeper than 10 feet from finish grade, subject to governing agency approval, or may be crushed to reduce their size for standard fill placement. Considering the thickness of proposed fills and the proximity of groundwater below existing grade, there are limited areas on the project that will accommodate the hold-down distance of 10 feet below finish grade, and that have significant volume for oversize material placement. Thus, onsite crushing of oversize materials to less than 12 inches may be necessary. This condition will need value engineering to evaluate the feasibility of either oversize rock placement and/or crushing oversize materials onsite.

- Representative samples of near surface site soils were tested for expansion potential. The Expansion Index (E.I.) test was performed in general accordance with ASTM Standard D 4829. The laboratory test results indicate that the soil expansion potentials are generally very low (E.I. 0 to 20). However, this does not preclude the presence of higher expansive soils locally onsite.

- Representative samples of site material has also been evaluated for corrosion, soluble sulfate, etc. Laboratory testing indicates that site soils generally have a negligible (not applicable) sulfate exposure to concrete, per Table 4.2.1 of ACI 318-11 (per the 2013 CBC [CBSC, 2013]), and the use of Type V cement is not required. Corrosion testing (pH/resistivity) indicates that the soils are slightly alkaline (pH of 6.45 to 6.99) with respect to soil acidity/alkalinity, and is mildly corrosive to ferrous metals when saturated (saturated resistivity of 1,800 to 3,400 ohm-cm [California Highway Design Manual, 2012]). Chloride content of the soil was measured as 122 to 192 ppm, which is slightly elevated. Alternative testing methods and additional comments should be obtained from a qualified corrosion engineer with regard to foundations, piping, etc. Additional corrosion testing should be performed at the completion of site grading to further evaluate geotechnical pad characteristics.

- A settlement analysis was performed for three (3) general, as-built conditions anticipated onsite, in consideration of both static and dynamic settlement. Group 1 areas (i.e., lower elevations of Planning Area PA-5, and a portion of Planning Area PA-2) would consist of engineered fills placed over older alluvium, Group 2 areas (i.e., Planning Areas PA-1, PA-2, and PA-4, and the upper elevations of PA-5) would generally consist of engineered fills placed over granitic bedrock, and Group 3 areas (Planning Area PA-3) would be where portions of the site overly alluvium below the groundwater table. Group 3 areas may also display an increased potential to be affected by lateral spreading during a seismic event. A discussion of settlement potential for each general area is presented in the text of this report. Due to high estimated settlements within Planning Area PA-3, additional review and field investigation is recommended.

- It should be kept in mind that drainage reversals could occur, when considering post-construction static and seismic settlement, if relatively flat yard drainage gradients are not periodically maintained in areas underlain by alluvium. Similarly, gravity flow utilities in areas underlain by alluvium are also subject to possible drainage reversals or deflections, considering the magnitude and angular distortions of settlement reported herein.

- The treatment of existing ground prior to fill placement for specific areas of the site will vary according to each of the following two (2) general cases:

  > *Case I* - Areas underlain with near surface, older alluvium, and/or granitic bedrock.
  >
  > *Case II* - Areas underlain with loose alluvium and a shallow groundwater table (i.e., alluvium left in place below the groundwater table).

  A discussion of specific recommendations for each case is included in the text of this report.

- Given the potential for settlement, expansive soils and lateral movement due to the design basis earthquake, Planning Area PA-5 should be further evaluated using a truck mounted drill rig.

- All existing structures, utilities, deleterious debris, and vegetation should be removed from the site and properly disposed, should settlement-sensitive improvements be proposed within their influence. It should be noted that the 2013 CBC (CBSC, 2013) indicates that for fill placed under the purview of the grading permit, removals of unsuitable soils be performed across all areas to be graded, not just within the influence of the 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 to mitigate site perimeter conditions or existing utilities.

Thus, any settlement-sensitive improvements (walls, curbs, flatwork, etc.), constructed within this zone, may require deepened foundations, reinforcement, etc., or will retain some potential for settlement and associated distress. Current conditions indicate compressible colluvium, alluvium, weathered older alluvium, and bedrock, which should be included in remedial grading efforts.

- In general, support of the new building(s) and structures may be provided entirely by engineered and compacted fill. As discussed herein, onsite soils appear to be very low, to possibly low expansive. However, the potential for higher expansive soils cannot be precluded locally.

- Based on the underlying conditions supporting engineered fills onsite, the as-built conditions will likely result in at least three (3) different foundation design/construction scenarios. Refer to the foundation recommendations sections of this report. These foundations will require various ground treatments (recompaction, improvement, overexcavation) prior to placement, and discussed herein.

- Retaining wall design and construction recommendations are provided herein. Onsite soils are generally very low expansive, to possibly low expansive, and appear suitable for wall backfill, without select import, subject to verification testing.

- Recommendations for concrete (PCC) and asphaltic concrete (AC) pavements are be provided. The majority of site soils anticipated at finish subgrade elevations are anticipated to be relatively sandy, and are considered to provide relatively good subgrade support for roadways. As such, County minimum pavement sections should be anticipated.

- Storm water infiltration feasibility was evaluated for each of the three (3) dominant geologic units onsite (alluvium, older alluvium, and granitic substrates). Based on our evaluation, hydraulic conductivities generally allow for full infiltration within alluvium, and older alluvium substrates, with partial infiltration feasible for bio basin design in granitic substrates.

- Adverse geologic structures that would preclude project feasibility were not encountered. However, the potentially liquefiable and compressible deposits of alluvium will require more investigation in order to develop a program of ground mitigation and/or specialized foundation/infrastructure designs, as discussed herein.

- The project design features presented in this report should be incorporated into the design and construction considerations of the project. If the design information and/or assumptions used as a basis for the geotechnical recommendations do not reflect current design information, GSI suggests a review of the current design(s) and modification of the geotechnical recommendations as needed.

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.

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Distribution: (3) Addressee (wet signed)

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GEOTECHNICAL EVALUATION FOR OCEAN BREEZE RANCH BONSALL, SAN DIEGO COUNTY, CALIFORNIA

SCOPE OF SERVICES

The scope of our services has included the following:

1. Review of available soils and geologic data for the site and site area, including in-house documents, and other referenced material, as well as our previous feasibility evaluation for the site (see Appendix A).

2. Review of the current 100-scale "preliminary grading plan," prepared by Project Design Consultants (PDC, 2016).

3. Geologic reconnaissance and geologic mapping of the site.

4. Subsurface exploration consisting of the excavation of eight (8) supplemental exploratory test pits with a rubber tire backhoe, four (4) supplemental cone Penetration Test (CPT) soundings, and 11 hollow stem auger borings. Samples were retrieved from the test pits and hollow stem auger borings for laboratory testing. The logs of the test pits, borings, and soundings are presented in Appendix B, with exploration locations presented on Plate 1. The supplemental test pits, borings, and CPT's were performed in order to augment data from existing test pits and CPT's completed in preparation of GSI (2015).

5. Site-specific seismic hazard evaluation and seismicity analysis (see Appendix C).

6. Completion of three (3) supplemental seismic refraction survey profiles for the evaluation of rock hardness within areas of the site underlain at the near surface by granitic rock (see Plate 1 and Appendix D). As with the test pits and CPT's, the supplemental seismic survey profiles were performed in order to augment data from existing surveys completed in preparation of GSI (2015).

7. Obtained representative samples of site soil for laboratory testing. Testing included: moisture-density determinations; compaction standards; soil expansion; Atterberg limits; direct shear; sieve and hydrometer analyses; consolidation; R-value; and corrosion potential (see Appendix E).

8. Analysis of data, including preliminary liquefaction, and settlement analysis (Appendix F).

9. Construction of geologic cross sections depicting the subsurface data. Plate 1 shows the cross section locations. The cross sections are provided as Plate 2.

10. An evaluation of storm water infiltration for several planned "bio-basins" distributed throughout the site. (Appendix G).

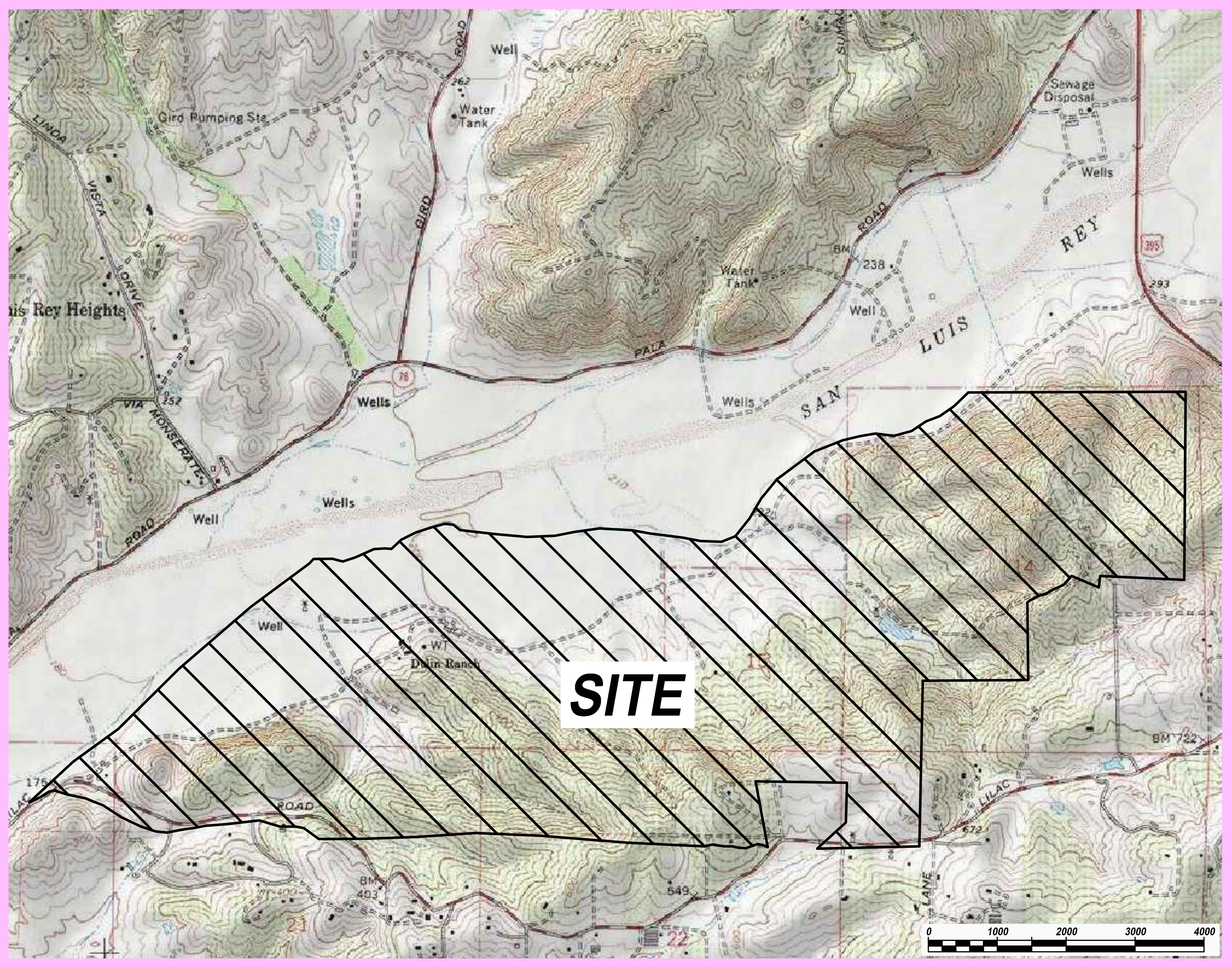
11. Preparation of this geotechnical report that includes: descriptions of site specific and regional geology, subsurface soil characteristics, the logs and/or soundings of the explorations; laboratory test results; earthwork factors; evaluation of seismic hazards; preliminary conclusions and recommendations related to project planning, preliminary foundation design, and grading guidelines (see Appendix H).

PROPERTY DESCRIPTION/PROPOSED DEVELOPMENT

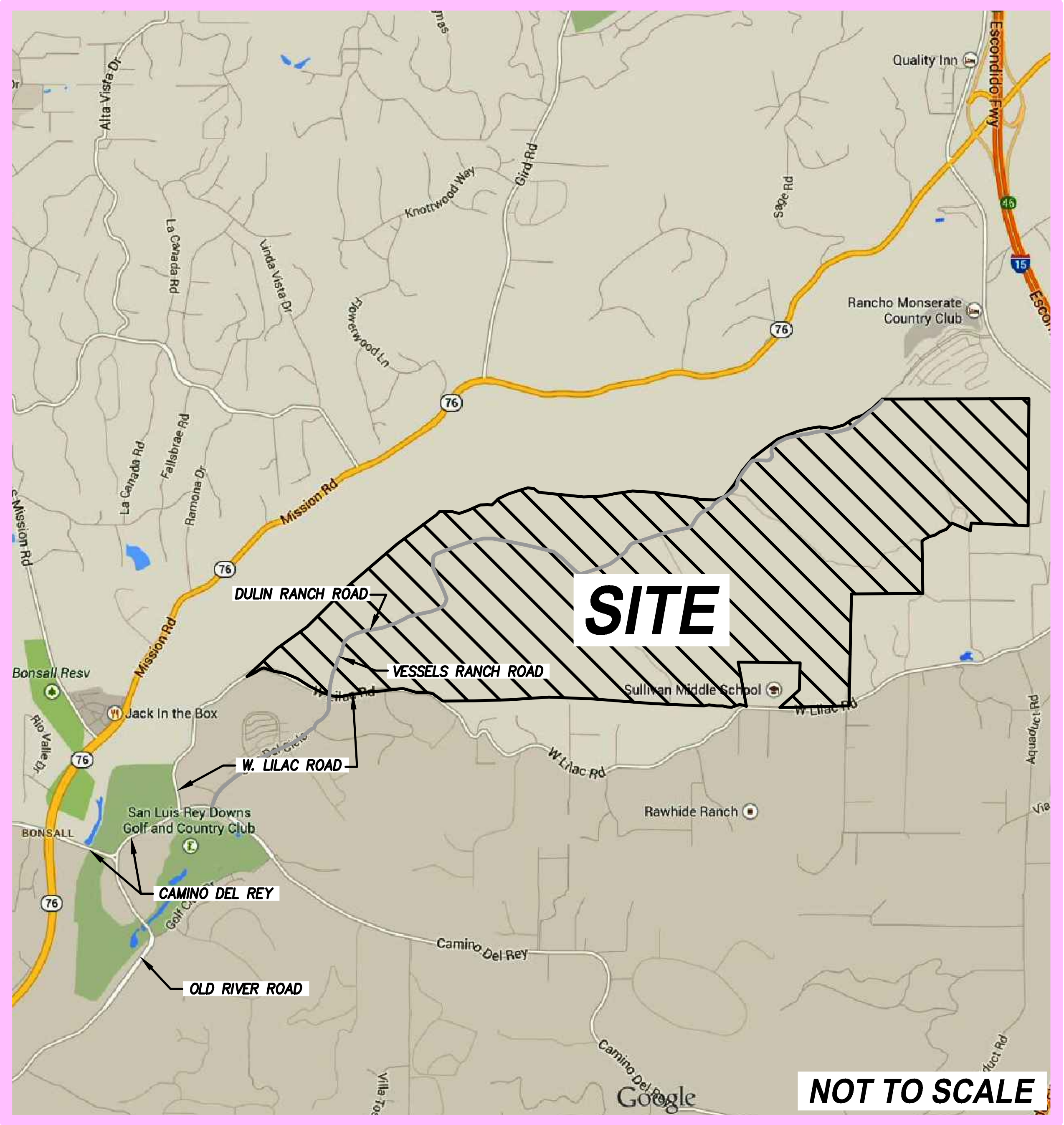
Based upon the data provided, GSI understands that the irregularly-shaped property consists of about 1,400 acres (gross), located along the southern margin of the San Luis Rey River Valley, in the vicinity of Dulin Ranch Road, including hilly and more rugged terrain generally between Dulin Ranch Road and West Lilac Road, in the community of Bonsall, San Diego County, California (see Figure 1).

Topographically, portions of the property (Planning Area PA-3) within the San Luis Rey Valley floor area are generally flat-lying/low gradient. South of the river valley (generally south of Dulin Ranch Road), the westernmost third of the property ascends from the valley floor to somewhat more rugged, inclined terrain, with slope gradients generally steeper than about 4:1 (horizontal to vertical [h:v]), that form a roughly east-west trending ridgeline across the southern portion of the site (Planning Areas PA-1 and PA-2). Within the remaining, easternmost portion of the property, the relatively flat-lying river valley floor transitions to moderately sloping terrain, with north facing slopes at gradients generally on the order of 4:1 (h:v), or less (lower elevations of Planning Area PA-5). As with the western portion of the property, these low/moderate gradient slopes ascend to somewhat more rugged, craggy terrain along the southern portion of the property (Planning Area PA-4, and the upper elevations of Planning Area PA-5). Drainage is generally directed northward, from the crest of the east-west trending ridgeline, toward the San Luis Rey River, via tributary drainages incised into the north facing slope. On the backside, or south side of the ridge, drainage is generally directed offsite to the south.

The relatively flat-lying valley floor portion of the site has elevations ranging from about 180 to 225 feet Mean Sea Level (MSL), with the area of low gradient slopes, south of the valley floor, ranging from 180 to 225 feet MSL at the valley floor/margin, up to approximately 300 feet MSL. The somewhat rugged, steeper terrain that ascends to the south, range from about 200 to as much as 747 feet MSL. Thus, overall relief across the site is on the order of about 567 feet. Portions of the site (i.e., valley floor), generally within the low/flat-lying portions of the site, lie within a San Diego County 100-year flood plain.



Base Map: TOPO!® ©2003 National Geographic, U.S.G.S. Bonsall Quadrangle, California -- San Diego Co., 7.5 Minute, dated 1975, current, 1975.



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

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SITE LOCATION MAP

Figure 1

The property is currently used for both equestrian and agricultural purposes. Existing improvements generally consist of an equestrian facility located within the low lying, northerly portions of the site (east of Planning Area PA-3), with an existing residence, located within Planning Area PA-2, overlooking the equestrian facility. Scattered outbuildings were also noted throughout, and generally located in close proximity to the equestrian facility. Vegetation generally consists of some native trees, planted trees, grass pasture, areas of irrigated row crops, groves, and also areas with native grasses and brush.

GSI understands that proposed development includes several Planning Areas (PAs) with different product anticipated. Current plans (PDC, 2016) indicate at least five (5) planning areas, with associated roadway, underground, and storm water (BMP) improvements, and is summarized as follows:

PLANNING AREA	APPROX. NUMBER OF LOTS	COMMENTS
PA-1	44 Lots	Graded Pads Indicated *
PA-2	98 lots	Graded Pads Indicated*
PA-3	218 Lots	Graded Pads Indicated*
PA-4	14 Lots	Raw Land, No Grading* Indicated
PA-5	16 Lots	Raw Land, No Grading* Indicated

* per PDC (2016)

Cut and fill grading techniques are anticipated to bring Planning Areas PA-1, PA-2, and PA-3 to the desired grades. Based on a review of PDC (2016), maximum cuts and fills on the order of 42 feet, and 35 feet, respectively, are anticipated within Planning Area PA-1, with graded slopes ranging from about 82 feet (cut), and 42 feet (fill) in height, at gradients ranging from 1.5:1 (h:v), or flatter, for cut slopes, and 2:1 (h:v), or flatter, for fill slopes. Within Planning Area PA-2, maximum cuts and fills on the order of 50 feet, and 46 feet, respectively, are anticipated, with graded slopes ranging from about 50 feet (cut), and 35 feet (fill) in height, at gradients ranging from 1.5:1 (h:v), or flatter, for cut slopes, and 2:1 (h:v), or flatter, for fill slopes. Within Planning Area PA-3, maximum fills on the order of 5 to 12 feet, are anticipated, with graded slopes ranging up to about 15 feet, or less, in height, at gradients of 2:1 (h:v), or flatter.

We anticipate that structures will be one- or two-story buildings utilizing typical foundations on grade, with wood frame and/or masonry block construction. Building loads are assumed to be typical for this type of relatively light construction. Sewage disposal for is understood to be accommodated by tying into the regional sewage system. The need for import soils is unknown, based upon the data provided. The approximate limits of each planning area are shown on PDC (2016), and are also indicated on Plate 1 included herein, which uses a 400-scale version of (PDC, 2016) as a base.

PREVIOUS WORK AND CURRENT FIELD STUDIES

GSI conducted a previous phase of subsurface investigation during March, 2014 (GSI, 2015). This feasibility level investigation consisted of 11 exploratory test pits excavated with a rubber tire backhoe, four (4) CPT soundings, four (4) seismic refraction surveys, and geologic reconnaissance mapping of the site.

Field work performed in preparation of this report was performed periodically during May/June/July, 2016, associated with plan changes, and consisted of eight (8) supplemental exploratory test pits excavated with a rubber tire backhoe, four (4) supplemental CPT soundings, three (3) supplemental seismic refraction surveys, and 11 hollow stem auger borings, in addition to additional geologic mapping of the site.

The approximate location of the previous (GSI, 2015) and current (this study) exploratory test pits, soundings, and seismic lines, and auger borings, are presented on the Geotechnical Map (see Plate 1), which uses a 400-scale version of the 100-scale, preliminary grading plan, prepared by PDC (2016), as a base. A GSI geologist observed the test pit excavations/borings, and collected bulk and undisturbed samples of materials encountered for visual examination and subsequent laboratory testing. The Cone Penetration Test (CPT) soundings were directed and observed by a GSI geologist. A discussion of the seismic refraction survey is presented in a later section of this report.

REGIONAL GEOLOGY

The subject property is located within the Peninsular Ranges geomorphic province, which is characterized by steep, elongated mountain ranges and valleys that trend northwesterly (Norris and Webb, 1990). The Peninsular Ranges Geomorphic Province extends north to the base of the east-west aligned Santa Monica - San Gabriel Mountains, and south into Baja California. The province is bounded by the east-west trending Transverse Ranges Geomorphic Province to the north and northeast, by the Colorado Desert Geomorphic Province to the southeast, and by the Continental Borderlands Geomorphic Province to the west. The mountain ranges are underlain by basement rocks consisting of pre-Cretaceous metasedimentary rocks, Jurassic metavolcanic rocks, and Cretaceous plutonic (granitic) rocks, which have been uplifted, tilted, faulted, eroded and deeply incised since their formation.

In the Bonsall area during the mid to late Pleistocene (within the Quaternary-age), the granitic rocks belonging to the Peninsular Ranges Batholith have been eroded and alluvial deposits have since filled the lower valleys. Regional mapping by Tan (2000) indicates that the site is underlain by Cretaceous-age granitic rock referred to as the Couser Canyon Tonalite. Pleistocene-age older alluvium also occurs in the site vicinity (Tan, 2000).

Flat-lying ground in the vicinity of Planning Area PA-3, primarily north of Dulin Ranch Road, and generally within the 100-year flood plain, is underlain with Holocene alluvial sediments. Lower slopes descending to the flood plan and flatter than about 4:1 (h:v) are developed on deposits of older alluvium (stream terrace deposits) and located primarily within the lower elevations of Planning Areas PA-2 and PA-5. Steeper slopes and upland areas are underlain with granitic bedrock, primarily within the limits of Planning Areas PA-1, PA-2, PA-4, and the upper elevations of PA-5.

SITE GEOLOGIC UNITS

General

Geologic units encountered during our current site investigation included, undocumented artificial fill, colluvium, Quaternary-age, younger alluvium, older alluvium (stream terrace deposits), and Cretaceous-age granitic bedrock. The surficial earth materials are generally described below from the youngest to oldest.

Undocumented Artificial Fill (Map Symbol - afu)

Small embankments of existing undocumented fill occur throughout the property and appear associated with the existing improvements (i.e., building pads, corrals, etc) onsite, and are likely less than approximately 10 feet in thickness. While not directly observed in any of our test pits, existing fill may be generally characterized as a brown silty sand to sand, dry/damp, and loose. Undocumented fill is considered potentially compressible in its existing state and therefore should be removed and recompacted, if settlement-sensitive improvements and/or planned fills are proposed within its influence.

Colluvium (Not Mapped)

Colluvium (topsoil) was noted to generally mantle deposits of older alluvium and granitic bedrock throughout the site. Where observed, colluvial soil generally consists of brown, and dark brown silty sand, and is typically damp, loose, and porous, with few roots. Where encountered, colluvium is on the order of approximately 1 to 7 feet in thickness. Within areas actively cultivated throughout the site, the upper 1 to 2 feet has likely been periodically reprocessed for agricultural purposes. Colluvium is considered potentially compressible in its existing state and therefore should be removed and recompacted, if settlement-sensitive improvements and/or planned fills are proposed within its influence.

Quaternary-age Alluvium (Map Symbol - Qal)

Alluvium was observed within the northern portions of the site, in areas of flat lying ground, primarily north of Dulin Ranch Road, and generally within the 100-year flood plain, including all of Planning Area PA-3, the lower elevations of PA-2, and along the extreme northern edge of PA-5.

Alluvium generally consists of light brown and very dark brown, interbedded sands, and silty sands, with silts and clays indicated at depth, based on CPT data. The thickness of this deposit generally varies from a daylight contact, adjacent to deposits of older alluvium and granitic bedrock, thickening northward to depths on the order of 42 to 62 feet below existing grades, based on field mapping, test pit, and CPT data. Within a tributary drainage located within a portion of Planning Area PA-5, alluvium was encountered to a depth of at least 17 feet below existing grades. Based on a review of California Department of Water Resources Bulletin No. 106-2 (State, 1967), the maximum thickness of alluvium along the nearby reach of the San Luis Rey river valley is less than 100 feet.

Alluvium above the current groundwater was slightly moist to moist, becoming saturated near, and below the groundwater table, and generally was noted to be loose near existing surface grades, becoming denser with depth. Alluvium is considered potentially compressible in its existing state and therefore should be removed and recompacted (where feasible), if settlement-sensitive improvements and/or planned fills are proposed within their influence. Alluvial soils will likely remain in place in areas of relatively high groundwater. Recommendations for the treatment of left-in-place alluvium in these areas is presented in a later section of this report.

Quaternary-age Older Alluvium (Map Symbol - Qoa)

Deposits of Quaternary (Pleistocene)-age older alluvium (less than ±500,000 years old) were generally encountered at/near the surface, typically underlying the lower elevations of PA-2 and PA-5, forming the moderate slopes located between the valley floor and the southern highland ridge areas. Based on the distribution of these materials in plan view, and in cross section, the thickness of these sediments may be on the order of up to 30 to 50 feet locally. The older alluvium (stream terrace deposits) generally consists of interbedded silty sand, with lessor amounts of silty sand with some clay. Where observed, stream terrace deposits are light brown, brown, and yellowish brown, damp, and medium dense. Stream terrace deposits are considered suitable for the support of engineered fills, and/or structures in its existing state, provided that the recommendations presented in this report are properly incorporated into the design and construction of the project.

Cretaceous-age Granitic Bedrock (Map Symbol - Kcc)

Cretaceous-age granitic bedrock, referred to as the Couser Canyon Tonalite (Tan, 2000), was encountered near the surface, and at depth throughout the site. Where encountered, bedrock consists of fractured rock, disintegrating to sand and silty sand with brittle gravel- to cobble-size rock fragments in near surface excavations. Bedrock was generally observed to be brown to olive brown, brownish yellow to yellowish brown, dry to moist, and dense.

Practical refusal on hard rock with a rubber tire backhoe was encountered at depths varying from approximately 2 to 8½ feet below existing grades. Relatively unweathered bedrock is considered suitable for the support of settlement-sensitive improvements and/or planned fill in its existing state.

Structural Geology

Based on our observations and available published geologic maps of the site and surrounding area, bedding within alluvium and older alluvium appears to be relatively flat-lying. Bedrock is fractured, with fractures generally steeply inclined to the northwest, southeast, southwest, and northeast (i.e., in all four quadrants).

GROUNDWATER

In preparation of GSI (2015), during the month of March, 2014, the regional groundwater table was encountered at depths on the order of 13½ to 15½ feet below existing grades, within the relatively flat-lying, alluviated areas underlying Planning Area PA-3, and within the flood plain area north of Planning Area PA-5. These depths generally correspond to approximate elevations ranging from about 189½ feet above Mean Sea Level (MSL), up gradient, within alluvial areas north of Planning Area PA-5, to approximately 178 feet MSL, down gradient, toward the western end of the property, within the current boundaries of Planning Area PA-3.

During the current study, groundwater was encountered in both the additional CPT soundings, and the hollow stem auger borings, at depths on the order of 11½ to 18 feet below existing grades within Planning Area PA-3, or at corresponding elevations of 178 to 179 feet MSL. Up gradient from Planning Area PA-3, within a tributary drainage located within the lower elevations of Planning Area PA-5, groundwater was encountered at a depth of about 21 feet below the surface grade, or at an approximate elevation of 213 feet MSL.

Over a two-year period, groundwater levels appear relatively constant within Planning Area PA-3. The variable levels noted near, and within portions of Planning Area PA-5 are considered to be due to perched water conditions, and variations in geology, and geomorphology.

Perched groundwater may occur in or along zones of contrasting permeability (i.e., between contrasting soil types in the underlying deposits/bedrock or discontinuities) due to migration from adjacent drainage areas, and during and after periods of above normal or heavy precipitation or irrigation. Thus, perched groundwater conditions may occur in the future, after construction, and should be anticipated. Groundwater observations reflect site conditions at the time of this report and do not preclude changes in local groundwater conditions in the future.

FAULTING AND REGIONAL SEISMICITY

Regional Faults

Our review indicates that there are no known active faults crossing this site (Jennings and Bryant, 2010), 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 Temecula segment of the Elsinore fault is closest known active fault to the site (located at a distance of approximately 11.1 miles [17.8 kilometers]). However, the Julian segment of the Elsinore fault (located at a distance of approximately 11.7 miles [18.9 kilometers]) should have the greatest effect on the site in the form of strong ground shaking, should the design earthquake occur. A list and the location of the Elsinore fault and other major faults relative to the site is provided 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 (PHGA) 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 Elsinore fault may be on the order of 0.306g, for portions of the site underlain with alluvial soil, and 0.34g for portions of the site underlain with granitic rock. 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 July 2013). 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 July 2013. Based on the selected acceleration-attenuation relationship, a peak horizontal ground acceleration is estimated, which may have affected the site during the specific event listed. Based on the available data and the attenuation relationship used, the estimated maximum (peak) site acceleration during the period 1800 through July 2013 was about 0.077g to 0.09g, for alluvial, and rock areas, respectively. A historic earthquake epicenter map and a seismic recurrence curve are also estimated/generated from the historical data. Computer printouts of the EQSEARCH program are presented in Appendix C.

For the evaluation of liquefaction potential onsite, and in general accordance with California Department of Conservation (2008), a probabilistic seismic hazards analysis was performed using a PSHA Interactive Deaggregation computer program provided by the USGS (2012). Based on a review of these data, and considering the relative seismic activity of the southern California region, a probabilistic horizontal site acceleration (PHSA) of 0.29g was considered. This value corresponds to a 10 percent probability of exceedance in 50 years. For seismic aspects of site design and construction, a probabilistic seismic hazards analysis was performed using the computer program “Seismic Design Maps,” provided by the United States Geologic Survey (USGS, 2014).

Seismic Shaking Parameters

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

2013 CBC SEISMIC DESIGN PARAMETERS

PARAMETER	ALLUVIUM/ OLDER ALLUVIUM	GRANITIC BEDROCK	2013 CBC AND/OR REFERENCE
Risk Category	I, II, or III	I, II, or III	Table 1604.5
Site Class	D	C (>10' of fill)	Section 1613.3.2/ ASCE 7-10 (Chapter 20)
Spectral Response - (0.2 sec), S_s	1.137 g	1.151g	Figure 1613.3.1(1)
Spectral Response - (1 sec), S_1	0.443g	0.447g	Figure 1613.3.1(2)

2013 CBC SEISMIC DESIGN PARAMETERS			
PARAMETER	**ALLUVIUM/ OLDER ALLUVIUM**	**GRANITIC BEDROCK**	**2013 CBC AND/OR REFERENCE**
Site Coefficient, F_a	1.045	1.00	Table 1613.3.3(1)
Site Coefficient, F_v	1.557	1.357	Table1613.3.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec), S_{MS}	1.188g	1.137g	Section 1613.3.3 (Eqn 16-37)
Maximum Considered Earthquake Spectral Response Acceleration (1 sec), S_{M1}	0.689g	0.601g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	0.792g	0.758g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.460g	0.401g	Section 1613.3.4 (Eqn 16-40)
PGA_M	0.46g	0.43g	ASCE 7-10 (Eqn 11.8.1)
Probabilistic Horizontal Ground Acceleration ([PHGA] 10 percent probability of exceedance in 50 years)	0.29g	N/A	USGS (2012)
Seismic Design Category	D	D	Section 1613.3.5/ ASCE 7-10 (Table 11.6-1 or 11.6-2)

GENERAL SEISMIC PARAMETERS	
PARAMETER	**VALUE**
Distance to Seismic Source (Elsinore-Julian segment) "B" Fault (1)	11.7 mi (18.9 km)(2)
Upper Bound Earthquake (Elsinore-Julian segment)	$M_W = 7.1$(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.

OTHER GEOLOGIC HAZARDS

Liquefaction

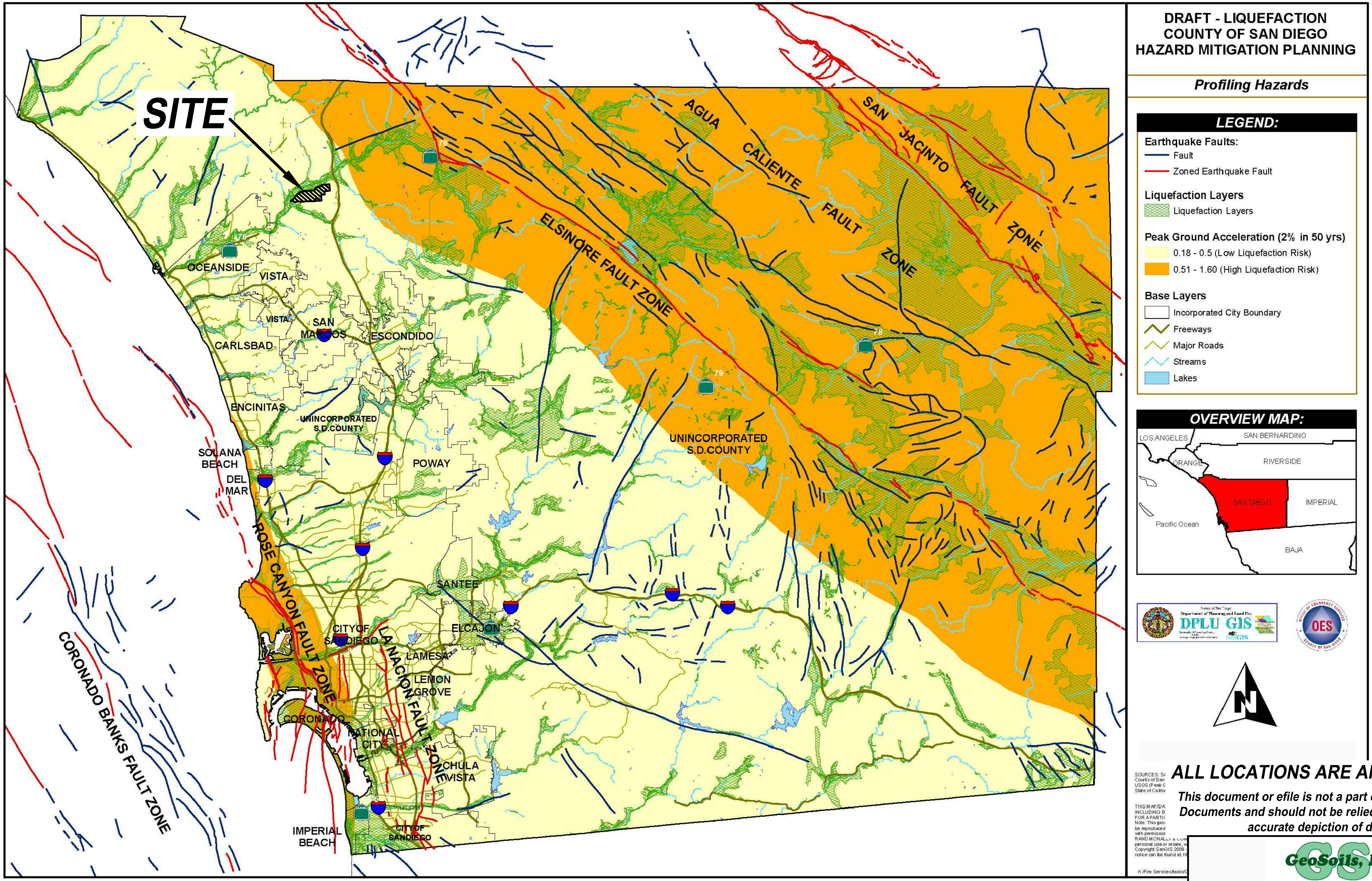
Our evaluation indicates that some of the younger alluvial materials (Qal) are liquefiable and have the potential for vertical and horizontal deformations when subjected to the design basis earthquake. Previous work by the County (County, 2009) also indicates that alluvial areas of the site are susceptible to liquefaction (see Figure 2). Groundwater ranges in depth from approximately 11½ to 21 feet below existing grades across the site. In the vicinity of Planning Area PA-3, groundwater elevations are about 178 feet to 179 feet MSL. Mitigation will typically include, but not necessary include, fill surcharging, ground improvement, and/or relatively onerous foundation design.

Seismically-induced liquefaction is a phenomenon in which cyclic stresses, produced by earthquake induced ground motion, create excess pore pressures in relatively cohesionless soils. These soils may thereby acquire a high degree of mobility, which can lead to sand boils, lateral movement/sliding, volumetric consolidation and settlement of loose sediments, and other damaging deformations as pore pressures dissipate. This phenomenon occurs only below the water table, but after liquefaction has developed, it can propagate upward into overlying, non-saturated soil, as excess pore water dissipates. Thus, one of the primary factors controlling liquefaction potential is the depth to groundwater.

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

Liquefaction susceptibility is related to numerous factors and the following conditions must generally exist, or have the potential to exist, for liquefaction to occur: 1) sediments must be relatively young in age and not have developed a large amount of cementation; 2) sediments must consist mainly of medium to fine grained, relatively cohesionless sands; 3) the sediments must have low relative density; 4) free groundwater must be present in the sediment; and, 5) the site must have a potential for a design seismic event of a sufficient duration and magnitude, to induce straining of soil particles.

In general, subsurface and background data indicate that the requisite five concurrent conditions exist, or have the potential to exist, within areas of the site underlain with alluvial soils for this project. Thus, it would appear that significant layers of alluvium underlying the northern portion of the site are relatively susceptible to liquefaction. Given the intended development, the potential site accelerations, the relatively low density soils occurring along the margins of the site, where younger alluvium is on the order of 40 to 60 feet thick, and the elevation of groundwater conditions at the site, GSI has performed a liquefaction



ALL LOCATIONS ARE APPROXIMATE

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

GSI GeoSoils, Inc.

LIQUIFACTION HAZARD MAP

Figure 2

W.O. 6960-A-SC	DATE: 10/16	SCALE: 1" = 8 miles

analysis for the proposed development, assuming current groundwater elevations. Seismic differential settlement was evaluated to be the most likely to induce damaging deformations along the alluvium/bedrock contact in Planning Areas PA-3 and PA-5. The margin of Planning Area PA-3 along the areas to remain undeveloped are considered to be susceptible to lateral and vertical seismic induced deformations.

The condition of liquefaction has two principal effects. One is the volumetric strain or "consolidation" of loose sediments with resultant settlement of the ground surface. The other effect is lateral sliding. Significant permanent lateral movement generally occurs only when there is significant differential loading, such as fill or natural ground slopes within susceptible materials. As such, any planned fill slopes constructed within existing alluvial areas present a potential for lateral spread to affect perimeter fill slopes underlain with unmitigated alluvial soils below the groundwater table, and should be further evaluated once grading plans are developed.

The evaluation of whether or not surface manifestation of liquefaction, such as sand boils, ground fissures, foundation tilt and cracking, etc., will occur at a site can be made using Special Publication 117A "Guidelines for Analyzing and Mitigating Liquefaction in California" (CGS, 2008). Based on the thickness of the potentially liquefiable layer, the thickness of the non-liquefiable soil (fill) cover, and ground acceleration for the design earthquake, an evaluation of these "liquefied" soils was made. Based on our evaluation, the potential for sand boils on the graded and mitigated site, is considered low, due to the thickness of overlying, non-liquefiable materials, consisting primarily of remediated alluvium (i.e., removal/recompact) and plan fill. However, the potential for densification and settlement of any near surface alluvium left in place (unmitigated) is considered high, and is discussed in a later section of this report. Potential lateral deformation (lateral spread) of fill over left-in-place younger alluvium (Qal) at the margin of Planning Area PA-3 adjacent to areas that will remain undeveloped is also moderate to high.

Seismic Densification

Seismic densification is a phenomenon that typically occurs in low relative density granular soils (i.e., USCS classified as SP, SW, and SM) that are above the groundwater table and are significantly dry of optimum moisture content. During the seismic-induced ground shaking, these natural sediments deform under loading and volumetrically strain, resulting in ground surface settlements. Additionally, some densification of the adjoining un-mitigated areas may also influence improvements at the perimeter of the site. These unsaturated granular soils are susceptible if left in their original density (unmitigated), and are significantly drier than the optimum water content (as defined by the ASTM D 1557). Some of the layers of alluvium onsite that was encountered above the water table may be considered susceptible to seismic densification. However, due to the relatively shallow groundwater table (i.e., 11 to 21 feet) in alluvial areas, mitigation of this material is feasible using conventional removal and recompaction techniques during grading in most areas.

Seismic Settlement

The results of the analyses indicate that densification and settlement of the underlying alluvium would occur in the event of the design earthquake, due to liquefaction.

The magnitude of potential seismic settlement for both "free-field" as well as under the anticipated foundation loads were evaluated using various methods within the LiquefyPro program in general accordance with Special Publication 117A "Guidelines for Evaluating and Mitigating Seismic Hazards in California" (California Department of Conservation, California Geological Survey [CGS], 2008) and ASCE 7-10, Section 11.8.3 (ASCE, 2010). GSI has provided these analyses with a consideration for a factor-of-safety (FOS) of 1.0 and 1.3. The results of our analysis are presented in our liquefaction and lateral spread analysis, presented in a later section of this report.

Lateral Spreading

Lateral spread phenomenon is described as the lateral movement of stiff, surficial, mostly intact blocks of sediment or compacted fill displaced downslope towards a free face along a shear zone that has formed within the liquefied sediment. The resulting ground deformation typically has extensional fissures at the head of the failure, shear deformations along the side margins, and compression or buckling of the soil at the toe. The extent of lateral displacement typically ranges from less than an inch to several feet. Two types of lateral spread can occur: 1) lateral spread towards a free face (e.g., river channel or embankment); and 2) lateral spread down a gentle ground slope where a free face is absent. Factors such as earthquake magnitude, distance from the seismic energy source, thickness of the liquefiable layers, depth of ground improvement grating, amount of slope confinement with rip-rap, the slope of the underlying bedrock surface and the fines content and particle size of those sediments also correlate with ground displacement.

Areas underlain with alluvium along the northern portion of the project will have one "free face" (planned north facing fill slope), where this slope "toe's out"into the adjacent undeveloped area where groundwater was observed to be as shallow as about 11½ to 21 feet or greater, below existing grades. The evaluation (Appendix F) indicates that the outer 25 feet of the development adjacent to the undeveloped area, may be subject to lateral spread and may significantly affect improvements, drainage, and top-of-slope stability in these areas, if no mitigation is used. Seismic deformations noted herein may be exceeded in this outer margin area, adjacent to the residential areas.

Other Seismic Hazards

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

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

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

Subsidence

The effects of areal subsidence generally occur at the transition or boundaries between low-lying areas and adjacent hillside terrain, where materials of substantially different engineering properties (i.e., alluvium/older alluvium - bedrock [granitic]) are present. Subsidence may occur at any time when site conditions change, including groundwater or fluid withdrawal, loading or heavy vibrations, etc., but is most noticeable during large-scale seismic events. Increased groundwater withdrawal at the northern portion of the site, PA-3 and PA-5, is possible and is beyond the present scope of this study.

Provided the guidelines presented in this report are properly incorporated into the design and construction of the project, the potential for significant areal subsidence is considered low.

Landslides

Landslide deposits were not noted during our review of Tan, et al. (2000), or Tan and Kennedy (2005). Landslide deposits, and/or geomorphology indicative of landslide deposits (i.e., humocky topography, scarps, lobate soil deposits, etc.) were not noted in the field. Given the site's relatively gentle relief (i.e., slope gradients on the order of 4:1 [h:v], or less), the absence of adverse geologic structure, and dense/resistant nature of the underlying bedrock, the potential for significant landslides to affect the proposed site development is considered low. The potential for debris flows is considered low for PA-1, PA-2, PA-3, and PA-4. The potential in PA-5 is elevated and would need to be further evaluated when grading plans for this area are available.

ROCK HARDNESS EVALUATION

A seismic refraction survey was performed in selected areas of deep cuts where the site is underlain with near surface granitic bedrock. To date, the survey consists of seven (7) seismic refraction lines, with four (4) completed within the vicinity of Planning Area PA-1 and PA-2. A summary of methodology and procedures is presented in the referenced report.

Layer boundaries tend to mimic the surface topography, although variations are common depending upon the depth of weathering, fracturing, etc. In general, the survey indicated a near surface layer (Layer 1) thickness (i.e., undocumented fill, colluvium, weathered bedrock), ranging from about ±1 to ±7 feet. The average velocity of Layer 1 material is about ±1,000 fps, and is considered typical for such near surface material. The depth to the Layer 1/Layer 2 transition (bedrock) also ranges from about ±1 to ±7 feet below existing grades. The average velocity of Layer 2 is about ±2,900 fps, with some variability. Layer 3 is inferred at depths on the order of ±9 to possibly 38 feet, with average velocities in Layer 3 (relatively unweathered bedrock) likely greater than 4,700 fps. At depths where velocities are greater than about 6,000 fps, rippability is ambiguous and blasting usually is required.

An evaluation has been made of the seismic refraction line data to estimate the approximate depth to non-rippable trenching (i.e., utility excavation) and to non-rippable bedrock. Approximate cut-off velocities of ±3,800 and ±6,000 fps are generally used as a basis for non-rippable trenching (assuming a Cat 235 Hoe [a large trackhoe], or equivalent), and non-rippable bedrock (assuming a D9L, or equivalent), respectively. It should be noted that a conventional rubber-tired backhoe can experience non-productive trenching at seismic velocities much less than ±2,000 to 2,500 fps.

Bedrock excavatability with respect to trenching shallower than the approximate ±3,800 fps cut-off depth is expected to vary from easy to very difficult and the necessity for localized areas requiring rock breaking, or blasting should be anticipated. Similarly, bedrock rippability shallower than the approximate ±6,000 fps cut-off depth is expected to vary from easy to very difficult, and the necessity for localized areas requiring rock breaking and/or blasting cannot be entirely precluded.

Variations should be expected. As such, bedrock excavations from the surface downward may generate oversize rock. Isolated "floaters" or corestones may also be encountered. The bulk of the materials derived from the weathered portion of the bedrock (up to and including the ±3,800 to 6,000 fps cut-off) are anticipated to disintegrate to approximately 12 to 24 inches and smaller constituents. Any oversize materials ($\geq$ 12 inches) generated would require special handling for use in fills, and may not be placed within 10 feet of finish grade or used as backfill in utility trenches. Oversize materials typically become commonplace during excavation into 5,000 fps materials, usually requiring specialized placement techniques during grading.

Based upon our experience in this area, and the seismic refraction data obtained, the following table reflects our preliminary estimates of the rippability and trenchability at the locations of the seismic refraction survey lines; other interpretations are possible:

SEISMIC LINE NO.	GENERAL RIPPABILITY (ASSUMING A D9L DOZER OR CAT 235 HOE, OR EQUIVALENT)
ST-1 (PA-5)	Rippable and trenchable to depths explored of 30 feet. Difficult trenching below depths of 2 to 4 feet. Localized blasting and/or rock breaking may not be precluded below depths of 10 feet.
ST-2 (open space between (PA-2 and PA-4)	Rippable and trenchable to depths explored of 30 feet. Difficult trenching below depths of 2½ to 3 feet. Localized blasting and/or rock breaking may not be precluded below depths of 10 feet.
ST-3 (PA-4)	Rippable and trenchable to depths explored of 30 feet. Moderate to difficult trenching below depths of 3½ feet. Localized blasting and/or rock breaking may not be precluded below depths of 30 feet.
ST-4 (PA-2)	Rippable to depths explored of 30 feet. Not trenchable below depths of 3 to 4 feet. Localized blasting and/or rock breaking may not be precluded below depths of 10 feet. Oversize material is significant.
ST-101 (PA-2)	Rippable and trenchable to depths explored of ±30 feet. Difficult trenching below depths of 2½ to 5½ feet. Localized blasting and/or rock breaking may not be precluded below depths of 30 feet.
ST-102 (PA-2)	Rippable and trenchable to depths explored of ±30 feet. Difficult trenching below a depth of 2½ feet. Localized blasting and/or rock breaking may not be precluded below depths of 30 feet.
ST-103 (PA-2)	Rippable and trenchable to depths explored of ±38 feet. Difficult trenching below depths of 4½ to 7 feet. Localized blasting and/or rock breaking may not be precluded below depths of 30 feet.

Rock Hardness Summary

In general, utilizing the seismic data, it appears that the site area in the vicinity of our seismic lines may be characterized as being underlain by a surficial soils (fill, colluvium, weathered rock) to depths ranging from about ±1 to about ±7 feet in thickness, with less weather bedrock below those depths. At depths inferred to be approximately 30 feet or more, relatively fresh and very dense granitic bedrock likely exists. Based on all of the above, the need for overexcavation, blasting and/or line shooting would be anticipated on the site, should proposed cut grades exceed the depths indicated herein, in areas underlain with granitic bedrock (see Plate 1), and may be required near the surface. It should be noted that a conventional rubber-tired backhoe will experience non-productive trenching at seismic velocities much less than ±2,000 to 2,500 fps. The seismic refraction data presented herein should be further reviewed in conjunction with final grading plans (when available). It should be noted that due to the variability of bedrock weathering, and the potential for local boulders, or less weathered bedrock, very difficult ripping, rock breaking, and/or blasting cannot be entirely precluded at shallower depths, even at or near the surface.

LABORATORY TESTING

General

Laboratory tests were performed on representative samples of the onsite earth materials collected from the subsurface geotechnical investigation summarized in GSI (2015), and this study, in order to evaluate their physical characteristics and engineering properties with respect to anticipated site development. The test procedures used and subsequent results are presented below.

Classification

Soils were classified with respect to the U.S.C.S. in general accordance with ASTM D 2487 and D 2488. The soil classification is presented with the Exploration Logs (see Appendix B).

Field Moisture and Density

Field moisture content and dry unit weight were determined for relatively "undisturbed" samples of earth materials obtained from GSI's exploratory excavation. The dry unit weight was evaluated in general accordance with ASTM D 2937, in pounds per cubic foot (pcf), and the field moisture content was evaluated as a percentage of the dry weight. Water contents were measured in general accordance with ASTM D 2216. Results of these tests are summarized on the Exploration Logs (see Appendix B).

Laboratory Standard

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

LOCATION AND DEPTH	SOIL TYPE	MAXIMUM DENSITY (PCF)	OPTIMUM MOISTURE CONTENT (%)
TP-7 @ 4' (GSI, 2015)	Silty Sand, Gray Brown	128.0	11.0
HSA-4 @ 1'-2'	Silty Sand, Brown	122.0	12.0
TP-102 @ 3'-4' (GSI, 2015)	Silty Sand, Brown	128.5	10.5

Expansion Index

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

LOCATION AND DEPTH	EXPANSION INDEX	EXPANSION POTENTIAL
TP-1 @ 1-4' (GSI, 2015)	<5	Very Low
TP-2 @ 8' (GSI, 2015)	<5	Very Low
TP-102 @ 3-4'	<20	Very Low

Our evaluation to date indicates that site soils appear to be very low expansive. Soils derived from excavation in bedrock are anticipated to also be very low expansive; however, residual soils developed on bedrock would typically be more expansive.

Direct Shear

Strain-controlled direct shear tests were performed on representative soil samples in general conformance with the ASTM D 3080 test method. The test results are presented in Appendix E.

Particle-Size Analysis

An evaluation was performed on selected representative soil samples in general accordance with ASTM D 422. Particle size analyses were performed on selected samples from our exploratory borings. The grain-size distribution curves are presented in Appendix E. These test results were utilized in evaluating the soil classifications in accordance with the Unified Soil Classification System.

Consolidation Test

A consolidation test was performed on selected undisturbed samples obtained from hollow stem auger borings completed within PA-3. Testing was performed in general accordance with ASTM Test Method D 2435. Test results are presented as in Appendix E.

Resistance Value

R-value testing was performed on a representative soil sample in general accordance with the latest revisions to the Department of Transportation, State of California, Material & Research Test Method No. 301. Based on this test, an R-Value of 74 was evaluated (Appendix E). It should be noted that an R-value of 38 was utilized in preparation of GSI (2015).

Corrosivity Testing

Corrosivity testing, performed on representative samples of site soil (GSI, 2015 and current study) indicates a pH range of 6.45 to 6.99 (which is considered relatively neutral); a soluble sulfate content of 0.001 to 0.011 percent by weight (which is considered negligible to moderate [Class S0 and S_1, respectively] per Table 4.2.1 of ACI 318-11 (per 2013 CBC [CBSC, 2013]); a chloride content of 122 to 182 parts per million (ppm), which is considered elevated; and a saturated resistivity of 1,800 to 3,400 ohm-cm (which is considered mildly corrosive to ferrous metals). While it is our understanding that typical structural concrete (f'c $\geq$ 3,000 to 4,500 psi) with minimal design cover is generally sufficient mitigation for such conditions, GSI recommends consultation with a corrosion engineer. Additional tests are warranted. Test results are presented in Appendix E.

PRELIMINARY SETTLEMENT, LIQUEFACTION AND LATERAL SPREAD ANALYSIS

GSI has estimated the potential magnitudes of total settlement, differential settlement, and angular distortion for the site. The analyses were based on laboratory test results and subsurface data collected from test pits, borings, and CPT soundings completed in preparation of this study. Site specific conditions affecting settlement potential include depositional environment, grain size and lithology of sediments, cementing agents, stress history, moisture history, material shape, density, void ratio, etc. The following discussion is preliminary. Additional studies are recommended once plans are developed.

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

Due to the varied geologic conditions, and for the purposes of this evaluation, at least three (3) general, as-built conditions are anticipated, and summarized into the following groups, as follows:

- **Group 1** - Areas where the complete removal of surficial deposits of alluvium, colluvium, and any unsuitable older alluvium are removed to dense older alluvium, (PA-5, and Portions of PA-2). This condition also includes overexcavated cut lots exposing older alluvium that meet the criteria indicated herein.

- **Group 2** - Areas where the complete removal of surficial deposits of alluvium, colluvium, and any unsuitable older alluvium are removed to granitic bedrock (PA-1, PA-2, and PA-4). This condition also includes overexcavated cut lots exposing suitable granitic bedrock.

- **Group 3** - Areas of alluvium left in place below the regional groundwater table, located approximately 11 to 18 feet below the existing ground surface. This condition would generally occur within PA-3, located within the alluviated valley floor.

Static Settlement of Fill Areas

On a preliminary basis, and based on a review of PDC (2016) maximum fill thicknesses are anticipated to range on the order of 35 to 46 feet within Planning Areas PA-1 and PA-2, and up to about 13 feet within PA-3.

Group 1

Group 1 lots are anticipated to consist of fill over dense older alluvium, and primarily occur within a small portion of PA-2, and the lower elevations of PA-5. The evaluation of older alluvium exposed in our subsurface explorations indicate the natural older alluvium is not prone to excessive post-construction compression although additional borings and review are necessary. The total post-construction "static" settlement may be approximately 1¼ inch, and differential settlement on the order of ⅝ inch in 40 lateral feet, with overlying fills up to 25 feet in thickness, on a preliminary basis. For fills between 25 and 30 feet, post-construction "static" settlement may be approximately 1½ inch, and differential settlement on the order of ¾ inch in 40 lateral feet. For fills between 30 and 50 feet, post-construction "static" settlement may be up to approximately 2¼ inch, and differential settlement on the order of 1½ inch in 40 lateral feet. Static settlement may be reduced by increasing the minimum relative compaction to at least 95 percent, per ASTM D 1557, for fills greater than 30 feet deep.

Group 2

Group 2 lots, primarily located within proposed Planning Areas PA-1, PA-2, PA-4, and the upper elevations of PA-5 are anticipated to be graded with fill over bedrock. Subdrainage and slope of overexcavation cuts is important to the reduction of potential perched water and subsequent compression. The total post-construction "static" settlement may be approximately 1¼ inch, and differential settlement on the order of less than ¾ inch in 40 lateral feet, with overlying fills up to 25 feet in thickness, on a preliminary basis. For fills between 25 and 30 feet, post-construction "static" settlement may be approximately 1½ inches, and differential settlement on the order of ¾ inch in 40 lateral feet. For fills between 30 and 50 feet, post-construction "static" settlement may be up to approximately 2¼ inches, and differential settlement on the order of 1⅛- inch in 40 lateral feet. Static settlement may be reduced by increasing the minimum relative compaction to at least 95 percent, per ASTM D 1557, for fills greater than 30 feet deep. In order to produce a higher level of performance and given the limited area of thicker fills in PA-1 and PA-2, consideration be given to increasing the minimum relative compaction in those planning areas to 95 perfect (per ASTM D 1557).

Group 3

Group 3 lots (PA-3) are likely to have left in place younger alluvium over bedrock beneath these areas at the conclusion of grading. Remedial grading has been estimated as approximately 10 to 17 feet below existing grades. That is, from the existing ground surface approximately 10 to 17 feet of dry to wet, loose alluvium will be removed up to and including the material at about 1 to 2 feet from the water table. The static settlement evaluation was based on the compression of up to about 15 to 25 feet of planned and remedial grading (10 to 17 feet of remedial plus 3 to 13 feet of fill above grade). This may result in compressions of the loose/soft layers of alluvium (long-term 10 to 30 years) with the potential to induce angular distortions in excess of 1/480 in this area (see below).

The static settlement estimates do not include the effects of expansive soils (shrink and swell) and the loading of soils under foundations, as well as top-of-slope creep effects, which are described in a later section of this report.

Seismic-Induced Settlement, Liquefaction and Densification

General

Following a review of the boring and CPT data, and laboratory testing, the boring and CPT data were evaluated for liquefaction potential within the alluvial areas. The liquefaction analyses were performed using the LiquefyPro computer program (Civiltech Software, 2015 [version 5.9b]), field boring/laboratory data, and the data from the recent CPTs. Computer printouts from the analysis are presented in Appendix F. The analysis was conducted in general accordance with Special Publication 117A "Guidelines for Evaluating and Mitigating Seismic Hazards in California" (California Department of Conservation, California Geological Survey [CGS], 2008) and American Society of Civil Engineers (ASCE) Manual 7-10 (ASCE, 2010). For the analyses, GSI utilized a groundwater depth of about 7 to 18 feet below the existing grade in alluvial areas, to account for an anticipated groundwater level at the time of the design seismic event. For ground acceleration, GSI used a PHGA value based on PGAM, in accordance with Section 11.8.3 of the ASCE Manual 7-10 (ASCE, 2010). The PHGA used in the liquefaction analysis ranged from 0.29g to 0.46g. Lastly, the design earthquake magnitude of 7.1 on the Julian strand of the Elsinore fault (Cao, et al., 2003) was also used. A review of the CPT and hollow stem auger data generally indicate that alluvium typically consists of interlayered sands, silty sands, silts, with minor amounts of clay with fines content (<0.075 mm) generally less than 20 percent.

The results of the liquefaction analyses indicate that liquefaction may occur within areas underlain by younger alluvium occurring below the groundwater table. We have evaluated this potential for seismic induced deformation using fill thicknesses of up to approximately 5 feet to 12 feet above existing grades. Therefore, it is the opinion of GSI, that liquefaction and its corresponding secondary effects including seismic settlement and lateral spreading, are considered potential secondary seismic hazards in alluvial areas. As a result, mitigation measures will be necessary to reduce the impact of earthquake induced liquefaction. Liquefaction mitigation at the site requires either special foundation design and ground improvements. Due to the presence of one free-face condition (i.e. perimeter fill slopes) along the northern part of Planning Area PA-3, the potential for lateral spreading exists with respect to the performance of perimeter fill slopes that "toe out" into the adjacent river valley floor.

Seismic densification of volumetric strain of loose, relatively dry (significantly drier than optimum moisture), granular soils above the groundwater table may occur when considering the seismic loading of the design basis earthquake. For this review, we have assumed a lack of regional groundwater within the fill (planned and mitigated). Based on our observations, the planned relative density (D_r) of 65 to 80 percent of the fill with relative compaction of 90 to 95 percent is indicative of low susceptible soils. Seismic densification of planned and remedial compacted granular fill placed beneath foundations and behind some site walls may be subject to some low magnitude seismic densification and result in limited surface settlement. The anticipated contribution strain or surface settlement due to seismic densification is approximately 1 inch or less, assuming up to 30 feet of granular fill in Planning Area PA-3 and up to 50 feet in Planning Area PA-2. Differential settlement due to seismic densification influence may be from 50 percent to 100 percent of this estimate, and should be considered by the designer.

The magnitude of potential seismic settlement for both "free-field" as well as under the anticipated fill loads were evaluated using various methods within the LiquefyPro program in general accordance with Special Publication 117A "Guidelines for Evaluating and Mitigating Seismic Hazards in California" (CGS, 2008) and ASCE 7-10, Section 11.8.3 (ASCE, 2010). GSI has provided these analyses with a consideration for a factor-of-safety (FOS) of 1.0 to 1.3 using the PGA_M to model horizontal site acceleration.

Based upon the assumed, current design configuration and the results of our seismic deformation analysis, the total free-field ground settlement in alluvial areas during the design seismic event may be summarized as follows:

Seismic Settlement Groups 1 and 2 (Planning Area PA-1, PA-2, PA-4, and PA-5)

Seismic settlement within settlement Groups 1 and 2 are not anticipated to be more than ½ inch total, and ¼ inch differential over a lateral distance of 40 feet, with greater values evaluated for deeper fills and up to 1¼- inch on fills (not affected by groundwater) to 50 feet thick differential of ⅝ inch in 40 lateral feet. Settlement values are presented in the foundation design section of this report. Seismic settlement may be reduced by increasing the minimum relative compaction to at least 95 percent, per ASTM D 1557, where fills are greater than 30 feet deep. Considering the area, volume of fill and number of lots affected, GSI recommends that the deep fill be compacted form top to bottom in PA-1 and PA-2, to 95 percent.

Seismic Settlement Group 3 (Planning Area PA-3)

Settlement Group 3 primarily includes all of Planning Area PA-3, where the planning area is underlain with relatively loose alluvial sediments and a shallow ground water table. Seismic settlement was evaluated using site accelerations ranging from 0.29g to 0.46g, inconsideration of plan fills, and typical removal/recompaction to near the groundwater table. Settlements were evaluated using a factor of safety (FOS) ranging from 1.0 to 1.3. Our evaluation generally indicates that seismically induced settlements decrease with an increase in fill loading/thickness.

Based upon the anticipated graded configuration, and the results of our seismic settlement analysis, the total free-field ground settlement during the design seismic event will be on the order of about 1 to 6 inches and about 2 to 4 inches, over a distance of about 40 feet, using site accelerations ranging from 0.29g to 0.46g, with a minimum fill surcharge of at least 2 to 13 feet. As the dynamic differential settlement (liquefaction plus densification) is simply calculated as that portion of the total settlement over a 40-foot span, potential differential settlements on the order of about 2 to 4 inches over 40 feet horizontally, or an angular distortion (seismic only) on the order of 1/240 to 1/120, may be anticipated. This potential damaging seismic deformation in untreated alluvium should be considered in foundation and improvement designs and expected performance, provided that the recommended earthwork is performed. The results of the seismic settlement analysis are provided in Appendix C. These seismic deformations are for existing conditions for free-field, as well as under the building and do not include edge effects at the perimeter of the site nor ground improvements. In general, a seismic differential settlement in excess of 2 inches in 40 lateral feet is typically considered excessive for shallow residential foundations.

Monitoring

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

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

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

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

Foundation Settlement Due to Structural Loads (Building Pads)

The settlement of the structures supported on strip and/or spread footings founded on compacted fill will depend on the actual footing dimensions, the thickness and compressibility of fill and natural soil deposits below the bottom of the footing, and the imposed structural loads. Provided the thickness of fill below the bottom of the footing is at least equal to twice the width of the footing, and based on a maximum allowable bearing pressure of 2,500 pounds per square foot (psf) or less, provided in this report, local post-construction settlement due to applied structural loads of less than 1 inch should be anticipated; however, this assumes all fill is properly compacted. Given this condition, the majority of the foundation settlement should occur as the building loads are applied during construction. Post construction differential settlement between the lightest and heaviest loaded footings, due to applied loads, may occur if the foundation is of the conventional type, and is anticipated to minimally be on the order of ¼ to ½ inch. Further review will be needed once draft foundation plans and building loads are provided.

Lateral Spreading

Lateral spread phenomenon is described as the lateral movement of stiff, surficial, mostly intact blocks of sediment or compacted fill displaced downslope towards a free face along a shear zone that has formed within the liquefied sediment. The resulting ground deformation typically has extensional fissures at the head of the failure, shear deformations along the side margins, and compression or buckling of the soil at the toe. The extent of lateral displacement typically ranges from less than an inch to several feet. Two types of lateral spread can occur: 1) lateral spread towards a free face (e.g., river channel or embankment); and 2) lateral spread down a gentle ground slope where a free face is absent. Factors such as earthquake magnitude, distance from the seismic energy source, thickness of the liquefiable layers, the slope of the underlying bedrock surface (Group 2 lots) and the fines content and particle size of those sediments also correlate with ground displacement.

Areas underlain with alluvium along the northern side of the project (Planning Area PA-3) will likely have one "free face" (north facing fill slope), where this slope "toe's out" into the adjacent flood plain area where groundwater was observed to be as shallow as 11½ to 18 feet below existing grades. On a preliminary basis, the outer 15 to 50 feet of the pads/improvement areas adjacent to unmitigated flood plain areas may be subject to lateral spread and may significantly affect improvements, drainage, and top-of-slope stability in these areas, if no mitigation is used. Mitigation may include grading (i.e., removal and recompaction of fill), ground impacted use of rip rap.

Subsidence

The effects of areal subsidence generally occur at the transition or boundaries between low-lying areas and adjacent hillside terrain, where materials of substantially different engineering properties (i.e., alluvium/older alluvium - bedrock [granitic]) are present. Subsidence may occur at any time when site conditions change, including groundwater or fluid withdrawal, loading or heavy vibrations, etc., but is most noticeable during large-scale seismic events.

Provided the guideline presented in this report are properly incorporated into the design and construction of the project, the potential for significant areal subsidence is considered low. A review of the long-term effects of increase/decrease in the groundwater of the river valley on Planning Areas PA-3 and PA-5 is beyond the scope of this study.

Ancillary Improvements

Ancillary improvements, such as utilities, pavements, and concrete flatwork will be subject to potential deformations and repair costs due to surface manifestation of liquefaction, such as sand boils, ground fissures, etc., In order to mitigate this potential, additional ground improvement methods, such as: soil cement treatment, drainage improvement with wick drains, compaction grouting, stone columns, geotextile reinforcement of removal bottoms and pavement subgrades, and/or increasing the depth of the currently removal/recompaction in alluvial areas, should be considered, and would also improve the performance of building foundations if performed throughout the site. Other methods, such as; dynamic compaction, compaction piles, and vibro floatation are not generally recommended, as these methods may induce settlement on adjacent, developed properties. Slurry backfill around vertical utility stand pipes should also be utilized.

PRELIMINARY SLOPE STABILITY EVALUATION

Gross Stability

Graded slopes are generally considered to be stable, up to gradients of 2:1 (h:v) or flatter, and bedrock slopes are considered suitable to gradients of 1.5:1, or flatter. However, mapping indicates some localized potential for dip slope oriented fractures/joints in bedrock that may require stabilization, and slope gradients of 2:1, or flatter. Natural slopes appear to be performing adequately. Additional geotechnical review of the seismic stability of those fill slopes is warranted at the 40-scale plan review stage.

All graded slope construction will require observation during grading in order to evaluate the findings and conclusions presented herein and in subsequent reports. Our analysis assumes that graded slopes are designed and constructed in accordance with guidelines provided by the County, the 2013 CBC (CBSC, 2013), the current edition of the "Greenbook," and recommendations provided by this office. These slopes are generally anticipated to be stable, assuming proper construction, maintenance, and normal climatic conditions.

If liquefaction occurs in unmitigated soils at the limit of fill slopes constructed within Group 3 settlement areas, the seismic FOS may be less than 1.1. Additional geotechnical review of the seismic stability of those fill slopes is warranted.

Temporary backcuts for construction slopes and keyways, are anticipated to be 1:1 (h:v) or flatter, and are anticipated to have a static FOS of 1.25. Should perched groundwater or other unexpected conditions be exposed during excavation, the project geotechnical consultant should review the conditions and revise recommendations as needed.

Surficial Stability

Surficial stability was evaluated for graded slopes constructed of compacted fills and/or formational soil. On a preliminary basis, our evaluation indicates that slopes should perform adequately against surficial failure, provided that the slopes are properly constructed and maintained, under normal rainfall.

Onsite soils are granular, sandy soils. If sandy soils with a cohesion of less than 200 psf are used on slope faces, the slopes may have surficial stability/erosion issues and perhaps a FOS against surficial instability of less than 1.5. Planting and management of surficial drainage is imperative to the surficial performance of slopes. Typically, similar to coastal bluff retreat, a surficial erosion rate (average) of up to about 1¼ inches/year for natural and unprotected sandy slopes may be assumed. Foot traffic and other activities that exacerbate surficial erosion should not be allowed to occur on slopes. Failure to adhere to these conditions may drastically increase any local surficial erosion, requiring mitigation, so that headward erosion does not result, and impact roadways, pads, and other improvements.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

General

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

- Depth to competent bearing material below existing pad grade.
- Expansion and corrosion potential of onsite soils.
- Perimeter conditions and the influence of onsite and offsite unmitigated soils.
- Seepage, drainage, and moisture transmission through foundations.
- Settlement potential (static and seismic).
- Groundwater.
- Lateral spreading potential.
- Regional seismic activity.
- Rock hardness excavation difficulty and utility installation/foundation construction.
- Potential for oversize rock exceeding 12 inches in long dimension.

The recommendations presented herein consider these as well as other aspects of the site. The engineering analyses, performed, concerning site preparation and the recommendations presented herein have been completed using the information provided and obtained during our field work. In the event that any significant changes are made to proposed site development, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the recommendations of this report are evaluated or modified in writing by this office. Foundation design parameters are considered preliminary until the foundation design, layout, and structural loads are provided to this office for review.

- Removals should consist of all surficial deposits of existing fill, colluvium, and near surface, weathered natural soils. Conventional removals of alluvium will be limited locally, due to the presence of a shallow groundwater table.

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

- Geologic observations should be performed during grading to observe and/or further evaluate site geologic conditions. Although unlikely, if adverse geologic structures are encountered, supplemental recommendations and earthwork may be warranted.

- A shallow groundwater table will be encountered during removals/excavation within alluvium, primarily within Planning Area PA-3. Within other areas of the site, regional groundwater is not expected to be encountered during excavation. However, seepage between layers of fill, fill/bedrock contacts, and in discontinuities within bedrock, cannot be precluded in all areas during, or after grading.

- Our laboratory test results and experience on this site indicate that soils with a very low, and possibly low expansion potential generally underlie the site. This should be considered during project design and construction. Preliminary foundation design and construction recommendations are provided herein for these soil conditions.

- Building foundations will need to be designed to accommodate the expansive soil conditions, corrosive soils, and potential settlements. Foundation alternatives including stiffened slabs, mat slabs, and post tensioned slabs, are provided.

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

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

Based on the findings of this study, the site is suitable for the proposed development from a geotechnical engineering and geologic viewpoint, provided the recommendations presented herein are properly incorporated into the design and construction phases of development. Preliminary remedial earthwork and foundation recommendations are provided in the following sections.

EARTHWORK CONSTRUCTION RECOMMENDATIONS

General

Remedial earthwork will likely be necessary for the support of the proposed settlement-sensitive improvements. Remedial grading should conform to the guidelines presented in Appendix J of the 2013 CBC (CBSC, 2013), the requirements of the County, and the General Earthwork, Grading Guidelines, and Preliminary Criteria presented in Appendix H, except where specifically superceded in the text of this report. In case of conflict, the more onerous code or recommendations should govern. Prior to grading, a GSI representative should be present at the pre-construction meeting to provide additional grading guidelines, if needed, and review the earthwork schedule.

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

Demolition/Grubbing

1. Vegetation, and any miscellaneous deleterious debris generated from the demolition of existing site improvements should be removed from the areas of proposed grading/earthwork.

2. Cavities or loose soils remaining after demolition and site clearance should be cleaned out and observed by the geotechnical consultant. 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.

3. Any septic systems encountered should be removed and disposed of per County guidelines.

Treatment of Existing Ground

The treatment of existing ground will vary by area/geologic conditions onsite, and may be subdivided into at least three (3) general cases, as follows:

Case I - Areas underlain with near surface, older alluvium and/or granitic bedrock.

Case II - Areas underlain with alluvium below a shallow groundwater table.

A discussion of existing ground treatment is presented for each case as follows:

Case I, Areas Underlain With Near Surface, Older Alluvium and/or Granitic Bedrock (Settlement Group Areas 1 and 2)

1. Areas underlain with near surface, older alluvium and/or granitic rock generally occur in the vicinity of Planning Areas PA-1, PA-2, PA-4, and PA-5.

2. Where not removed by the planned excavations, all undocumented fill, colluvium, alluvium, and weathered older alluvium/bedrock should be removed to competent older alluvium/bedrock, cleaned of deleterious materials, moisture conditioned, and recompacted within areas proposed for settlement-sensitive improvements. In general, the remedial removal excavations are anticipated to be on the order of 1½ to 5½ feet, to depths potentially as much as 17 to 18 feet locally (lower elevations of Planning Areas PA-2 and PA-5), where observed in our subsurface explorations. However, local deeper removal excavations elsewhere cannot be precluded and should be anticipated. Actual depths of removals will be evaluated in the field during grading by the soil engineer. This recommended earthwork does not include in-place ground improvement/treatment.

3. Subsequent to the above removals, the upper 8 inches of the exposed subsoils/bedrock should be scarified, brought to at least optimum moisture content, and recompacted to a minimum relative compaction of 90 percent of the laboratory standard (ASTM D 1557), prior to any fill placement.

4. Localized deeper removals may be necessary due to buried drainage channel meanders or dry porous materials. The project soils engineer/geologist should observe all removal areas during the grading.

Case II, Areas Underlain with Loose Alluvium and a Shallow Groundwater Table (Settlement Group Area 3):

1. Areas underlain with loose, surficial deposits of alluvium and a shallow groundwater table, generally occur in the vicinity of Planning Area PA-3.

2. Alluvium should be removed to near the existing groundwater table, cleaned of deleterious materials, moisture conditioned, and recompacted within areas proposed for settlement-sensitive improvements. In general, the remedial removal excavations are anticipated to near the groundwater table, at depths on the order of 10 to 17 feet below existing grades, and be completed to at least 15 feet outside the improvement. Excavations may generate wet materials that will require "drying back" to a workable moisture content prior to placement as compacted fill. In order to reduce damaging effects of liquefaction to tolerable levels an additional 5 to 15 feet below the groundwater may also be modified (in-place ground improvement) or using previously discussed grading techniques.

3. Yielding subgrades near the groundwater table may require bottom stabilization with stone prior to fill placement. In this case, stones consisting of gravel to cobble size material should be worked into the soil until a relatively firm bottom is achieved. The use of crushed rock and Mirafi HP 570 should be considered to stabilize removal bottoms.

4. For Planning Area PA-3, deep foundation would potentially mitigate residential foundation, but not reduce static/seismic pad settlement.

5. In order to mitigate the potential for adverse settlement/lateral spreading due to earthquake shaking, ground treatment options for alluvial soils are presented in the following table.

GROUND TREATMENT	DESCRIPTION	COMPATIBLE FOUNDATION TYPES	QUALITY AND COST
Partial Removal/ Recompaction (R&R)	R&R completed to near the groundwater table.	Structural mat*	Treats surficial, unsaturated soils. Foundation design must accommodate potential settlements due to differential settlement and liquefaction. Structural mats could potentially require re-leveling after event or after significant time.
Partial Removal/ Recompaction (R&R) with geotextile reinforcement	R&R completed to near the groundwater table. Placement of geotextile fabrics (Mirafi HP 570, or equivalent) along removal bottom. The use of geotextiles in slope construction potentially mitigates lateral spreading.	Structural mat Post-tension slab	Treats surficial, unsaturated soils. Geotextile reinforces fill embankment, further minimizing differential settlements. Foundation design must accommodate potential settlements due to differential settlement and Liquefaction. Potential for foundation re-leveling after event.
Complete R&R	Complete R&R to suitable formation. Dewatering and perimeter shoring required	Structural mat Post-tension slab	Treats loose, near surface unsaturated and saturated soils below the groundwater table. Dewatering and shoring may be cost, or time prohibitive.
R&R with stone columns	R&R completed to near the groundwater table. Stone columns are vibrated stone columns, which are continuous vertical columns of dense interlocking aggregate, free of non-granular inclusions.	Structural mat Post-tension slab	Treats loose, near surface unsaturated soil. Stone columns reinforce cohesive soils and densify granular soils in order to increase bearing capacity, decrease total and differential settlement, provide vertical drainage pathways to increase the time-rate of consolidation settlement, and reduce the potential for liquefaction. A Cost/benefit evaluation vs. other methods will be needed.
R&R with Deep Soil Mixing	R&R completed to near the groundwater table. Deep soil mixing, or DSM is a process of mechanically blending the in situ soil with cementitious materials that are referred to as binders using a hollow stem auger and paddle arrangement. The intent of the soil mixing method is to achieve improved soil properties.	Structural mat Post-tension slab	Treats loose, near surface unsaturated soil. Deep soil mixing provides similar benefits as stone columns. A Cost/benefit evaluation vs. other methods will be needed.
R&R with compaction grouting.	R&R completed to near the groundwater table. Compaction grouting is a method of ground treatment that involves injecting a very stiff homogeneous grout mix in order to displace and compact soils. The injected grout pushes the soils to the side as it forms a grout column or bulb.	Structural mat Post-tension slab	Treats loose, near surface unsaturated soil. Compaction grouting provides similar benefits as stone columns. A Cost/benefit evaluation vs. other methods will be needed.
Vibro Compaction	The Vibro compaction technique is used in granular soils with limited fines content. It uses sustained vibrations to rearrange the soil particles of non-cohesive soils into a denser state. The action of the vibrator reduces the inter-granular forces between the soil particles, allowing them to move into a more compact configuration.	Structural mat Post-tension slab	This process is used in fully saturated and very weak soils. Water jetting removes soft materials, stabilizes the hole and allows the sand backfill to reach the bottom of the vibrator. This is then compacted and interlocked with the surrounding soil. A Cost/benefit evaluation vs. other methods will be needed.

GROUND TREATMENT	DESCRIPTION	COMPATIBLE FOUNDATION TYPES	QUALITY AND COST
Dynamic Deep Compaction	The process involves of dropping a heavy weight repeatedly on the ground at regularly spaced intervals. The weight and the height determine the amount of compaction that would occur. The weight that is used, depends on the degree of compaction desired and is between 8 ton to 36 tons. The height varies from about 3 to 90 feet.	Structural mat Post-tension slab	Most soil types can be improved with dynamic compaction. Soils that are below the water table have to be treated carefully to permit emission of the excess pore water pressure that is created when the weight is dropped onto the surface. A Cost/benefit evaluation vs. other methods will be needed.

* Deep foundations may be considered, but will not mitigate pad settlement in this condition.

Ground Improvement - Value Engineering

Based on subsurface information onsite from our work to date and the potential for settlement due to seismic induced vertical and lateral deformation on Planning Area PA-3, GSI recommends a value engineering review be conducted for the following alternatives.

Comparison of Alternatives

A. Grading

Current planned fill plus remedial within Planning Area PA-3 will yield about 10 to almost 23 feet of fill over the groundwater table. This will be sufficient to reduce the anticipated seismic induced vertical deformation to 2 inches in 40 lateral feet. Based on our analysis to date, an additional 5 to 15 feet below the groundwater would further reduce vertical deformation. Cost for diverting groundwater (or pumping) may exceed the costs of other alternatives.

B. Vibro Compaction

Vibro compaction is anticipated to be effective on most alluvial soils above and below the groundwater could be utilized to densify the upper 30 feet of alluvium (in place of and significantly reduce potential seismic deformation). Costs are generally anticipated to be on the order of $100,000.00 to $200,000.00 per acre, and should be reviewed by the appropriate ground improvement contractor.

C. Deep Dynamic Compaction

Deep dynamic compaction using a heavy weight allowed to free fall up to 90 feet, may be used to treat soils in the upper 30 feet at an approximate cost of approximately $150,000.00 an acre, and should be reviewed by the appropriate ground improvement contractor.

Miscellaneous

It should be noted, that the 2013 CBC (CBSC, 2013) indicates that removals of unsuitable soils be performed across all areas under the purview of the grading permit, not just within the influence of the structure. Relatively deep removals may also necessitate a special zone of consideration, on perimeter/confining areas or near existing utilities. This zone would be approximately equal to the depth of removals, if removals cannot 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 require proper disclosure to all future owners and interested/affected parties. Utilities that cross this zone between mitigated and unmitigated ground may require special details to reduce the potential for rupture during a seismic event.

Transitions/Overexcavation

In order to reduce the potential for differential settlement and facilitate trenching for foundations underground utilities, etc., the entire cut portion of the building pad(s), areas with planned fills less than 4 feet thick, and areas where the as-built fill thickness would be less than 4 feet after remedial removals have been performed should be overexcavated to a minimum depth of 4 feet below finish grade or 2 feet below the lowest foundation element (whichever is greater) and be replaced with compacted fill. The overexcavation subgrade bottom should be inclined to drain away from the structure(s), and into the street. Prior to fill placement, the overexcavation subgrade should be scarified at least 8 inches in depth, moisture conditioned as necessary, and be recompacted to at least 90 percent of the laboratory standard (ASTM D 1557). Overexcavation should be completed to a minimum lateral distance of 5 feet outside the outermost exterior foundation. Overexcavation for underground utilities may be completed to at least 1 foot below the lowest utility invert and be replaced with compacted fill. The undercut transition should not create a minimum to maximum of fill thickness variation of more than 3:1 (maximum to minimum) across any lot. In order to mitigate this condition, deeper undercuts may be necessary.

Fill Import

If the importation of fill soil is necessary, the import material should be reviewed by this office prior to delivery. In general, import fill should be very low to low expansive (E.I. less than 50), and contain 6-inch minus material.

Engineered Fill Placement

Engineered fill should be placed in thin (±6- to 8-inch) lifts, that have been cleaned of vegetation and debris, and moisture conditioned, and mixed to minimally achieve the soil's optimum moisture content, and then be compacted to at least 90 percent of the laboratory standard (ASTM D 1557). Onsite expansive soils may be placed in thin (±6- to 8-inch) lifts that have been cleaned of vegetation and debris, brought to at least 120 percent of (1.2 times) the soil's optimum moisture content, and compacted to achieve a minimum relative compaction of 90 percent of the laboratory standard (ASTM D 1557). Engineered fill placement should be observed and selectively tested for moisture content and compaction by the geotechnical consultant.

Fill Quality

Fill material produced from excavations within onsite soils (i.e., existing fills, colluvium, alluvium, and older alluvium) will generally generate mixtures of silty sand, sand and gravelly sand, with minor amounts of clayey sand, and produce good to fair quality fill material.

Excavations within the underlying granitic bedrock will generally produce good quality material near the surface, with poor quality fill material consisting of angular gravel to cobble to boulder size rock fragments becoming more abundant with depth of excavation.

Onsite soils may be reused as compacted fill provided that major concentrations of vegetation, miscellaneous debris, and oversize material (see below) are removed from the fill, prior to or during fill placement. General recommendations for the treatment of rock onsite is presented in a following section.

Monitoring

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

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

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

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

Slope Considerations and Slope Design

Graded Slopes

Graded slopes are generally considered to be stable, up to gradients of 2:1 (h:v) or flatter, and bedrock slopes may be suitable to gradients of 1.5:1, or flatter. However, mapping indicates some potential for dip slope oriented fractures/joints in bedrock that may require stabilization, and slope gradients of 2:1, or flatter. Natural slopes appear to be performing adequately.

All slopes should be designed and constructed in accordance with the minimum requirements of the County, the 2013 CBC (CBSC, 2013), the current "Greenbook," and the recommendations in Appendix H. Due to the predominantly granular nature of site soils, slopes are anticipated to have erosion and surficial instability issues if left unplanted, and without engineered surface drainage control, and as such, will require periodic and regular maintenance above and beyond what is normally performed for slopes in general.

Cut Slopes

Cut slopes are generally considered to be grossly stable. However, the dense nature of cut slopes constructed in granitic bedrock may present difficulties with respect to landscaping and planting. In order to enhance the plantability of these slopes, consideration may be given to reconstructing cut slopes as stability fill slopes, if desired. General stabilization fill slope design and construction is presented in Appendix H.

Planned Fill Slopes

Planned fill slopes are generally considered to be grossly stable to the anticipated heights and gradients shown on the plans. Fill slopes should performed adequately assuming that the slope are properly constructed, and maintained, under conditions of normal rain fall and climate.

Subdrains

The need for subdrainge within perimeter fill slope keyways will be evaluated during grading. Subdrains will be recommended at the base of any canyon fill. Subdrains will also be recommended within stabilization fill keyways, if constructed. If encountered, local seepage along the contact between the bedrock and overburden materials, or along jointing patterns of the bedrock may require a subdrain system. Typical subdrain design and construction details are presented in Appendix H.

Toe Drains

In order to mitigate perched water conditions associated with permeability contrast between fill and bedrock, and due to the potential for significant storm water runoff rom cut slopes, cut slopes in granitic bedrock should be provided with a toe of slope subdrain, or “toe drain” as discussed in the “Development Criteria” section of this report. Toe drains may be warranted at other locations as well.

Temporary Slopes

Temporary slopes completed in non-saturated, medium dense to dense, granular soils 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 is not encountered. Construction materials or soil stockpiles should not be placed within ‘H’ of any temporary slope where ‘H’ equals the height of the temporary slope.

For saturated soils encountered near the groundwater table, temporary slopes should conform to CAL-OSHA and/or OSHA requirements for Type “C” soils. Local dewatering may also be required.

All temporary slopes should be observed by a licensed engineering geologist and/or geotechnical engineer prior to worker entry into the excavation. Based on the exposed field conditions, inclining temporary slopes to flatter gradients or the use of shoring may be necessary if adverse conditions are observed.

Embankment Factors

Excavation into onsite earth materials, such as existing fills, colluvium, alluvium, and older alluvium, will generally generate mixtures of silty sand, sand and gravelly sand, with minor amounts of clayey sand, and produce good to fair quality fill material. Excavations within the underlying granitic bedrock will generally produce good quality material near the surface, with poor quality fill material consisting of angular gravel to cobble to boulder size rock fragments becoming more abundant with depth of excavation.

Embankment factors (shrinkage/bulking) for the site have been estimated based upon our experience with other sites in the general vicinity, as well as data obtained from ongoing site exploration. It is apparent that shrinkage would vary with depth and with areal extent over the site. The refraction data indicates variability between depths of about 4 feet to 20 feet below existing grades (b.e.g.), in addition to other variables, including vegetation, weed control, discing, and previous filling or exporting, etc. All these factors are difficult to define in a three-dimensional fashion, and the contractors compactive efforts may also contribute some variance. Therefore, the information presented below represents average shrinkage and bulking values, using the following assumptions.

Colluvium	10-15% Shrinkage
Alluvium	10-15% Shrinkage
Older Alluvium	2-5% Shrinkage
Existing Fill	2-5% Shrinkage
Bedrock (from Church, 1981)	
25% Rock/75% Earth (about 2½ to 4 feet b.e.g.)	8% Shrinkage
50% Rock/50% Earth (about 4 to 18 feet b.e.g.)	5% Shrinkage
75% Rock/25% Earth (about 18 to 30 feet b.e.g.)	12% Bulk
100% Rock (> ±30 feet b.e.g.)	12-33% Bulk

Please note that the depths assigned to the various bedrock zones are measured below existing grades (b.e.g.). We emphasize that the seismic refraction data obtained does not indicate the actual depth to 100% rock, but infers that it exists below a depth that ranges from about 10 to 38 feet (b.e.g.). Prior to grading and finalization of grading plans, additional rock hardness evaluation with an air track rig should be considered in this regard. Subsidence in bedrock areas should be nil. Subsidence in alluvial and older alluvial areas may be on the order of 0.1 feet and 0.05 feet, respectively. Some variation should be anticipated due to equipment haul rates.

Rock Crushing and/or Placement Guidelines

Crushing/Rock Disposal

GSI anticipates that some of the onsite soils to be utilized as fill material for the subject project may contain some rock, especially during grading operations in the vicinity of Planning areas PA-1, PA-2, PA-4, and the upper elevations of PA-5. Appropriately, the need for rock crushing and/or disposal may be necessary during grading operations on the site. The option for crushing rocks or oversize disposal should be value engineered. 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 rock 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 in occurring in cut or natural areas, would need to be disclosed to all interested/affected parties. Once approved by the governing agency, the hold-down depth for oversized rock (i.e., greater than 12 inches) in fills on this project is provided as 10 feet. The re-use of oversized materials around pools (next to or below) is not recommended.

General

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

Materials 8 Inches in Diameter or Less

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

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

2. The purpose for the 8-inch diameter limit is to allow reasonable sized rock fragments into the fill under selected conditions (optimum moisture or above) surrounded with compacted fines. The 8-inch diameter size also allows a greater volume of the rock fragments to be handled during grading, while staying in reasonable limits for later onsite excavation equipment (backhoes and trenchers) to excavate footings and utility line trenches. Please note that most utilities limit soil particles to 2 to 4 inches within trench backfill.

3. Fill materials 8 inches or less in diameter should be placed (but not limited to) within the hold-down depth on proposed fill pads, the upper 5 feet of overexcavated cut areas of cut/fill transition pads, and the entire street right-of-way width, including the proposed overexcavated areas and replacement fill areas, from the depth of the lowest utility (within the street and lot), to subgrade, or to the hold-down depth below finish grade. Overexcavation is discussed later in this report.

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

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

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

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

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

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

6. Minor amounts of rock material in this size range may also be placed a rock windrows (see below).

Substructures Placed in the Hold-down Depth Zone

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

RECOMMENDATIONS - FOUNDATIONS

Typical foundation design for very low to low expansive soil conditions is anticipated where support is provided by engineered fill overlying older alluvium or bedrock. Building areas underlain with alluvial deposits and shallow groundwater will require relatively more onerous foundation design, in addition to mitigative earthwork such as, but not necessarily limited to fill surcharging, and/or other ground improvement.

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 structure are made, the conclusions and recommendations contained in this report are for the subject site only and 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 considered minimums and are not meant to supercede design(s) by the project structural engineer or civil engineer specializing in structural design. Upon request, GSI could provide additional consultation regarding soil parameters, as related to foundation design. They are considered preliminary recommendations for proposed construction, in consideration of our field investigation, laboratory testing, and engineering analysis. We anticipate that the wall loads of 1.5 to 3.0 kips/foot, and column loads of 5 to 50 kips will be utilized.

As previously indicated, foundation systems will be supported by engineered fill bearing on older alluvium and/or granitic bedrock, left in-place alluvium below the groundwater table, or left in place alluvium that has been improved by methods such as stone columns, grouting, deep mixing, etc. Based on the as-built conditions, including area geology, soil expansion, treatment of existing ground, and/or ground improvement, etc., GSI recommends foundation design in accordance with the following categories:

> **Category I** - Conventional slabs. Limited to very low to low expansive soil conditions. Best suited for settlement Group Areas 1 and 2 (Planning Areas PA-1, PA-2, PA-4, and PA-5), excluding deep fill areas.
>
> **Category II** - Post-tension [PT] slab foundations. May be used for all expansive soil conditions onsite, and may be used for settlement Group Areas 1, and 2, including deep fill areas. May be used for structures within settlement Group Area 3, dependant upon method or extent of ground improvement.
>
> **Category III** - Structural mat slabs and/or stiffened slabs per WRI (1981, 1986). May be used for all expansive soil conditions onsite. May be used for settlement Group Areas 1 and 2. May be used for Group 3 areas, dependant upon method or extent of ground improvement.

Ancillary structures (benches, light poles, utility boxes) may use either these types, or conventional spread footings for support.

Foundation Design Parameters

General

1. The foundation systems should be designed and constructed in accordance with guidelines presented in the 2013 CBC (CBSC, 2013). All foundations should be embedded entirely into newly compacted or mitigated fill (90 percent of ASTM D 1557).

2. An allowable bearing value of 2,000 pounds per square foot (psf) may be used for design of footings that maintain a minimum width of 12 inches and a minimum depth of 12 inches, and founded in compacted fill. This value may be increased by

20 percent for each additional 12 inches in depth to a maximum value of 2,500 psf. In addition, this value may be increased by one-third when considering short duration wind or seismic loads. Isolated pad footings should have a minimum dimension of at least 24 inches square and minimum depth of 24 inches, and be connected in two directions back to the main portion of the foundation. The depth of embedment shall not include the slab thickness nor underlayment, and shall be below the lowest adjacent grade.

3. Passive earth pressure may be computed as an equivalent fluid having a density of 250 pounds per cubic foot (pcf), with a maximum lateral earth pressure of 2,000 psf. Lateral passive pressures for shallow foundations within 2013 CBC setback zones should be reduced following a review by the geotechnical engineer unless proper setback can be established.

4. An allowable coefficient of friction between soil and concrete of 0.35 may be used with the dead load forces.

5. For the evaluation of total lateral resistance on the foundation and combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third. For effect of shrink-swell soils on hillside foundations, the geotechnical consultant should review foundation designs when available. The addition of creep loads on top-of-slope or mid-slope foundations should be considered.

6. Seismic design parameter are presented in a previous section of this report.

Settlement Summary

For preliminary design purposes, a summary of potential foundation settlement is presented in the following table.

SETTLEMENT SUMMARY ESTIMATES*			
SETTLEMENT GROUP AREA	**STATIC***	**SEISMIC**	**STATIC PLUS SEISMIC DIFFERENTIAL SETTLEMENT**
Group 1 - Fill over older alluvium (PA-2, PA-5)	1¼-inch total, ⅝-inch differential in 40 feet for fills up to 25 feet 1½-inch total, ¾-inch differential in 40 feet for fills up to 30 feet 2¼-inch total, 1⅛-inch differential in 40 feet for fills between 30 to 50 feet	Less than ¾-inch total, less than ⅜-inch differential in 40 feet for fills up to 25 feet ¾-inch total, ⅜-inch differential in 40 feet for fills up to 30 feet 1¼-inch total, ⅝ -inch differential in 40 feet for fills between 30 to 50 feet.	¾ inch in 40 feet for fills up to 25 feet thick. 1⅛ inches in 40 feet for fills between 25 to 30 feet thick. May be reduced to less than 1 inch in 40 feet when minimum relative compaction for fill is 95 percent. 1¾-inch differential in 40 feet for fills between 30 to 50 feet. May be reduced to 1 inch in 40 feet when minimum relative compaction for fill is 95 percent.
Group 2 - Fill over Granitic bedrock (PA-1, PA-2, PA-4, PA-5)	1¼-inch total, less than ¾-inch differential in 40 feet for fills up to 25 feet 1½-inch total, ¾-inch differential in 40 feet for fills up to 30 feet. 2¼-inch total, 1⅛-inch differential in 40 feet for fills between 30 to 50 feet.	Less than ¾-inch total, less than ⅜-inch differential in 40 feet for fills up to 25 feet ¾-inch total, ⅜-inch differential in 40 feet for fills up to 30 feet 1¼-inch total, ⅝ -inch differential in 40 feet for fills between 30 to 50 feet	¾ inch in 40 feet for fills up to 25 feet thick. 1⅛ inches in 40 feet for fills between 25 to 30 feet thick. May be reduced to less than 1 inch in 40 feet when minimum relative compaction for fill is 95 percent. 1¾-inch differential in 40 feet for fills between 30 to 50 feet. May be reduced to 1 inch in 40 feet when minimum relative compaction for fill is 95 percent.
Group 3, fill over alluvium and shallow groundwater table. (PA-3)	Angular distortions of greater than 1/480. With wait periods on the order of at least 180 days, angular distortions could be reduced to 1/480 with ground improvements.	Up to ±6 inch total, and up to 2 to 4 inches differential over 40 feet. Seismic settlement reduced with increased fill surcharge (i.e., fill placed above existing grade) and ground improvement.	Reduce to 2 inches in 40 feet (with ground improvement)

* Does not include foundation settlement due to applied footing loads.

It should also be kept in mind that drainage reversals could occur in areas underlain with alluvium left in place below the groundwater table (Group 3 areas), when considering post-construction static and seismic settlement, if relatively flat yard drainage gradients are not periodically maintained by the maintenance department, owners, and/or other interested/affected parties. Similarly, gravity flow utilities in areas underlain by alluvium are also subject to possible drainage reversals or deflections, considering the magnitude and angular distortions of settlement reported herein.

Foundation Category I (i.e., Very Low Expansive Soils, Settlement Group Areas 1 and 2)

Conventional Slabs

The following foundation construction recommendations are presented as a minimum criteria from a soils engineering viewpoint for very low expansive soils consisting of engineered fill over older alluvium, or granitic bedrock only. Recommendations by the project's design/structural engineer or architect, which may exceed the soils engineer's recommendations, should take precedence over the following minimum requirements. These are for conventional foundations of ancillary structures (other than buildings) that need not comply with criteria for foundations on expansive soils per Code.

1. Continuous footings should be founded at a minimum depth of 12 and 18 inches below the lowest adjacent ground surface bearing properly compacted fill, for one- or two-story floor loads, respectively. All footings should be reinforced with a minimum of two No. 4 reinforcing bars at the top and two No. 4 reinforcing bars at the bottom (four bars total). Reinforcement of Isolated footings should be provided by the structural engineer. The depth of embedment is measured from the lowest adjacent grade, and does not include slab underlayment or the landscape zone.

2. A grade beam, reinforced as above, and at least 12 inches square, should be provided across any large entrance (garage, etc.). The base of the reinforced grade beam should be at the same elevation as the adjoining footings.

3. Concrete slabs should be a minimum of 5 inches. Recommendations for floor slab construction and the mitigation of moisture vapor transmission are presented in a later section of this report.

4. Concrete slabs, including large building entrance areas, should be minimally reinforced with No. 3 reinforcement bars placed on 18-inch centers, in two horizontally perpendicular directions (i.e., long axis and short axis). 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.

5. The slab and footing subgrade should be free of loose and uncompacted material prior to placing concrete.

6. Soils generated from footing excavations to be used onsite should be compacted to a minimum relative compaction 90 percent of the laboratory standard (ASTM D 1557), whether it is 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.

7. Footings should maintain a horizontal distance, X, between any adjacent descending slope face and the bottom outer edge of the footing. The horizontal distance, X, may be calculated by using X = H/3, where "H" is the height of the slope. X should not be less than 7 feet, nor need not be greater than 40 feet. X may be maintained by deepening the footings. Setbacks should minimally conform to Section 1808.7.2, and 1808.7.3 of the 2013 CBC (CBSC, 2013) guidelines as applicable, unless specifically superceded herein.

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, including the above recommendations for conventional slabs.

For a typical slab designed with interior ribs, or stiffeners, the slab should minimally be at least 5 inches thick. The ribs should be provided in both transverse and longitudinal directions. The interior rib spacing and depth should be provided by the project structural engineer. The perimeter beams, however, should be embedded as specified in the post-tension slabs section of this report, 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. Please note that stiffener beams will tend to make water vapor retarder installation more complex.

Foundation Category II - Post-tension Slab Foundations, Settlement Group Areas 1 and 2. Group 3 with Ground Improvement

Post-tension (PT) slab foundation may also be used to support the structure. 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). 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).

TABLE - POST-TENSION FOUNDATION DESIGN(3)	
DESIGN PARAMETER(3)	**VERY LOW TO LOW EXPANSION POTENTIAL**
e_m center lift	9.0 feet
e_m edge lift	5.2 feet
y_m center lift	0.3 inches
y_m edge lift	0.7 inch
Bearing Value (1)	1,000 psf
Lateral Pressure	250 psf
Subgrade Modulus (k)	100 pci/inch
Minimum Perimeter Footing Embedment (2)	12 inches

(1) Internal bearing values within the perimeter of the post-tension slab may be increased to 1,500 psf for a minimum embedment of 18 inches, then by 20 percent for each additional foot of embedment to a maximum of 2,000 psf.

(2) As measured below the lowest adjacent compacted subgrade surface.

(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 maintenance staff, owners, affected/interested parties. The values tabulated above may not be appropriate to account for possible differential settlement of the slab due to other factors, such as excessive settlements. If a stiffer slab is desired, alternative Post-Tensioning Institute ([PTI] third edition) parameters may be recommended.

<u>Foundation Category III - Structural Mat Foundations, Settlement Group Areas 1 and 2. Group 3 with Ground Improvement</u>

As previously, indicated soils within the influence of the proposed structures are generally considered to be very low to possibly low expansive. However, settlement potentials due to the presence of left in place alluvium in settlement area 3 (Planning Area PA-3) generally exceed the tolerance of a typical slab on grade foundation system. As such, a mat slab foundation may be considered in these areas.

A structural mat-type foundation slabs should be a minimum of 10 inches in thickness, and in accordance with the structural engineer, and also be reinforced with a double mat of rebars at the spacing recommended by the structural engineer. Footings should be embedded as indicated herein, below the lowest adjacent grade into properly compacted fill, unless expansive soil conditions dictate deeper embedments as discussed in a following section. The need and arrangement of grade beams will be in accordance with the structural consultant's recommendations. Alternative uniform thickness mat slabs may be used in the design if the structural consultant can demonstrate that the alternative is equivalent to the recommended mat slab/footing. All mat-type designs should resist differential settlement and expansive soil conditions as explained herein.

Recommended design parameters used in the design of WRI foundations (WRI, 1996) and slabs-on-grade are provided in the following table.

WRI DESIGN PARAMETERS	
Effective Plasticity Index*	20
Unconfined Compressive Strength*	1,000 psf (0.5 tsf)
Modulus of Subgrade Reaction	100 pci
Settlement Potential	see Text
Resistance Value (R-value)*	38
Minimum Slab Thickness	6 inches
Minimum Steel Reinforcement per Structural Engineer	Double Mat of Steel Reinforcement Bars per Structural Consultant

* To be re-evaluated upon completion of grading.

For this method, either a uniform thickness foundation (UTF) or mat may be used. Alternatively, the slab (in plan view) may be divided up into at least quarters and grade beams should be used to enhance the strength of the slab to resist the expansive soil forces. The foundation bearing capacity and other geotechnical parameters previously provided in this report are still applicable.

Perimeter cut-off walls may be incorporated into the UTF design and should be 18 inches deep for the medium to highly expansive soil conditions evaluated onsite. The cut-off walls may be integrated into the slab design or independent of the slab. The cut-off walls should be a minimum of 6 inches thick. The bottom of the perimeter cut-off wall should be designed to resist tension, using reinforcement per the structural engineer.

Slab Subgrade Pre-Soaking

Pre-moistening of the slab subgrade soil is recommended for these soil conditions. The moisture content of the subgrade soils should be equal to or greater than optimum moisture to a depth equivalent to the exterior footing depth in the slab areas (typically 12 inches for very low to low expansive soils). Pre-moistening and/or pre-soaking should be evaluated by the soils engineer 72 hours prior to vapor retarder placement. In summary:

EXPANSION INDEX	PAD SOIL MOISTURE	CONSTRUCTION METHOD	SOIL MOISTURE RETENTION
Very Low (0-20)	Upper 12 inches of pad at or above soil optimum moisture	Wetting and/or reprocessing	Periodically wet or cover with plastic after trenching. Evaluation 72 hours prior to placement of concrete.
Low (21-50)	Upper 12 inches of pad soil moisture 2 percent over optimum	Wetting and/or reprocessing	Periodically wet or cover with plastic after trenching. Evaluation 72 hours prior to placement of concrete.

SOIL MOISTURE CONSIDERATIONS

GSI has evaluated the potential for vapor or water transmission through the slabs, in light of typical floor coverings and improvements. Generally, slab moisture emission rates range from about 2 to 27 lbs./1,000 square feet from a typical slab (Kanare, 2005), while most floor covering manufacturers recommend about 3 lbs./24 hours as an upper limit. Thus, the client will need to evaluate the following in light of a cost versus benefit analysis (tenant complaints and repairs/replacement), along with disclosure to owners.

Considering the proximity of groundwater, potential for perched groundwater to occur, E.I. test results, anticipated typical water vapor transmission rates, and floor coverings and improvements (to be chosen by the client) that can tolerate those rates without distress, the following alternatives are provided:

- Concrete slabs should be thicker than the minimum specified herein.

- Concrete slab underlayment should consist of a 15-mil vapor retarder, or equivalent, with all laps sealed per the 2013 CBC (CBSC, 2013) 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 D 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.).

- The vapor retarder should be underlain with 2 inches of washed sand, and should be overlain by a 2-inch thick layer of washed sand (SE>30).

- Concrete should have a maximum water/cement ratio of 0.50. This does not supercede the 2013 CBC (CBSC, 2013) 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 above, 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.

- Owner(s) should be specifically advised which areas are suitable for tile flooring, wood 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, and areas potentially using moisture sensitive floor coverings and/or moisture sensitive storage, should be identified 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.

Corrosion and Concrete Mix

Upon completion of grading, laboratory testing should be performed of site materials for corrosion to concrete and corrosion to steel. Soils with negligible to moderate levels of sulfate content are present near the surface. As such, the use of Type V concrete is not required per 2013 CBC, as well as ACI 318-11, on a preliminary basis. Additional comments may be obtained from a qualified corrosion engineer.

WALL DESIGN PARAMETERS

Conventional Retaining Walls

The design parameters provided below assume that either very low expansive soils (typically Class 2 permeable filter material or Class 3 aggregate base) or native onsite materials with an expansion index up to 20 are used to backfill any retaining wall. The type of backfill (i.e., select or native), should be specified by the wall designer, and clearly shown on the plans. Building walls, below grade, should be water-proofed. Waterproofing may also be provided for site retaining walls in order to reduce the potential for efflorescence staining. Recommendations for specialty walls, or mechanically stabilized earth (MSE) walls (i.e., crib, earthstone, geogrid, etc.) can be provided on request.

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 any topsoil/colluvium, or landscape layer [upper 6 inches]), into suitable bedrock.

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 older alluvium/bedrock or engineered fill (no transitions). 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 120 pcf and 125 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 earth material 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 (level backfill). The design should include any applicable surcharge loading. For areas of male or re-entrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall (2H) laterally from the corner.

Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls up to 10 feet high. Design parameters for 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. (SELECT NATIVE BACKFILL)(3)
Level(1)	38	50
2 to 1	55	65

(1) Level backfill behind a retaining wall is defined as compacted earth materials, properly drained, without a slope for a distance of 2H behind the wall, where H is the height of the wall.
(2) SE $\geq$ 30, P.I. < 15, E.I. < 21, and $\leq$ 10% passing No. 200 sieve.
(3) E.I. = 0 to 30, SE $\geq$ 20, P.I. < 20, and $\leq$ 20% passing No. 200 sieve; confirmation testing required.

Earthquake Loads (Seismic Surcharge)

For engineered retaining walls, GSI recommends that the walls be evaluated for a seismic surcharge (in general accordance with 2013 CBC requirements). The site walls in this category should maintain an overturning Factor-of-Safety (FOS) of approximately 1.25 when the seismic surcharge (increment), is applied. For restrained walls, the seismic surcharge should be applied as a uniform surcharge load from the bottom of the footing (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 20H 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 20H. Reference for the seismic surcharge is Section 1802.2 of the 2013 CBC. Please note this is for local wall stability only.

The 20H 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° - ϕ/2 plane away from the back of the wall. The 20H seismic surcharge is derived from the formula:

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

Where:
P_h = Seismic increment
a_h = Probabilistic horizontal site acceleration with a percentage of "g"
γ_t = Total unit weight (125 to 130 pcf for site soils at 90 percent relative compaction).
H = Height of the wall from the bottom of the footing or point of pile fixity.

Retaining Wall Backfill and Drainage

Positive drainage must be provided behind all retaining walls in the form of gravel wrapped in geofabric and outlets. A backdrain system is considered necessary for retaining walls that are 2 feet or greater in height. Details 1, 2, and 3, present the backdrainage options discussed below. Backdrains should consist of a 4-inch diameter perforated PVC or ABS pipe encased in either Class 2 permeable filter material or ¾-inch to 1½-inch gravel wrapped in approved filter fabric (Mirafi 140 or equivalent). For low expansive backfill, the filter material should extend a minimum of 1 horizontal foot behind the base of the walls and upward at least 1 foot. This material should be continuous (i.e., full height) behind the wall, and it should be constructed in accordance with the enclosed Detail 1 (Typical Retaining Wall Backfill and Drainage Detail). For limited access and confined areas, (panel) drainage behind the wall may be constructed in accordance with Detail 2 (Retaining Wall Backfill and Subdrain Detail Geotextile Drain). Any materials (if encountered) with an expansion index (E.I.) potential of greater than 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).

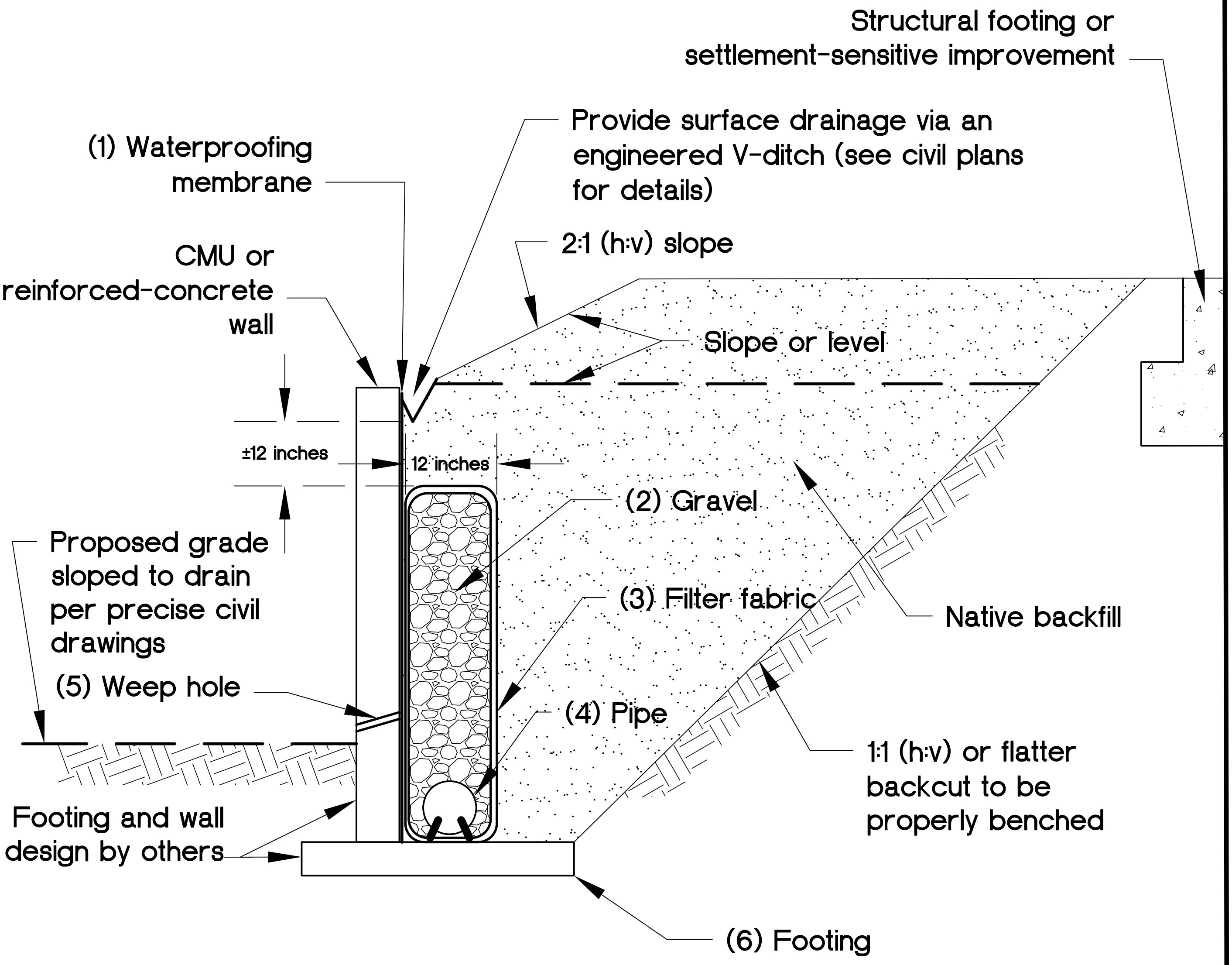
Outlets should consist of a 4-inch diameter solid PVC or ABS pipe spaced no greater than ±100 feet apart, with a minimum of two outlets, one on each end. The use of weep holes, only, in walls higher than 2 feet, is not recommended. The surface of the backfill should be sealed by pavement or the top 18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

Wall/Retaining Wall Footing Transitions

Site walls are anticipated to be founded on footings designed in accordance with the recommendations in this report. Should wall footings transition from 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 an angular distortion of 1/360 for a distance of 2H on either side of the transition may be accommodated. Expansion joints should be placed no greater than 20 feet on-center, in accordance with the structural engineer's/wall designer's recommendations, regardless of whether or not transition conditions exist. Expansion joints should be sealed with a flexible, non-shrink grout.



(1) Waterproofing membrane.

(2) Gravel: Clean, crushed, $\frac{3}{4}$ to $1\frac{1}{2}$ inch.

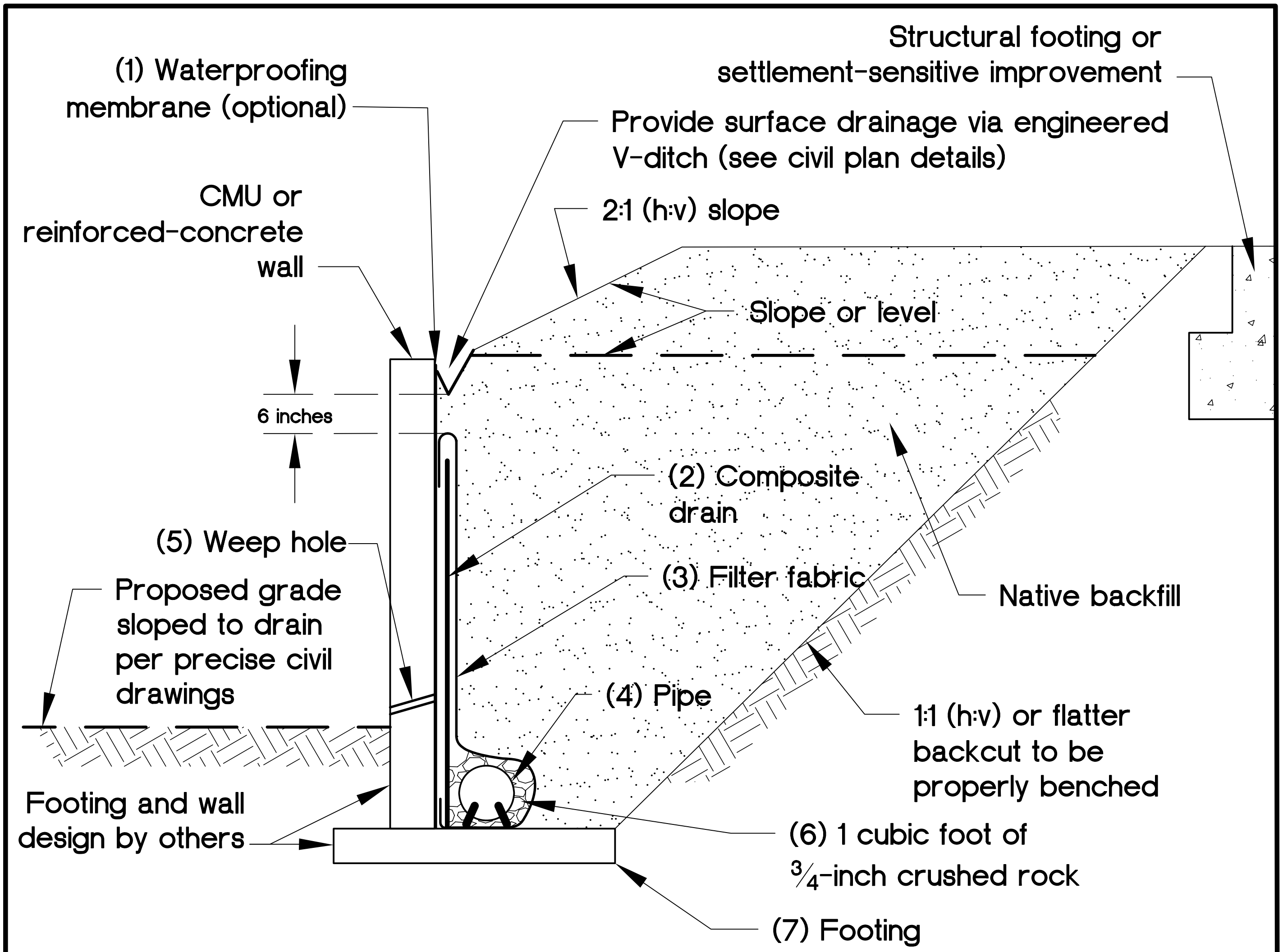
(3) Filter fabric: Mirafi 140N or approved equivalent.

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

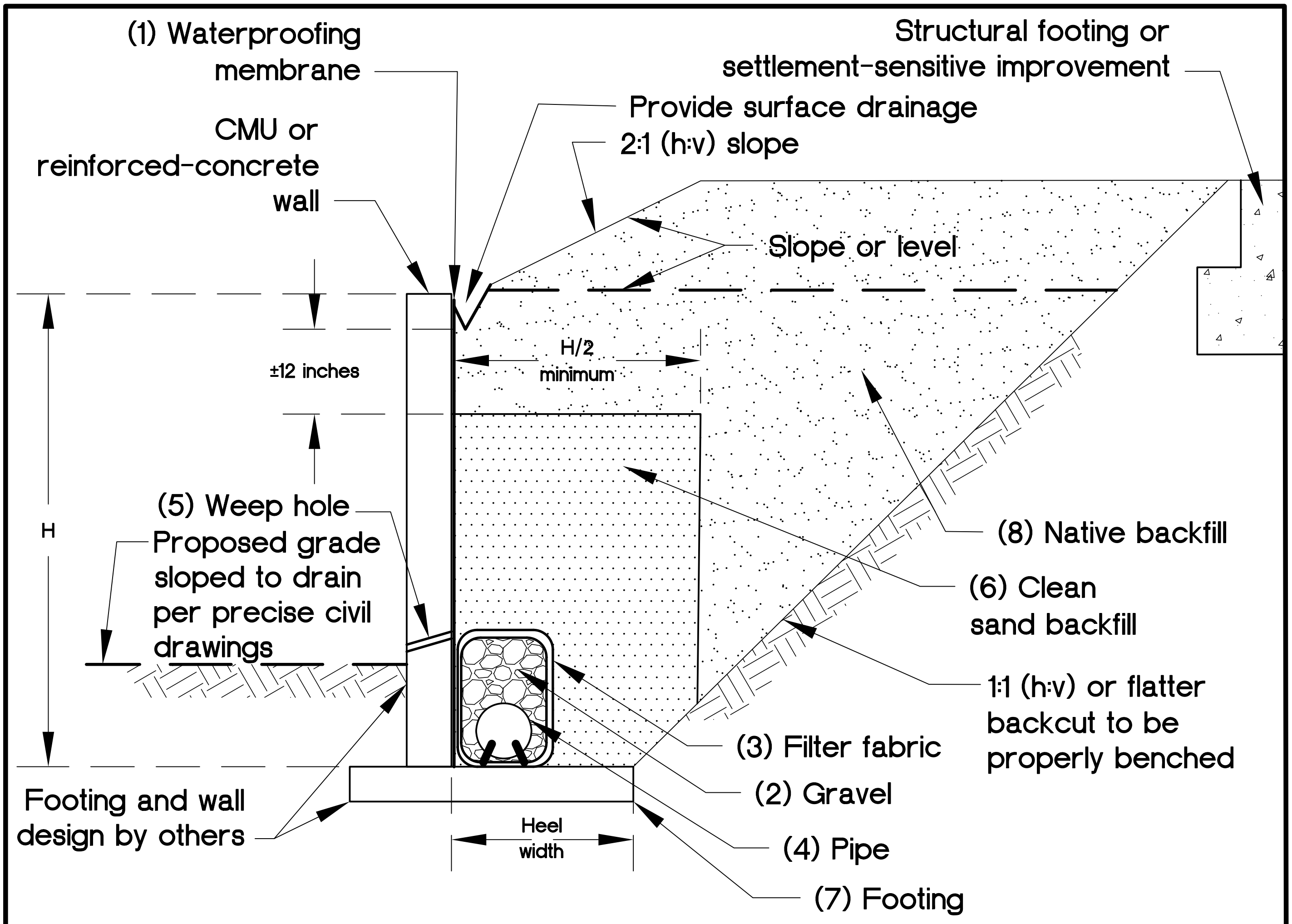
(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

(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, $\frac{3}{4}$ to $1\frac{1}{2}$ inch.

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

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

(5) Weep hole: Minimum 2-inch diameter placed at 20-foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

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

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

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

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

c) Embed the footings entirely into native formational material (i.e., deepened footings).

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

PRELIMINARY MECHANICALLY STABILIZED RETAINING WALL RECOMMENDATIONS

General

Based on the granular nature of site soils, Mechanically Stabilized Earth, or MSE retaining walls, may be considered. The MSE retaining wall design parameters, provided herein, assume that either non-expansive soils (typically Class 2 permeable filter material or Class 3 aggregate base) or select soil import materials (up to and including an E.I. of < 20, a P.I. $\leq$15, and $\leq$20 percent passing the No. 200 sieve, and 100 percent 3-inch minus material), or suitable native soils meeting select criteria, are used to backfill any MSE retaining walls within the “reinforced zone.”

Onsite Soil Suitability

Onsite earth materials primarily consist of localized undocumented fill, topsoil, colluvium, alluvium, older alluvium, and granitic bedrock. These materials appear to predominantly consist of silty sands, and sands, with variable amounts of rock fragments and are generally considered non-detrimentally expansive (i.e., expansion index [E.I.] less than 20 and a plasticity index [P.I.] less than 15). Based on a review of laboratory testing completed on selected samples, granular soils generated from planned excavations may generate potentially suitable materials. However, due to the variability of earth materials throughout the site, the related soil parameters should be anticipated as non-uniform. Wall backfill derived from surficial soil deposits (topsoil, colluvium, alluvium, etc.) will have a greater potential for increased fines and lower strengths, locally, and the selected materials should be adequately evaluated for suitability prior to use. Soils generated from excavation into granitic bedrock will generate variable amounts of rock fragments, with the overall percentage and average size of the fragments increasing with the depth of excavation. Suitable soils (generated from onsite bedrock) for use in wall construction should consist of 3-inch minus material, with enough granular fines to fill any voids. Select grading to stockpile suitable, granular soils generated during grading of the site may be performed, and is recommended.

GSI does not recommend the use of onsite soils in the MSE retaining wall construction until testing is performed and the materials are approved in writing by the geotechnical consultant as well as the MSE retaining wall designer, prior to construction and/or use.

This will need to be considered during project planning, design, and construction. If not considered as described above, it may result in wall re-design during or after site grading.

Guidelines for MSE Retaining Wall Design/Construction

General

MSE retaining walls are, by nature, a flexible system and, as such, not suited for every slope support condition. This will need to be considered and ultimately determined by the project design civil engineer and client.

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

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

Design

As part of MSEW (Mechanically Stabilized Earth Wall design, MSE retaining wall design may be generally subdivided into three components: 1.) Foundation Zone, 2.) Retained Zone, and 3.) Reinforced Zone (i.e., the zone containing the grid reinforcement layers).

- On a preliminary basis, a cohesion of c=0, a phi angle of 30 degrees, and a unit weight of 131 pcf, may be used in the design of the “foundation” zone.
- On a preliminary basis, a cohesion of c=0, a phi angle of 30 degrees, and a unit weight of 131 pcf, may be used in the design of the “retained” zone.
- On a preliminary basis, a cohesion of c=0, a phi angle of 30 degrees, and a unit weight of 131 pcf, may be used in the design of the “reinforced” zone.
- A site acceleration (PGA_M) of 0.46g (site class “D”) was evaluated using USGS (2014) methodology and may be used for the evaluation of seismic stability.

- The recommended equivalent fluid pressure for design of the MSE retaining walls should be 45 pcf for level backfill and 65 pcf for 2:1 backfill, assuming the use of granular backfill material (E.I. $\leq$20, P.I. $\leq$ 15, ϕ $\geq$30 degrees, c = 0 psf, and $\leq$20 percent passing the No. 200 sieve). These equivalent fluid pressures are based solely on static soil conditions and do not include seismic loading, expansive soil pressures, earthwork surcharge, or traffic loading which will need to be included, as necessary.

- An evaluation of global stability (i.e., static and seismic) is typically not performed by the wall designer. Once preliminary plans/calculations are completed, the geotechnical consultant should review global stability with respect to a factor of safety of 1.5 (static) and 1.1 (seismic).

Foundation Construction

1. Prior to excavation for the wall base, the alignment and grade for the wall should be established in the field by the project civil engineer or project surveyor.

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

3. Defective segments or wall units should not be utilized.

4. The project surveyor should spot-check wall gradient (face-of-wall slope) and alignment and using this data, the civil design/wall designer should evaluate if the wall installation is per plan.

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

6. Walls should be founded on engineered fill approved by this office, or dense, suitable bedrock. GSI recommends that the MSE wall footings be embedded at least 1 foot into suitable bearing material for adequate lateral support.

7. Prior to placement of the MSE wall units, the excavation for the wall base should be observed by representatives of this firm, and should be a minimum of 12 inches into approved engineered fill or bedrock. However, deeper excavation may be necessary due to the depth of removals , and or setbacks from the face of adjacent, descending slopes.

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

9. If it is necessary to locally deepen the wall base to obtain suitable bearing materials, the contractor should consult the project design engineer to determine if the wall location or design of the wall is affected.

10. MSE retaining wall height at the terminal ends of the wall should not exceed 4 feet, unless lateral support is provided.

Backfill

1. Fill placed within the geotextile reinforcement zone, and in front of the MSE retaining walls, should be compacted to a minimum of 90 percent relative compaction unless otherwise specified by the manufacturer. Any backfill other than the "unit core fill (¾ inch crushed rock or stone)" should be placed in controlled lifts not to exceed 6 inches in thickness, and moisture-conditioned as necessary to achieve at least optimum moisture content. Backfill within and immediately behind the walls should also be as indicated on the (precise and rough) grading plans.

2. Backfill materials should be free draining, and free from organic materials, with an E.I. less than 20, a P.I. less than 15, and a maximum of 20 percent fines passing the No. 200 sieve. Lifts should be placed horizontally and compaction equipment should not be allowed to damage the geotextile reinforcement, where utilized.

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

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

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

Wall Back Drains

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

Materials and Wall Construction

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

Structural Setbacks from Proposed MSE Retaining Walls

Slope and structural setbacks from the heel of walls and/or geogrid will be necessary, owing to potential deflection/movement. The necessary setbacks should be defined by the various project consultants and approved by the governing agencies prior to final design. At a minimum, the building setback should be up at a 1:1 (h:v) projection from the heel of the MSE wall foundation or the reinforced (grid) zone whichever is greater, and should be shown on the precise grading plans by the design civil engineer. Building setback mitigation may be accomplished by deepening any adjoining foundations through this zone of 1:1 projection, provided this does not disturb any geogrid. Appurtenant structures, including pools, utilities, and landscaping, should not disrupt the geogrid behind the walls. All structures proposed within the setback zone will be subject to both horizontal and vertical deflections and potential distressed. All construction proposed within the setback area should be reviewed by the design civil engineer and geotechnical consultant. This review should be provided in writing to the Client prior to installation in the field. Homeowners and all interested parties should be notified of the setback zones.

Other Considerations

- Surcharge loads (slopes, traffic, etc.) should be applied by the design engineer as necessary.

- GSI recommends that geotextile reinforcement is not placed beneath appurtenant structures or improvements that require significant excavation that could damage the geotextile reinforcement. Appurtenant structures, should not disrupt the geotextile reinforcement behind the walls. Relatively deep underground utilities (i.e., greater than 2 to 3 feet in depth) should be located above a 1½:1 (h:v) projection down and away from the rear of the uppermost layer of geotextile reinforcement such that any future trenching for repairs would not damage the geotextile reinforcement. All structures proposed within the setback zones will be subject to both horizontal and vertical deflections and potential distress. All construction proposed within the setback area should be reviewed by the design civil engineer and geotechnical consultant. This review should be provided in writing to the Client, prior to installation in the field.

- The alternative use of paver stone flatwork should be considered where Portland Cement Concrete (PCC) concrete hardscape (walkways, patios, etc.) are planned above a 1:1 (h:v) projection up from the heel of MSE retaining walls reinforced zone. Paver stone flatwork is more capable of tolerating the ground deformations related to MSE retaining walls. Structures/improvements that are settlement sensitive should not be placed in the setback zone.

- Wall drainage should be reviewed by this office as plans become available. The gravel pad provided for the support of the base course should be adequately drained.

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

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

Review of MSE Retaining Wall Plans and Structural Calculations

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

Additional Testing

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

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

Expansive Soils and Slope Creep

Soils at the site are likely to be expansive (i.e., E.I. > 0) and therefore, become desiccated when allowed to dry. Such soils are susceptible to surficial slope creep, especially with seasonal changes in moisture content. Typically in southern California, during the hot and dry summer period, these soils become desiccated and shrink, thereby developing surface cracks. The extent and depth of these shrinkage cracks depend on many factors such as the nature and expansivity of the soils, temperature and humidity, and extraction of moisture from surface soils by plants and roots. When seasonal rains occur, water percolates into the cracks and fissures, causing slope surfaces to expand, with a corresponding loss in soil density and shear strength near the slope surface. With the passage of time and several moisture cycles, the outer 3 to 5 feet of slope materials experience a very slow, but progressive, outward and downward movement, known as slope creep. For slope heights greater than 10 feet, this creep related soil movement will typically impact all rear yard flatwork and other secondary improvements that are located within about 15 feet from the top of slopes, such as concrete flatwork, etc., and in particular top of slope fences/walls. This influence is normally in the form of detrimental settlement, and tilting of the proposed improvements. The dessication/swelling and creep discussed above continues over the life of the improvements, and generally becomes progressively worse. Accordingly, the developer should provide this information to all interested/affected parties.

In addition, surficial slope failures occurring along the slope descending from the subject site have the potential to affect improvements (walls, flatwork, etc.) constructed within about 5 feet from the top of this slope. To that end, improvements located within this zone should be supported by CIDH piles (caissons).

Top of Slope Walls/Fences

Due to the potential presence of loose/soft bearing soils along property lines, some settlement and tilting of the walls/fence with the corresponding distresses, should be expected. Furthermore, due to the potential for slope creep for slopes higher than about 10 feet, some settlement and tilting of the walls/fence with corresponding distress, should be expected. To mitigate the tilting of top of slope walls/fences, we recommend that the walls/fences be constructed on a combination of grade beam and caisson foundations. The grade beam should be at a minimum of 12 inches by 12 inches in cross section, supported by drilled caissons, 12 inches minimum in diameter, placed at a maximum spacing of 6 feet on center, and with a minimum embedment length of 7 feet below the bottom of the grade beam. The strength of the concrete and grout should be evaluated by the structural engineer of record. The proper ASTM tests for the concrete and mortar should be provided along with the slump quantities. The concrete used should be appropriate to mitigate severe sulfate exposure. The design of the grade beam and caissons should be in accordance with the recommendations of the project structural engineer, and include the utilization of the following geotechnical parameters:

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

Creep Load: The creep load projected on the area of the grade beam should be taken as an equivalent fluid approach, having a density of 60 pcf. For the caisson, embedded into low to highly expansive soil, it should be taken as a uniform 900 pounds per linear foot of caisson's depth, located above the creep zone.

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

Passive Resistance: Passive earth pressure of 300 psf per foot of depth per foot of caisson diameter, to a maximum value of 4,000 psf may be used to determine caisson depth and spacing, provided that they meet or exceed the minimum requirements stated above. To determine the total lateral resistance, the contribution of the creep prone zone above the point of fixity, to passive resistance, should be disregarded.

Allowable Axial Capacity:

Shaft capacity:	350 psf applied below the point of fixity (in formational soil) over the surface area of the shaft.
Tip capacity:	4,000 psf (clear of loose soil, bearing into dense formational soil).

CONCRETE FLATWORK AND OTHER IMPROVEMENTS

The soil materials on site are expansive (i.e., E.I. > 0). 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 recommended that the developer should notify any interested/affected parties of this long-term potential for distress. To reduce the likelihood of distress, the following recommendations are presented for all exterior flatwork:

1. Concrete slabs should be founded entirely on properly compacted fill. The subgrade area for concrete slabs should be compacted to achieve a minimum 90 percent relative compaction, and then be presoaked to 2 to 3 percentage points above (or 130 percent of) the soils' optimum moisture content, to a depth of 18 inches below subgrade elevation. Refer to slab subgrade pre-soaking recommendation a previous section of this report. This moisture content should be maintained in the subgrade soils during concrete placement to promote uniform curing of the concrete and minimize the development of unsightly shrinkage cracks. If very low expansive soils are present, only optimum moisture content, or greater, is required and specific presoaking is not warranted. The moisture content of the subgrade should be proof tested within 72 hours prior to pouring concrete.

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

3. Exterior slabs supporting pedestrian traffic only should be a minimum of 4 inches thick.

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

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

   In order to reduce the potential for unsightly cracks, slabs should be reinforced at mid-height with a minimum of No. 3 bars placed at 18 inches on center, in each direction. The exterior slabs should be scored or saw cut, ½ to ⅜ inches deep, often enough so that no section is greater than 10 feet by 10 feet. For sidewalks or narrow slabs, control joints should be provided at intervals of every 6 feet. The slabs should be separated from the foundations and sidewalks with expansion joint filler material.

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

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

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

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

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

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

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

13. Positive site drainage should be maintained at all times. Finish grades should be provided with a minimum of 1 to 2 percent fall to the street, or other approved area, as indicated herein. It should be kept in mind that drainage reversals could occur, including post-construction settlement, if relatively flat yard drainage gradients are not periodically maintained by the maintenance department, school, owners, and/or other interested/affected parties.

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.

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

PRELIMINARY PAVEMENT DESIGN/CONSTRUCTION

Structural Section

Traffic Indices (T.I.) were assumed to range from 4.5 to 6.0 for the subject traffic areas, and should be reviewed by the project civil engineer for comment, and any revisions, as necessary. An R-value of 74 was evaluated for some site soils, but an R-value of 38 was assumed for preliminary planning purposes to account for some variability. It should be noted that even with the down graded R-value, pavement sections will likely default to County minimums for a given pavement. The recommended preliminary pavement sections for both asphaltic concrete (A.C.) pavement over aggregate base (A.B.), and Portland concrete cement pavement (PCCP), are provided in the following tables:

APPROXIMATE TRAFFIC AREA	TRAFFIC INDEX(1)	SUBGRADE R-VALUE(2)	A.C. THICKNESS (INCHES)	A.B. THICKNESS(3) (INCHES)
Cul du Sac	4.5	38	3.0	4.0
Residential	5.0	38	3.0	4.0

APPROXIMATE TRAFFIC AREA	TRAFFIC INDEX(1)	SUBGRADE R-VALUE(2)	A.C. THICKNESS (INCHES)	A.B. THICKNESS(3) (INCHES)
Residential Collector	5.0	38	3.0	5.0
Light Collector	6.5	38	3.0	8.0

(1) The T.I. is an estimation based on the intended use. The T.I. should be review for comment by the project civil engineer. Trash disposal areas, entry areas, fire vehicle access may require special design detailing.
(2) Estimate, to be verified by the project civil engineer.
(3) Denotes Class 2 Aggregate Base R $\geq$78, SE $\geq$25)
(4) Designs should follow County guidelines for PCCP aprons in front of trash enclosures.

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 4-inch thick leveling course of compacted aggregate base, or crushed rock may be considered to improve performance. All PCCP should be properly detailed (jointing, etc.) per the industry standard. Pavements may be additionally reinforced with #4 reinforcing bars, placed 12 inches on center, each way, for improved performance. Trash truck loading pads shall be per County standards, and reinforced accordingly.

All pavement installation, including preparation and compaction of subgrade, compaction of base material, and placement and rolling of asphaltic concrete, etc., shall be done in accordance with the County guidelines, and under the observation and testing of the project geotechnical engineer and/or the County.

The recommended pavement sections are meant as minimums. If thinner or highly variable pavement sections are constructed, increased maintenance and repair may be needed. The recommended pavement sections provided above are 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 TI 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.

Pervious Pavements

Manufacturer's guidelines for paver installation should be strictly adhered to. GSI should review such guidelines for comment, prior to construction. Pervious asphaltic concrete (A.C.) or Portland Cement Concrete (PCC) pavements should be reviewed for location and anticipated vehicle loading. Use of the AC or PCC pavement sections for said porous pavements should not use the sections herein without additional review and analysis by GSI.

Aggregate Base Rock

Compaction tests are required for the recommended base section. Minimum relative compaction required will be 95 percent of the laboratory maximum density as evaluated by ASTM Test Designation D 1557. Base aggregate should be in accordance to the Caltrans Class 2 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 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 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 base course and paving and the time between completion of base and paving is reduced to three days, provided the base is free of loose soil or debris. Where prime coat has been omitted and rain occurs, traffic is routed over base course, or paving is delayed, measures shall be taken to restore base course, and subgrade to conditions that will meet specifications as directed by the County and/or geotechnical consultant.

STORM WATER TREATMENT BMPs AND HYDROMODIFICATION MANAGEMENT

USDA Study

A review of the United States Department of Agriculture database (USDA; 1973, 2015) indicates a broad range of infiltration rates, between 0.00 inches per hour, to 19.98 inches per hour for all soil types across the site. Based on the USDA data, the following table provides a summary of representative infiltration rates associated with the three main geologic units onsite.

GEOLOGIC UNIT	APPROXIMATE RANGE INFILTRATION RATES (INCHES PER HOUR)	HYDROLOGIC SOIL GROUP (HSG)	COMMENTS
Alluvium	1.98 to 19.98	A, B, D	HSG group D due to potentially shallow groundwater locally
Older Alluvium	0.00 to 0.60	C, D	Contains relatively high clay content within surficial weathered zones
Granitic Bedrock	0.02 to 5.95	C, D	HSG Group D due to shallow depths to rock

It should be noted that the USDA data generally characterizes surficial soil conditions. During the grading/construction process, in areas proposed for improvements, these surficial soils would generally be removed and exported, or recompacted during mass grading, and as such, are not considered entirely representative of “as-built” site conditions, or parent material at greater depths.

Infiltration Feasibility

In accordance with the BMP Design Manual (County, 2016), the infiltration feasibility for this site was evaluated. 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), for the various soil types encountered onsite. Based on the testing performed, *K* values ranging on the order of 0.3 to 7.8 inches per hour were evaluated, and are summarized in the following Table with respect to the corresponding bio basins. Bio basin locations are shown on the preliminary grading plan, prepared by Project Design Consultants (PDC, 2016).

REPRESENTATIVE BIO BASIN PER PDC (2016)	INFILTRATION RATE (INCHES PER HOUR)	GEOLOGIC UNIT	COMPARISON WITH USDA DATA
PA-1, Lot B	1.0	Older Alluvium/ Weathered Rock	Slightly Higher than USDA Data
PA-1, Lot E	No Infiltration	planned fill	N/A
PA-2, Lot P	0.3	Granitic Rock	Low End of USDA Data

REPRESENTATIVE BIO BASIN PER PDC (2016)	INFILTRATION RATE (INCHES PER HOUR)	GEOLOGIC UNIT	COMPARISON WITH USDA DATA
PA-3, Lot Z	7.8	Alluvium	Similar to USDA Data
PA-3, Lot BB	6.0	Alluvium	Similar to USDA Data
PA-3, Lot EE	6.0	Alluvium	Similar to USDA Data
PA-4, Lot YY	0.40	Granitic Rock	Low End of USDA Data
PA-4, Lot 368	0.40	Granitic Rock	Low End of USDA Data
PA-4, Lot 373	0.40	Granitic Rock	Low End of USDA Data
PA-5, Lot 397	0.40	Granitic Rock	Low End of USDA Data
PA-5, Lot EEE, Collector ‘C’ Station 107	1.0	Older Alluvium	Slightly Higher than USDA Data
PA-5, Lot PPP	1.0	Older Alluvium	Slightly Higher than USDA Data
PA-5, Lot MMM	6.0	Alluvium	Similar to USDA Data
PA-5, Lot NNN	1.0	Older Alluvium	Slightly Higher than USDA Data
PA-5, Lot HHH, Collector ‘C’ Station 736	6.0	Alluvium	Similar to USDA Data
PA-5, Lot HHH, Collector ‘C’ Station 778	6.0	Alluvium	Similar to USDA Data

The values presented are generally both below, and above the recommended feasibility threshold of 0.52 inches per hour per the EPA (Clar, et al., 2004), and 0.50 inches per hour per the County (2016) for full infiltration. Differences noted between the USDA data, and this evaluation are likely due to testing being performed on soils generally deeper in the soil profile than characterized in the USDA study. For instance, older alluvium contains relatively more clay in the near surface, than at depth. As such, the zones evaluated result in slightly higher rates than USDA data. Conversely, testing in granitic areas indicates infiltration rates relatively lower than USDA data, as testing was not performed within the near surface soil horizon and is due to decreased permeability with depth within granite.

Based on our review and engineering analysis, areas suitable for either full, or partial infiltration occur onsite. However, it should be noted that the infiltration rates evaluated are for undisturbed, near surface native soils. Infiltration rates for compacted fills, and for native earth materials exposed within deeper cuts, will be substantially less. Compacted fills are considered as belonging to Hydrologic Soil Group “D” (no infiltration). For hydromodification structures located within 10 feet of a residential structure, or settlement sensitive improvement, storm water treatment and hydromodification management should be designed for no infiltration. An additional discussion of infiltration feasibility for specific groups of basins is presented in Appendix G, which contains a Categorization of infiltration for each feasibility condition, Worksheet C.4.1 (Form I-8), provided by the County (2016), for the areas underlain by alluvium, or older alluvium, or granitic substrate (each work sheet separately).

Onsite Infiltration-Runoff Retention Systems

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

Should onsite infiltration-runoff retention systems (OIRRS) be planned for Best Management Practices (BMPs) or Low Impact Development (LID) principles for the project, some guidelines should 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 (sometimes 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 (granitic) bedrock, which is anticipated to have relatively very low vertical infiltration rate.

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 may now require this.

- 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.

- Where infiltration systems are located near slopes, or improvements, impermeable liners and subdrains should be used along the bottom of bioretention swales/basins located within the influence of such slopes and structures. Impermeable liners used in conjunction with bioretention basins should consist of a 30-mil polyvinyl chloride

(PVC) membrane that is covered by a minimum of 12 inches of clean soil, free from rocks and debris, with a maximum 4:1 (h:v) slope inclination, or flatter, and meets the following minimum specifications:

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

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

Final project plans (grading, precise grading, foundation, retaining wall, landscaping, etc.), should be reviewed by this office prior to construction, so that construction is in accordance with the conclusions and recommendations of this report. Based on our review, supplemental recommendations and/or further geotechnical studies may be warranted. It should be noted that structural and landscape plans were not available for review at this time.

DEVELOPMENT CRITERIA

Slope Maintenance and Planting

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

Drainage

Adequate lot surface drainage is a very important factor in reducing the likelihood of adverse performance of foundations, hardscape, and slopes. Surface drainage should be sufficient to prevent ponding of water anywhere on a pad, and especially near structures and tops of slopes. Pad surface drainage should be carefully taken into consideration during fine grading, landscaping, and building construction. Therefore, care should be taken that future landscaping or construction activities do not create adverse drainage conditions. Positive site drainage within lots and common areas should be provided and maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond and/or seep into the ground. In general, the area within 5 feet around a structure should slope away from the structure. We recommend that unpaved lawn and landscape areas have a minimum gradient of 1 percent sloping away from structures, and whenever possible, should be above adjacent paved areas. Consideration should be given to avoiding construction of planters adjacent to structures (buildings, ancillary slabs, etc.). Pad drainage should be directed toward the street or other approved area(s). Although not a geotechnical requirement, roof gutters, down spouts, or other appropriate means may be utilized to control roof drainage. Down spouts, or drainage devices should outlet a minimum of 5 feet from structures or into a subsurface drainage system. Areas of seepage may develop due to irrigation or heavy rainfall, and should be anticipated. Minimizing irrigation will lessen this potential. If areas of seepage develop, recommendations for minimizing this effect could be provided upon request.

Toe of Slope Drains/Toe Drains

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

- Is there a source of irrigation above or on the slope that could contribute to saturation of soil at the base of the slope?

- Are the slopes hard rock and/or impermeable, or relatively permeable, or; do the slopes already have or are they proposed to have subdrains (i.e., stabilization fills, etc.)?

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

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

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

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

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

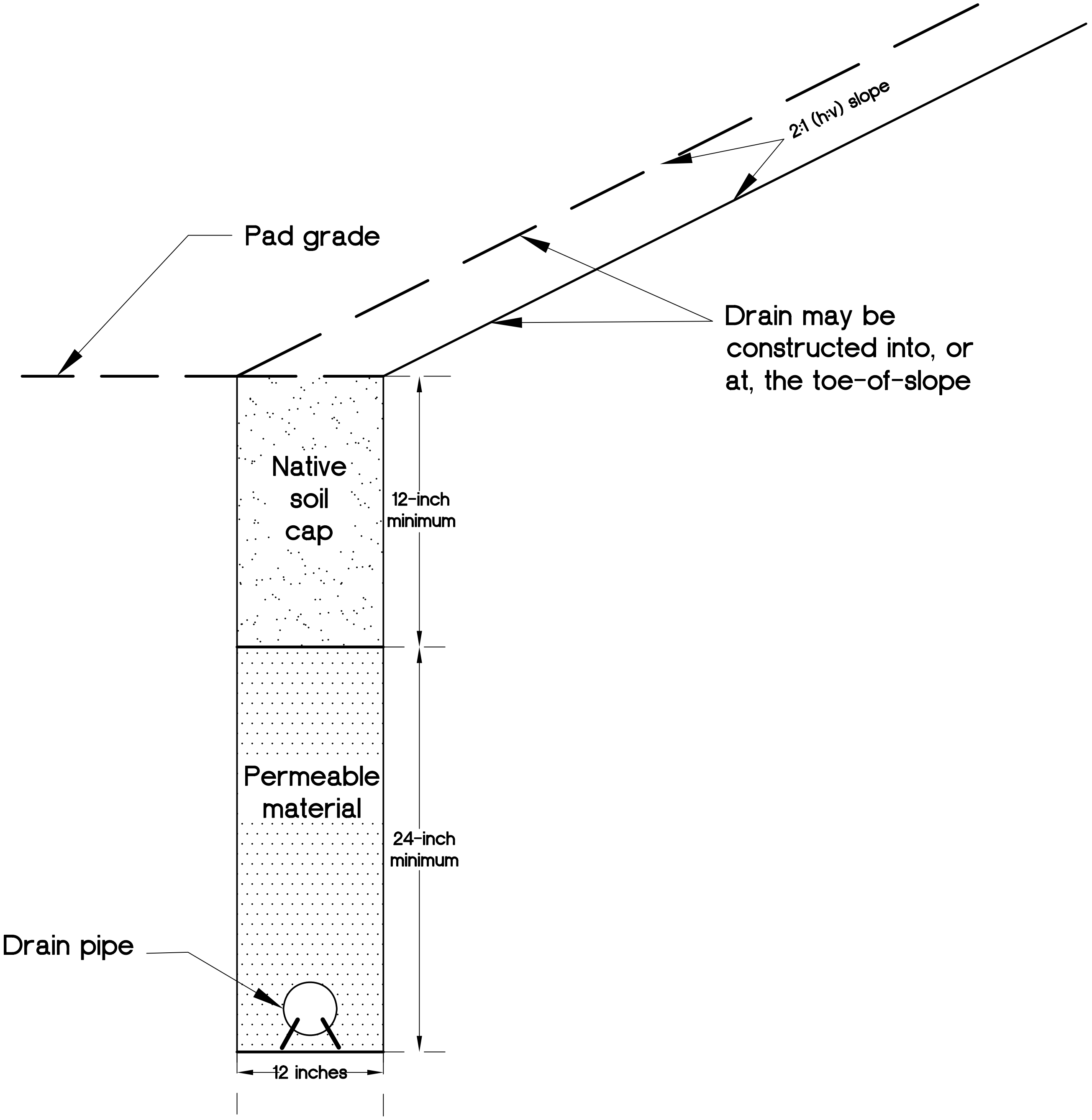
Erosion Control

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

Landscape Maintenance

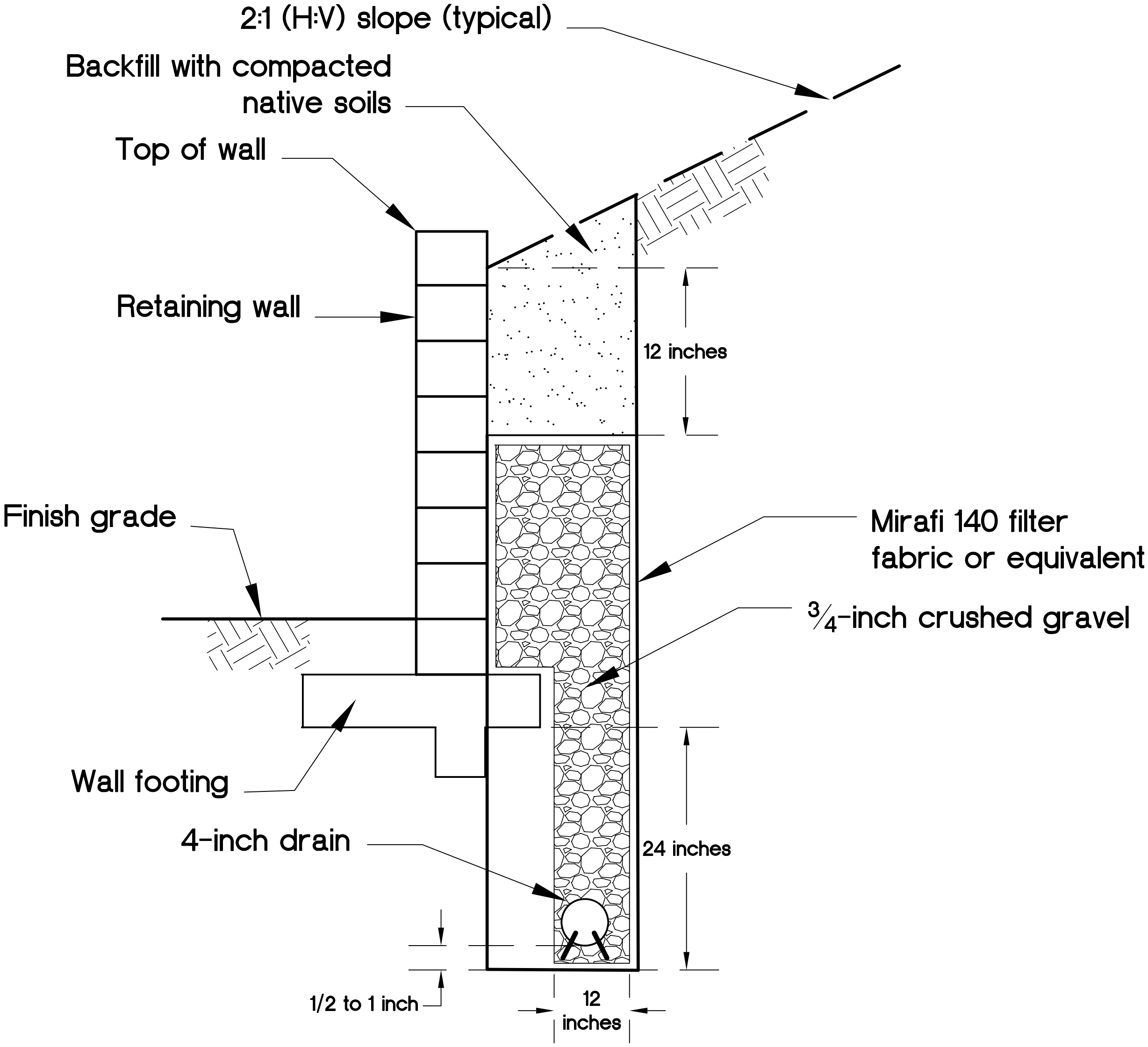
Only the amount of irrigation necessary to sustain plant life should be provided. Over-watering the landscape areas will adversely affect proposed site improvements. We would recommend that any proposed open-bottom planters adjacent to proposed structures be eliminated for a minimum distance of 10 feet. 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.,



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

GeoSoils, Inc.	SCHEMATIC TOE DRAIN DETAIL	Detail 4



NOTES:

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

GeoSoils, Inc.	SUBDRAIN ALONG RETAINING WALL DETAIL	Detail 5

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.

Subsurface and Surface Water

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

Site Improvements

If in the future, any additional improvements (e.g., wall, enclosures, 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/spas should not be constructed without specific geotechnical studues/review. This office should be notified in advance of any fill placement, grading of the site, or trench backfilling after rough grading has been completed. This includes any grading, utility trench and retaining wall backfills, flatwork, etc. This information should be provided to all interested/affected parties.

Additional Grading

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

Footing Trench Excavation

All footing excavations should be observed by a representative of this firm subsequent to trenching and prior to concrete form and reinforcement placement. The purpose of the observations is to evaluate that the excavations have been made into the recommended bearing material and to the minimum widths and depths recommended for construction. If loose or compressible materials are exposed within the footing excavation, a deeper footing or removal and recompaction of the subgrade materials would be recommended at that time. Footing trench spoil and any excess soils generated from utility trench excavations should be compacted to a minimum relative compaction of 90 percent (ASTM D 1557), if not removed from the site.

Trenching/Temporary Construction Backcuts

Considering the nature of the onsite earth materials, it should be anticipated that caving or sloughing could be a factor in subsurface excavations and trenching into onsite granular soils. Shoring or excavating the trench walls/backcuts at a maximum angle of 45 degrees (except as specifically superceded within the text of this report), should be anticipated. All excavations should meet a minimum FOS for temporary slope, backcut, shoring conditions of at least 1.25, and be observed by a geologist or 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 (such as groundwater) exist, appropriate recommendations would be offered at that time. The above recommendations should be provided to any contractors and/or subcontractors, etc., that may perform such work. If water is present or exposed during the trench excavation trench shields, shoring and dewatering should be used to complete excavations. Depending on the height of the groundwater above the trench shoring on trench shield bottom heave of sands may occur.

Utility Trench Backfill

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

2. Exterior trenches adjacent to, and within areas extending below a 1:1 plane projected from the outside bottom edge of the footing, and all trenches beneath hardscape features and in slopes, should be compacted to at least 90 percent of the laboratory standard. Sand backfill, unless excavated from the trench, should not be used in these backfill areas. Compaction testing and observations, along with probing, should be accomplished to evaluate the desired results.

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

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

Monitoring of Structures

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

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

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

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

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

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

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

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

SUMMARY OF RECOMMENDATIONS REGARDING GEOTECHNICAL OBSERVATION AND TESTING

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

- During grading/recertification.
- During excavation.
- During the excavation and placement of drilled piers (CIDH piles).
- During placement of subdrains, toe drains, or other subdrainage devices, prior to placing fill and/or backfill.
- After excavation of retaining wall footings/foundations, and free standing walls footings, prior to the placement of reinforcing steel or concrete.
- Prior to pouring any slabs or flatwork, after presoaking/presaturation of building pads and other flatwork subgrade, before the placement of concrete, reinforcing steel, capillary break (i.e., sand, pea-gravel, etc.), or vapor retarders (i.e., visqueen, etc.).
- During retaining wall subdrain installation, prior to backfill placement.
- During placement of backfill for area drain, interior plumbing, utility line trenches, and retaining wall backfill.
- During slope construction/repair.
- When any unusual soil conditions are encountered during any construction operations, subsequent to the issuance of this report.
- When any improvements, such as flatwork, walls, etc., are constructed, prior to construction. GSI should review and approve such plans prior to construction.
- A report of geotechnical observation and testing should be provided at the conclusion of each of the above stages, in order to provide concise and 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 any foundation. The structural engineer/foundation and/or slab designer should provide recommendations to not allow water or vapor to enter into the structure so as to cause damage to another building component, or so as to limit the installation of the type of flooring materials typically used for the particular application, as appropriate.

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

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

PLAN REVIEW

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

ADDITIONAL STUDIES

Given the likelihood of significant seismic induced settlement on Planning Area PA-3, variable thickness and potential for steep buried contact(s) in Planning Area PA-5, GSI recommends that additional CPTs be performed in both areas. Additional borings are recommended in Planning Area PA-3 to delineate: a) depth of alluvium (Qal); b) shape of buried formation/bedrock and alluvial contact; c) presence of fine grained soils or oversized earth materials; and d) groundwater.

LIMITATIONS

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

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

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

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APPENDIX B

TEST PIT, BORING LOGS, AND CPT LOGS
(CURRENT STUDY AND GSI, 2015)

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.

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-101	274'	0'-1'	SM				**COLLUVIUM:** SILTY SAND, grayish brown, dry, loose; porous, few roots and burrows encountered.
		1'-4'	SM				SILTY SAND, brown, slightly moist, loose.
							Total Depth = 4' No Groundwater Encountered Backfilled 06/7/16 Infiltration Test Zone Between Approximately -2' to -4'
TP-102	228'	0'-3½’	SM				**COLLUVIUM:** SILTY SAND, grayish brown, dry, loose; porous, few roots encountered.
		3½’-6½’	SM				**OLDER ALLUVIUM:** SILTY SAND, brown, slightly moist, medium dense; slightly porous from 3½’ to 4½’.
		6½’-8½’	SC				CLAYEY SAND, dark yellowish brown, moist, medium dense to dense.
							Total Depth = 8½’ No Groundwater Encountered Backfilled 06/7/16

GeoSoils, Inc.

W.O.6960-A-SC
Ocean Breeze Farms, LLC
Ocean Breeze Ranch
Logged By: RGC
June 7, 2016

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-103	232'	0'-3½'	SM				**UNDOCUMENTED FILL:** SILTY SAND with few angular gravels, brown, dry, loose; very porous.
		3½'-4½'	SW				**BEDROCK:** GRANITIC ROCK breaking to SAND upon excavation, medium gray, dry, dense.
							Total Depth = 4½' Practical Refusal No Groundwater Encountered Backfilled 06/7/16 Infiltration Test Performed Within Depth Interval of Approximately 2½'-4½'
TP-104	234'	0'-2½'	SM				**COLLUVIUM:** SILTY SAND, gray brown, dry, loose; porous, few roots encountered.
		2½'-3'	SM				**OLDER ALLUVIUM:** SILTY SAND, dark yellow brown, slightly moist, loose to medium dense; slightly porous.
		3'-5½'	SM				SILTY SAND, dark yellowish brown, slightly moist, medium dense to dense.
							Total Depth = 5½' No Groundwater Encountered Backfilled 06/7/16 Infiltration Test Performed at Approximately 3'-5½'

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-105	253'	0'-2'	SM				**COLLUVIUM:** SILTY SAND, grayish brown, dry, loose; porous, burrowed.
		2'-6'	SM				SILTY SAND, grayish brown to brown, dry, loose to medium dense; porous, weakly cemented.
		6'-8'	SM				**OLDER ALLUVIUM:** SILTY SAND, dark yellowish brown, moist, medium dense to dense; few sub-angular cobbles encountered.
							Total Depth = 8' No Groundwater Encountered Backfilled 06/7/16
TP-106	250'	0'-2'	SM				**COLLUVIUM:** SILTY SAND, dark brown, dry, loose; porous.
		2'-3½'	SM				SILTY SAND with some CLAY, brown, moist, loose; porous.
		3½'-5'	SM				**OLDER ALLUVIUM:** SILTY SAND, brown, slightly moist, medium dense; weakly cemented.
							Total Depth = 6' No Groundwater Encountered Backfilled 06/7/16

GeoSoils, Inc.

W.O.6960-A-SC
Ocean Breeze Farms, LLC
Ocean Breeze Ranch
Logged By: RGC
June 7, 2016

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-107	226'	0'-3½'	SM				**COLLUVIUM:** SILTY SAND, brown, dry, loose; porous, few roots encountered.
		3½'-7'	SM				**OLDER ALLUVIUM:** SILTY SAND, brown, slightly moist, medium dense.
							Total Depth = 7' No Groundwater Encountered Backfilled 06/7/16 Infiltration Test Within Interval Approximately 3½'-6'
TP-108	610'	0'-2½'	SM				**COLLUVIUM:** SILTY SAND, grayish brown, dry, loose; porous, few roots encountered.
		2½'-6'	SM/SC				**WEATHERED BEDROCK:** SILTY SAND to CLAYEY SAND, slightly moist, medium dense.
		6'-7'	SM				**BEDROCK:** GRANITIC ROCK breaking to SILTY SAND upon excavation, brown, moist, dense.
							Total Depth = 7' No Groundwater Encountered Backfilled 06/7/16

BORING LOG

GeoSoils, Inc.

W.O. 6960-A-SC

PROJECT: OCEAN BREEZE RANCH, LLC
5820 West Lilac Road, Bonsall

BORING HSA-1 *SHEET* 1 *OF* 1

DATE EXCAVATED 5-19-16

SAMPLE METHOD: 140 Lb. Hammer @ 30" Drop

Approx. Elevation: 189'

▩ *Standard Penetration Test*

▨ *Undisturbed, Ring Sample*

▽ *Groundwater*

≋ *Seepage*

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)
				SM			
1		▨	8	SP	97.3	5.9	22.4
2							
3							
4							
5							
6							
7							
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							
21							
22							
23							
24							
25							
26							
27							
28							
29							

Description of Material

COLLUVIUM (TOPSOIL):
@ 0' SILTY SAND, grayish brown, dry , loose; few roots, burrowed.

QUATERNARY ALLUVIUM:
@ 1' SAND with SILT, brown, slightly moist, loose.

Total Depth = 5½'
No Groundwater Encountered
Backfilled 05/19/16
Infiltration Test Performed From 3' to 5½'

BORING LOG

GeoSoils, Inc.

W.O. 6960-A-SC

PROJECT: OCEAN BREEZE RANCH, LLC
5820 West Lilac Road, Bonsall

BORING HSA-2 *SHEET* 1 *OF* 1

DATE EXCAVATED 5-19-16

SAMPLE METHOD: 140 Lb. Hammer @ 30" Drop

Approx. Elevation: 190'

▦ *Standard Penetration Test*

▨ *Undisturbed, Ring Sample*

▽ *Groundwater*

≋ *Seepage*

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
0–1				SM				**<u>COLLUVIUM (TOPSOIL):</u>** @ 0 ' SILTY SAND, grayish brown, dry, loose; few roots, burrowed.
2		▨	8	SP	96.0	7.8	28.4	**<u>QUATERNARY ALLUVIUM:</u>** @ 1½' SAND with SILT, brown, dry, loose.
3								
4								
5		▨	9		88.7	7.6	23.2	@ 5' As per 1½'.
6								
7								
8								
9								
10		▦	10					@ 10' SAND with SILT, brown, moist, loose to medium dense; micaceous.
11								
12								
13 ▽								@ 13' Groundwater encountered.
14								
15		▦	13	SW				@ 15' SAND, brown, saturated, medium dense; fine to medium grained.
16								
17								
18								
19								
20		▦	22	SP				@ 20' SAND, dark gray, saturated, medium dense; medium grained.
21								
22								Total Depth = 21½' Groundwater Encountered @ 13' (EL = 177' MSL) Backfilled 05/19/16
23								
24								
25								
26								
27								
28								
29								

GeoSoils, Inc.

BORING LOG

W.O. 6960-A-SC

PROJECT: OCEAN BREEZE RANCH, LLC
5820 West Lilac Road, Bonsall

BORING HSA-3 *SHEET* 1 *OF* 2

DATE EXCAVATED 5-19-16

SAMPLE METHOD: 140 Lb. Hammer @ 30" Drop

Approx. Elevation: 190'

▩ *Standard Penetration Test*

▨ *Undisturbed, Ring Sample*

∇ *Groundwater*

≈ *Seepage*

Depth (ft.)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
	Bulk	Undisturbed	Blows/Ft.					
				SM				**COLLUVIUM (TOPSOIL):** @ 0' SILTY SAND, grayish brown, slightly moist, loose; few roots and many burrows.
1								
2				SM/SP				**QUATERNARY ALLUVIUM:** @ 2' SILTY SAND to SAND, brown, dry, loose.
3								
4								
5		▩	8					@ 5' As per 2'.
6								
7								
8								
9								
10		▨	14	SP	94.4	14.0	49.5	@ 10' SAND, brown, moist, loose.
11 ∇								@ 11½' Groundwater encountered.
12								
13								
14								
15		▩	14					@ 15' SAND, dark grayish brown, saturated, medium dense; fine to medium grained.
16								
17								
18								
19								
20		▨	26		108.9	15.3	78.1	@ 20' SAND, dark grayish brown, saturated, medium dense; medium to coarse grained.
21								
22								
23								
24								
25		▩	20					@ 25' SAND, medium to dark gray, saturated, medium dense; medium grained.
26								
27								
28								
29								

BORING LOG

GeoSoils, Inc.

W.O. 6960-A-SC

PROJECT: OCEAN BREEZE RANCH, LLC
5820 West Lilac Road, Bonsall

BORING HSA-3 *SHEET* 2 *OF* 2

DATE EXCAVATED 5-19-16

SAMPLE METHOD: 140 Lb. Hammer @ 30" Drop

Approx. Elevation: 190'

▩ *Standard Penetration Test*

▨ *Undisturbed, Ring Sample*

▽ *Groundwater*

≋ *Seepage*

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	**Description of Material**
		▨	39	SP	102.7	21.7	100	@ 30' SAND, dark gray, saturated, medium dense to dense; medium grained.
31								
32								
33								
34								
35		▩	10					@ 35' SAND, dark gray, saturated, medium dense; fine to medium grained.
36								
37								
38								
39								
40		▨	21		103.0	23.0	100	@ 40' As per 35'.
41								
42								
43								
44								
45		▩	20					@ 45' As per 40'.
46								
47								
48								
49								
50		▩	26		134.0	17.6	100	@ 50' SAND, dark gray brown, saturated, dense.
51								Total Depth = 51' Groundwater Encountered @ 11½' (EL = 178½' MSL) Backfilled 05/19/16
52								
53								
54								
55								
56								
57								
58								
59								

GeoSoils, Inc.

BORING LOG

W.O. 6960-A-SC

PROJECT: OCEAN BREEZE RANCH, LLC
5820 West Lilac Road, Bonsall

BORING HSA-4 *SHEET* 1 *OF* 2

DATE EXCAVATED 5-19-16

SAMPLE METHOD: 140 Lb. Hammer @ 30" Drop

Approx. Elevation: 193'

▩ *Standard Penetration Test*

▨ *Undisturbed, Ring Sample*

▽ *Groundwater*

≈ *Seepage*

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
				SM				**COLLUVIUM (TOPSOIL):**
1								@ 0' SILTY SAND, light brown, dry, loose; few roots, burrowed.
2				SW				**QUATERNARY ALLUVIUM:**
3								@ 2 ' SAND with SILT, brown, slightly moist, loose.
4								
5		▨	15			6.8		@ 5' SAND with SILT, brown, slightly moist, loose.
6								
7								
8								
9								
10		▩	8					@ 10' SAND with SILT, brown, moist, loose.
11								
12								
13 ▽								
14								@ 13½' Groundwater encountered.
15		▨	19		108.8	20.4	100	@ 15' SAND, dark to medium gray, saturated, medium dense; fine grained.
16								
17								
18								
19								
20		▩	23	SP				@ 20' SAND, medium gray, saturated, medium dense; fine to medium grained.
21								
22								
23								
24								
25		▨	41		109.2	19.4	100	@ 25' SAND, medium gray, saturated, medium dense; fine to medium grained.
26								
27								
28								
29								

BORING LOG

W.O. 6960-A-SC

PROJECT: OCEAN BREEZE RANCH, LLC
5820 West Lilac Road, Bonsall

BORING HSA-4 *SHEET* 2 *OF* 2

DATE EXCAVATED 5-19-16

SAMPLE METHOD: 140 Lb. Hammer @ 30" Drop

Approx. Elevation: 193'

▩ *Standard Penetration Test*

▨ *Undisturbed, Ring Sample*

▽ *Groundwater*

≋ *Seepage*

Depth (ft.)	Sample Bulk	 Undisturbed	Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	**Description of Material**
		▩	12	SP				@ 30' SAND, dark gray, saturated, medium dense; fine grained.
31		▩						
32								
33								
34								
35		▨	24	SP/SM	97.5	26.4	100	@ 35' SAND with SILT, very dark gray, saturated, medium dense; fine grained, micaceous.
36								
37								
38								
39								
40		▩	34	SP				@ 40' SAND, very dark gray, saturated, dense; fine grained.
41		▩						
42								
43								
44								
45		▨	51		113.6	13.6	100	@45 SAND, medium gray to dark gray, saturated, dense; fine to medium grained.
46								
47								
48								
49								
50		▩	49					@ 50' SAND, dary gray, saturated, dense; fine to medium grained.
51		▩						
52								Total Depth = 51½' Groundwater Encountered @ 13½' (EL = 179½' MSL) Backfilled With Bentonite 05/19/16
53								
54								
55								
56								
57								
58								
59								

GeoSoils, Inc.

BORING LOG

W.O. 6960-A-SC

PROJECT: OCEAN BREEZE RANCH, LLC
5820 West Lilac Road, Bonsall

BORING HSA-5 *SHEET* 1 *OF* 2

DATE EXCAVATED 5-18-16

SAMPLE METHOD: 140 Lb. Hammer @ 30" Drop

Approx. Elevation: 197'

▩ *Standard Penetration Test*

▨ *Undisturbed, Ring Sample*

▽ *Groundwater*

≋ *Seepage*

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
				SM				**COLLUVIUM (TOPSOIL):** @ 0' SILTY SAND, grayish brown, dry, loose; few roots, burrowed.
1								
2				SM				**QUATERNARY ALLUVIUM:** @ 2' SILTY SAND, brown, slightly moist, loose; fine.
3								
4								
5		▩	10					@ 5' As per 2'.
6								
7								
8								
9								
10		▨	11	SP	93.1	11.6	39.5	@ 10' SAND with SILT, dark brown, slightly moist, loose.
11								
12								
13								
14								
15		▩	12					@ 15' As Per 10', moist, medium dense.
16								
17								
18 ▽								@ 18' Groundwater encountered.
19								
20		▨	15		No Recovery			@ 20' No recovery.
21								
22								
23								
24								
25		▩	12					@ 25' SAND with SILT, dark brown, saturated, medium dense.
26								
27								
28								
29								

BORING LOG

GeoSoils, Inc.

W.O. 6960-A-SC

PROJECT: OCEAN BREEZE RANCH, LLC
5820 West Lilac Road, Bonsall

BORING HSA-5 *SHEET* 2 *OF* 2

DATE EXCAVATED 5-18-16

SAMPLE METHOD: 140 Lb. Hammer @ 30" Drop

Approx. Elevation: 197'

▩ *Standard Penetration Test*

▨ *Undisturbed, Ring Sample*

▽ *Groundwater*

≈ *Seepage*

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)		**Description of Material**
31 32 33 34		▨	37	SW	112.0	17.7	100		@ 30 SAND with SILT, dark gray, saturated, medium dense; fine to coarse grained.
35 36 37 38 39		▩	15	SP/ML					@ 35' SAND with SILT, dark gray brown, saturated, medium dense; and SANDY SILT, dark gray, saturated, stiff.
40 41 42 43 44		▨	44	SP	104.1	24.5	100		@ 40' SAND with SILT, gray, saturated, medium dense to dense.
45 46 47 48 49		▩	28	SP/SW					@ 45' SAND with SILT and gravel, dark gray, saturated, medium dense to dense.
50		▨	30	SP	108.7	18.1	100		@ 50' SAND with SILT, brown, saturated, medium dense.
51 52 53 54 55 56 57 58 59									Total Depth = 51' Groundwater Encountered @ 18' (EL = 179' MSL) Backfilled 05/18/16

BORING LOG

W.O. 6960-A-SC

PROJECT: OCEAN BREEZE RANCH, LLC
5820 West Lilac Road, Bonsall

BORING HSA-6 SHEET 1 OF 2

DATE EXCAVATED 5-18-16

SAMPLE METHOD: 140 Lb. Hammer @ 30" Drop

Approx. Elevation: 195'

▩ Standard Penetration Test

▨ Undisturbed, Ring Sample

▽ Groundwater

≈ Seepage

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
1				SM				**COLLUVIUM (TOPSOIL):** @ 0' SILTY fine SAND, dark gray, slightly moist, loose; burrowed, roots.
2				SM				**QUATERNARY ALLUVIUM:** @ 2' SILTY SAND, dark brown, slightly moist, loose; fine grained, micaceous.
3								
4								
5		▨	13		97.4	7.7	29.1	@ 5' As per 2', moist, loose to medium dense.
6								
7								
8								
9								
10		▩	12					@ 10' As per 5', medium dense.
11								
12								
13								
14								
15		▨	17	SP	104.2	21.1	95	@ 15' SAND, gray brown, moist to wet, medium dense; medium grained, few fines.
16								
17 ▽								@ 17' Groundwater encountered.
18								
19								
20		▩	18					@ 20' SAND with SILT, dark gray, saturated, medium dense.
21								
22								
23								
24								
25		▨	29					@ 25' As per 20'.
26								
27								
28								
29								

BORING LOG

GeoSoils, Inc.

W.O. 6960-A-SC

PROJECT: OCEAN BREEZE RANCH, LLC
5820 West Lilac Road, Bonsall

BORING HSA-6 *SHEET* 2 *OF* 2

DATE EXCAVATED 5-18-16

SAMPLE METHOD: 140 Lb. Hammer @ 30" Drop

Approx. Elevation: 195'

▩ *Standard Penetration Test*

▨ *Undisturbed, Ring Sample*

∇ *Groundwater*

≈ *Seepage*

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
		▩	19	SP				@ 30' SAND with SILT, dark gray, saturated, medium dense; fine to coarse grained.
31		▩						
32								
33								
34								
35		▨	47		113.7	16.1	100	@ 35' As per 30', dense.
36								
37								
38								
39								
40		▩	8					@ 40' As per 35', loose; some gravel.
41		▩						
42								
43								
44								
45		▨	46		107.8	19.2	100	@ 45' As per 40', dense; no gravel.
46								
47								
48								
49								
50		▩	50					@ 50' As per 45'.
51		▩						
52								Total Depth = 51½' Groundwater Encountered @ 17' (EL = 178' MSL) Backfilled with Bentonite Clay 05/18/16
53								
54								
55								
56								
57								
58								
59								

BORING LOG

GeoSoils, Inc.

W.O. 6960-A-SC

PROJECT: OCEAN BREEZE RANCH, LLC
5820 West Lilac Road, Bonsall

BORING HSA-7 *SHEET* 1 *OF* 1

DATE EXCAVATED 7-5-16

SAMPLE METHOD: 140 Lb. Hammer @ 30" Drop

Approx. Elevation: 202'

▩ *Standard Penetration Test*

▨ *Undisturbed, Ring Sample*

∇ *Groundwater*

≈ *Seepage*

Depth (ft.)	**Sample** Bulk	Undisturbed	Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	**Description of Material**
				SM				**COLLUVIUM (TOPSOIL):** @ 0' SILTY SAND, dark grayish brown, wet, loose.
1								
2				SM				**QUATERNARY ALLUVIUM:** @ 2' SILTY SAND, grayish brown, wet, loose.
3								
4								
5		▨		SM	94.0	14.0	50	
6								Total Depth = 6' No Groundwater Encountered Backfilled 07/5/16
7								
8								
9								
10								
11								
12								
13								
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								
26								
27								
28								
29								

BORING LOG

GeoSoils, Inc.

W.O. 6960-A-SC

PROJECT: OCEAN BREEZE RANCH, LLC
5820 West Lilac Road, Bonsall

BORING HSA-8 *SHEET* 1 *OF* 1

DATE EXCAVATED 7-5-16

SAMPLE METHOD: 140 Lb. Hammer @ 30" Drop

Approx. Elevation: 224'

▩ *Standard Penetration Test*

▨ *Undisturbed, Ring Sample*

∇ *Groundwater*

≈ *Seepage*

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
1 2 3 4				ML				**ALLUVIUM:** @ 0' SANDY SILT, dark gray brown, dry, loose.
5 6 7 8		▨	35	SM/ML	105.8	2.8	13.4	@ 5' SANDY SILT to SILTY fine SAND, dark gray brown, slightly moist, medium dense.
9 10 11 12 13 14 15 16 17 18 19		▨	72	BDRX	116.8	5.4	34.2	**BEDROCK:** @ 8½' GRANITIC ROCK (decomposed), breaking to SAND upon excavation, dark brown, moist, dense.
20 21 22 23 24 25 26 27 28 29								Total Depth = 19½' No Groundwater Encountered Backfilled 07/5/16

BORING LOG

GeoSoils, Inc.

W.O. 6960-A-SC

PROJECT: OCEAN BREEZE RANCH, LLC
5820 West Lilac Road, Bonsall

BORING HSA-9 *SHEET* 1 *OF* 2

DATE EXCAVATED 7-5-16

SAMPLE METHOD: 140 Lb. Hammer @ 30" Drop

Approx. Elevation: 225'

▦ *Standard Penetration Test*

▨ *Undisturbed, Ring Sample*

▽ *Groundwater*

≈ *Seepage*

Depth (ft.)	Bulk	Undisturbed	Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
				SM				**ALLUVIUM:** @ 0' SILTY SAND, brown, dry, loose; few roots.
1								
2								
3								
4								
5		▨	5		103.8	6.4	28.6	@ 5' SILTY SAND, dark brown, slightly moist, loose.
6								
7								
8								
9								
10		▦	11	SP				@ 10' SAND with SILT, brown, moist, loose.
11								
12								
13								
14								
15		▨	16		106.9	4.2	20.1	@ 15' SAND, yellowish brown, moist, loose to medium dense.
16								
17								
18								
19								
20		▦	5					@ 20' No recovery.
21 ▽								@ 21' Groundwater ecountered.
22								
23								
24								
25		▨	13		111.3	16.6	100	@ 25' SAND, brown, saturated, loose.
26								
27								
28								
29								

BORING LOG

GeoSoils, Inc.

W.O. 6960-A-SC

PROJECT: OCEAN BREEZE RANCH, LLC
5820 West Lilac Road, Bonsall

BORING HSA-9 *SHEET* 2 *OF* 2

DATE EXCAVATED 7-5-16

SAMPLE METHOD: 140 Lb. Hammer @ 30" Drop

Approx. Elevation: 225'

▩ *Standard Penetration Test*

▨ *Undisturbed, Ring Sample*

▽ *Groundwater*

≋ *Seepage*

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)		Description of Material
		▩	24	CH				▨	@ 30' CLAY, olive brown, wet, stiff.
31		▩						▨	
32								▨	
33								▨	
34								▨	
35		▨	29	CL				▨	@ 35' SANDY CLAY, mottled olive brown to strong brown, moist, very stiff.
36								▨	
37		▨	50-2"	BDR					**BEDROCK:** @ 37' GRANITIC ROCK, very dense (practical refusal). Total Depth = 37¼' Groundwater Encountered @ 21' (EL = 204' MSL) Backfilled 07/5/16 Infiltration Test Location
38									
39									
40									
41									
42									
43									
44									
45									
46									
47									
48									
49									
50									
51									
52									
53									
54									
55									
56									
57									
58									
59									

BORING LOG

GeoSoils, Inc.

W.O. 6960-A-SC

PROJECT: OCEAN BREEZE RANCH, LLC
5820 West Lilac Road, Bonsall

BORING HSA-10 *SHEET* 1 *OF* 1

DATE EXCAVATED 7-5-16

SAMPLE METHOD: 140 Lb. Hammer @ 30" Drop

Approx. Elevation: 222'

▩ *Standard Penetration Test*

▨ *Undisturbed, Ring Sample*

∇ *Groundwater*

≈ *Seepage*

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
1				SM				**COLLUVIUM (TOPSOIL):** @ 0' SILTY SAND, gray brown, dry, loose; few roots, porous.
2								
3				BDRX				**BEDROCK:** @ 2½' GRANITIC ROCK (decomposed granite) breaking to SAND upon excavation. @ 3½' becomes dense.
4								
5		▨	50-4"					
6								
7								
8								
9								
10		▨	50-3"					
11								Total Depth = 10¼' No Groundwater Encountered Backfilled 07/5/16
12								
13								
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								
26								
27								
28								
29								

BORING LOG

GeoSoils, Inc.

W.O. 6960-A-SC

PROJECT: OCEAN BREEZE RANCH, LLC
5820 West Lilac Road, Bonsall

BORING HSA-11 *SHEET* 1 *OF* 1

DATE EXCAVATED 7-5-16

SAMPLE METHOD: 140 Lb. Hammer @ 30" Drop

Approx. Elevation: 227'

▩ *Standard Penetration Test*

▨ *Undisturbed, Ring Sample*

▽ *Groundwater*

≈ *Seepage*

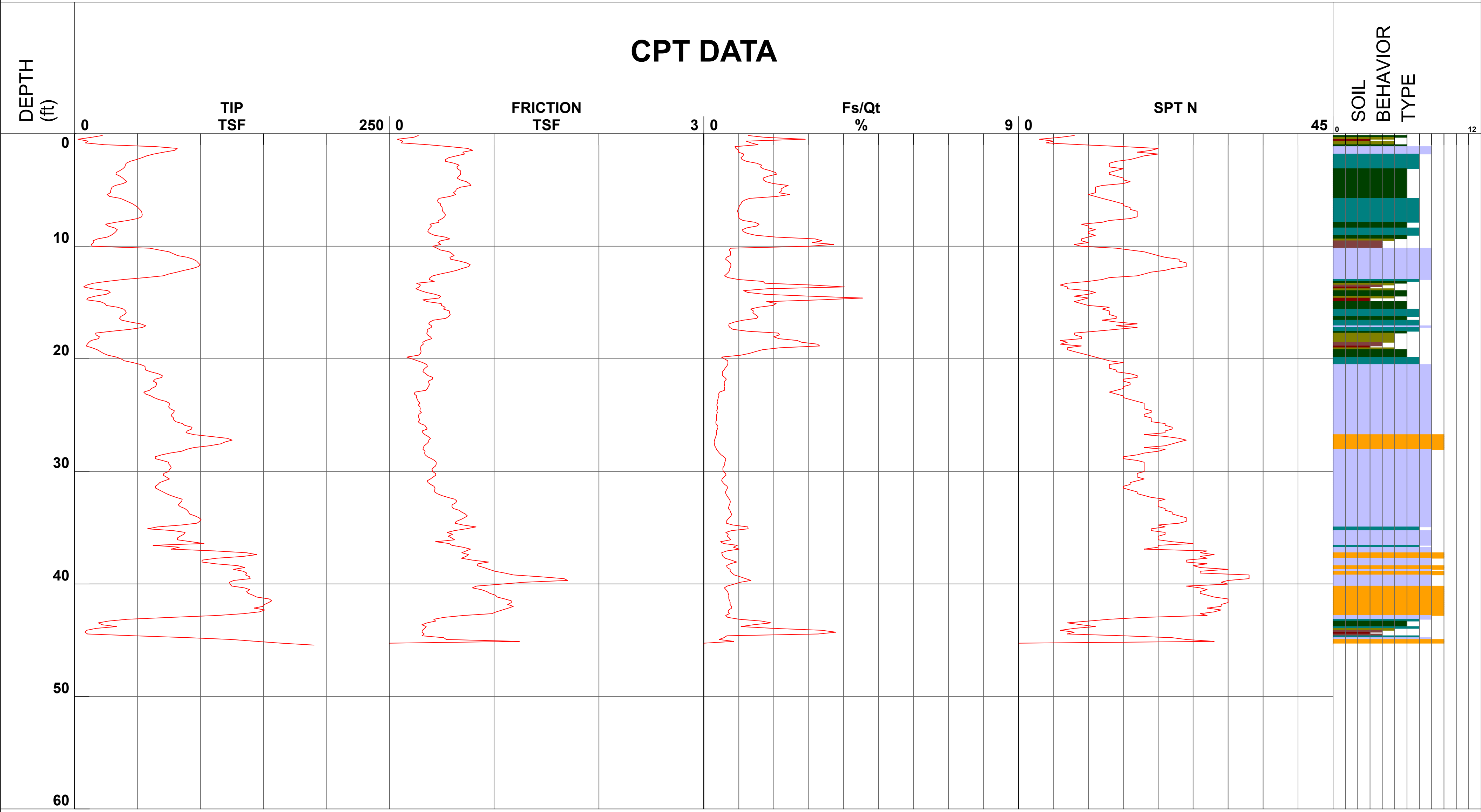
Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
0				SM				**COLLUVIUM (TOPSOIL):** @ 0' SILTY SAND, dark brown, dry, loose; porous, many roots.
3				SM				**OLDER ALLUVIUM:** @ 3' SILTY SAND, brown, slightly moist, loose to medium dense; few gravels.
5		▨	19		102.2	3.3	14.1	@ 5' SILTY SAND with few gravels, brown, slightly moist, loose to medium dense.
7								@ 7' becomes medium dense.
10		▨	28					@ 10' SILTY SAND, brown moist, medium dense.
15		▨	26	SP	111.5	2.6	14.4	@ 15' SAND, brown, slightly moist, medium dense, few sub-horizontal SILTY SAND inter-layers.
17				SP/SM				**BEDROCK:** @ 17' GRANITIC ROCK (decomposed granite) breaking to SAND/SILTY SAND, brown, moist, dense.
20		▨	52					@ 20' As per 17'.
25		▨	50-5"		120.3	10.7	75.5	
26								Total Depth = 25½' (EL = 201½' MSL) No Groundwater Encountered Backfilled 07/5/16

Geosoils

Project	Ocean Breeze Ranch	**Operator**	DG-RC	**Filename**	SDF(481).cpt
Job Number	6960-A-SC	**Cone Number**	DDG1268	**GPS**	
Hole Number	CPT-101	**Date and Time**	5/20/2016 11:55:34 AM	**Maximum Depth**	45.44 ft
EST GW Depth During Test		13.00 ft			

Net Area Ratio .8

CPT DATA



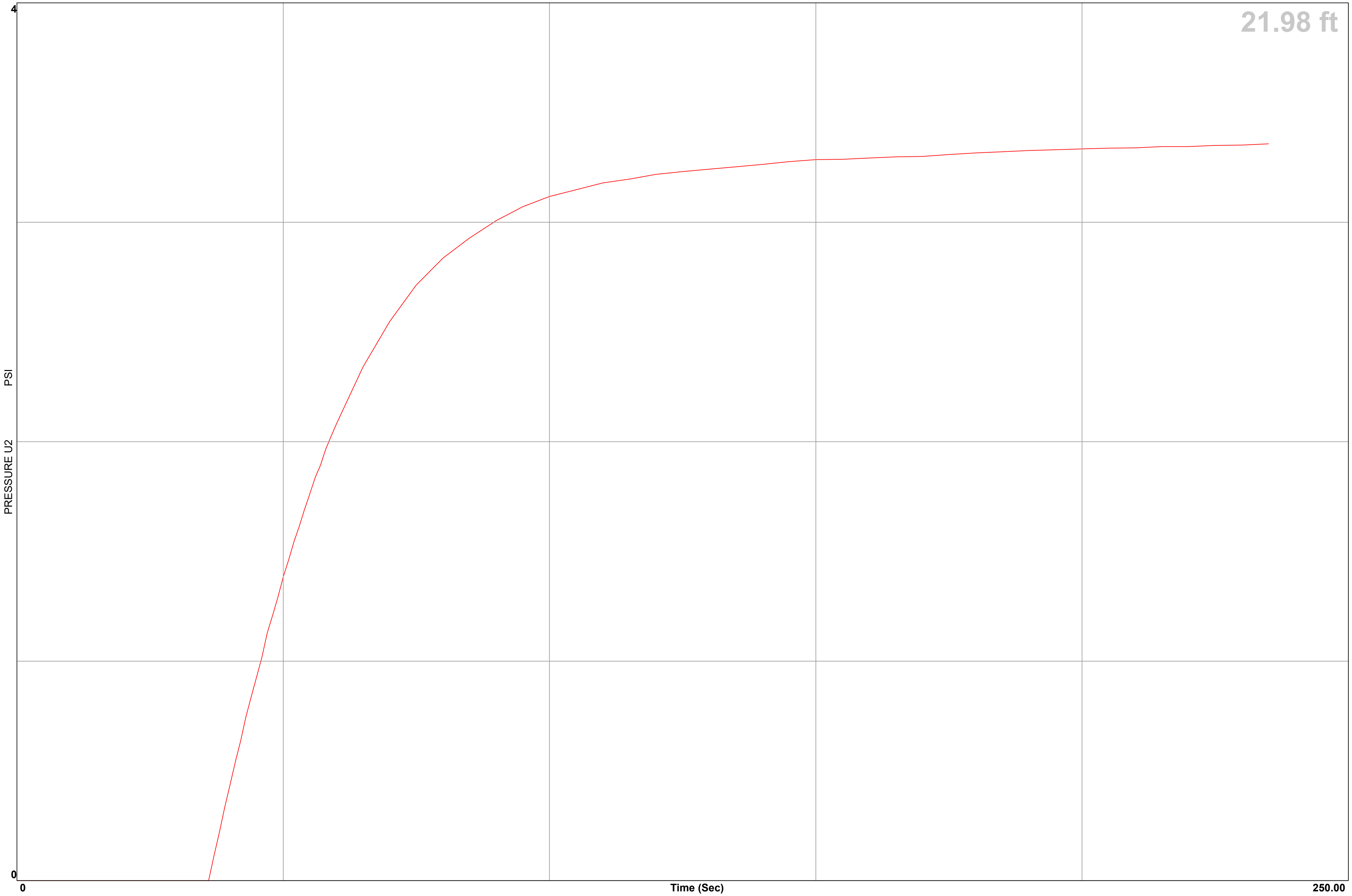
■ 1 - sensitive fine grained	■ 4 - silty clay to clay	■ 7 - silty sand to sandy silt	■ 10 - gravelly sand to sand
■ 2 - organic material	■ 5 - clayey silt to silty clay	■ 8 - sand to silty sand	■ 11 - very stiff fine grained (*)
■ 3 - clay	■ 6 - sandy silt to clayey silt	■ 9 - sand	■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Geosoils

Location	Ocean Breeze Ranch	**Operator**	DG-RC		
Job Number	6960-A-SC	**Cone Number**	DDG1268	**GPS**	
Hole Number	CPT-102	**Date and Time**	5/20/2016 12:32:11 PM		
Equilized Pressure	3.3	**EST GW Depth During Test**	14.2		



Page 1 of 1

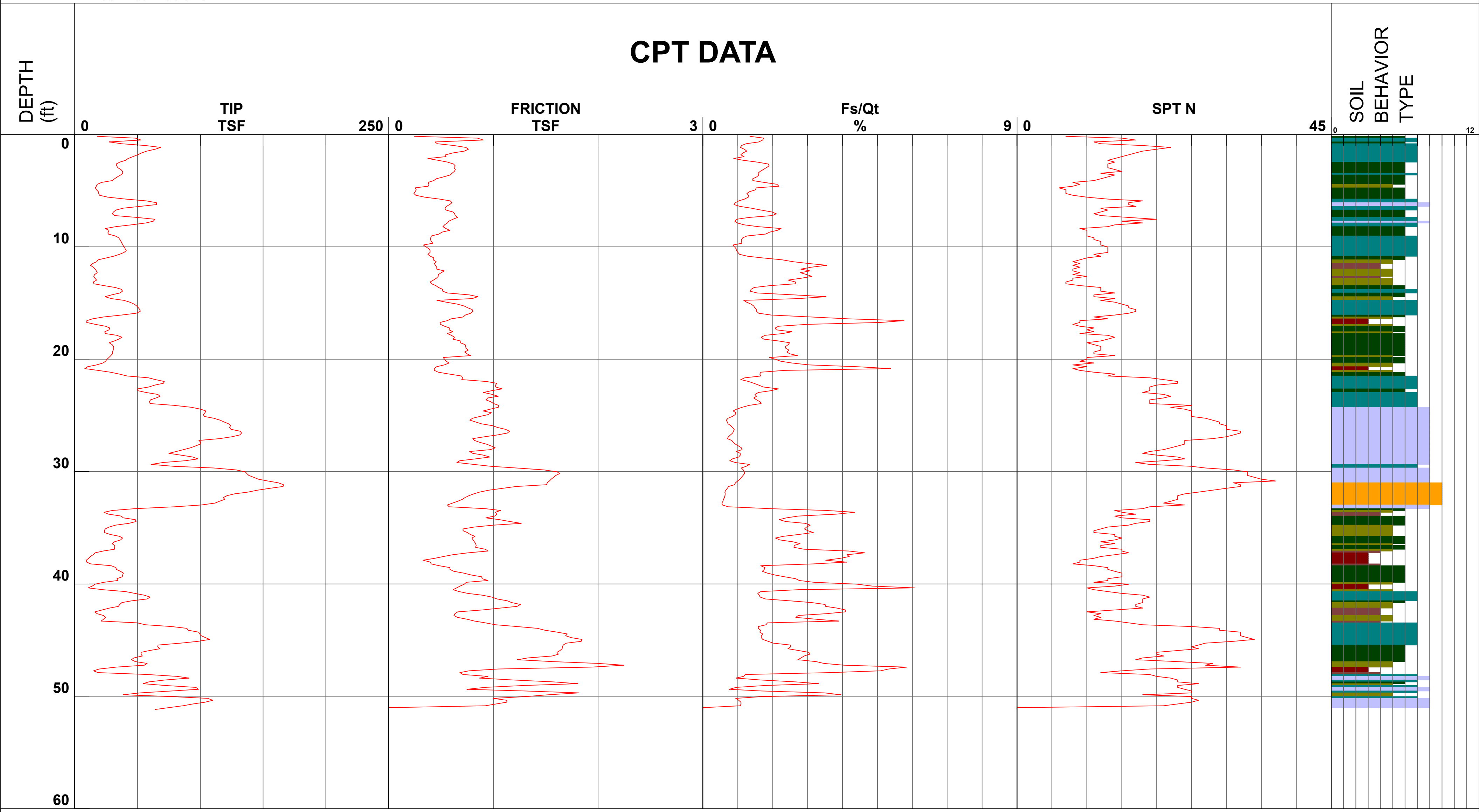
Geosoils

Project Ocean Breeze Ranch
Job Number 6960-A-SC
Hole Number CPT-102
EST GW Depth During Test 14.20 ft

Operator DG-RC
Cone Number DDG1268
Date and Time 5/20/2016 12:32:11 PM

Filename SDF(482).cpt
GPS
Maximum Depth 51.18 ft

Net Area Ratio .8



- ■ 1 - sensitive fine grained
- ■ 2 - organic material
- ■ 3 - clay
- ■ 4 - silty clay to clay
- ■ 5 - clayey silt to silty clay
- ■ 6 - sandy silt to clayey silt
- ■ 7 - silty sand to sandy silt
- ■ 8 - sand to silty sand
- ■ 9 - sand
- ■ 10 - gravelly sand to sand
- ■ 11 - very stiff fine grained (*)
- ■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

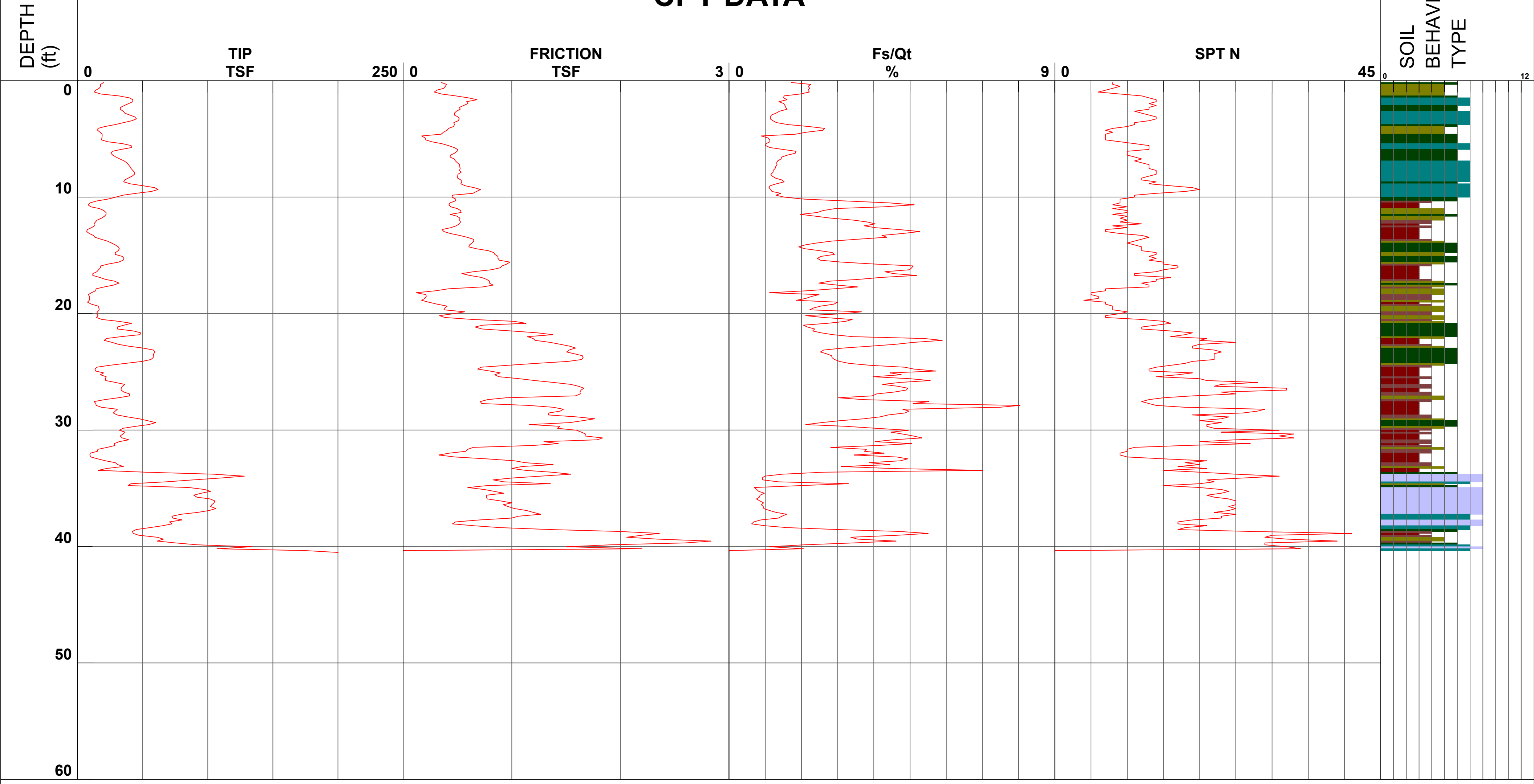
S*Soil behavior type and SPT based on data from UBC-1983

Geosoils

Project	Ocean Breeze Ranch	Operator	DG-RC	Filename	SDF(483).cpt
Job Number	6960-A-SC	Cone Number	DDG1268	GPS	
Hole Number	CPT-103	Date and Time	5/20/2016 1:14:09 PM	Maximum Depth	40.52 ft
EST GW Depth During Test		13.00 ft			

Net Area Ratio .8

CPT DATA



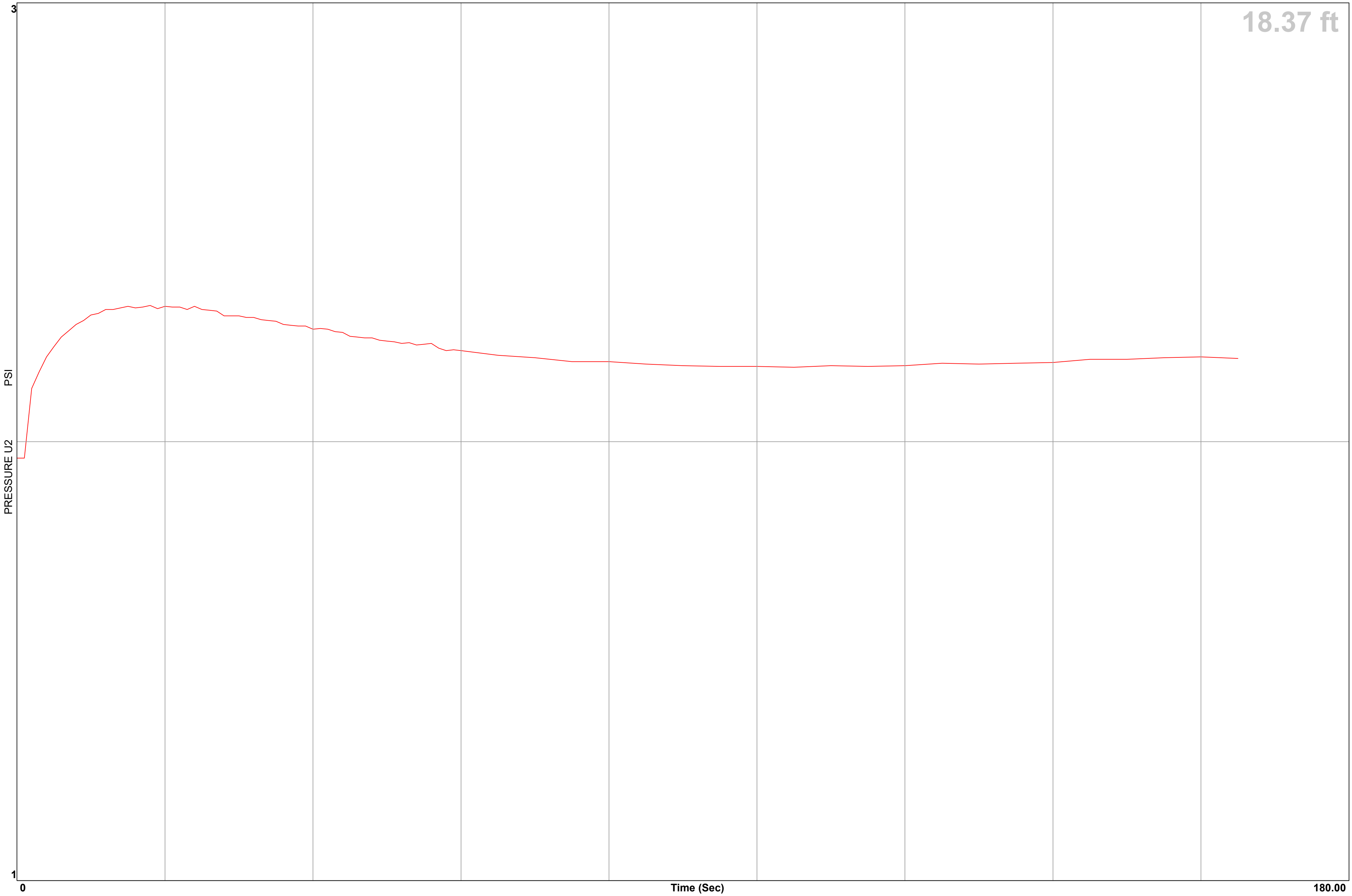
■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Geosoils

Location	Ocean Breeze Ranch	**Operator**	DG-RC	**GPS**	
Job Number	6960-A-SC	**Cone Number**	DDG1268		
Hole Number	CPT-104	**Date and Time**	5/20/2016 10:45:44 AM		
Equilized Pressure	2.1	**EST GW Depth During Test**	13.3		

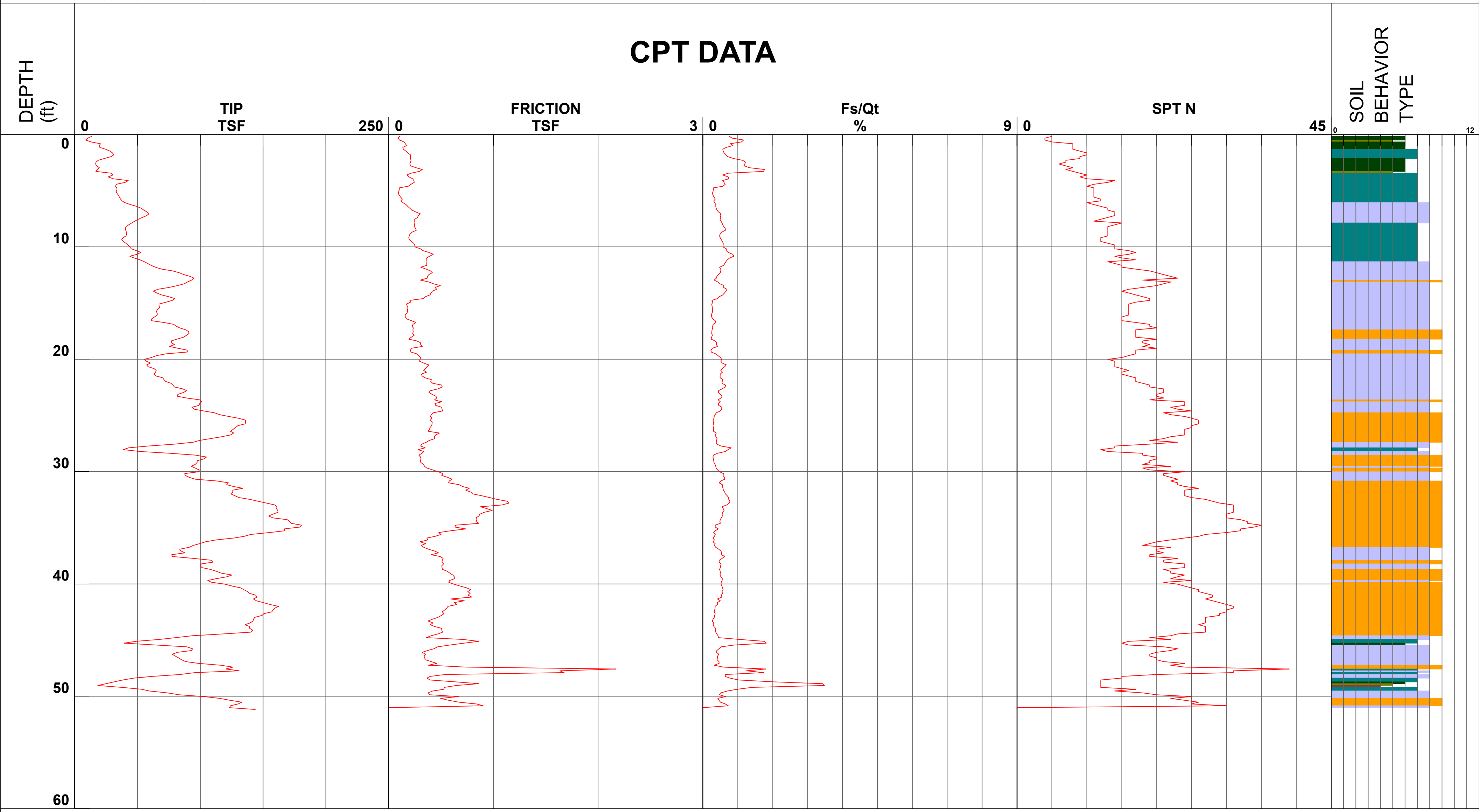


Geosoils

Project	Ocean Breeze Ranch	Operator	DG-RC	Filename	SDF(480).cpt
Job Number	6960-A-SC	Cone Number	DDG1268	GPS	
Hole Number	CPT-104	Date and Time	5/20/2016 10:45:44 AM	Maximum Depth	51.18 ft
EST GW Depth During Test		13.30 ft			

Net Area Ratio .8

CPT DATA



- ⬜ 1 - sensitive fine grained
- ⬛ 2 - organic material
- 🟥 3 - clay
- 🟫 4 - silty clay to clay
- 🟨 5 - clayey silt to silty clay
- 🟩 6 - sandy silt to clayey silt
- 🟦 7 - silty sand to sandy silt
- 🟪 8 - sand to silty sand
- 🟧 9 - sand
- 🟥 10 - gravelly sand to sand
- 🟪 11 - very stiff fine grained (*)
- 🟦 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

APPENDIX B

TEST PIT, BORING LOGS, AND CPT LOGS
(GSI, 2015)

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. 6688-A-SC
Vessels Stallion Ranch
Vessels Stallion Ranch, Bonsall
Logged By: RGC
March 7, 2014, March 26, 2014

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-1	±232' MSL	0-2	SM	1-2			**COLLUVIUM:** SILTY SAND, very dark brown, dry, loose.
		2-3½	SM/SC	2-3½			SILTY SAND with some CLAY, brown, moist, loose; porous.
		3½-5	SM	3½-4			**TERRACE DEPOSITS:** SILTY SAND, brown, damp, medium dense; slightly porous, weakly cemented.
							Total Depth = 5' No Groundwater/Caving Encountered Backfilled 3-7-2014
TP-2	±225' MSL	0-1½	SW				**ALLUVIUM:** SAND, light brown, damp, loose; few roots in upper 2".
		1½-3½	SM	Ring @ 3	108.7	5.5	SILTY SAND, very dark brown, moist, loose to medium dense.
		3½-17	SM				SILTY SAND, dark brown, moist, loose.
							Total Depth = 17' No Groundwater/Caving Encountered Backfilled 3-26-2014

GeoSoils, Inc.

W.O. 6688-A-SC
Vessels Stallion Ranch
Vessels Stallion Ranch, Bonsall
Logged By: RGC
March 7, 2014, March 26, 2014

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-3	±292' MSL	0-½	SM				**<u>COLLUVIUM:</u>** SILTY SAND, grayish brown, damp, loose; many roots, porous.
		½-4	SM	2	103.1	5.7	SILTY SAND, brown, dry, loose; very porous (pores to 1/8").
		4-7	SM				SILTY SAND, brown, damp, loose; few pores.
		7-8	SM				**<u>BEDROCK:</u>** GRANITIC ROCK breaking to SILTY SAND upon excavation, olive brown to dark brown, damp, medium dense to dense @ 8'.
							Total Depth = 8' No Groundwater/Caving Encountered Backfilled 3-26-2014
TP-4	±315' MSL	0-1	SM				**<u>COLLUVIUM:</u>** SILTY SAND, grayish brown, dry, loose; few roots, porous.
		1-2	SW				**<u>HIGHLY WEATHERED BEDROCK:</u>** GRANITIC ROCK breaking to SAND upon excavation, brown, damp, loose.
		2-5	SW				**<u>BEDROCK:</u>** GRANITIC ROCK breaking to SAND, damp/dry, medium dense becoming dense at 4'; joint sets: N40°W, 65°NE.
							Total Depth = 5' (Practical Refusal) No Groundwater/Caving Encountered Backfilled 3-26-2014

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-5	±230' MSL	0-1	SM				**COLLUVIUM:** SILTY SAND, brown, dry, loose; few roots, porous.
		1-2	SM				SILTY SAND, dark brown, damp, loose; very porous.
		2-3	SM	2	102.4	5.1	SILTY SAND, brown, dry, loose to medium dense; slightly porous, disseminated, carbonates, weakly cemented.
		3-5	SM				**TERRACE DEPOSITS:** SILTY SAND, light brown, to yellowish brown, damp to dry, medium dense; slightly porous, disseminated carbonates, moderately cemented.
		5-6	SM				As per 3', no visible pores, to few pinhole pores.
							Total Depth = 6' No Groundwater/Caving Encountered Backfilled 3-26-2014
TP-6	±225' MSL	0-2	SM				**COLLUVIUM:** SILTY SAND, brown, damp, loose; few roots.
		2-14	SP	Ring 3'	93.4	4.6	**ALLUVIUM:** SAND, brownish gray, damp, loose; fine grained.
		13½-14					**BEDROCK:** GRANITIC ROCK breaking to SILTY SAND and brittle gravel to cobble-size rock fragments upon excavation, yellowish brown-brownish yellow, moist, dense.
							Total Depth = 14' No Groundwater/Caving Encountered Backfilled 3-26-2014

GeoSoils, Inc.

W.O. 6688-A-SC
Vessels Stallion Ranch
Vessels Stallion Ranch, Bonsall
Logged By: RGC
March 7, 2014, March 26, 2014

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-7	±700' MSL	0-1½	SM				**COLLUVIUM:** SILTY SAND, dark brown, damp, loose; few roots.
		1½-7	SW				**BEDROCK:** GRANITIC ROCK breaking to SAND and brittle ground to cobble-size rock fragments upon excavation, yellowish brown to light grayish brown, dry, dense; practical refusal at 7'.
							Total Depth = 7' (practical refusal) No Groundwater/Caving Encountered Backfilled 3-26-2014
TP-8	±650' MSL	0-2	SM				**COLLUVIUM:** SILTY SAND with angular cobble to small boulder-size rock fragment, dark brown, damp, loose; porous, few roots in upper; 6".
		2-6	SW				**BEDROCK:** GRANITIC ROCK breaking to SAND and brittle cobble-size rock fragments upon excavation, yellowish brown, dry, dense; practical refusal at 6' on hard rock.
							Total Depth = 6' (Practical Refusal) No Groundwater/Caving Encountered Backfilled 3-26-2014

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-9	±260' MSL	0-3	SM				**COLLUVIUM:** SILTY SAND, brown, damp, loose; porous, some angular rock fragments.
		3-4	SM				**HIGHLY WEATHERED BEDROCK:** GRANITIC ROCK breaking to SILTY SAND, yellowish brown, damp, loose to medium dense; highly weathered, relict bedrock structure.
		4-12					**BEDROCK:** GRANITIC ROCK breaking to SILTY SAND upon excavation, yellowish brown to brownish yellow, damp, medium dense; fractured and brittle gravel to cobble-size rock fragments.
							Total Depth = 12' No Groundwater/Caving Encountered Backfilled 3-26-2014
TP-10	±225' MSL	0-1	SW				**FILL:** SAND, gray brown, dry, loose.
		1-2	SM				**COLLUVIUM:** SILTY SAND, dark brown, damp, loose; porous.
		2-3½	SC				CLAYEY SAND to SAND with CLAY, brown, damp, loose; porous, blocky.
		3½-4	SM				**TERRACE DEPOSITS:** SILTY SAND, brown, damp, medium dense.
							Total Depth = 4' No Groundwater/Caving Encountered Backfilled 3-7-2014

GeoSoils, Inc.

W.O. 6688-A-SC
Vessels Stallion Ranch
Vessels Stallion Ranch, Bonsall
Logged By: RGC
March 7, 2014, March 26, 2014

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-11	±265' MSL	0-2					**COLLUVIUM:** SILTY SAND, very dark brown, damp, loose.
		2-3					SILTY SAND with CLAY, brown, damp, loose; porous.
		3-5					**TERRACE DEPOSITS:** SILTY SAND, brown, damp, medium dense.
		5-6					**BEDROCK:** GRANITIC ROCK breaking to SILTY SAND and SAND with trace CLAY, olive brown, moist, medium dense.
							Total Depth = 5' No Groundwater/Caving Encountered Backfilled 3-7-2014

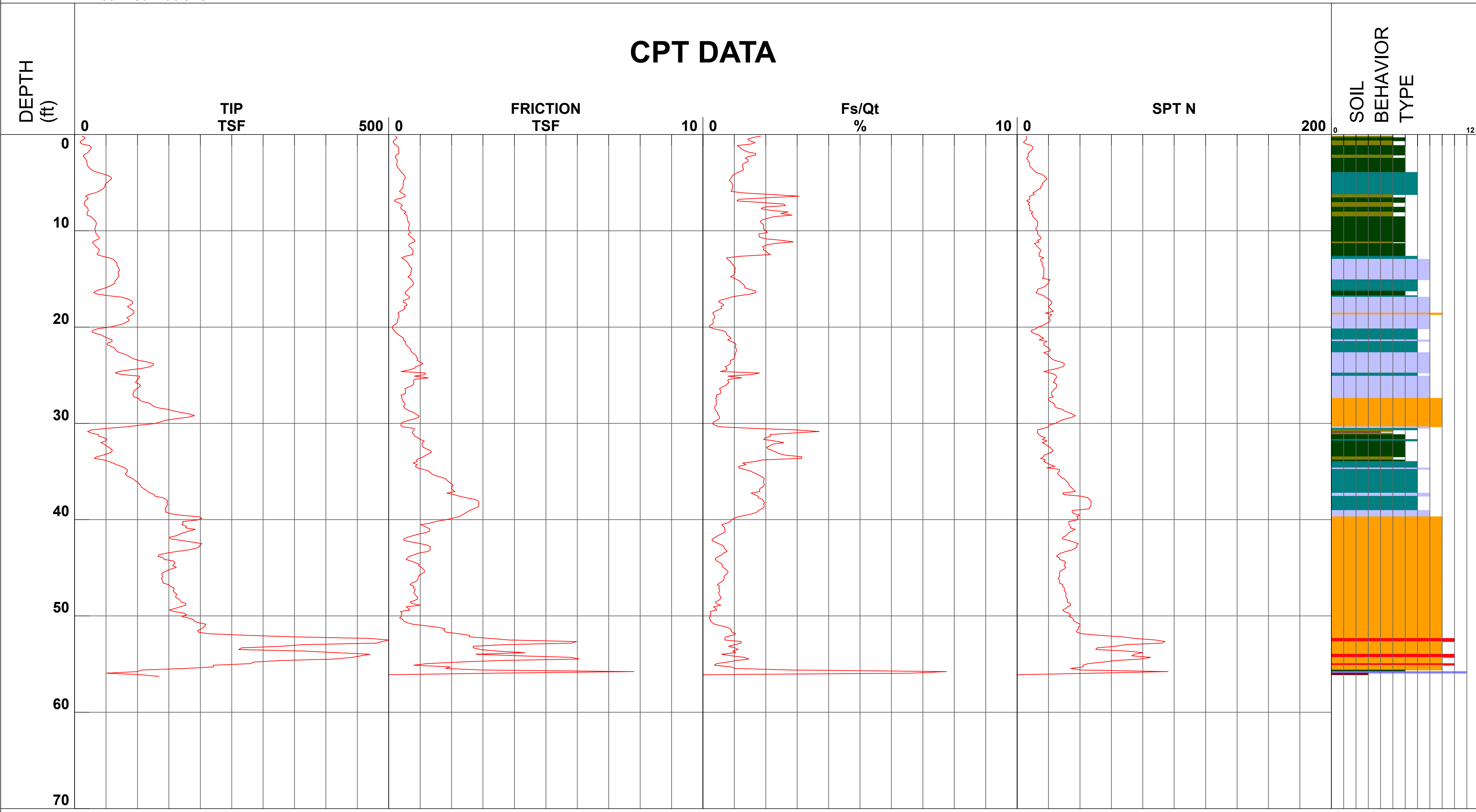
Middle Earth
Geo Testing Inc.

Geosoils Inc

Project Vessels
Job Number 6688-A
Hole Number CPT-01
EST GW Depth During Test 14.00 ft
Operator BH-MM
Cone Number DDG1281
Date and Time 3/7/2014 8:45:55 AM
Filename SDF(580).cpt
GPS
Maximum Depth 56.27 ft

Net Area Ratio .8

CPT DATA



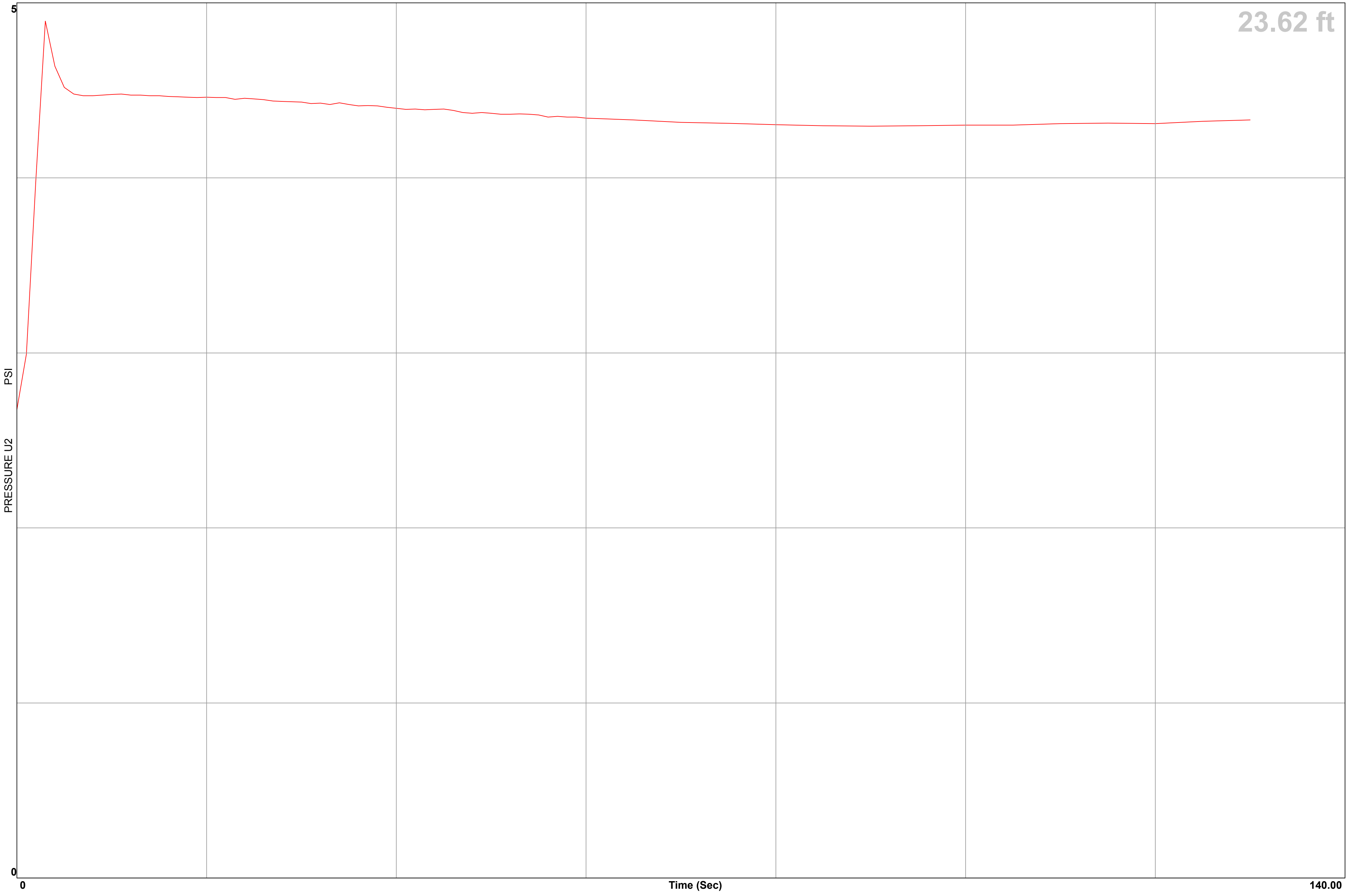
■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

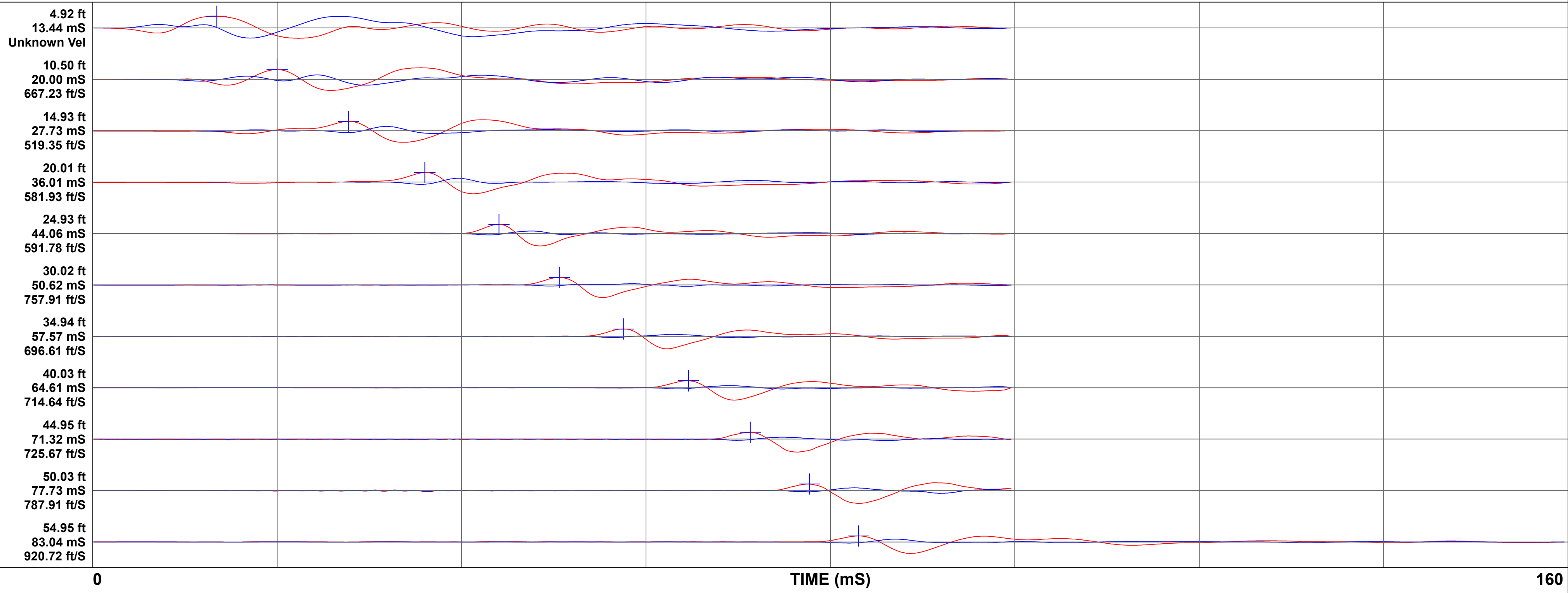
Geosoils Inc

Location	Vessels	Operator	BH-MM		
Job Number	6688-A	Cone Number	DDG1281	GPS	
Hole Number	CPT-01	Date and Time	3/7/2014 8:45:55 AM		
Equilized Pressure	4.3	EST GW Depth During Test	13.6		



Geosoils Inc

Location Vessels
Job Number 6688-A
Hole Number CPT-01
Operator BH-MM
Cone Number DDG1281
Date and Time 3/7/2014 8:45:55 AM
GPS

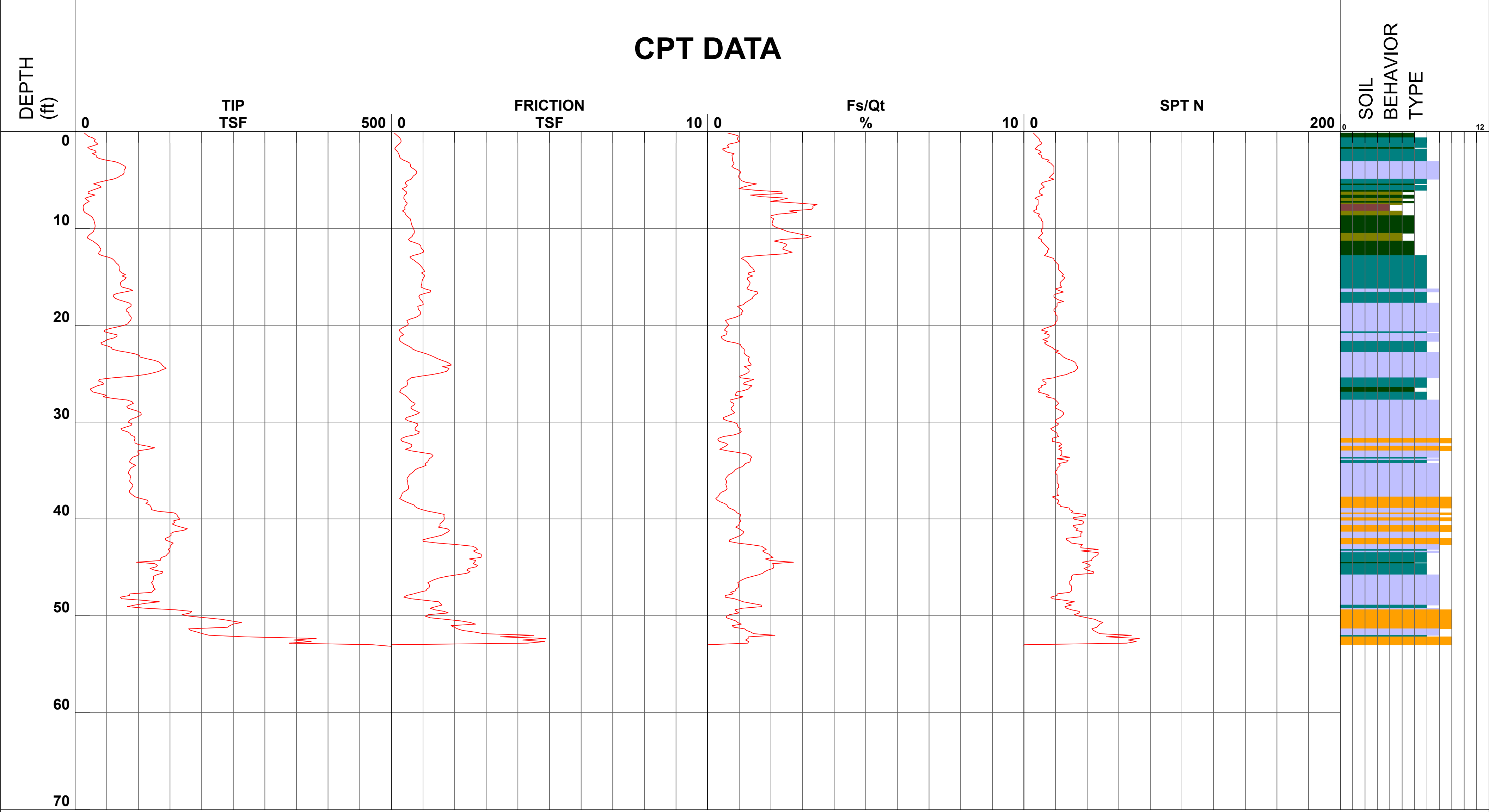


Geosoils Inc

Project Vessels
Job Number 6688-A
Hole Number CPT-01A
EST GW Depth During Test 14.00 ft
Operator BH-MM
Cone Number DDG1281
Date and Time 3/7/2014 9:41:55 AM
Filename SDF(581).cpt
GPS
Maximum Depth 53.15 ft

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained
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■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

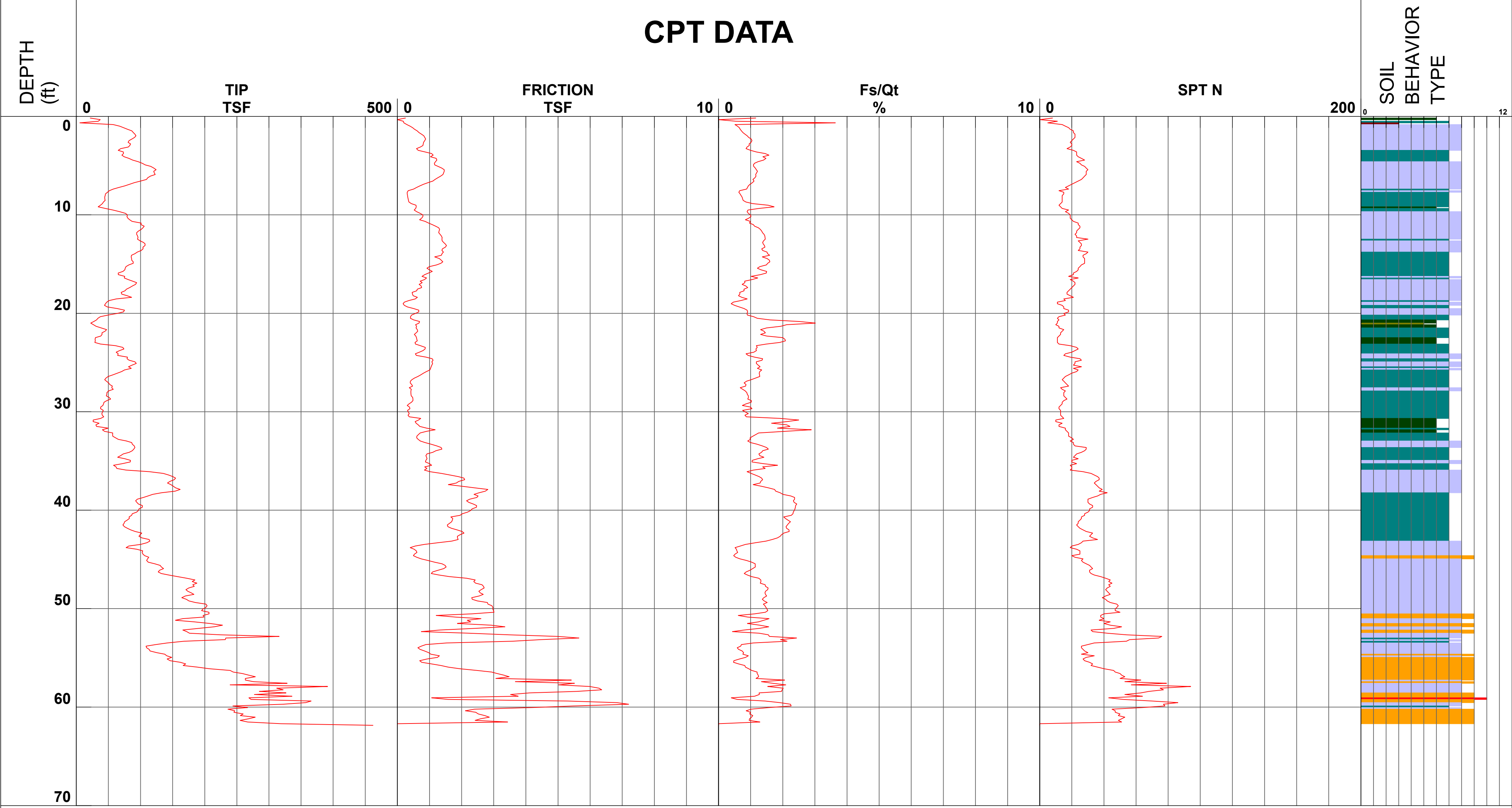
S*Soil behavior type and SPT based on data from UBC-1983

Geosoils Inc

Project: Vessels
Job Number: 6688-A
Hole Number: CPT-02
EST GW Depth During Test: 14.00 ft
Operator: BH-MM
Cone Number: DDG1281
Date and Time: 3/7/2014 10:32:26 AM
Filename: SDF(582).cpt
GPS:
Maximum Depth: 61.84 ft

Net Area Ratio .8

CPT DATA



- ■ 1 - sensitive fine grained
- ■ 2 - organic material
- ■ 3 - clay
- ■ 4 - silty clay to clay
- ■ 5 - clayey silt to silty clay
- ■ 6 - sandy silt to clayey silt
- ■ 7 - silty sand to sandy silt
- ■ 8 - sand to silty sand
- ■ 9 - sand
- ■ 10 - gravelly sand to sand
- ■ 11 - very stiff fine grained (*)
- ■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

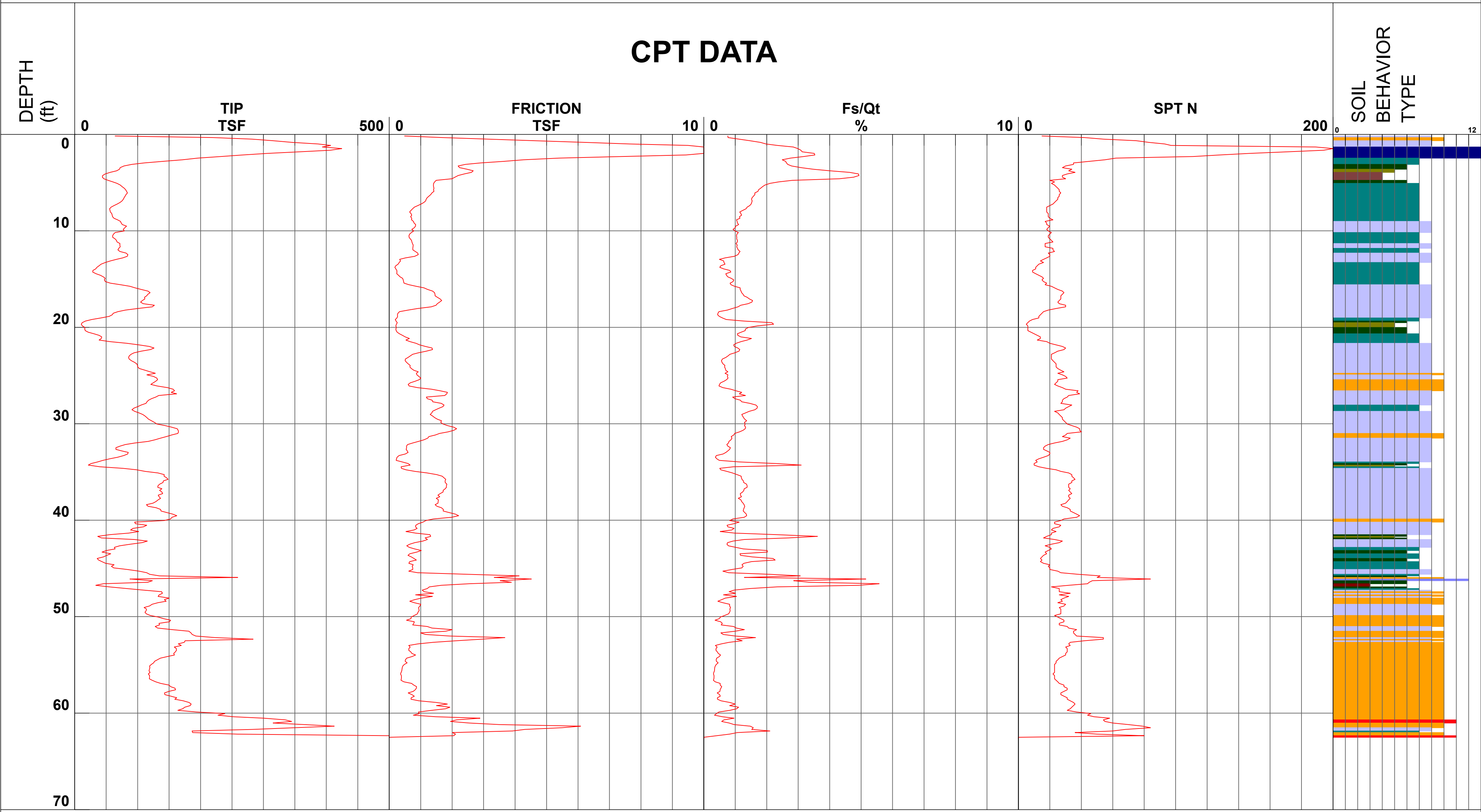
S*Soil behavior type and SPT based on data from UBC-1983

Geosoils Inc

Project: Vessels
Job Number: 6688-A
Hole Number: CPT-03
EST GW Depth During Test: 15.00 ft
Operator: BH-MM
Cone Number: DDG1281
Date and Time: 3/7/2014 11:28:59 AM
Filename: SDF(583).cpt
GPS:
Maximum Depth: 62.66 ft

Net Area Ratio .8

CPT DATA



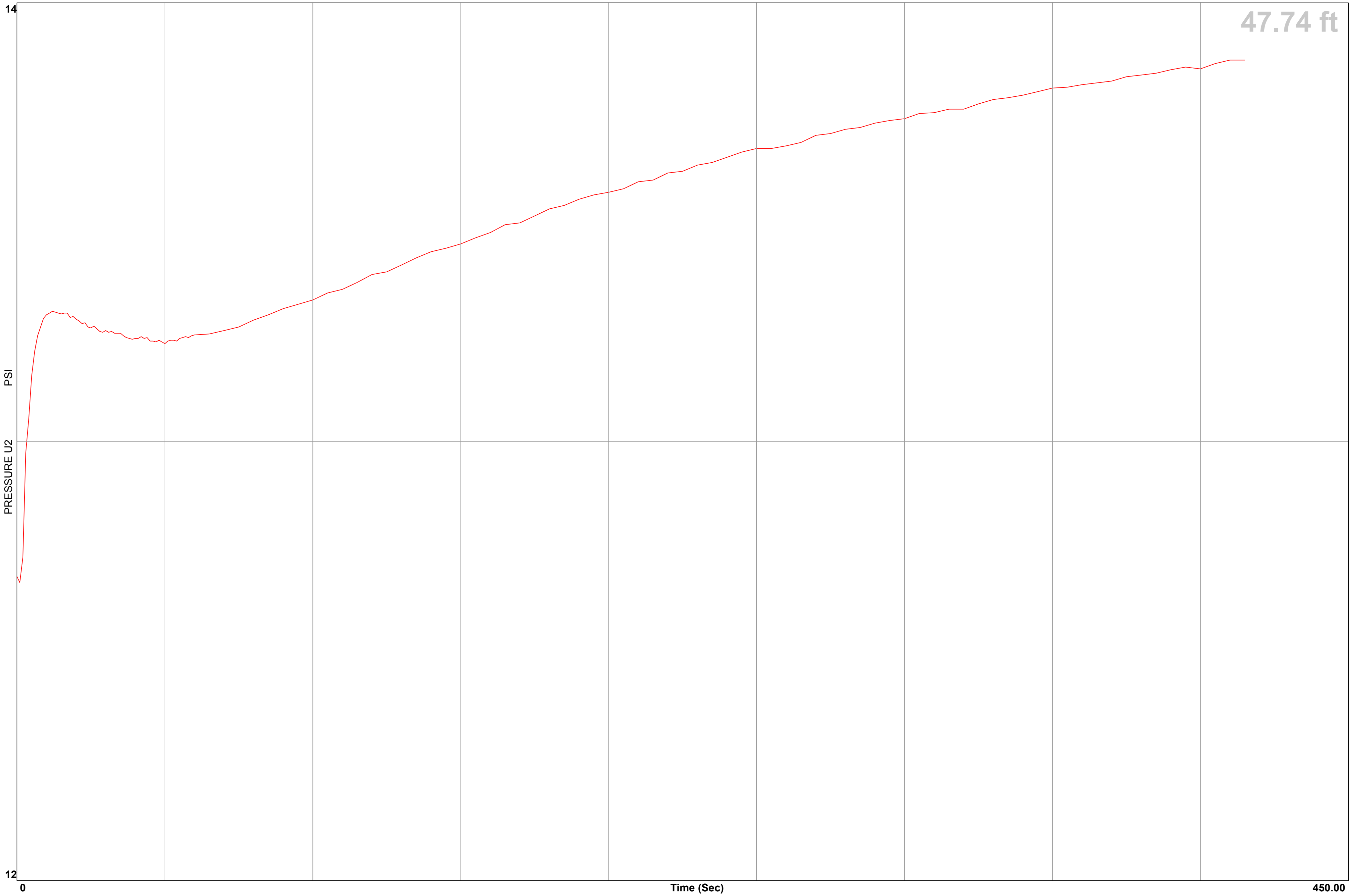
- ■ 1 - sensitive fine grained
- ■ 2 - organic material
- ■ 3 - clay
- ■ 4 - silty clay to clay
- ■ 5 - clayey silt to silty clay
- ■ 6 - sandy silt to clayey silt
- ■ 7 - silty sand to sandy silt
- ■ 8 - sand to silty sand
- ■ 9 - sand
- ■ 10 - gravelly sand to sand
- ■ 11 - very stiff fine grained (*)
- ■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Geosoils Inc

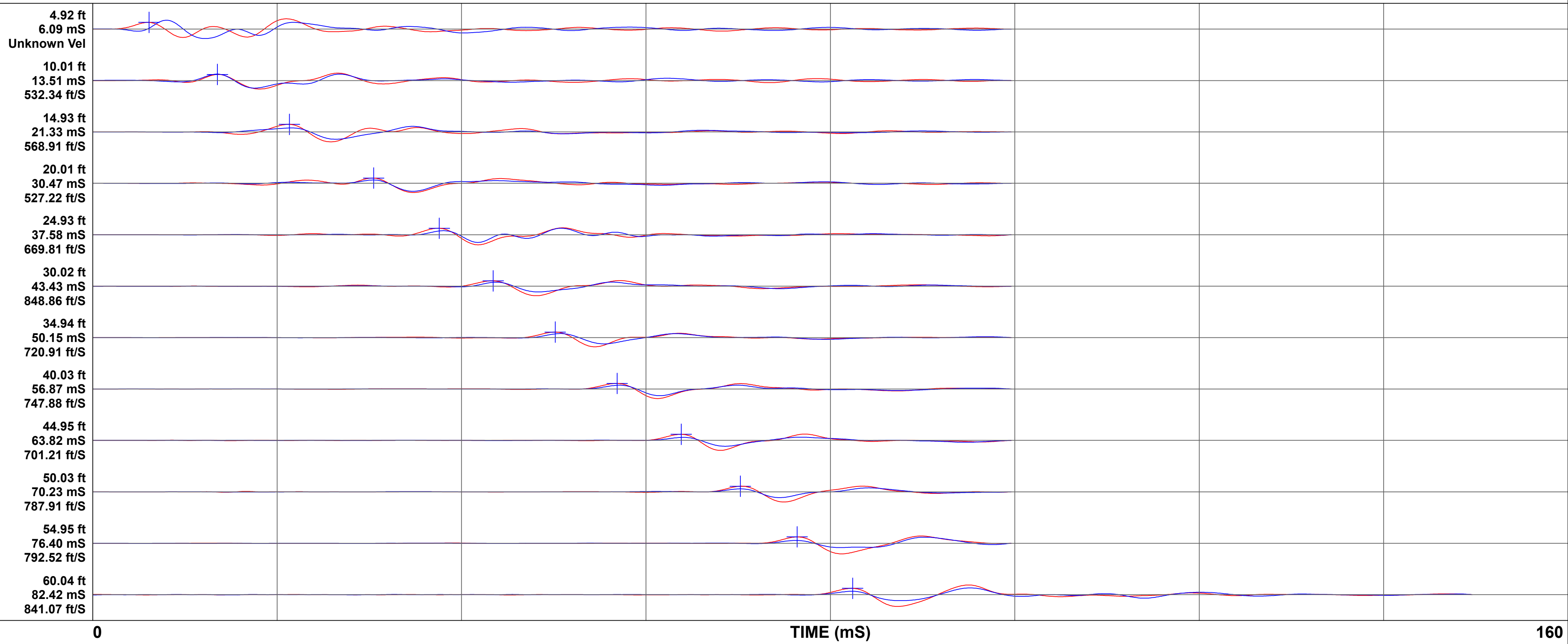
Location	Vessels	**Operator**	BH-MM		
Job Number	6688-A	**Cone Number**	DDG1281	**GPS**	
Hole Number	CPT-03	**Date and Time**	3/7/2014 11:28:59 AM		
Equilized Pressure	13.8	**EST GW Depth During Test**	15.7		



Middle Earth
Geo Testing Inc.

Geosoils Inc

Location Vessels
Job Number 6688-A
Hole Number CPT-03
Operator BH-MM
Cone Number DDG1281
Date and Time 3/7/2014 11:28:59 AM
GPS

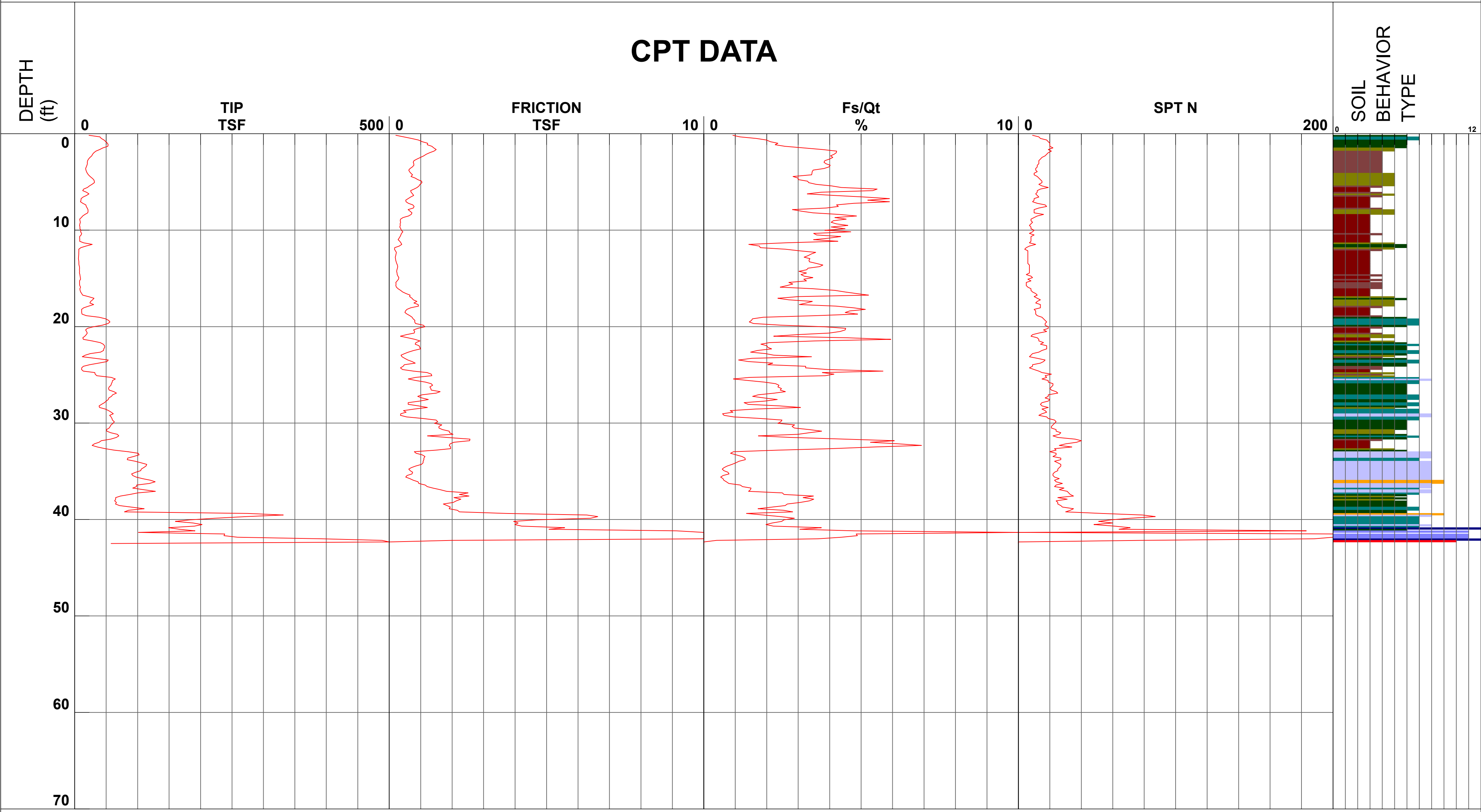


Middle Earth
Geo Testing Inc.

Geosoils Inc

Project	Vessels	Operator	BH-MM	Filename	SDF(584).cpt
Job Number	6688-A	Cone Number	DDG1281	GPS	
Hole Number	CPT-04	Date and Time	3/7/2014 12:54:37 PM	Maximum Depth	42.49 ft
EST GW Depth During Test		15.00 ft			

Net Area Ratio .8



■ 1 - sensitive fine grained
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■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

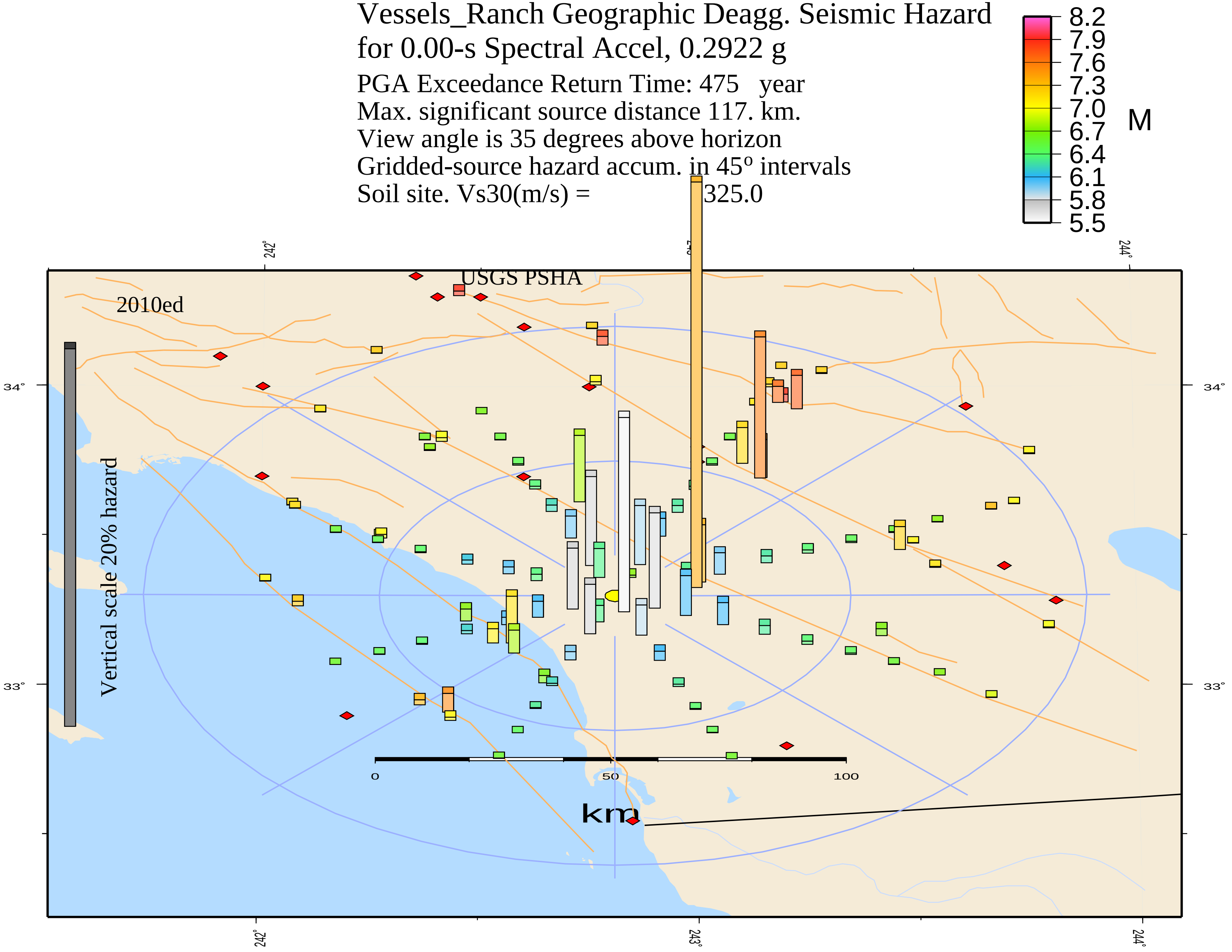
S*Soil behavior type and SPT based on data from UBC-1983

APPENDIX C

SEISMICITY ANALYSIS

Vessels_Ranch Geographic Deagg. Seismic Hazard
for 0.00-s Spectral Accel, 0.2922 g

PGA Exceedance Return Time: 475 year
Max. significant source distance 117. km.
View angle is 35 degrees above horizon
Gridded-source hazard accum. in 45° intervals
Soil site. Vs30(m/s) = 325.0



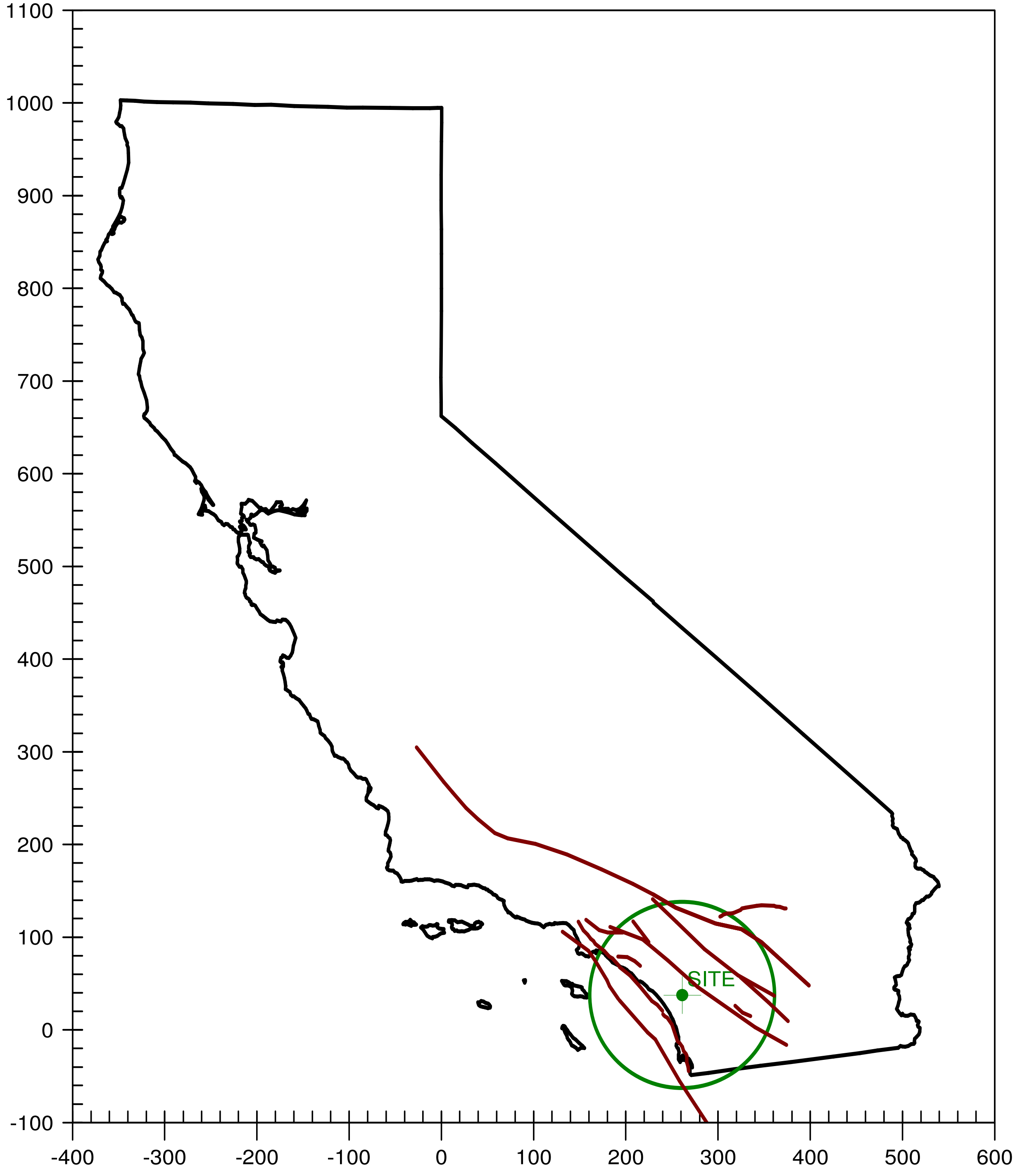
W.O. 6688-A-SC
PLATE C-1

GMT 2014 Apr 14 19:42:37

Site Coords:-117.193 33.3027 (yellow disk) Vs30= 325.0. Max annual ExcdRate .4527E-03 (column height prop. to ExRate). Red diamonds: historical earthquakes, M>6

CALIFORNIA FAULT MAP

Vessels non-rock areas



W.O. 6688-A-SC
PLATE C-2

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

                    DETERMINISTIC ESTIMATION OF
              PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 6688
                                                      DATE: 01-27-2015

JOB NAME: Vessels  non rock

CALCULATION NAME: Test Run Analysis

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


SITE COORDINATES:
   SITE LATITUDE:  33.3027
   SITE LONGITUDE:  117.1933

SEARCH RADIUS:   62.4  mi

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

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


MINIMUM DEPTH VALUE (km):  3.0
```
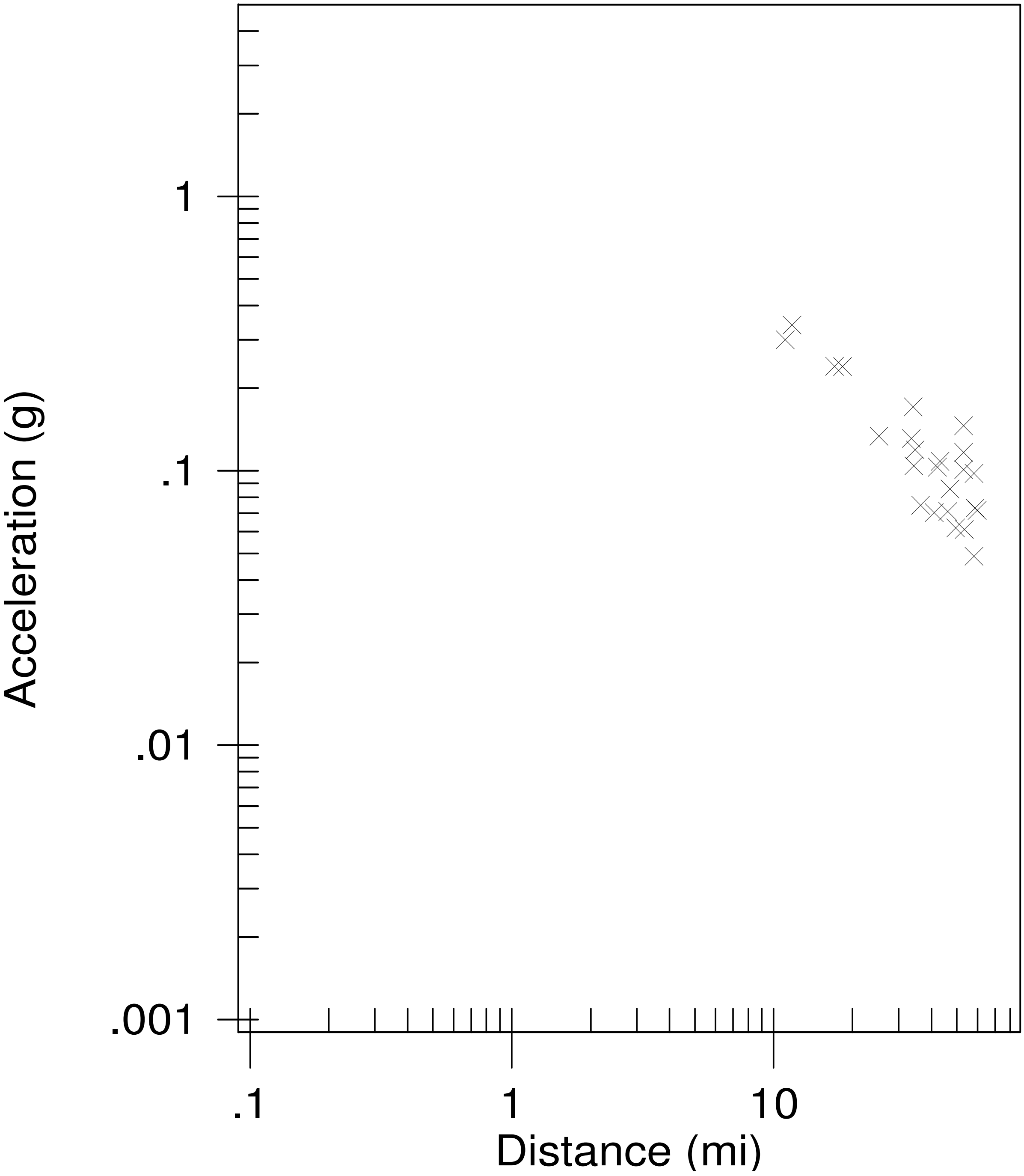

```
                                 ---------------
                                 EQFAULT SUMMARY
                                 ---------------



                           -----------------------------
                           DETERMINISTIC SITE PARAMETERS
                           -----------------------------

Page  1
-------------------------------------------------------------------------------
                                |              |ESTIMATED MAX. EARTHQUAKE EVENT
                                | APPROXIMATE  |-------------------------------
          ABBREVIATED           |  DISTANCE    | MAXIMUM  |   PEAK   |EST. SITE
          FAULT  NAME           |  mi   (km)   |EARTHQUAKE|   SITE   |INTENSITY
                                |              | MAG.(Mw) | ACCEL. g |MOD.MERC.
================================|==============|==========|==========|=========
ELSINORE (TEMECULA)             |  11.1( 17.8)|   6.8    |  0.301   |   IX
ELSINORE (JULIAN)               |  11.7( 18.9)|   7.1    |  0.340   |   IX
NEWPORT-INGLEWOOD (Offshore)    |  17.1( 27.5)|   7.1    |  0.241   |   IX
ROSE CANYON                     |  18.3( 29.5)|   7.2    |  0.240   |   IX
ELSINORE (GLEN IVY)             |  25.2( 40.6)|   6.8    |  0.134   |  VIII
SAN JACINTO-ANZA                |  33.5( 53.9)|   7.2    |  0.131   |  VIII
CORONADO BANK                   |  34.1( 54.9)|   7.6    |  0.171   |  VIII
SAN JACINTO-SAN JACINTO VALLEY  |  34.2( 55.1)|   6.9    |  0.104   |  VII
SAN JOAQUIN HILLS               |  34.7( 55.8)|   6.6    |  0.119   |  VII
EARTHQUAKE VALLEY               |  36.4( 58.5)|   6.5    |  0.075   |  VII
SAN JACINTO-COYOTE CREEK        |  41.0( 66.0)|   6.6    |  0.071   |  VI
CHINO-CENTRAL AVE. (Elsinore)   |  42.2( 67.9)|   6.7    |  0.103   |  VII
PALOS VERDES                    |  43.2( 69.5)|   7.3    |  0.108   |  VII
WHITTIER                        |  46.2( 74.4)|   6.8    |  0.071   |  VI
NEWPORT-INGLEWOOD (L.A.Basin)   |  47.2( 75.9)|   7.1    |  0.086   |  VII
SAN JACINTO-SAN BERNARDINO      |  49.5( 79.6)|   6.7    |  0.062   |  VI
SAN ANDREAS - Whole M-1a        |  53.1( 85.4)|   8.0    |  0.146   |  VIII
SAN ANDREAS - San Bernardino M-1|  53.1( 85.4)|   7.5    |  0.101   |  VII
SAN ANDREAS - SB-Coach. M-1b-2  |  53.1( 85.4)|   7.7    |  0.117   |  VII
SAN ANDREAS - SB-Coach. M-2b    |  53.1( 85.4)|   7.7    |  0.117   |  VII
ELSINORE (COYOTE MOUNTAIN)      |  53.4( 86.0)|   6.8    |  0.061   |  VI
PUENTE HILLS BLIND THRUST       |  58.2( 93.6)|   7.1    |  0.098   |  VII
SAN JACINTO - BORREGO           |  58.2( 93.6)|   6.6    |  0.049   |  VI
PINTO MOUNTAIN                  |  58.7( 94.5)|   7.2    |  0.073   |  VII
SAN ANDREAS - Coachella M-1c-5  |  59.8( 96.3)|   7.2    |  0.072   |  VI
*******************************************************************************









-END OF SEARCH-   25 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ELSINORE (TEMECULA)             FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 11.1 MILES (17.8 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.3400 g
```

MAXIMUM EARTHQUAKES

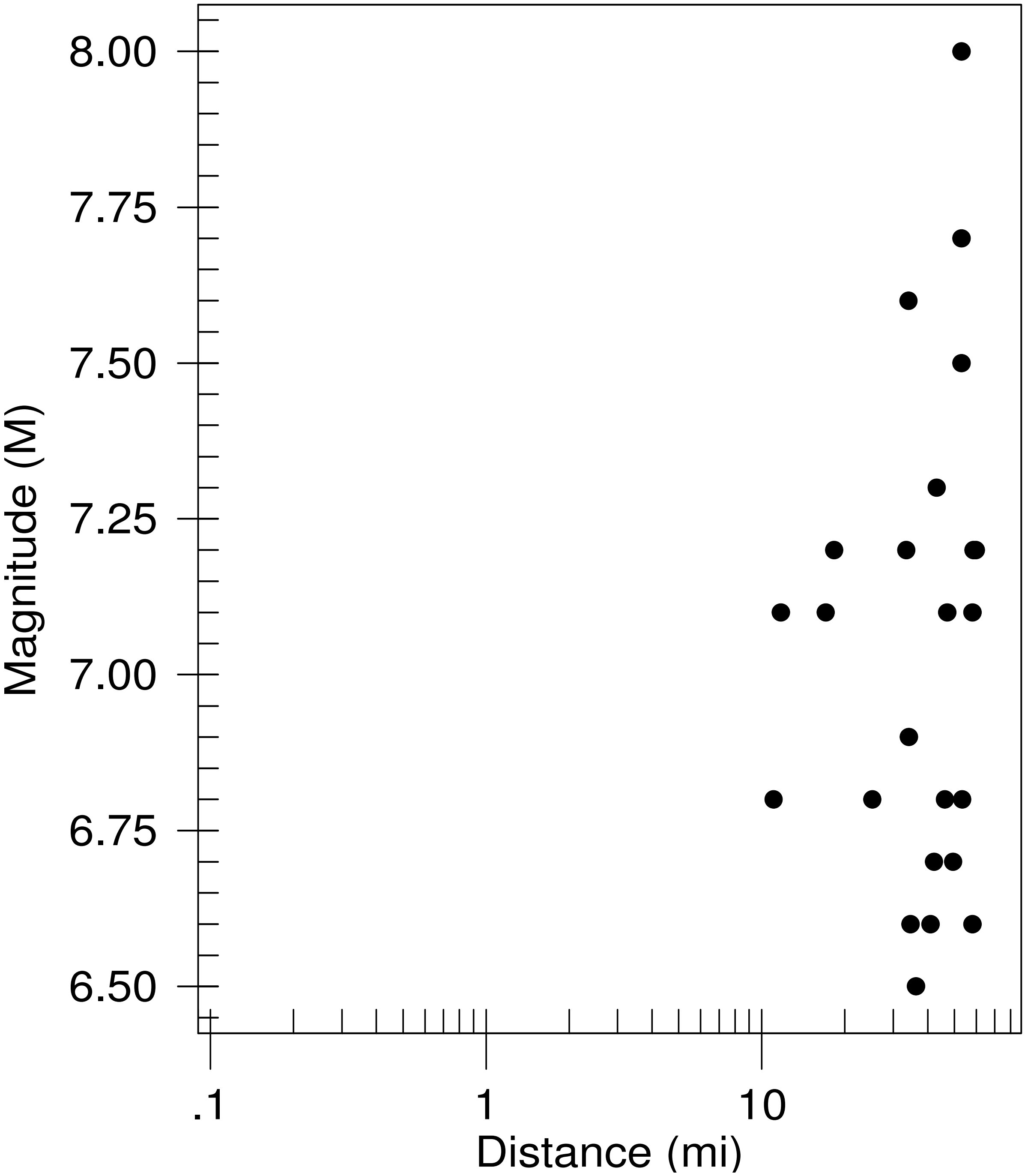
Vessels non-rock areas



W.O. 6688-A-SC
PLATE C-5

EARTHQUAKE MAGNITUDES & DISTANCES

Vessels non-rock areas



W.O. 6688-A-SC
PLATE C-6

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

JOB NUMBER: 6688
                                                     DATE: 01-27-2015

JOB NAME: Vessels  rock

CALCULATION NAME: Test Run Analysis

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


SITE COORDINATES:
   SITE LATITUDE:  33.3027
   SITE LONGITUDE:  117.1933

SEARCH RADIUS:   62.4  mi

ATTENUATION RELATION:  13) Bozorgnia Campbell Niazi (1999) Hor.-Hard Rock-Cor.
   UNCERTAINTY (M=Median, S=Sigma): S       Number of Sigmas:  1.0
   DISTANCE MEASURE:  cdist
   SCOND:   1
   Basement Depth:  .01 km      Campbell SSR:  0     Campbell SHR:  1
   COMPUTE PEAK HORIZONTAL ACCELERATION

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


MINIMUM DEPTH VALUE (km):  3.0
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                    EQFAULT SUMMARY
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              DETERMINISTIC SITE PARAMETERS
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Page  1
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                                |              |ESTIMATED MAX. EARTHQUAKE EVENT
                                | APPROXIMATE  |-------------------------------
          ABBREVIATED           |   DISTANCE   | MAXIMUM  |   PEAK   |EST. SITE
          FAULT  NAME           |  mi    (km)  |EARTHQUAKE|   SITE   |INTENSITY
                                |              | MAG.(Mw) | ACCEL. g |MOD.MERC.
================================|==============|==========|==========|=========
ELSINORE (TEMECULA)             |  11.1(  17.8)|   6.8    |  0.267   |   IX
ELSINORE (JULIAN)               |  11.7(  18.9)|   7.1    |  0.306   |   IX
NEWPORT-INGLEWOOD (Offshore)    |  17.1(  27.5)|   7.1    |  0.210   |  VIII
ROSE CANYON                     |  18.3(  29.5)|   7.2    |  0.210   |  VIII
ELSINORE (GLEN IVY)             |  25.2(  40.6)|   6.8    |  0.114   |  VII
SAN JACINTO-ANZA                |  33.5(  53.9)|   7.2    |  0.112   |  VII
CORONADO BANK                   |  34.1(  54.9)|   7.6    |  0.147   |  VIII
SAN JACINTO-SAN JACINTO VALLEY  |  34.2(  55.1)|   6.9    |  0.089   |  VII
SAN JOAQUIN HILLS               |  34.7(  55.8)|   6.6    |  0.102   |  VII
EARTHQUAKE VALLEY               |  36.4(  58.5)|   6.5    |  0.064   |  VI
SAN JACINTO-COYOTE CREEK        |  41.0(  66.0)|   6.6    |  0.060   |  VI
CHINO-CENTRAL AVE. (Elsinore)   |  42.2(  67.9)|   6.7    |  0.088   |  VII
PALOS VERDES                    |  43.2(  69.5)|   7.3    |  0.092   |  VII
WHITTIER                        |  46.2(  74.4)|   6.8    |  0.060   |  VI
NEWPORT-INGLEWOOD (L.A.Basin)   |  47.2(  75.9)|   7.1    |  0.073   |  VII
SAN JACINTO-SAN BERNARDINO      |  49.5(  79.6)|   6.7    |  0.053   |  VI
SAN ANDREAS - Whole M-1a        |  53.1(  85.4)|   8.0    |  0.125   |  VII
SAN ANDREAS - San Bernardino M-1|  53.1(  85.4)|   7.5    |  0.086   |  VII
SAN ANDREAS - SB-Coach. M-1b-2  |  53.1(  85.4)|   7.7    |  0.100   |  VII
SAN ANDREAS - SB-Coach. M-2b    |  53.1(  85.4)|   7.7    |  0.100   |  VII
ELSINORE (COYOTE MOUNTAIN)      |  53.4(  86.0)|   6.8    |  0.052   |  VI
PUENTE HILLS BLIND THRUST       |  58.2(  93.6)|   7.1    |  0.083   |  VII
SAN JACINTO - BORREGO           |  58.2(  93.6)|   6.6    |  0.041   |   V
PINTO MOUNTAIN                  |  58.7(  94.5)|   7.2    |  0.062   |  VI
SAN ANDREAS - Coachella M-1c-5  |  59.8(  96.3)|   7.2    |  0.061   |  VI
*******************************************************************************



-END OF SEARCH-   25 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ELSINORE (TEMECULA)              FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 11.1 MILES (17.8 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.3057 g
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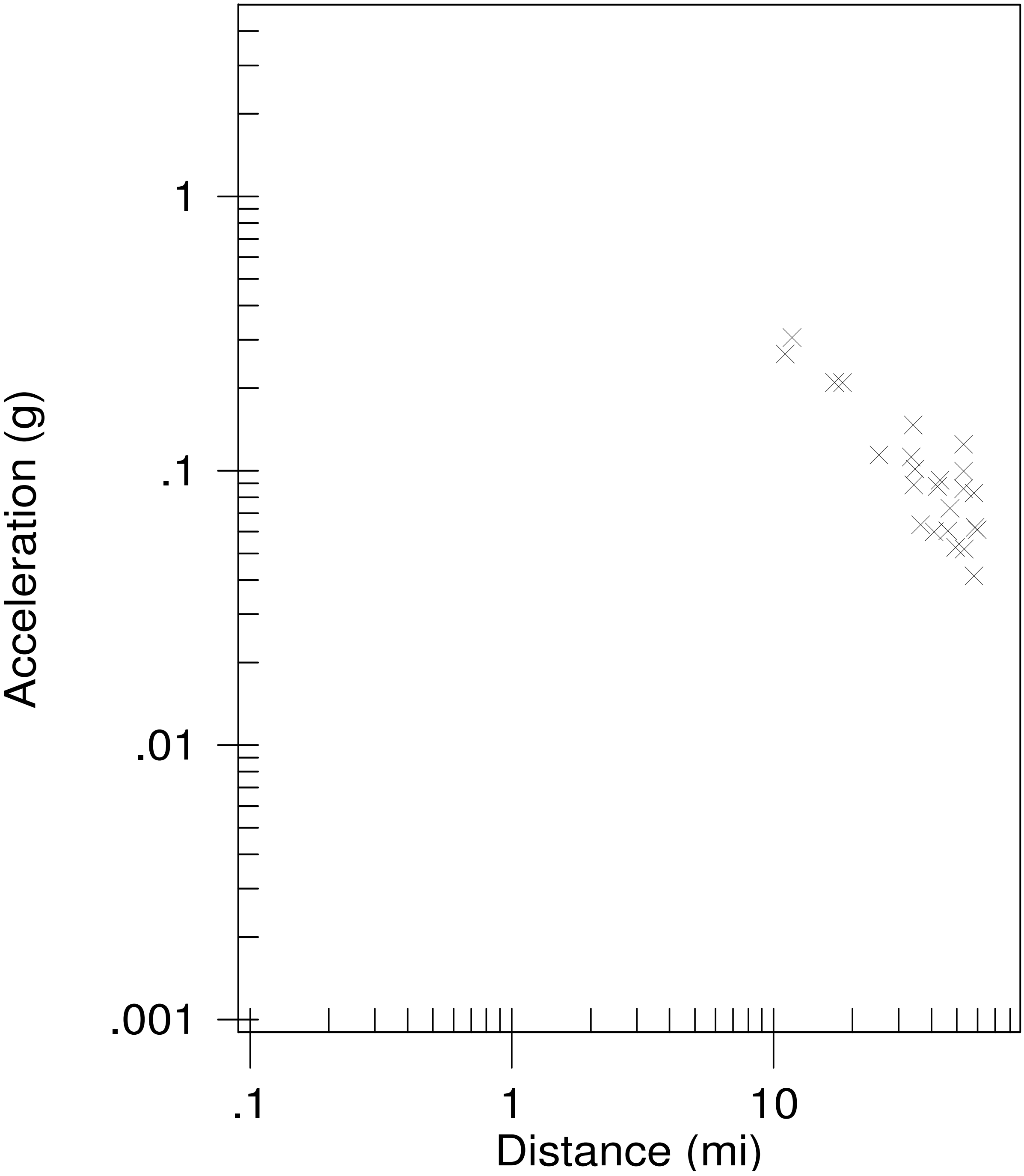
MAXIMUM EARTHQUAKES

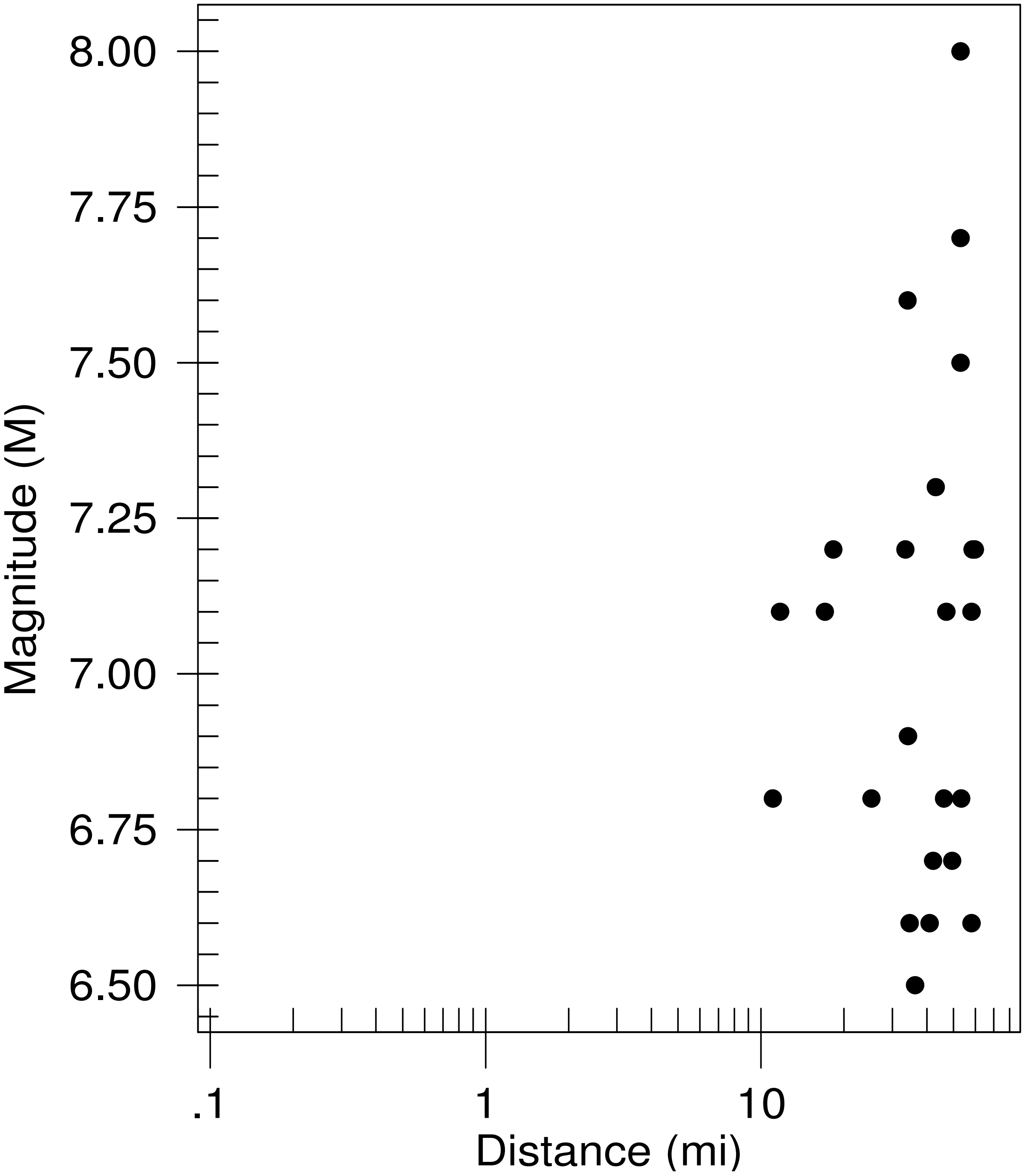
Vessels rock



W.O. 6688-A-SC
PLATE C-9

EARTHQUAKE MAGNITUDES & DISTANCES

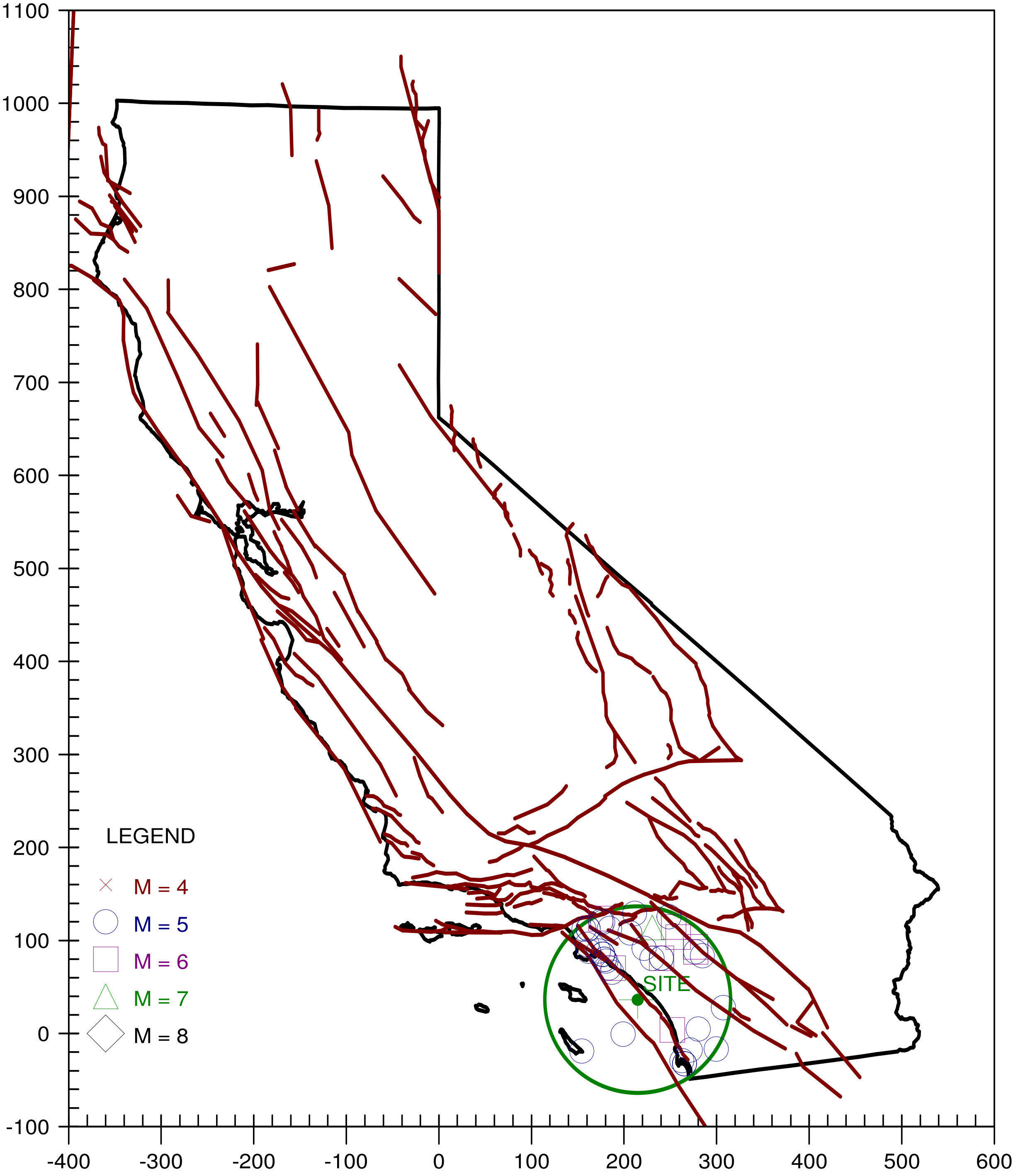
Vessels rock



W.O. 6688-A-SC
PLATE C-10

EARTHQUAKE EPICENTER MAP

vessels non rock



W.O. 6688-A-SC
PLATE C-11

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                           *    E Q S E A R C H    *
                           *                       *
                           *     Version 3.00      *
                           *                       *
                           *************************

                                 ESTIMATION OF
                            PEAK ACCELERATION FROM
                        CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 6688-A
                                                     DATE: 01-27-2015

JOB NAME: vessels non rock

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT


MAGNITUDE RANGE:
   MINIMUM MAGNITUDE:  5.00
   MAXIMUM MAGNITUDE:  9.00

SITE COORDINATES:
   SITE LATITUDE:  33.3027
   SITE LONGITUDE:  117.6917

SEARCH DATES:
           START DATE:   1800
           END DATE:   2014

SEARCH RADIUS:
           62.4 mi
           100.4 km

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

MINIMUM DEPTH VALUE (km):  3.0
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                         EARTHQUAKE SEARCH RESULTS
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Page  1
-------------------------------------------------------------------------------
    |       |        |          |  TIME    |     |     |  SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC)   |DEPTH|QUAKE|  ACC.  | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec  | (km)| MAG.|   g    |INT.|  mi  [km]
----+-------+--------+----------+----------+-----+-----+--------+----+-----------
PAS |32.9710|117.8700|07/13/1986|1347 8.2|  6.0| 5.30| 0.052  | VI | 25.1( 40.4)
DMG |33.5750|117.9830|03/11/1933| 518 4.0|  0.0| 5.20| 0.049  | VI | 25.2( 40.6)
DMG |33.6170|117.9670|03/11/1933| 154 7.8|  0.0| 6.30| 0.090  | VII| 26.9( 43.2)
DMG |33.6170|118.0170|03/14/1933|19 150.0|  0.0| 5.10| 0.040  |  V | 28.7( 46.1)
DMG |33.6990|117.5110|05/31/1938| 83455.4| 10.0| 5.50| 0.050  | VI | 29.3( 47.1)
DMG |33.0000|117.3000|11/22/1800|2130 0.0|  0.0| 6.50| 0.089  | VII| 30.8( 49.6)
DMG |33.7000|117.4000|05/15/1910|1547 0.0|  0.0| 6.00| 0.062  | VI | 32.2( 51.8)
DMG |33.7000|117.4000|04/11/1910| 757 0.0|  0.0| 5.00| 0.034  |  V | 32.2( 51.8)
DMG |33.7000|117.4000|05/13/1910| 620 0.0|  0.0| 5.00| 0.034  |  V | 32.2( 51.8)
DMG |33.6830|118.0500|03/11/1933| 658 3.0|  0.0| 5.50| 0.043  | VI | 33.4( 53.7)
MGI |33.8000|117.6000|04/22/1918|2115 0.0|  0.0| 5.00| 0.031  |  V | 34.7( 55.9)
DMG |33.7000|118.0670|03/11/1933| 85457.0|  0.0| 5.10| 0.033  |  V | 34.9( 56.2)
DMG |33.7000|118.0670|03/11/1933| 51022.0|  0.0| 5.10| 0.033  |  V | 34.9( 56.2)
DMG |33.7500|118.0830|03/11/1933| 323 0.0|  0.0| 5.00| 0.028  |  V | 38.2( 61.5)
DMG |33.7500|118.0830|03/11/1933| 2 9 0.0|  0.0| 5.00| 0.028  |  V | 38.2( 61.5)
DMG |33.7500|118.0830|03/11/1933| 230 0.0|  0.0| 5.10| 0.030  |  V | 38.2( 61.5)
DMG |33.7500|118.0830|03/13/1933|131828.0|  0.0| 5.30| 0.033  |  V | 38.2( 61.5)
DMG |33.7500|118.0830|03/11/1933| 910 0.0|  0.0| 5.10| 0.030  |  V | 38.2( 61.5)
DMG |33.7830|118.1330|10/02/1933| 91017.6|  0.0| 5.40| 0.032  |  V | 41.8( 67.2)
GSG |33.9530|117.7610|07/29/2008|184215.7| 14.0| 5.30| 0.028  |  V | 45.1( 72.5)
MGI |33.0000|117.0000|09/21/1856| 730 0.0|  0.0| 5.00| 0.024  | IV | 45.1( 72.6)
DMG |33.7830|118.2500|11/14/1941| 84136.3|  0.0| 5.40| 0.029  |  V | 46.2( 74.3)
MGI |32.8000|117.1000|05/25/1803| 0 0 0.0|  0.0| 5.00| 0.022  | IV | 48.8( 78.5)
MGI |34.0000|117.5000|12/16/1858|10 0 0.0|  0.0| 7.00| 0.076  | VII| 49.4( 79.5)
DMG |33.9000|117.2000|12/19/1880| 0 0 0.0|  0.0| 6.00| 0.039  |  V | 50.0( 80.5)
DMG |33.8500|118.2670|03/11/1933|1425 0.0|  0.0| 5.00| 0.021  | IV | 50.2( 80.8)
DMG |33.7500|117.0000|04/21/1918|223225.0|  0.0| 6.80| 0.065  | VI | 50.4( 81.1)
DMG |33.7500|117.0000|06/06/1918|2232 0.0|  0.0| 5.00| 0.021  | IV | 50.4( 81.1)
DMG |32.7000|117.2000|05/27/1862|20 0 0.0|  0.0| 5.90| 0.036  |  V | 50.4( 81.1)
DMG |32.8170|118.3500|12/26/1951| 04654.0|  0.0| 5.90| 0.036  |  V | 50.7( 81.7)
MGI |34.0000|118.0000|12/25/1903|1745 0.0|  0.0| 5.00| 0.021  | IV | 51.3( 82.6)
DMG |33.7100|116.9250|09/23/1963|144152.6| 16.5| 5.00| 0.020  | IV | 52.3( 84.2)
DMG |33.8000|117.0000|12/25/1899|1225 0.0|  0.0| 6.40| 0.048  | VI | 52.6( 84.6)
T-A |32.6700|117.1700|10/21/1862| 0 0 0.0|  0.0| 5.00| 0.020  | IV | 53.1( 85.5)
T-A |32.6700|117.1700|05/24/1865| 0 0 0.0|  0.0| 5.00| 0.020  | IV | 53.1( 85.5)
T-A |32.6700|117.1700|12/00/1856| 0 0 0.0|  0.0| 5.00| 0.020  | IV | 53.1( 85.5)
DMG |34.0000|117.2500|07/23/1923| 73026.0|  0.0| 6.25| 0.042  |  V | 54.4( 87.6)
PAS |34.0610|118.0790|10/01/1987|144220.0|  9.5| 5.90| 0.032  |  V | 56.9( 91.5)
DMG |33.2000|116.7000|01/01/1920| 235 0.0|  0.0| 5.00| 0.018  | IV | 57.7( 92.8)
GSP |34.1400|117.7000|02/28/1990|234336.6|  5.0| 5.20| 0.020  | IV | 57.8( 93.0)
T-A |34.0000|118.2500|01/10/1856| 0 0 0.0|  0.0| 5.00| 0.018  | IV | 57.9( 93.1)
T-A |34.0000|118.2500|09/23/1827| 0 0 0.0|  0.0| 5.00| 0.018  | IV | 57.9( 93.1)
T-A |34.0000|118.2500|03/26/1860| 0 0 0.0|  0.0| 5.00| 0.018  | IV | 57.9( 93.1)
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PAS |34.0730|118.0980|10/04/1987|105938.2|  8.2| 5.30| 0.022 | IV | 58.1( 93.5)
MGI |34.1000|117.3000|07/15/1905|2041 0.0|  0.0| 5.30| 0.021 | IV | 59.5( 95.7)
MGI |34.0000|118.3000|09/03/1905| 540 0.0|  0.0| 5.30| 0.021 | IV | 59.5( 95.7)
MGI |34.1000|118.1000|07/11/1855| 415 0.0|  0.0| 6.30| 0.039 |  V | 59.8( 96.3)
DMG |32.8000|116.8000|10/23/1894|23 3 0.0|  0.0| 5.70| 0.025 |  V | 62.2(100.1)

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-END OF SEARCH-   48 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH:   1800  TO  2014

LENGTH OF SEARCH TIME:   215  years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 25.1 MILES (40.4 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.090 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
  a-value=  1.062
  b-value=  0.392
  beta-value=  0.902


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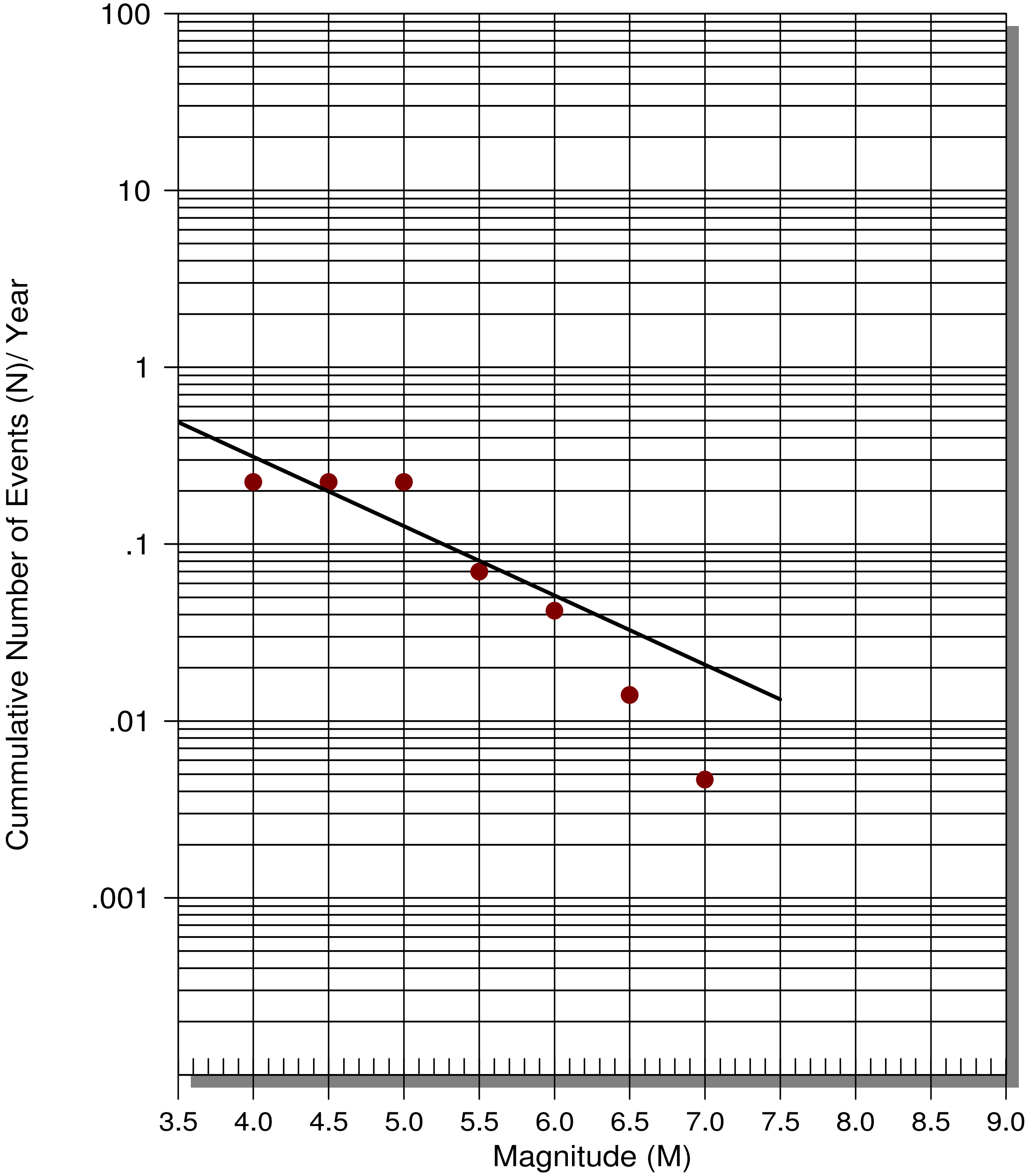
TABLE OF MAGNITUDES AND EXCEEDANCES:
------------------------------------

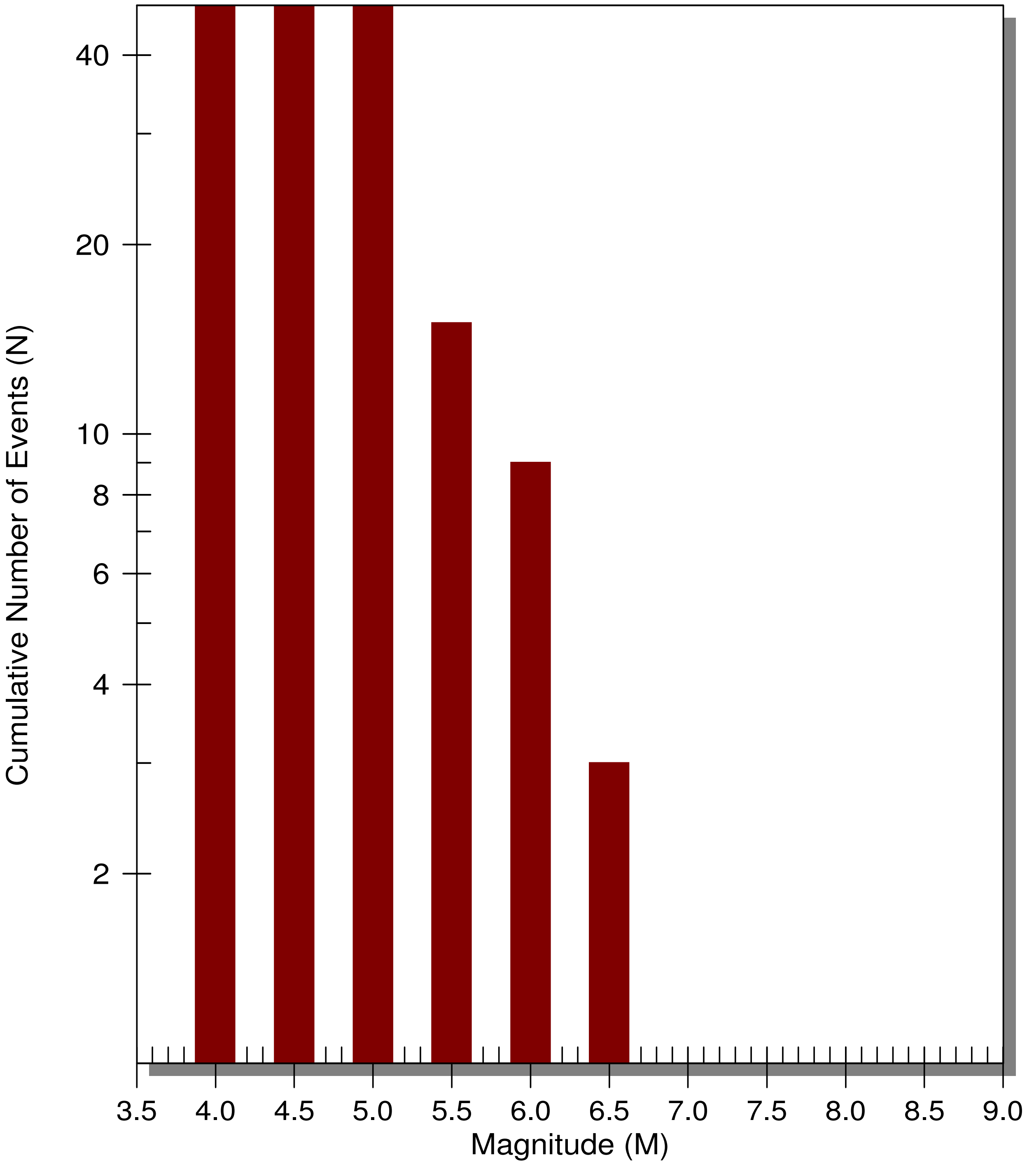
  Earthquake | Number of Times | Cumulative
   Magnitude |    Exceeded     | No. / Year
  -----------+-----------------+-----------
     4.0     |       48        |   0.22326
     4.5     |       48        |   0.22326
     5.0     |       48        |   0.22326
     5.5     |       15        |   0.06977
     6.0     |        9        |   0.04186
     6.5     |        3        |   0.01395
     7.0     |        1        |   0.00465
```

EARTHQUAKE RECURRENCE CURVE

vessels non rock



W.O. 6688-A-SC
PLATE C-15



Number of Earthquakes (N) Above Magnitude (M)

vessels non rock

W.O. 6688-A-SC
PLATE C-16

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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: 6688-A
                                                      DATE: 01-27-2015

JOB NAME: vessels rock

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT


MAGNITUDE RANGE:
   MINIMUM MAGNITUDE:  5.00
   MAXIMUM MAGNITUDE:  9.00

SITE COORDINATES:
   SITE LATITUDE:  33.3027
   SITE LONGITUDE:  117.6917

SEARCH DATES:
           START DATE:   1800
           END DATE:   2014

SEARCH RADIUS:
           62.4 mi
           100.4 km

ATTENUATION RELATION:  13) Bozorgnia Campbell Niazi (1999) Hor.-Hard Rock-Cor.
   UNCERTAINTY (M=Median, S=Sigma): S       Number of Sigmas:  1.0
   ASSUMED SOURCE TYPE:  SS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
   SCOND:   1  Depth Source:  A
   Basement Depth:  .01 km      Campbell SSR:  0     Campbell SHR:  1
   COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km):  3.0
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                         EARTHQUAKE SEARCH RESULTS
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Page  1
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    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   | (UTC)  |DEPTH|QUAKE| ACC.  | MM | DISTANCE
CODE| NORTH |  WEST  |          |H M Sec | (km)| MAG.|   g   |INT.| mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
PAS |32.9710|117.8700|07/13/1986|1347 8.2|  6.0| 5.30| 0.044 | VI | 25.1( 40.4)
DMG |33.5750|117.9830|03/11/1933| 518 4.0|  0.0| 5.20| 0.041 |  V | 25.2( 40.6)
DMG |33.6170|117.9670|03/11/1933| 154 7.8|  0.0| 6.30| 0.077 | VII| 26.9( 43.2)
DMG |33.6170|118.0170|03/14/1933|19 150.0|  0.0| 5.10| 0.034 |  V | 28.7( 46.1)
DMG |33.6990|117.5110|05/31/1938| 83455.4| 10.0| 5.50| 0.042 | VI | 29.3( 47.1)
DMG |33.0000|117.3000|11/22/1800|2130 0.0|  0.0| 6.50| 0.076 | VII| 30.8( 49.6)
DMG |33.7000|117.4000|05/15/1910|1547 0.0|  0.0| 6.00| 0.052 | VI | 32.2( 51.8)
DMG |33.7000|117.4000|04/11/1910| 757 0.0|  0.0| 5.00| 0.029 |  V | 32.2( 51.8)
DMG |33.7000|117.4000|05/13/1910| 620 0.0|  0.0| 5.00| 0.029 |  V | 32.2( 51.8)
DMG |33.6830|118.0500|03/11/1933| 658 3.0|  0.0| 5.50| 0.037 |  V | 33.4( 53.7)
MGI |33.8000|117.6000|04/22/1918|2115 0.0|  0.0| 5.00| 0.026 |  V | 34.7( 55.9)
DMG |33.7000|118.0670|03/11/1933| 85457.0|  0.0| 5.10| 0.028 |  V | 34.9( 56.2)
DMG |33.7000|118.0670|03/11/1933| 51022.0|  0.0| 5.10| 0.028 |  V | 34.9( 56.2)
DMG |33.7500|118.0830|03/11/1933| 323 0.0|  0.0| 5.00| 0.024 | IV | 38.2( 61.5)
DMG |33.7500|118.0830|03/11/1933| 2 9 0.0|  0.0| 5.00| 0.024 | IV | 38.2( 61.5)
DMG |33.7500|118.0830|03/11/1933| 230 0.0|  0.0| 5.10| 0.025 |  V | 38.2( 61.5)
DMG |33.7500|118.0830|03/13/1933|131828.0|  0.0| 5.30| 0.028 |  V | 38.2( 61.5)
DMG |33.7500|118.0830|03/11/1933| 910 0.0|  0.0| 5.10| 0.025 |  V | 38.2( 61.5)
DMG |33.7830|118.1330|10/02/1933| 91017.6|  0.0| 5.40| 0.027 |  V | 41.8( 67.2)
GSG |33.9530|117.7610|07/29/2008|184215.7| 14.0| 5.30| 0.024 | IV | 45.1( 72.5)
MGI |33.0000|117.0000|09/21/1856| 730 0.0|  0.0| 5.00| 0.020 | IV | 45.1( 72.6)
DMG |33.7830|118.2500|11/14/1941| 84136.3|  0.0| 5.40| 0.025 |  V | 46.2( 74.3)
MGI |32.8000|117.1000|05/25/1803| 0 0 0.0|  0.0| 5.00| 0.019 | IV | 48.8( 78.5)
MGI |34.0000|117.5000|12/16/1858|10 0 0.0|  0.0| 7.00| 0.065 | VI | 49.4( 79.5)
DMG |33.9000|117.2000|12/19/1880| 0 0 0.0|  0.0| 6.00| 0.033 |  V | 50.0( 80.5)
DMG |33.8500|118.2670|03/11/1933|1425 0.0|  0.0| 5.00| 0.018 | IV | 50.2( 80.8)
DMG |33.7500|117.0000|04/21/1918|223225.0|  0.0| 6.80| 0.055 | VI | 50.4( 81.1)
DMG |33.7500|117.0000|06/06/1918|2232 0.0|  0.0| 5.00| 0.018 | IV | 50.4( 81.1)
DMG |32.7000|117.2000|05/27/1862|20 0 0.0|  0.0| 5.90| 0.031 |  V | 50.4( 81.1)
DMG |32.8170|118.3500|12/26/1951| 04654.0|  0.0| 5.90| 0.030 |  V | 50.7( 81.7)
MGI |34.0000|118.0000|12/25/1903|1745 0.0|  0.0| 5.00| 0.018 | IV | 51.3( 82.6)
DMG |33.7100|116.9250|09/23/1963|144152.6| 16.5| 5.00| 0.017 | IV | 52.3( 84.2)
DMG |33.8000|117.0000|12/25/1899|1225 0.0|  0.0| 6.40| 0.040 |  V | 52.6( 84.6)
T-A |32.6700|117.1700|10/21/1862| 0 0 0.0|  0.0| 5.00| 0.017 | IV | 53.1( 85.5)
T-A |32.6700|117.1700|05/24/1865| 0 0 0.0|  0.0| 5.00| 0.017 | IV | 53.1( 85.5)
T-A |32.6700|117.1700|12/00/1856| 0 0 0.0|  0.0| 5.00| 0.017 | IV | 53.1( 85.5)
DMG |34.0000|117.2500|07/23/1923| 73026.0|  0.0| 6.25| 0.035 |  V | 54.4( 87.6)
PAS |34.0610|118.0790|10/01/1987|144220.0|  9.5| 5.90| 0.027 |  V | 56.9( 91.5)
DMG |33.2000|116.7000|01/01/1920| 235 0.0|  0.0| 5.00| 0.016 | IV | 57.7( 92.8)
GSP |34.1400|117.7000|02/28/1990|234336.6|  5.0| 5.20| 0.017 | IV | 57.8( 93.0)
T-A |34.0000|118.2500|01/10/1856| 0 0 0.0|  0.0| 5.00| 0.015 | IV | 57.9( 93.1)
T-A |34.0000|118.2500|09/23/1827| 0 0 0.0|  0.0| 5.00| 0.015 | IV | 57.9( 93.1)
T-A |34.0000|118.2500|03/26/1860| 0 0 0.0|  0.0| 5.00| 0.015 | IV | 57.9( 93.1)
```
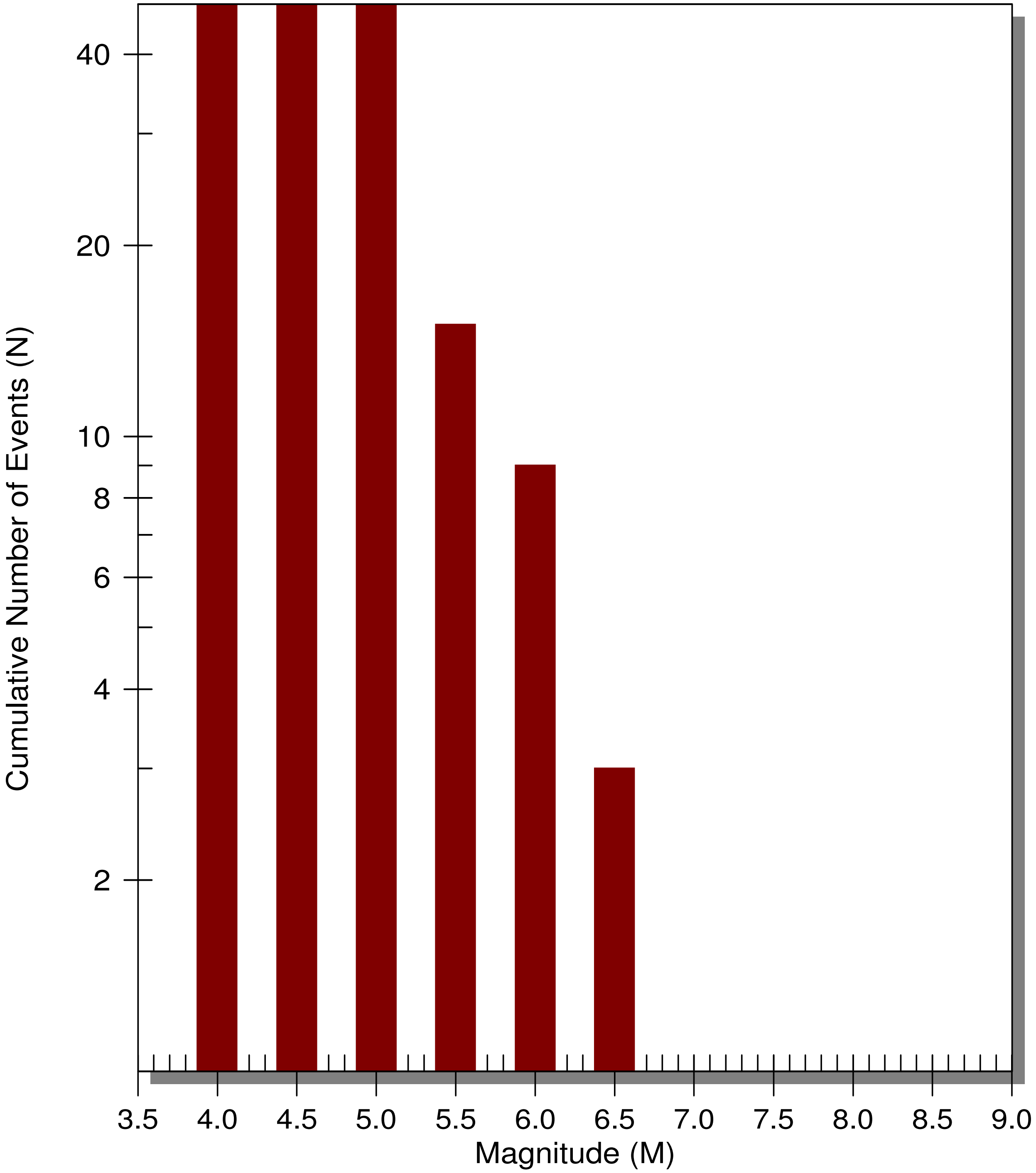

```
PAS |34.0730|118.0980|10/04/1987|105938.2|  8.2| 5.30| 0.018 | IV | 58.1( 93.5)
MGI |34.1000|117.3000|07/15/1905|2041 0.0|  0.0| 5.30| 0.018 | IV | 59.5( 95.7)
MGI |34.0000|118.3000|09/03/1905| 540 0.0|  0.0| 5.30| 0.018 | IV | 59.5( 95.7)
MGI |34.1000|118.1000|07/11/1855| 415 0.0|  0.0| 6.30| 0.033 |  V | 59.8( 96.3)
DMG |32.8000|116.8000|10/23/1894|23 3 0.0|  0.0| 5.70| 0.022 | IV | 62.2(100.1)

*******************************************************************************







-END OF SEARCH-   48 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH:   1800  TO  2014

LENGTH OF SEARCH TIME:   215  years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 25.1 MILES (40.4 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.077 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
  a-value=  1.062
  b-value=  0.392
  beta-value=  0.902


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

  Earthquake | Number of Times | Cumulative
   Magnitude |    Exceeded     | No. / Year
  -----------+-----------------+------------
     4.0     |       48        |   0.22326
     4.5     |       48        |   0.22326
     5.0     |       48        |   0.22326
     5.5     |       15        |   0.06977
     6.0     |        9        |   0.04186
     6.5     |        3        |   0.01395
     7.0     |        1        |   0.00465
```

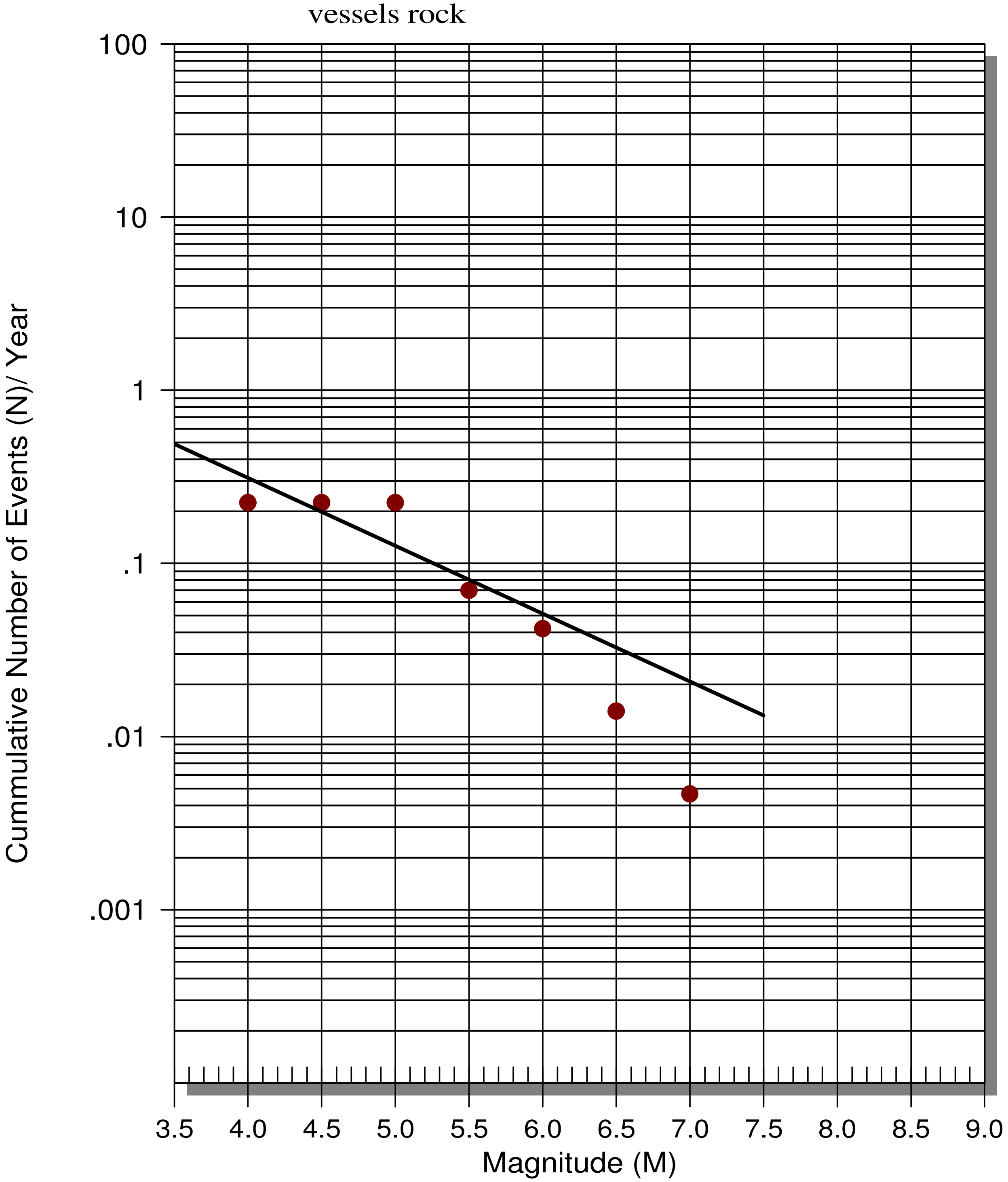
Number of Earthquakes (N) Above Magnitude (M)

vessels rock



W.O. 6688-A-SC
PLATE C-20

EARTHQUAKE RECURRENCE CURVE

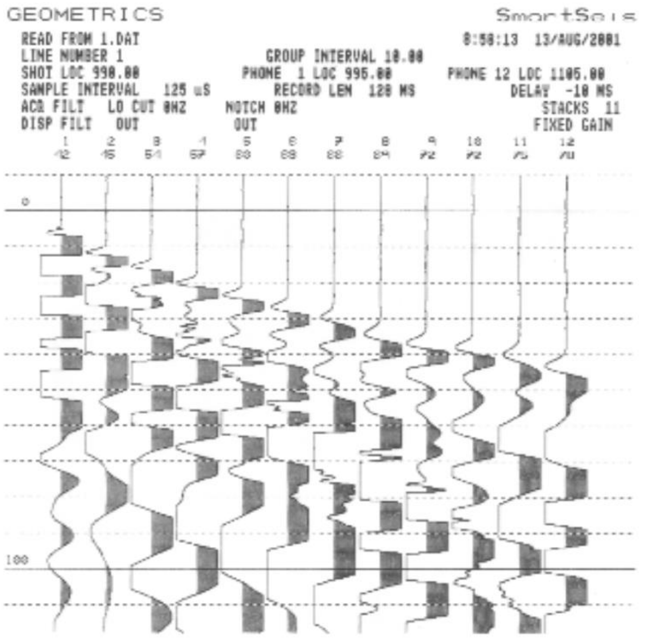


W.O. 6688-A-SC
PLATE C-21

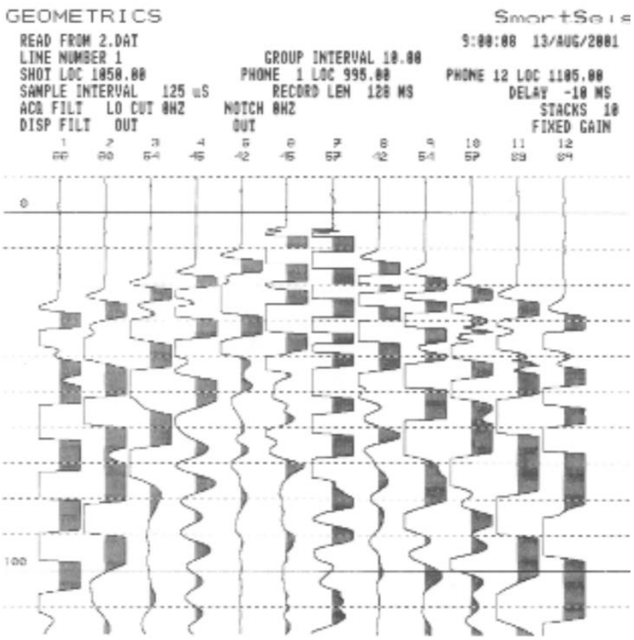
APPENDIX D

ROCK HARDNESS REFRACTION SURVEY

Example
Raw Seismic Data

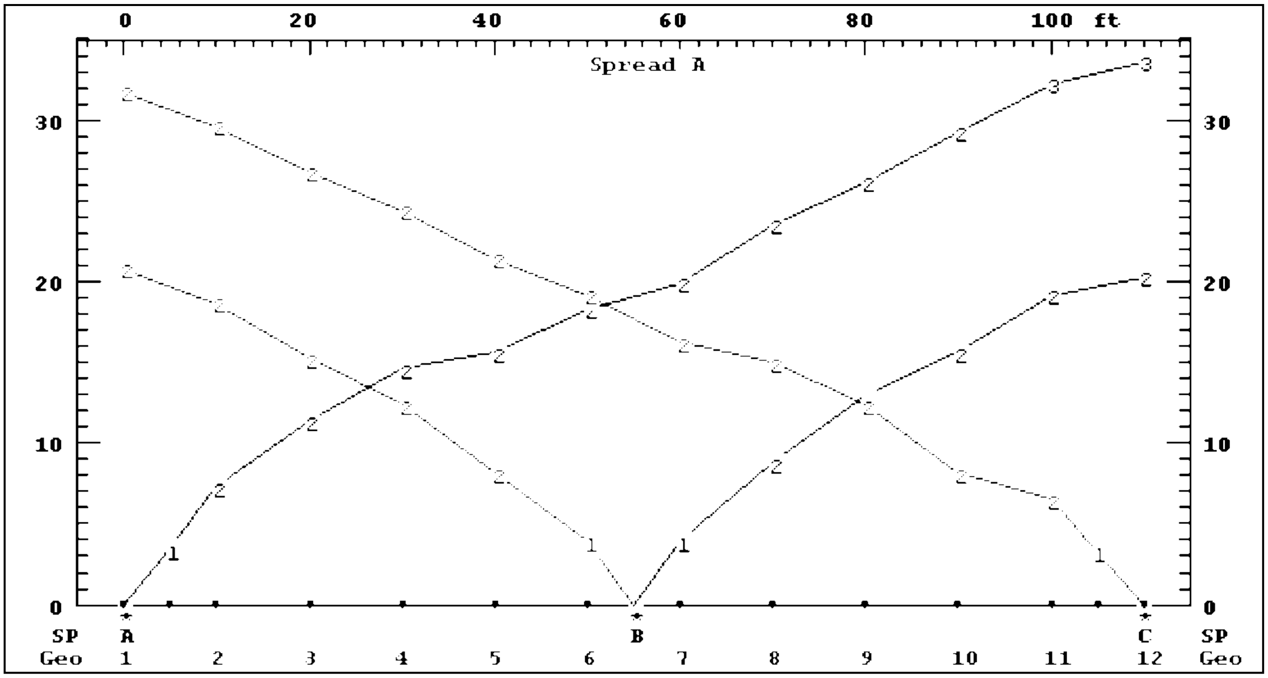


```
GEOMETRICS                                                    SmartSeis
READ FROM 1.DAT                                          8:58:13  13/AUG/2001
LINE NUMBER 1                  GROUP INTERVAL 10.00
SHOT LOC 990.00              PHONE  1 LOC 995.00       PHONE 12 LOC 1105.00
SAMPLE INTERVAL   125 uS       RECORD LEN  128 MS              DELAY  -10 MS
ACQ FILT   LO CUT 0HZ     NOTCH 0HZ                              STACKS  11
DISP FILT   OUT            OUT                                  FIXED GAIN
```

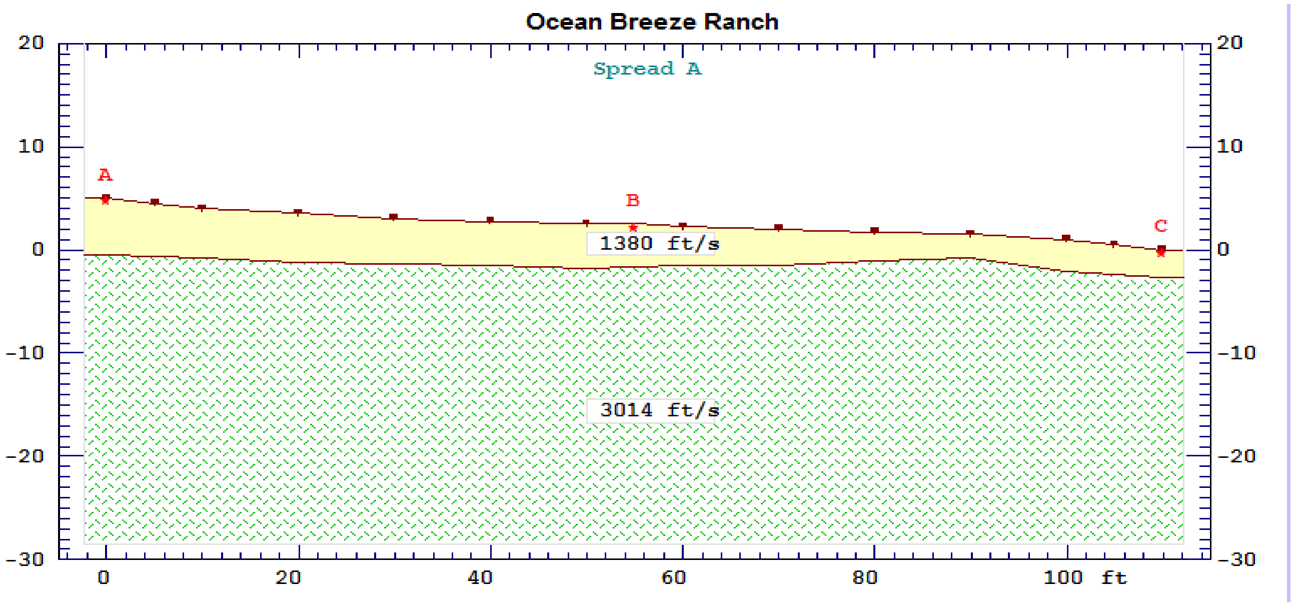


```
GEOMETRICS                                                    SmartSe
READ FROM 2.DAT                                          9:00:08  13/AUG/2001
LINE NUMBER 1                  GROUP INTERVAL 10.00
SHOT LOC 1050.00             PHONE  1 LOC 995.00       PHONE 12 LOC 1105.00
SAMPLE INTERVAL   125 uS       RECORD LEN  128 MS              DELAY  -10 MS
ACQ FILT   LO CUT 0HZ     NOTCH 0HZ                              STACKS  10
DISP FILT   OUT            OUT                                  FIXED GAIN
```

Example
Seismic Line



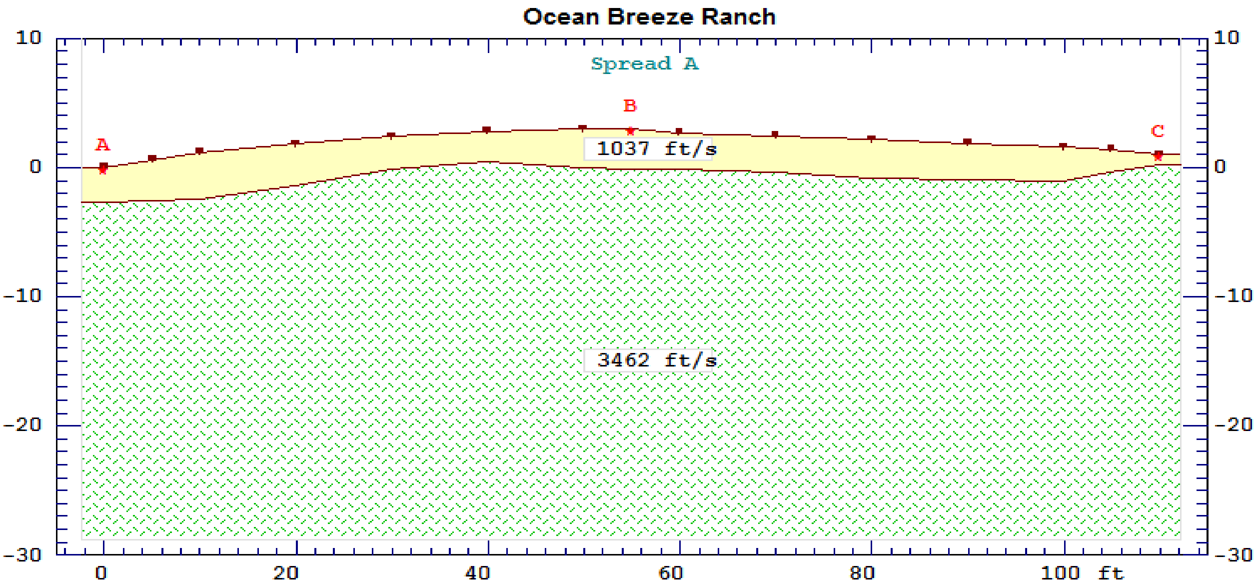
Seismic Line
ST-101



South

North

Seismic Line
ST-102



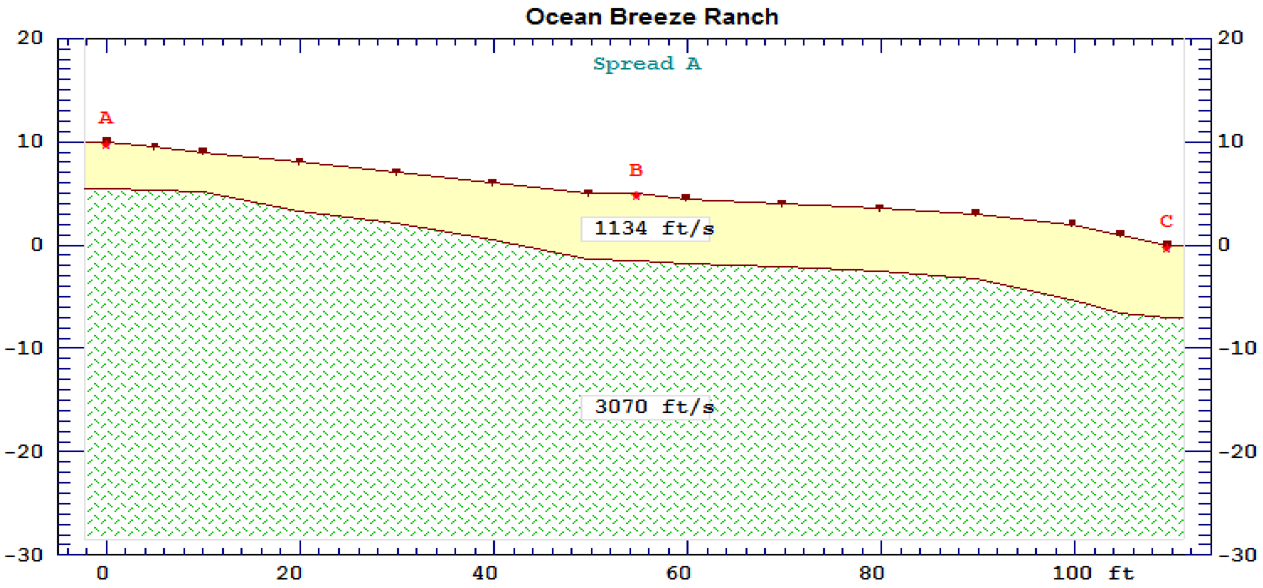
North

South

Ocean Breeze Ranch
Bonsall, California

W.O. 6960-A-SC
Plate D-4

Seismic Line
ST-103

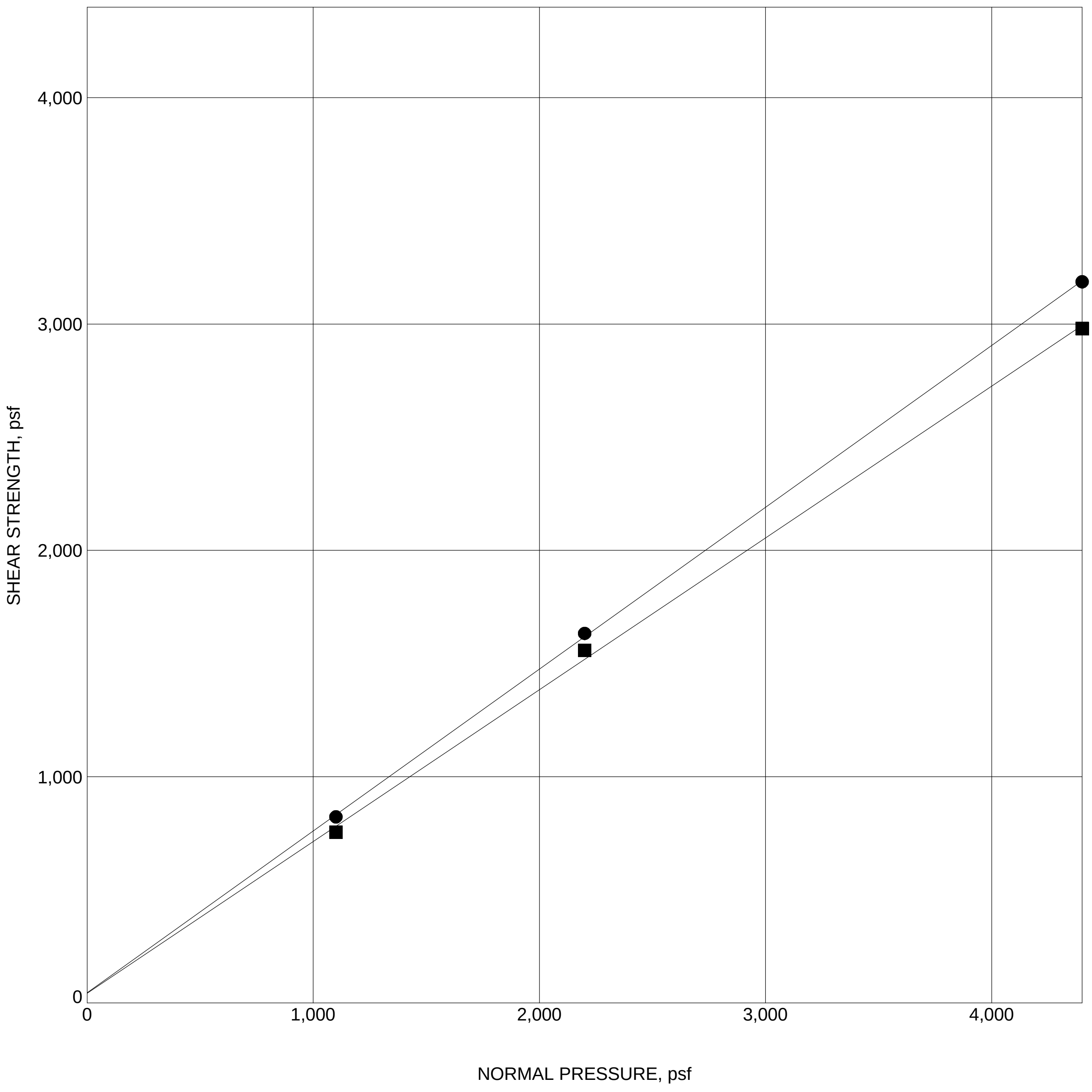


East

West

APPENDIX E

LABORATORY DATA
(CURRENT STUDY AND GSI, 2015)



	Sample	Depth/El.	Primary/Residual Shear	Sample Type	γ_d	MC%	c	ϕ
●	TP-102	4.0	Primary Shear	Remolded	121.5	8.5	45	36
■	TP-102	4.1	Residual Shear	Remolded	121.5	8.5	42	34

Note: Sample Innundated prior to testing

US DIRECT SHEAR 6960.GPJ US LAB.GDT 10/10/16

GeoSoils, Inc.

GeoSoils, Inc.
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Carlsbad, CA 92010
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Fax: 760-931-0915

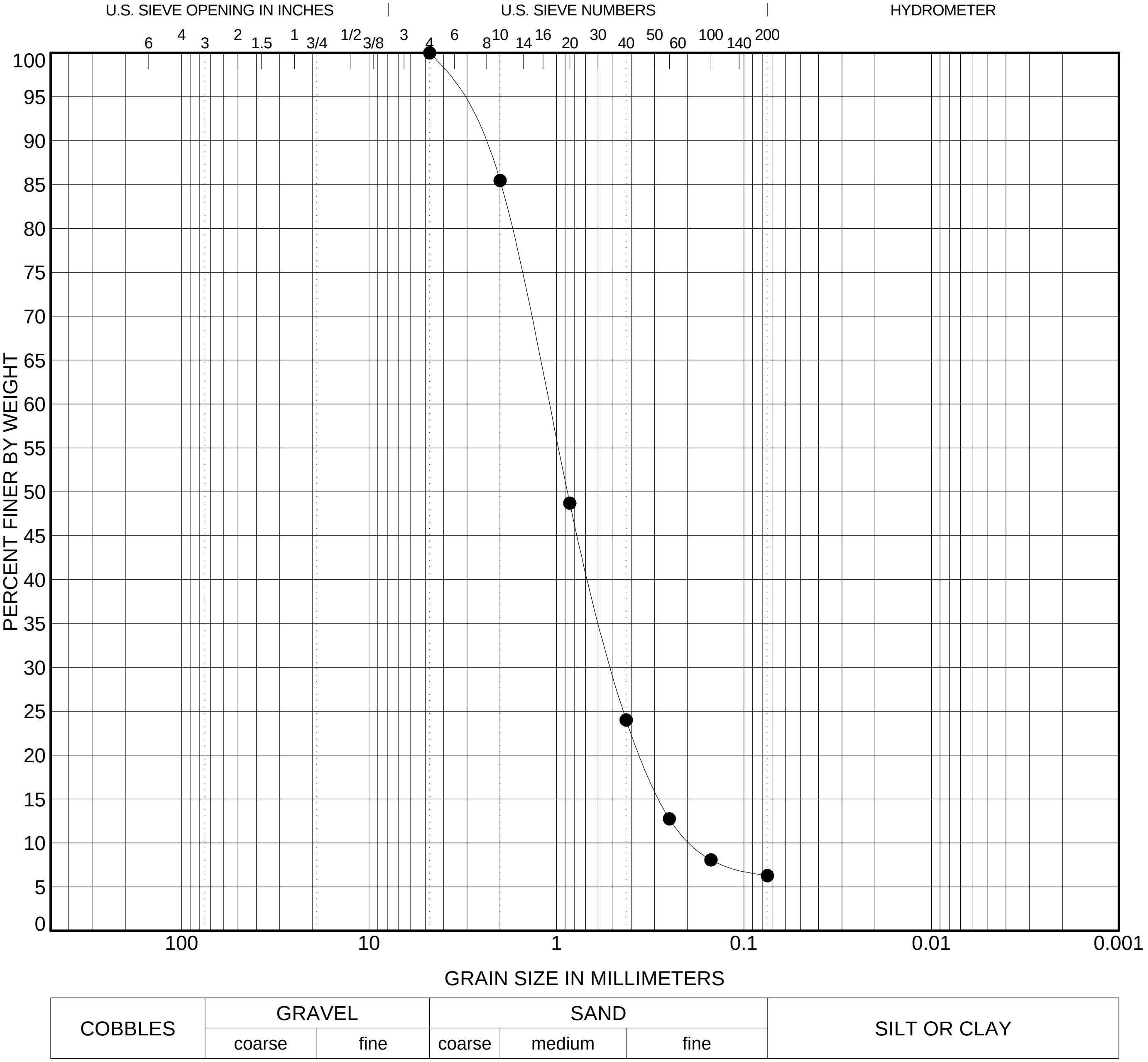
DIRECT SHEAR TEST

Project: Ocean Breeze Ranch

Number: 6960-A-SC

Date: October 2016

Figure: E - 1



	Sample	Depth	Classification	LL	PL	PI	Cc	Cu
●	HSA-3	20.0	Silty Sand				1.23	5.97

	Sample	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	HSA-3	20.0	4.75	1.106	0.503	0.185	0.0	93.7	6.3	

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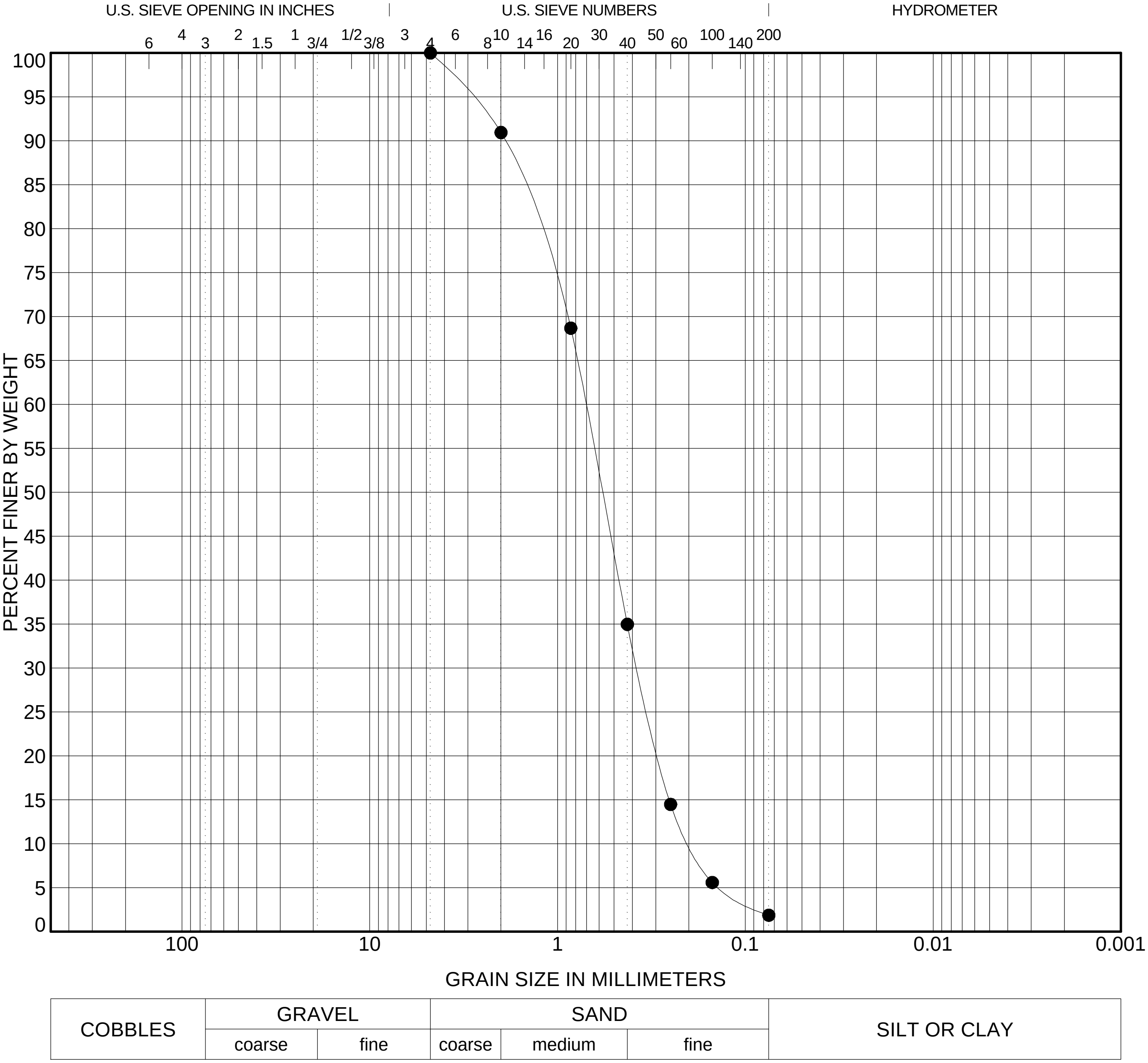
GRAIN SIZE DISTRIBUTION

Project: Ocean Breeze Ranch

Number: 6960-A-SC

Date: October 2016

Figure: E - 2



	Sample	Depth	Classification	LL	PL	PI	Cc	Cu
●	HSA-4	40.0	POORLY GRADED SAND(SP)				1.02	3.68

	Sample	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	HSA-4	40.0	4.75	0.711	0.374	0.193	0.0	98.1	1.9	

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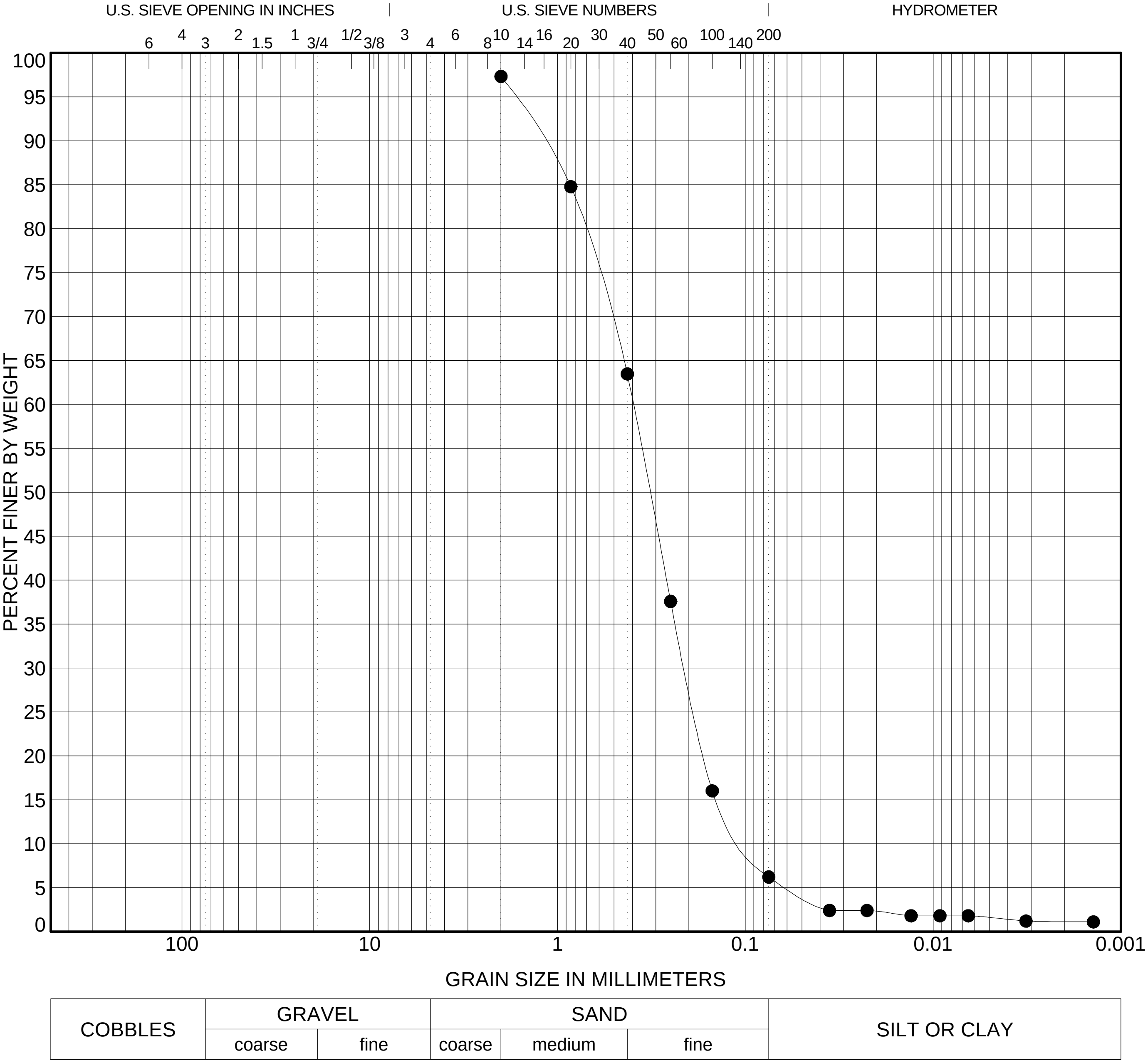
Project: Ocean Breeze Ranch

Number: 6960-A-SC

Date: October 2016

Figure: E - 3



	Sample	Depth	Classification	LL	PL	PI	Cc	Cu
●	HSA-5	10.0	Silty Sand				1.12	4.04

	Sample	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	HSA-5	10.0	2	0.396	0.209	0.098	0.0	91.1	4.6	1.6

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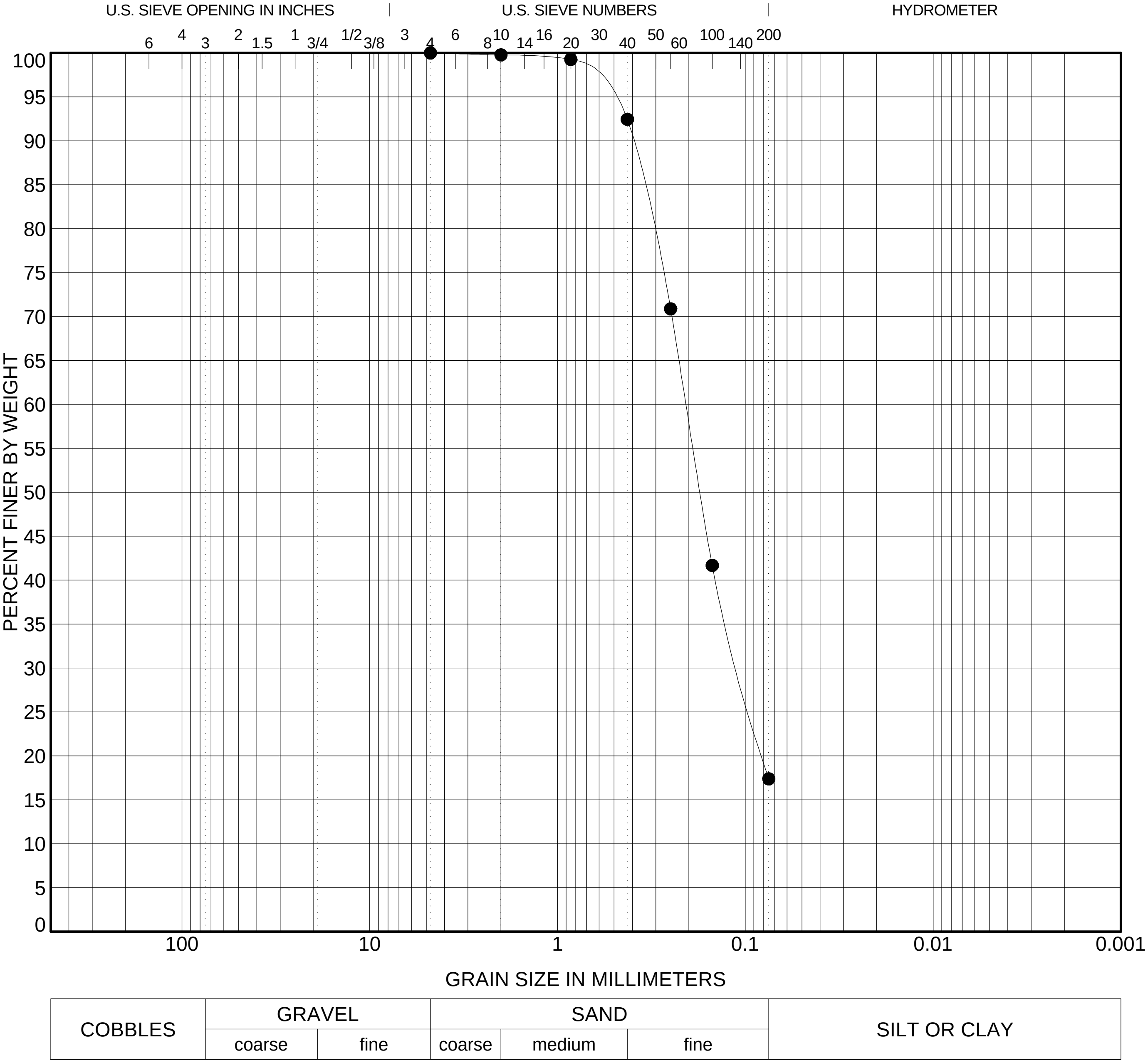
GRAIN SIZE DISTRIBUTION

Project: Ocean Breeze Ranch

Number: 6960-A-SC

Date: October 2016

Figure: E - 4



	Sample	Depth	Classification	LL	PL	PI	Cc	Cu
●	HSA-5	35.0						

	Sample	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	HSA-5	35.0	4.75	0.207	0.107		0.0	82.6	17.4	

US GRAIN SIZE 6960.GPJ US LAB.GDT 10/6/16

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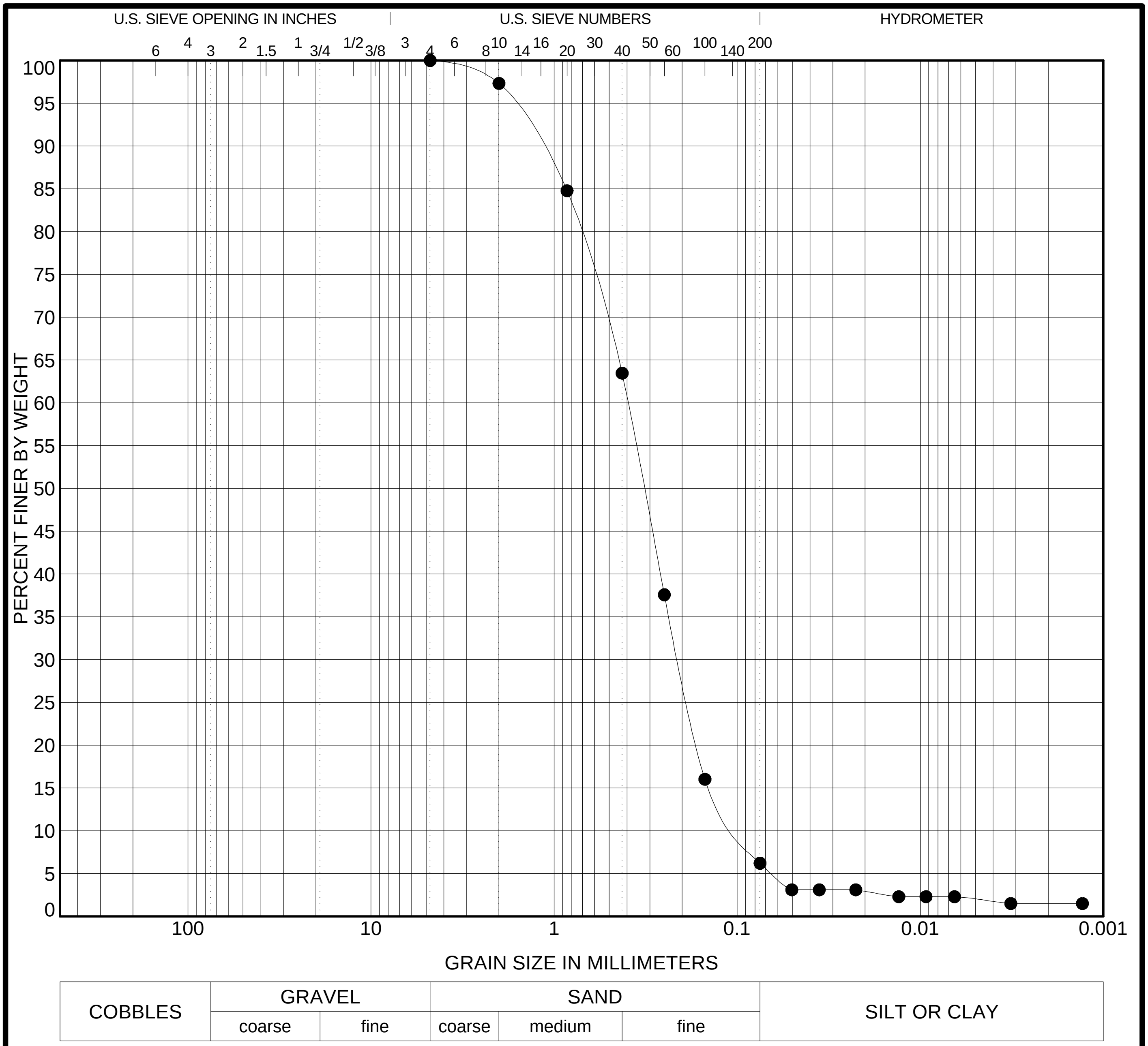
Number: 6960-A-SC

Date: October 2016

Figure: E - 5



	Sample	Depth	Classification	LL	PL	PI	Cc	Cu
●	HSA-6	20.0					1.12	4.04

	Sample	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	HSA-6	20.0	4.75	0.396	0.209	0.098	0.0	93.8	4.2	2.0

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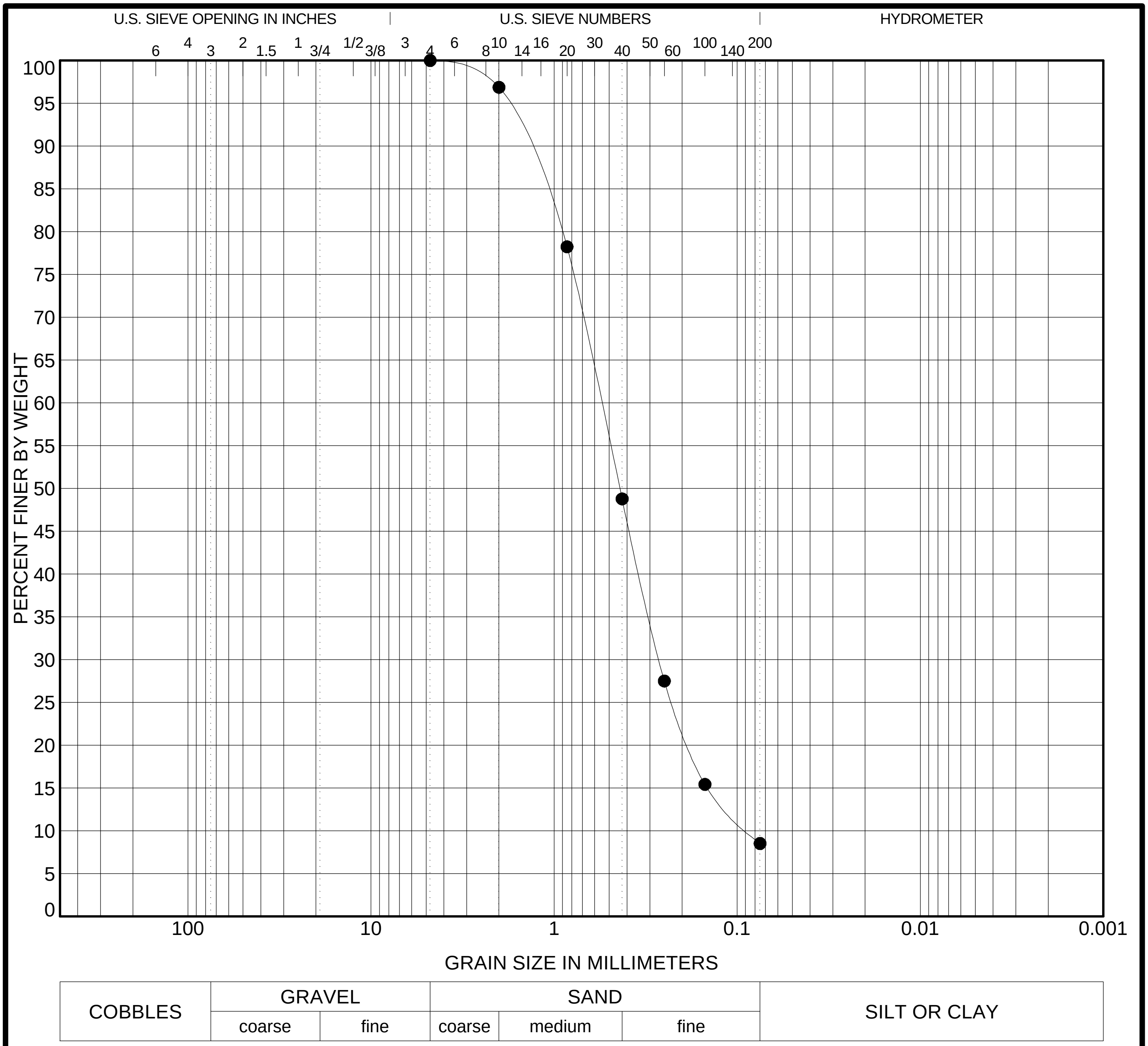
Number: 6960-A-SC

Date: October 2016

Figure: E - 6



	Sample	Depth	Classification	LL	PL	PI	Cc	Cu
●	HSA-6	30.0					1.47	6.36

	Sample	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	HSA-6	30.0	4.75	0.553	0.266	0.087	0.0	91.5	8.5	

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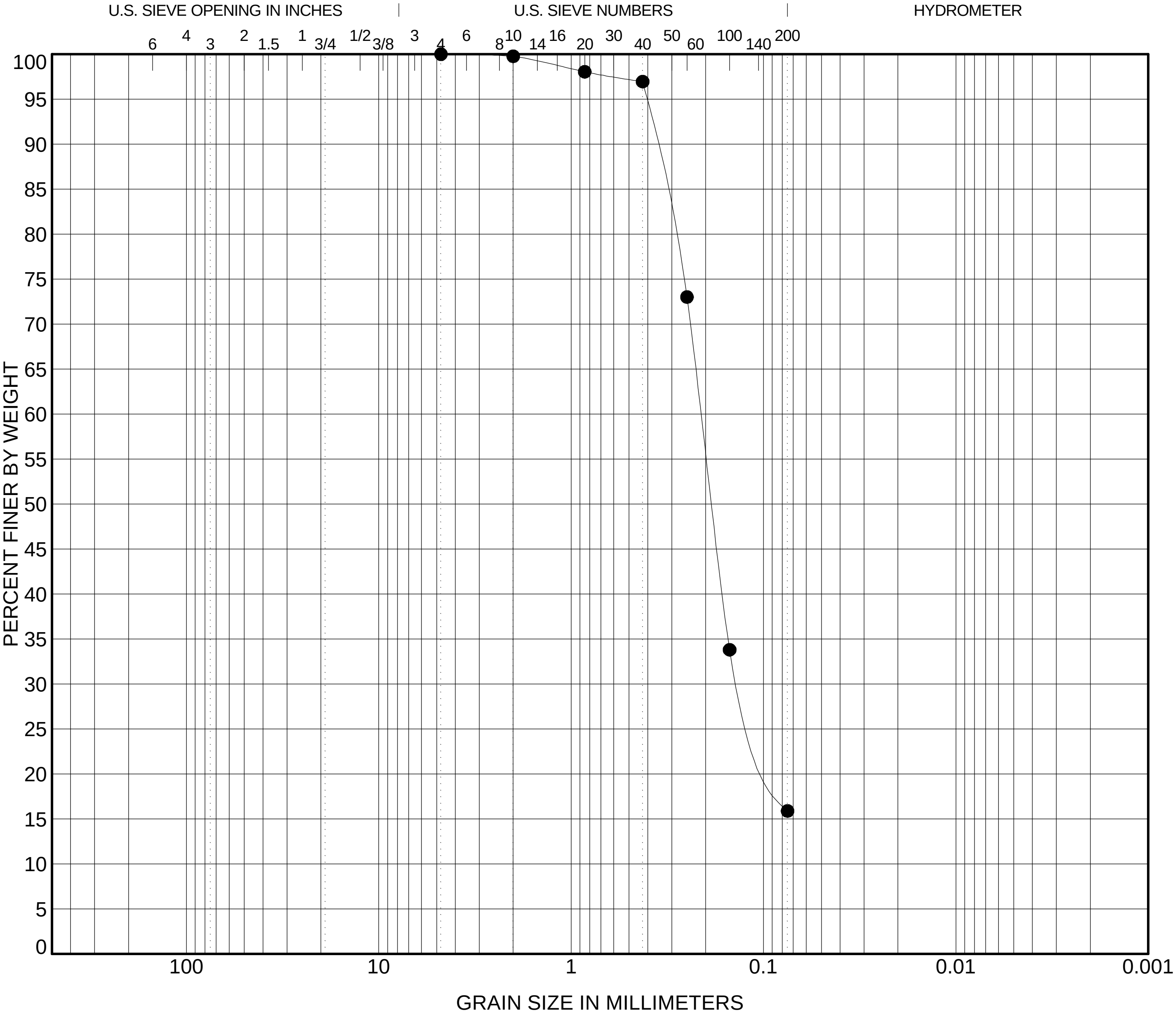
Number: 6960-A-SC

Date: October 2016

Figure: E - 7



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample	Depth	Classification	LL	PL	PI	Cc	Cu
●	HSA-6	40.0						

	Sample	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	HSA-6	40.0	4.75	0.211	0.129		0.0	84.1	15.9	

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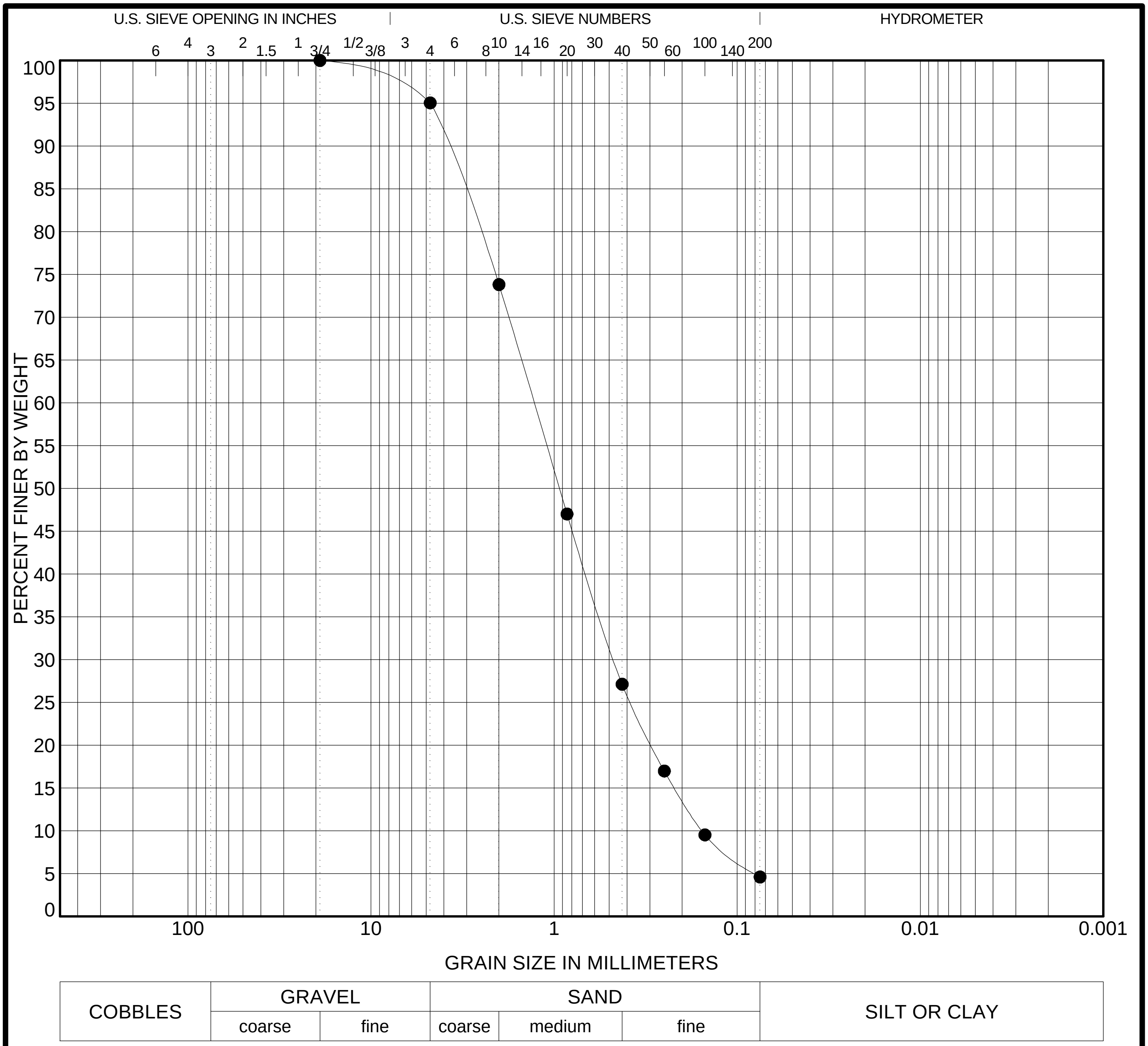
GRAIN SIZE DISTRIBUTION

Project: Ocean Breeze Ranch

Number: 6960-A-SC

Date: October 2016

Figure: E - 8



	Sample	Depth	Classification	LL	PL	PI	Cc	Cu
●	HSA- 9	15.0	Sand w/ Silt				1.11	8.30

	Sample	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	HSA- 9	15.0	19	1.287	0.47	0.155	5.0	90.4	4.6	

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GRAIN SIZE DISTRIBUTION

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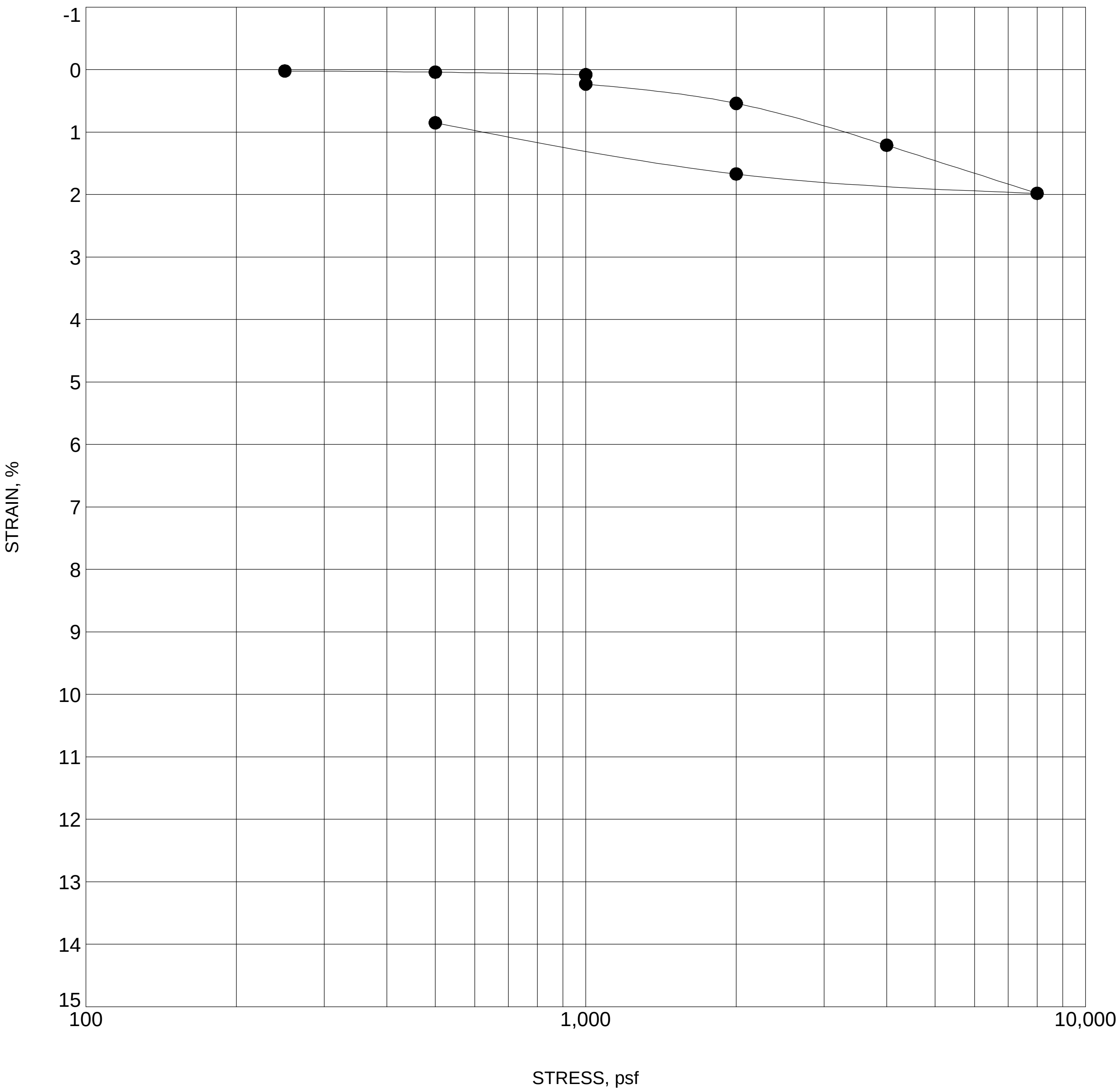
Number: 6960-A-SC

Date: October 2016

Figure: E - 9



	Sample	Depth/El.	Visual Classification	γ_d Initial	MC Initial	MC Final	H20
●	HSA- 8	5.0	Silty Sand	105.0	2.8	18.8	1000

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CONSOLIDATION TEST

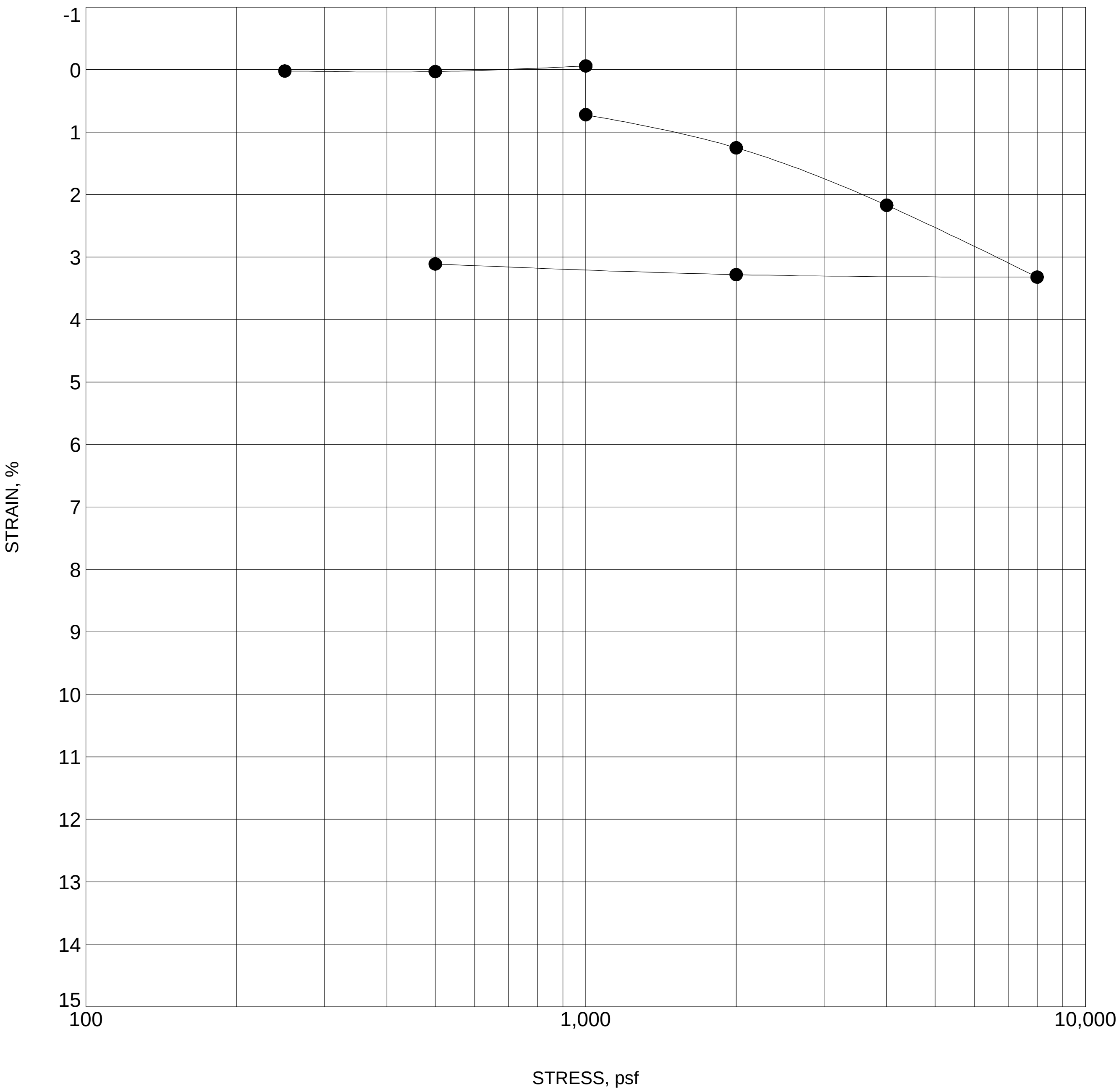
Project: OBR, LLC.

Number: 6960-A-SC

Date: October 2016

Figure: E - 10

US CONSOL STRAIN 6960.MUR.GPJ US LAB.GDT 10/6/16



	Sample	Depth/El.	Visual Classification	γ_d Initial	MC Initial	MC Final	H20
●	HSA- 9	15.0	Sand w/ Silt	104.1	4.1	18.1	1000

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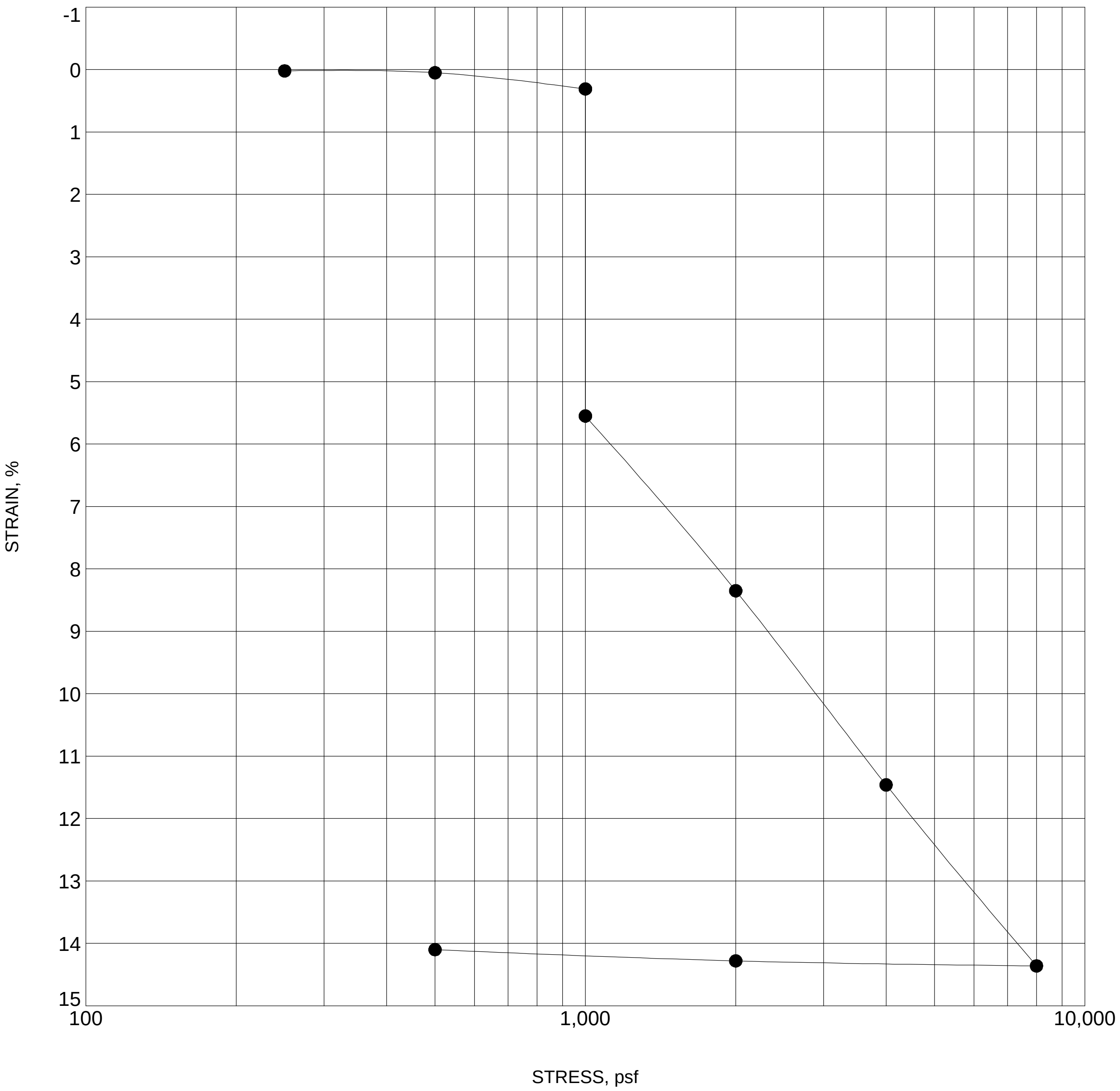
CONSOLIDATION TEST

Project: OBR, LLC.

Number: 6960-A-SC

Date: October 2016

Figure: E - 11



Sample		Depth/El.	Visual Classification	γ_d Initial	MC Initial	MC Final	H20
●	HSA-11	5.0	Clayey Sand	100.9	3.3	14.2	1000

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GeoSoils, Inc.
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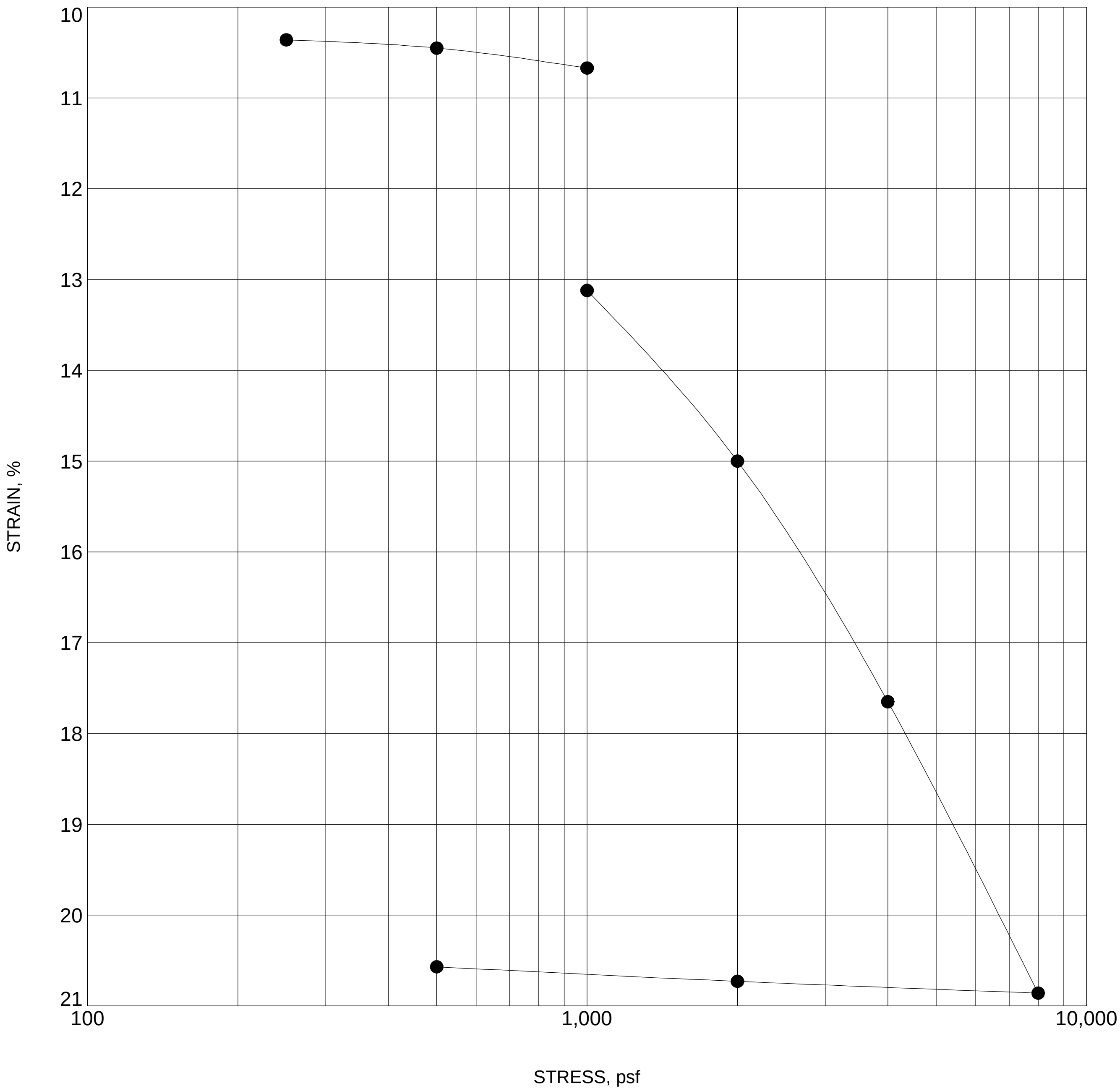
CONSOLIDATION TEST

Project: OBR, LLC.

Number: 6960-A-SC

Date: October 2016

Figure: E - 12



Sample		Depth/El.	Visual Classification	γ_d Initial	MC Initial	MC Final	H20
●	HSA-11	10.0		103.6	11.6		1000

US CONSOL STRAIN 6960.GPJ US LAB.GDT 10/6/16

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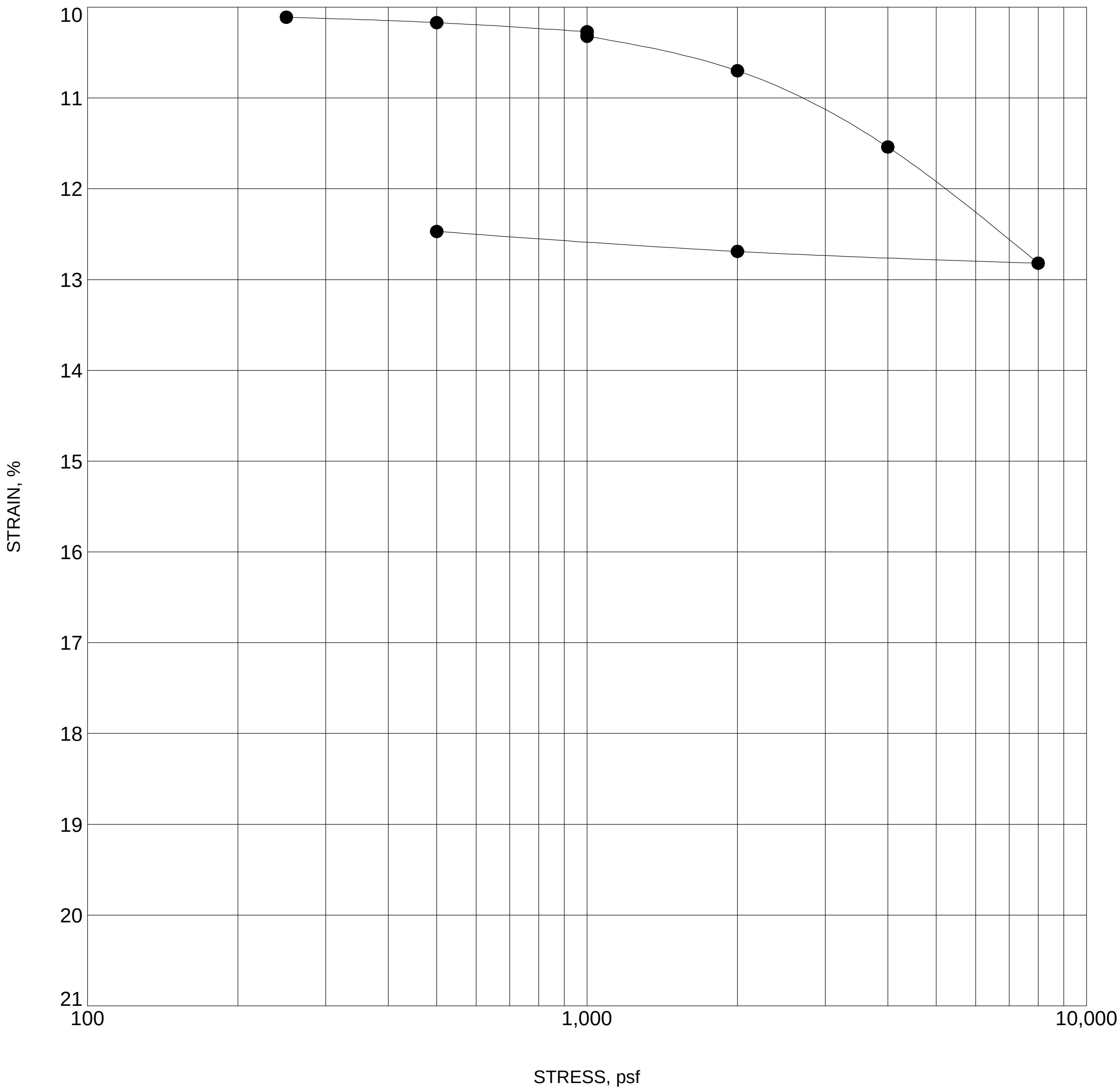
CONSOLIDATION TEST

Project: Ocean Breeze Ranch

Number: 6960-A-SC

Date: October 2016

Figure: E - 13



	Sample	Depth/El.	Visual Classification	γ_d Initial	MC Initial	MC Final	H20
●	HSA-5	10.0	Silty Sand	92.8	11.6	22.7	1000

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CONSOLIDATION TEST

Project: Ocean Breeze Ranch

Number: 6960-A-SC

Date: October 2016

Figure: E - 14

US CONSOL STRAIN 6960.GPJ US LAB.GDT 10/6/16

TEST SPECIMEN

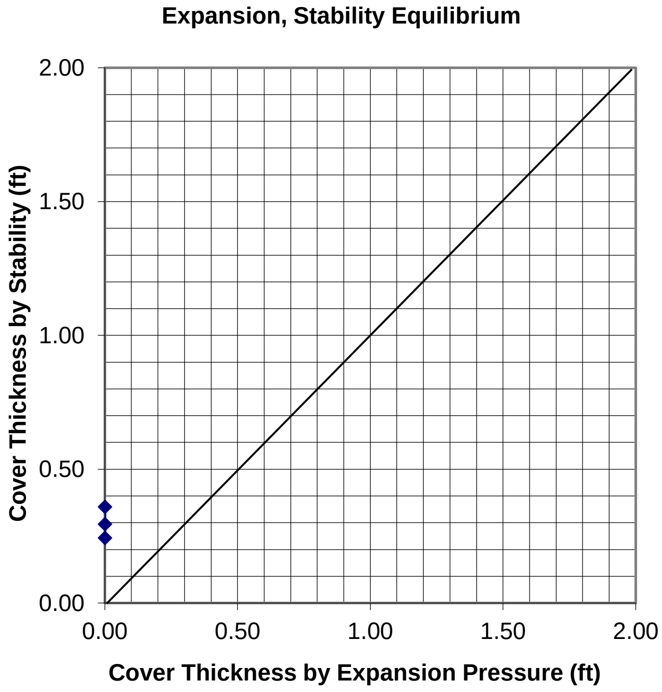
		A	B	C	D
Compactor air pressure	**PSI**	350	350	350	
Water added	**%**	0.6	1.4	1.9	
Moisture at compaction	**%**	8.2	9.0	9.5	
Height of sample	**IN**	2.54	2.51	2.55	
Dry density	**PCF**	126.4	125.4	124.3	
R-Value by exudation		81	77	72	
R-Value by exudation, corrected		81	77	72	
Exudation pressure	**PSI**	600	415	216	
Stability thickness	**FT**	0.24	0.29	0.36	
Expansion pressure thickness	**FT**	0.00	0.00	0.00	

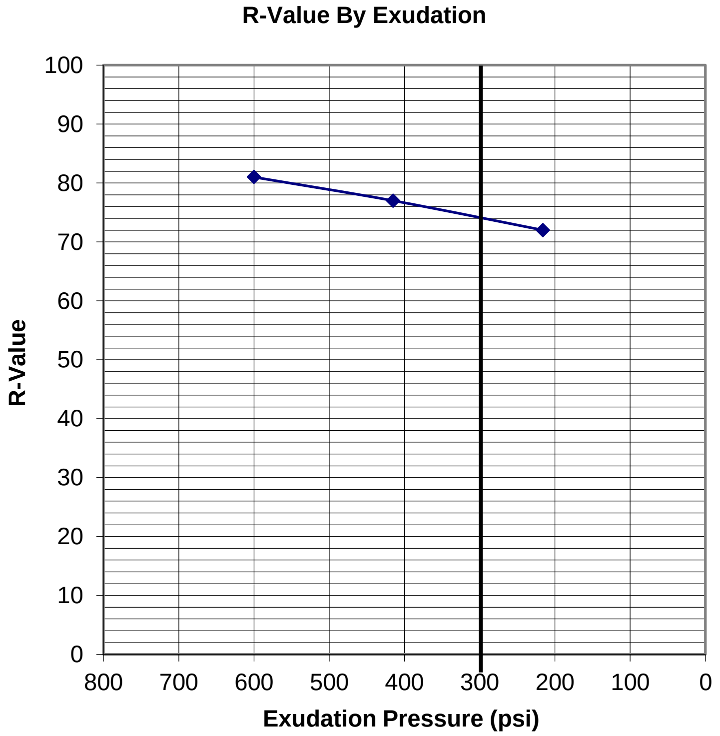
DESIGN CALCULATION DATA

Traffic index, assumed	5.0
Gravel equivalent factor, assumed	1.25
Expansion, stability equilibrium	0
R-Value by expansion	NA
R-Value by exudation	74
R-Value at equilibrium	**74**

SAMPLE INFORMATION

Sample Location: TP-102 @ 3-4ft.
Sample Description: Yellow Brown Silty Sand
Notes: Ocean Breeze Ranch
0% Retained on 3/4 inch sieve
Test Method: Cal-Trans Test 301





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

9/2/2010

R - VALUE TEST RESULTS

Project: Ocean Breeze Ranch

Number: 6960-A-SC

Date: October 2016 Plate: E-15

SUMMARY OF LABORATORY TEST DATA

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

QCI Project No.: 16-029-008d
Date: August 17, 2016
Summarized by: DM

W.O. 6960-A-SC
Project Name: Ocean Breeze Ranch
Client: N/A

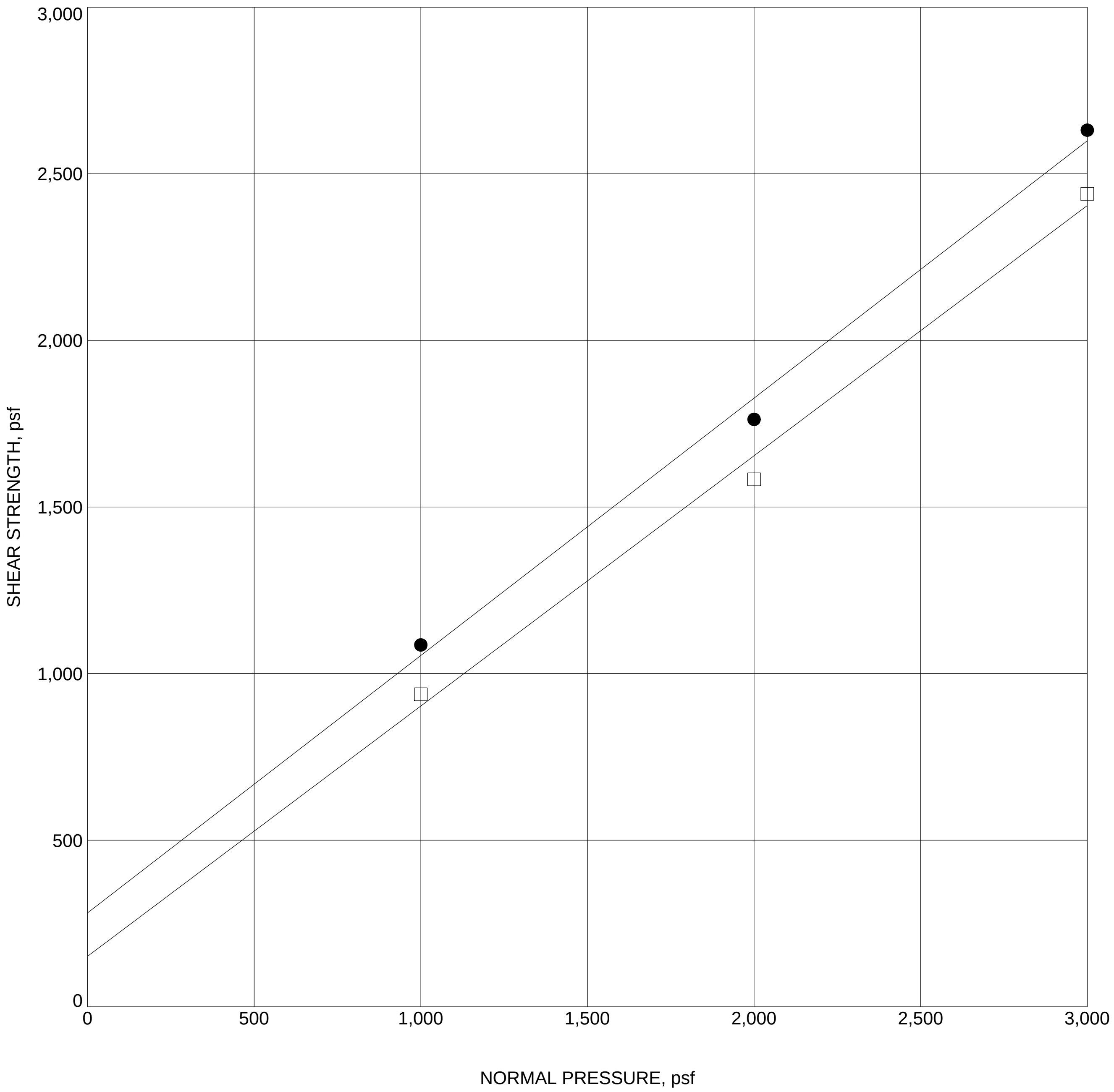
Corrosivity Test Results

Sample ID	Sample Depth (ft)	pH CT-532 (643)	Chloride CT-422 (ppm)	Sulfate CT-417 % By Weight	Resistivity CT-532 (643) (ohm-cm)
TP-102	3-4’	6.45	182	0.0010	3,400

W.O. 6960-A-SC
PLATE E-16

APPENDIX E

LABORATORY DATA
(GSI, 2015)



	Sample	Depth/El.	Range	Classification	Primary/Residual	Sample Type	γ_d	MC%	*c*	ϕ
●	TP-7	4.0		Silty Sand	Primary Shear	Remolded	115.2	11.0	282	38
☐	TP-7	4.0			Residual Shear	Remolded	115.2	11.0	152	37
					Reshear Shear	Remolded				
					Reshear Shear	Remolded				

Note: Sample Innundated Prior To Test

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

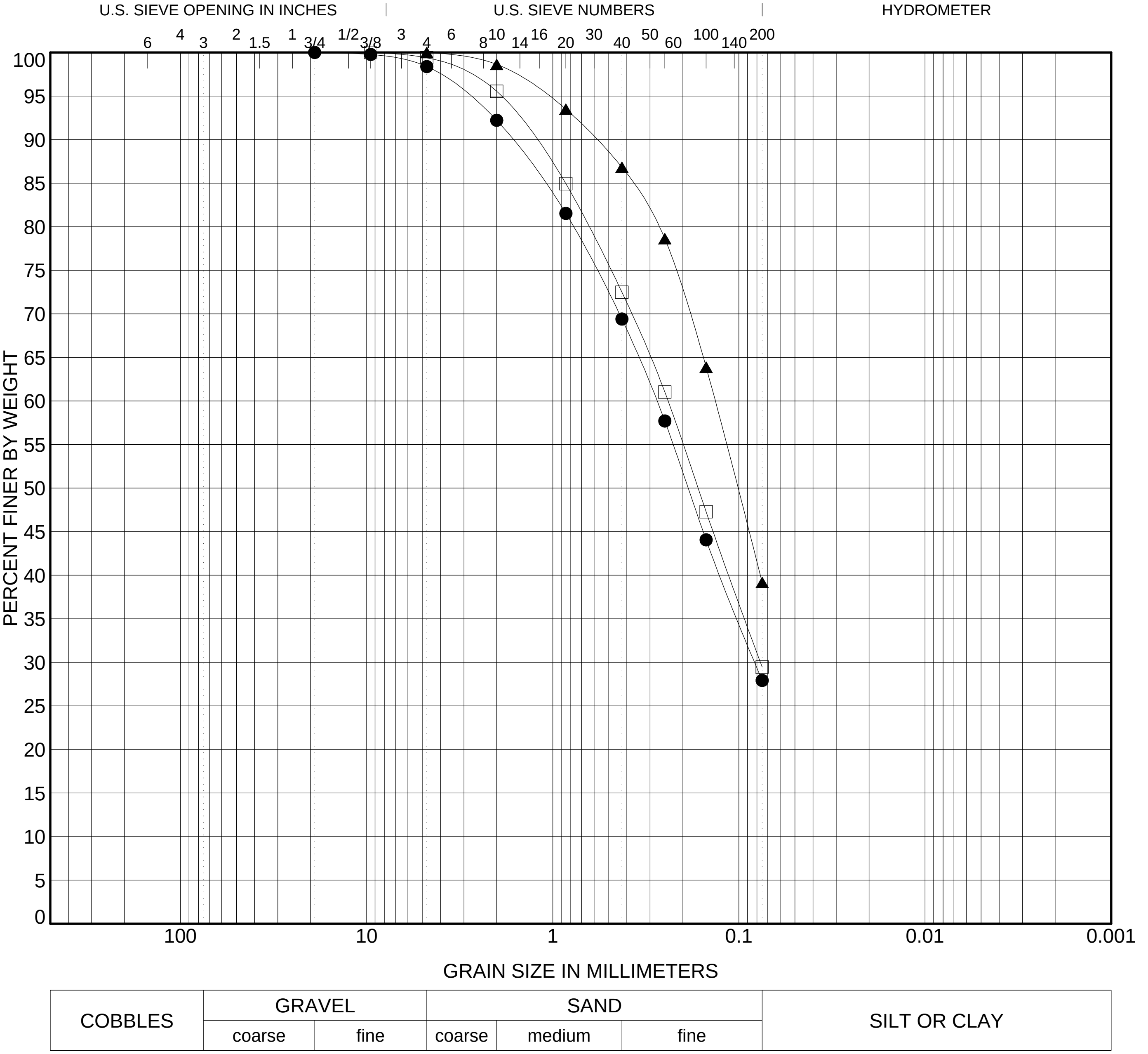
DIRECT SHEAR TEST

Project: VESSEL'S STALLION RANCH

Number: 6688-A-SC

Date: January 2015

Plate: E - 1



	Sample	Depth	Range	Visual Classification/USCS CLASSIFICATION	LL	PL	PI	Cc	Cu
●	TP-1	0.0		Silty Sand					
☐	TP-2	8.0		Silty Sand					
▲	TP-6	6.0		Silty Sand					

	Sample	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	TP-1	0.0	19	0.277	0.082		1.6	70.5	27.9	
☐	TP-2	8.0	9.5	0.241	0.077		0.6	69.9	29.5	
▲	TP-6	6.0	4.75	0.135			0.0	60.8	39.2	

US GRAIN SIZE 6688.GPJ US LAB.GDT 1/30/15

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

GRAIN SIZE DISTRIBUTION

Project: VESSEL'S STALLION RANCH

Number: 6688-A-SC

Date: January 2015

Plate: E - 2

SUMMARY OF LABORATORY TEST DATA

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

QCI Project No.: 14-029-03g
Date: March 20, 2014
Summarized by: ABK

Client: Vessel's Stallion Ranch
W.O. 6688-A- SC

Corrosivity Test Results

Sample ID	Sample Depth	pH CT-532 (643)	Chloride CT-422 (ppm)	Sulfate CT-417 % By Weight	Resistivity CT-532 (643) (ohm-cm)
TP-1	1' -.4'	6.99	122	0.0110	1,800

APPENDIX F

LIQUEFACTION ANALYSIS

SEISMIC VERTICAL DEFORMATION ANALYSIS

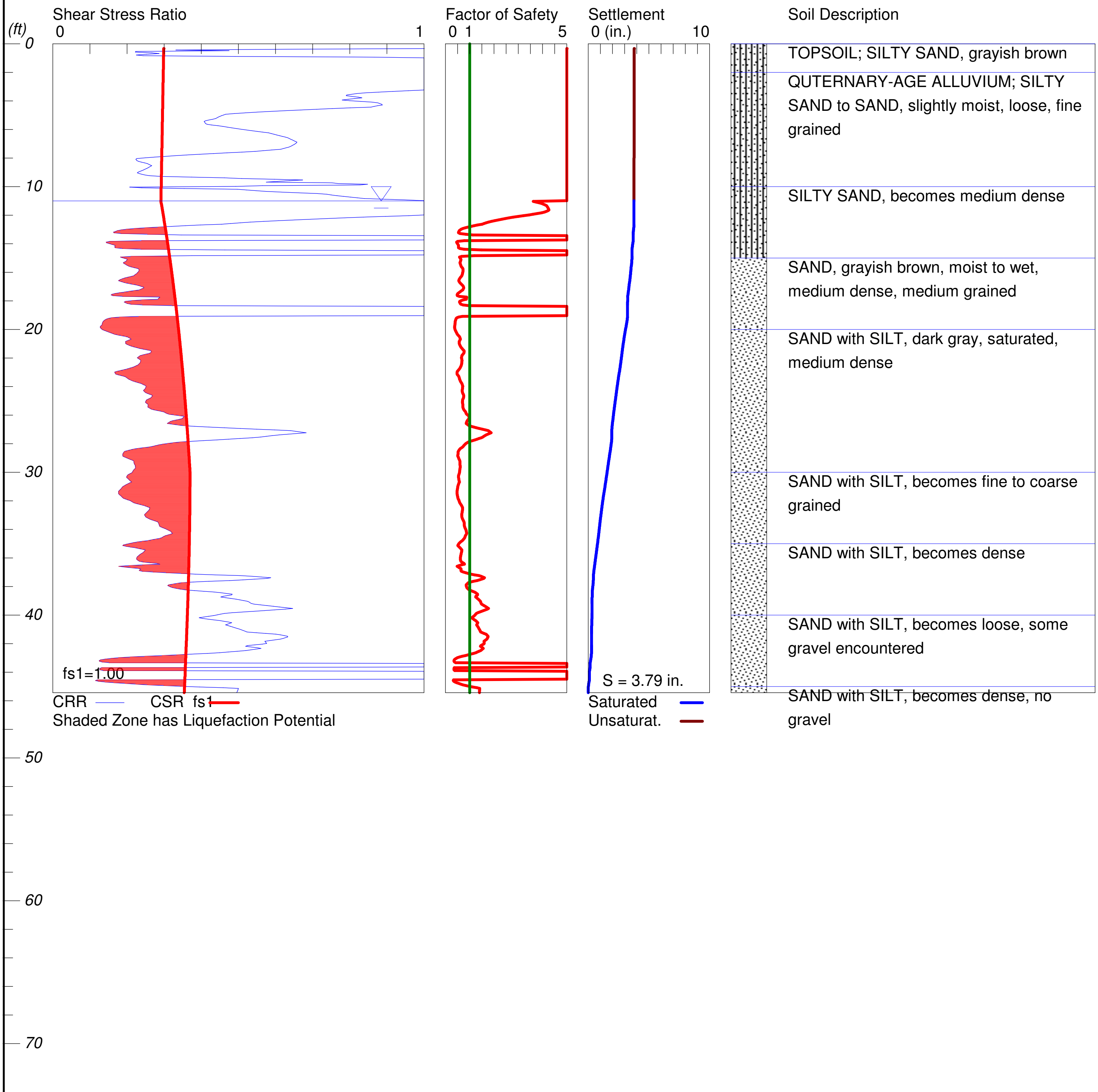
6960-A-SC, Ocean Breeze Ranch, FOS = 1.0

Hole No.=CPT-101   Water Depth=11 ft   Surface Elev.=189

Ground Improvement of Fill=9 ft

Magnitude=7.2

Acceleration=0.46g



TOPSOIL; SILTY SAND, grayish brown

QUTERNARY-AGE ALLUVIUM; SILTY SAND to SAND, slightly moist, loose, fine grained

SILTY SAND, becomes medium dense

SAND, grayish brown, moist to wet, medium dense, medium grained

SAND with SILT, dark gray, saturated, medium dense

SAND with SILT, becomes fine to coarse grained

SAND with SILT, becomes dense

SAND with SILT, becomes loose, some gravel encountered

SAND with SILT, becomes dense, no gravel

Liquefaction Analysis

Plate F-1

GeoSoils, Inc.

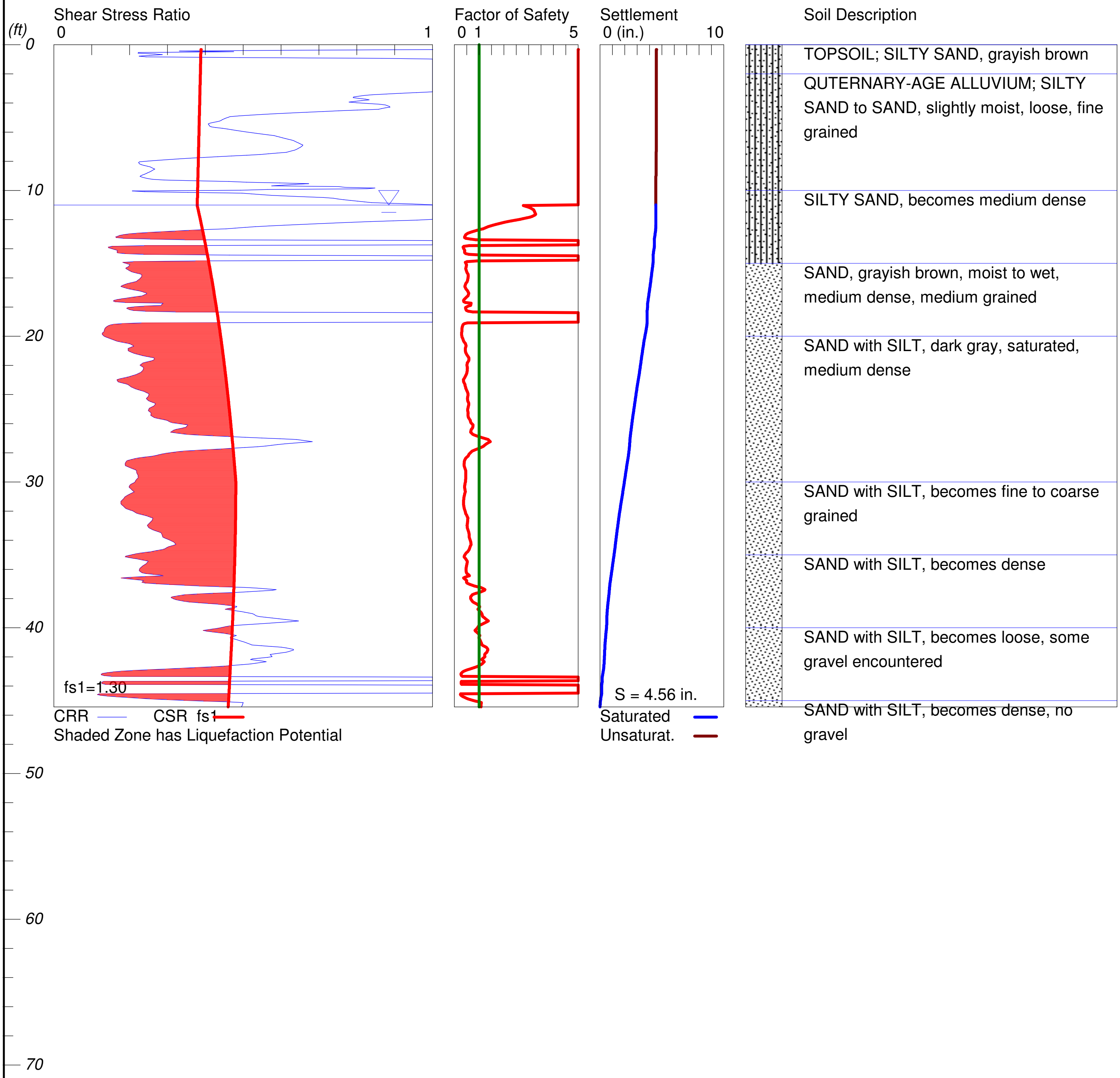
GeoSoils, Inc.

SEISMIC VERTICAL DEFORMATION ANALYSIS

6960-A-SC, Ocean Breeze Ranch, FOS = 1.3

Hole No.=CPT-101 ***Water Depth=11 ft*** ***Surface Elev.=189*** ***Magnitude=7.2***

Ground Improvement of Fill=9 ft ***Acceleration=0.46g***



Liquefaction Analysis **Plate F-2**

SEISMIC VERTICAL DEFORMATION ANALYSIS

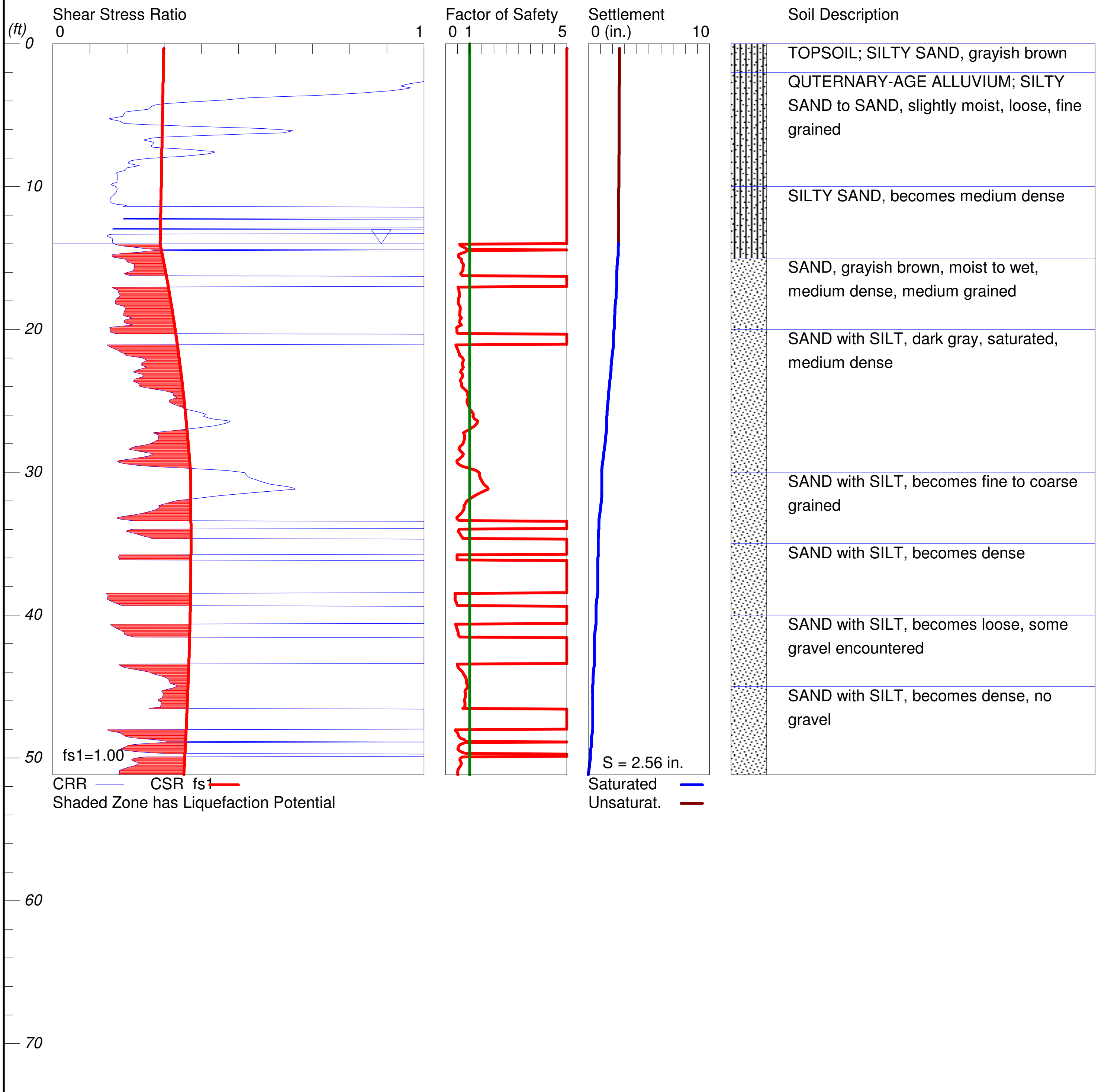
6960-A-SC, Ocean Breeze Ranch, FOS = 1.0

Hole No.=CPT-102 Water Depth=14 ft Surface Elev.=193

Ground Improvement of Fill=3 ft

Magnitude=7.2

Acceleration=0.46g



SEISMIC VERTICAL DEFORMATION ANALYSIS

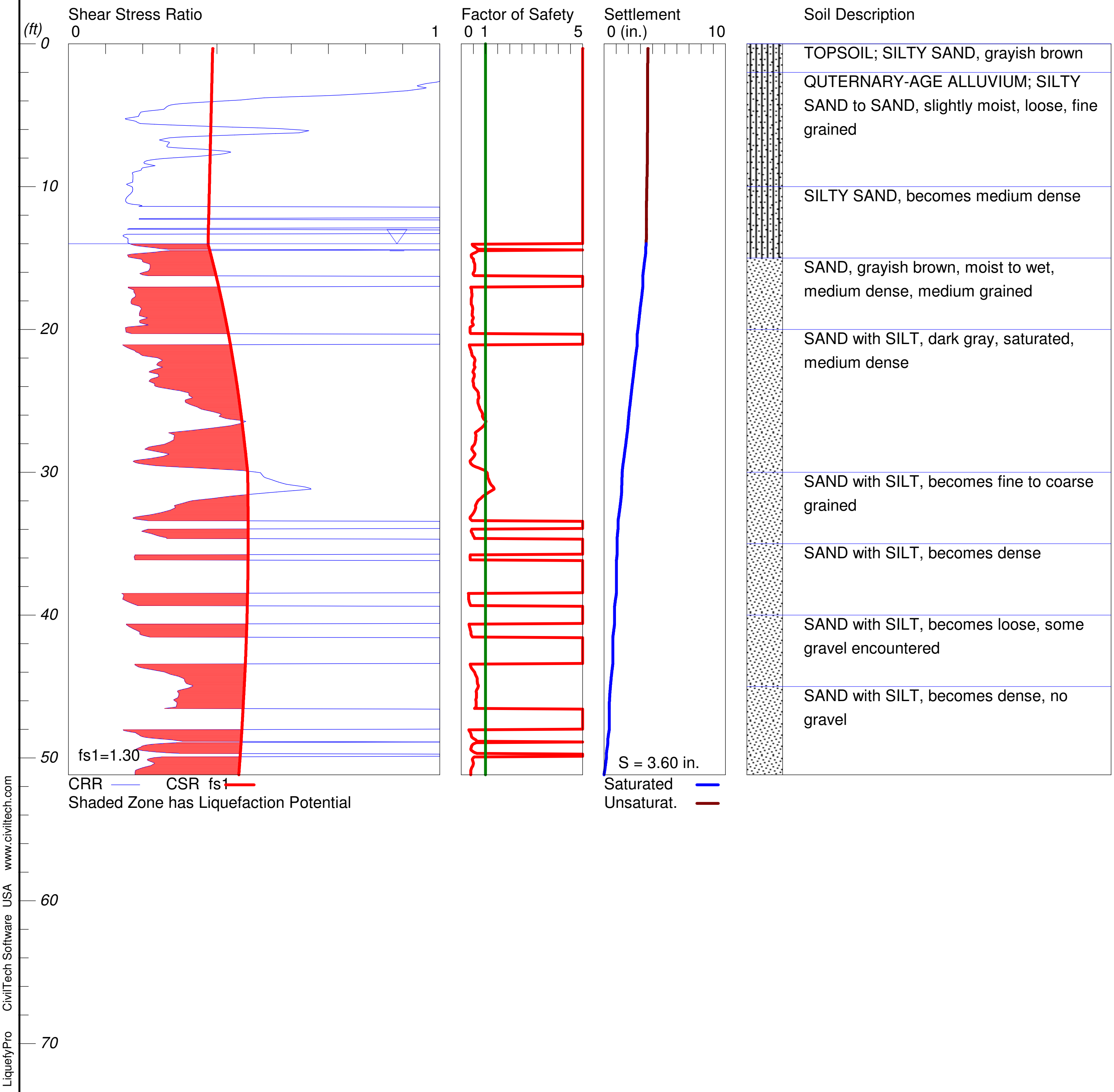
6960-A-SC, Ocean Breeze Ranch, FOS = 1.3

Hole No.=CPT-102   Water Depth=14 ft   Surface Elev.=193

Magnitude=7.2

Ground Improvement of Fill=3 ft

Acceleration=0.46g



SEISMIC VERTICAL DEFORMATION ANALYSIS

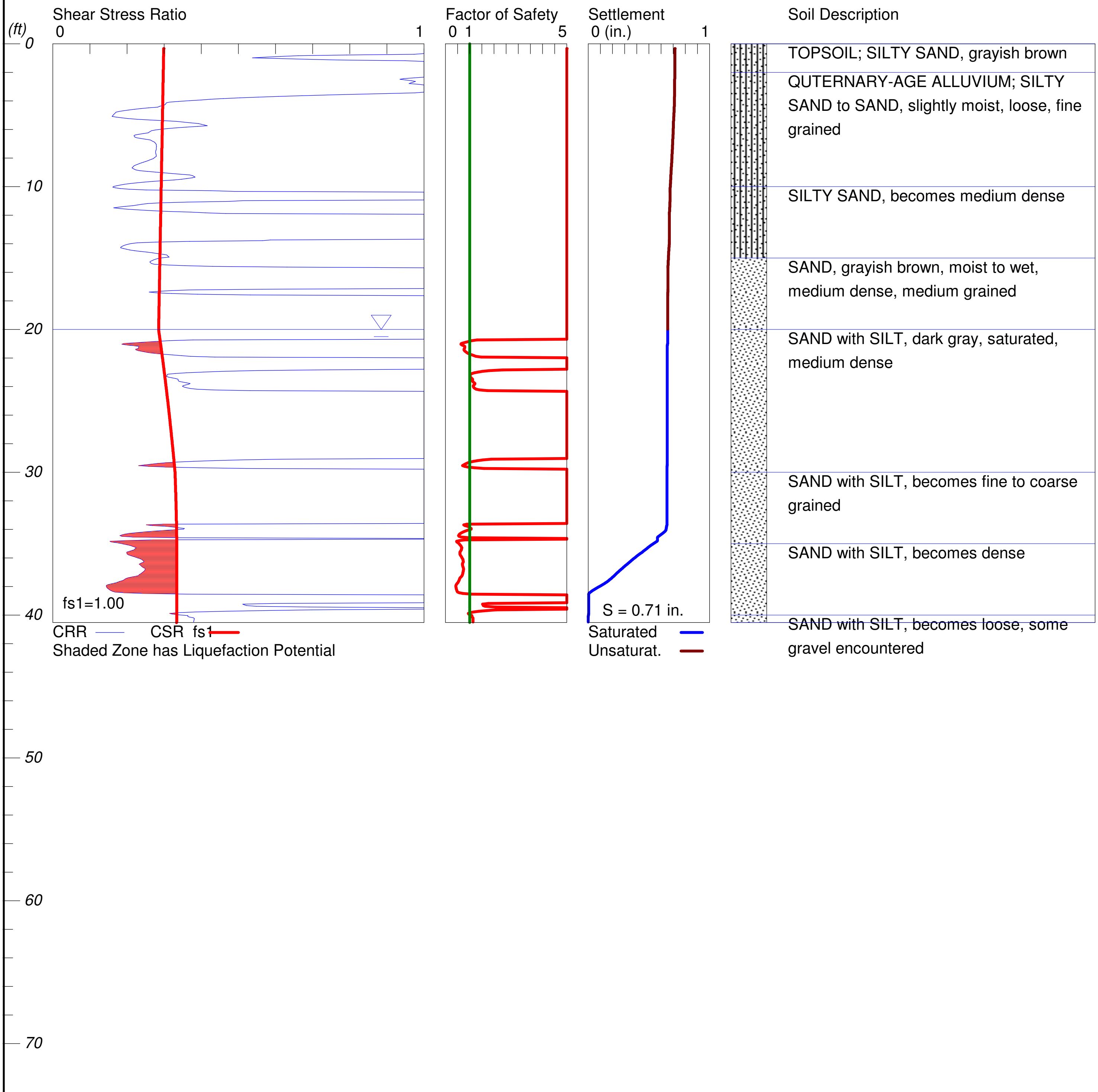
6960-A-SC, Ocean Breeze Ranch, FOS = 1.0

Hole No.=CPT-103 Water Depth=20 ft Surface Elev.=199

Ground Improvement of Fill=4 ft

Magnitude=7.2

Acceleration=0.46g



TOPSOIL; SILTY SAND, grayish brown

QUTERNARY-AGE ALLUVIUM; SILTY SAND to SAND, slightly moist, loose, fine grained

SILTY SAND, becomes medium dense

SAND, grayish brown, moist to wet, medium dense, medium grained

SAND with SILT, dark gray, saturated, medium dense

SAND with SILT, becomes fine to coarse grained

SAND with SILT, becomes dense

SAND with SILT, becomes loose, some gravel encountered

SEISMIC VERTICAL DEFORMATION ANALYSIS

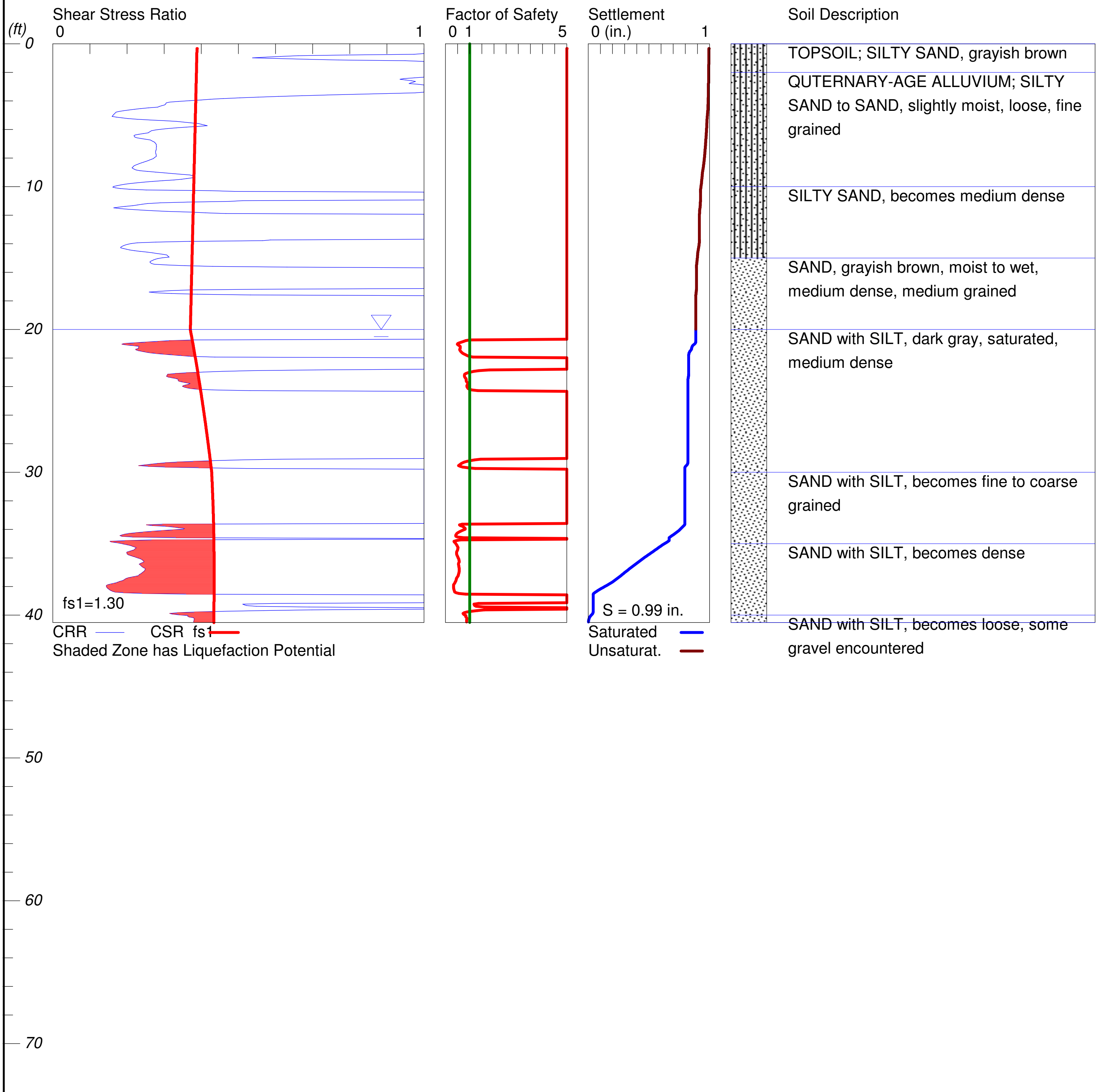
6960-A-SC, Ocean Breeze Ranch, FOS = 1.3

Hole No.=CPT-103 Water Depth=20 ft Surface Elev.=199

Ground Improvement of Fill=4 ft

Magnitude=7.2

Acceleration=0.46g



LiquefyPro CivilTech Software USA www.civiltech.com

Liquefaction Analysis

GeoSoils, Inc.

SEISMIC VERTICAL DEFORMATION ANALYSIS

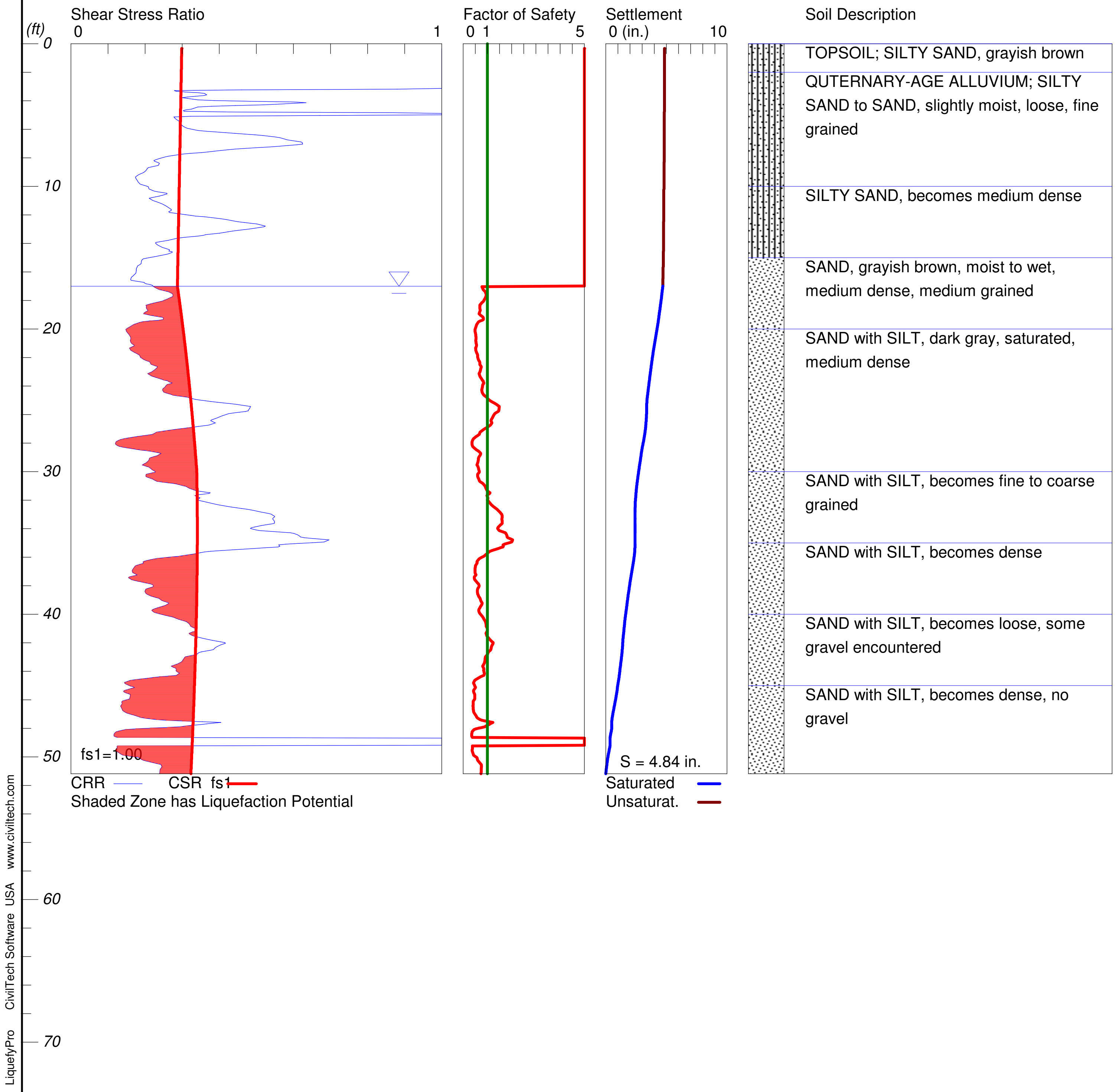
6960-A-SC, Ocean Breeze Ranch, FOS = 1.0

Hole No.=CPT-104 Water Depth=17 ft Surface Elev.=195

Ground Improvement of Fill=6 ft

Magnitude=7.2

Acceleration=.46g



Liquefaction Analysis

SEISMIC VERTICAL DEFORMATION ANALYSIS

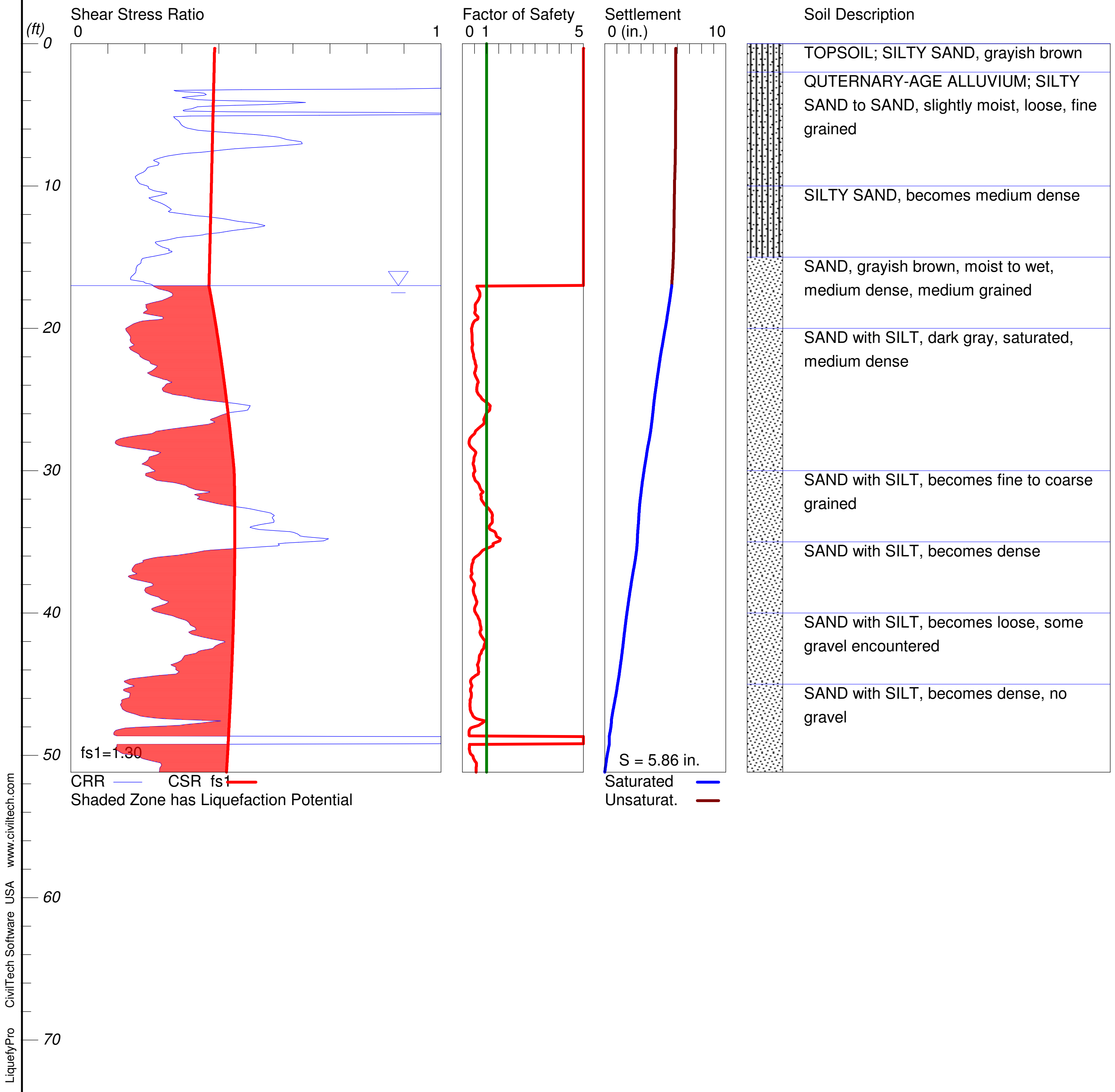
6960-A-SC, Ocean Breeze Ranch, FOS = 1.3

Hole No.=CPT-104   Water Depth=17 ft   Surface Elev.=195

Ground Improvement of Fill=6 ft

Magnitude=7.2

Acceleration=.46g



SEISMIC VERTICAL DEFORMATION ANALYSIS

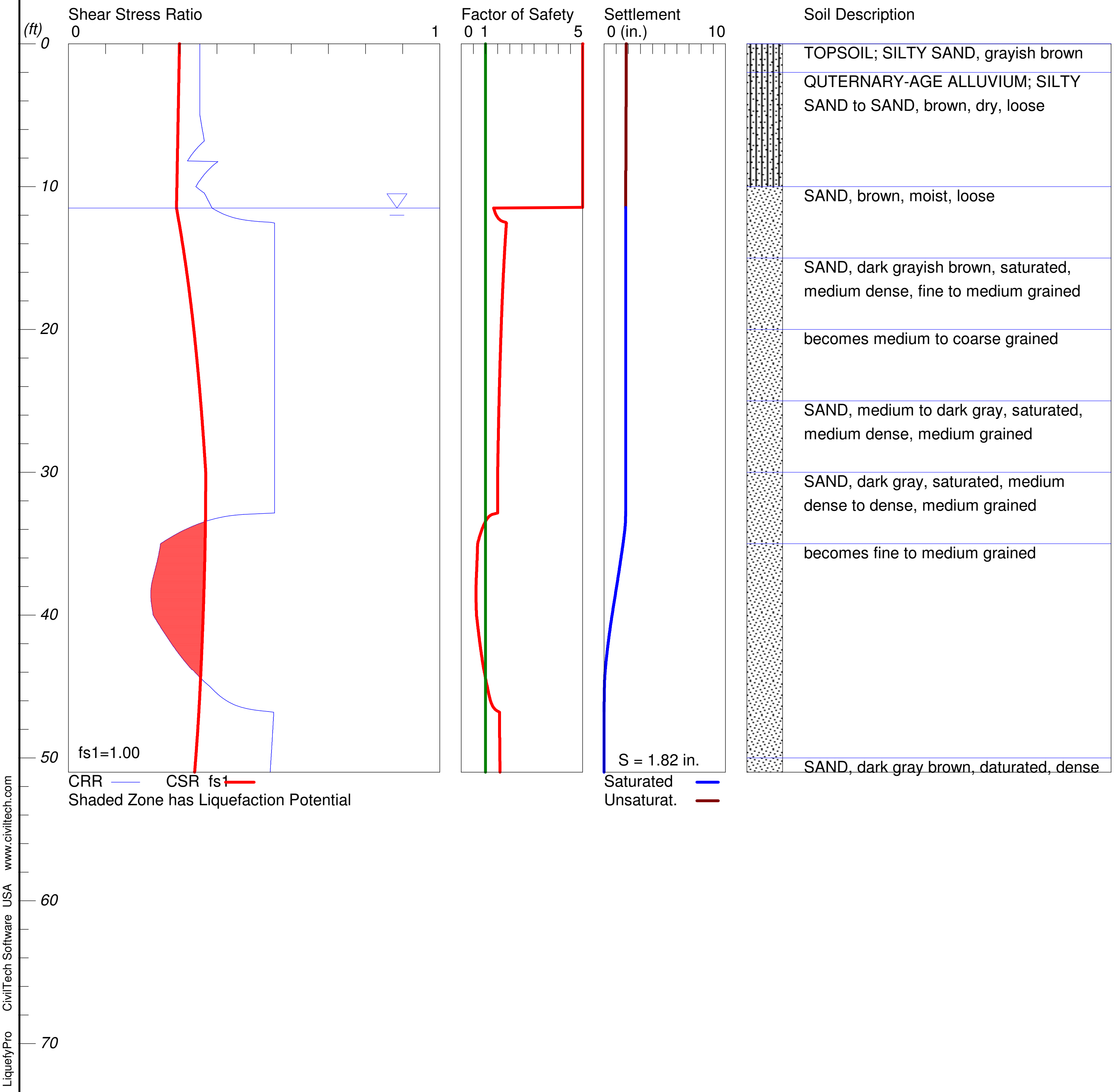
6960-A-SC, Ocean Breeze Ranch, FOS = 1.0

Hole No.=HSA-3 Water Depth=11.5 ft Surface Elev.=190

Ground Improvement of Fill=9 ft

Magnitude=7.2

Acceleration=0.46g



Liquefaction Analysis

Plate F-9

GeoSoils, Inc.

SEISMIC VERTICAL DEFORMATION ANALYSIS

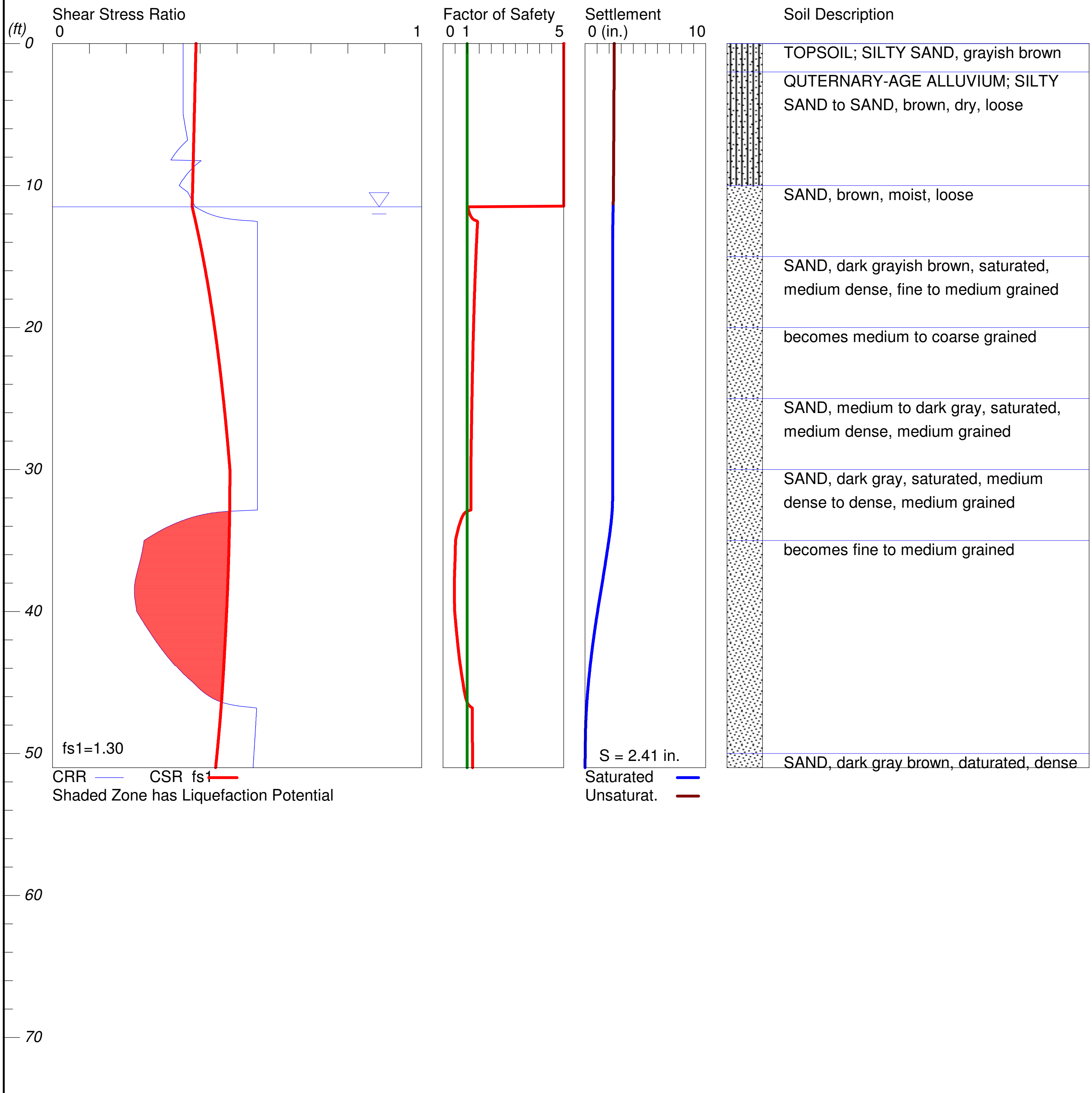
6960-A-SC, Ocean Breeze Ranch, FOS = 1.3

Hole No.=HSA-3 Water Depth=11.5 ft Surface Elev.=190

Ground Improvement of Fill=9 ft

Magnitude=7.2

Acceleration=0.46g



LiquefyPro CivilTech Software USA www.civiltech.com

Liquefaction Analysis

GeoSoils, Inc.

SEISMIC VERTICAL DEFORMATION ANALYSIS

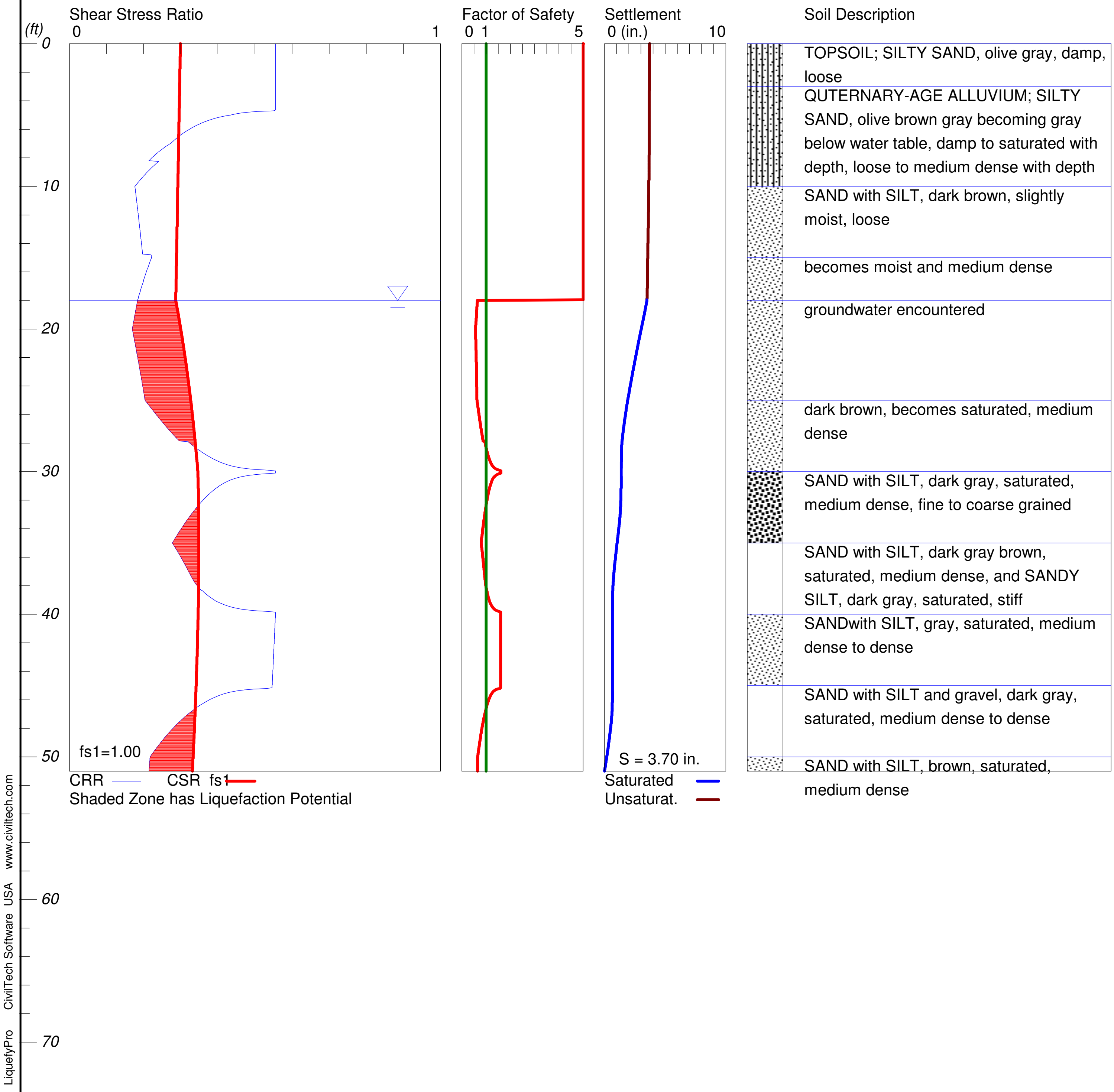
6960-A-SC, Ocean Breeze Ranch, FOS = 1.0

Hole No.=HSA-5   Water Depth=18 ft   Surface Elev.=197

Ground Improvement of Fill=3 ft

Magnitude=7.2

Acceleration=0.46g



Liquefaction Analysis

Plate F-11

GeoSoils, Inc.

SEISMIC VERTICAL DEFORMATION ANALYSIS

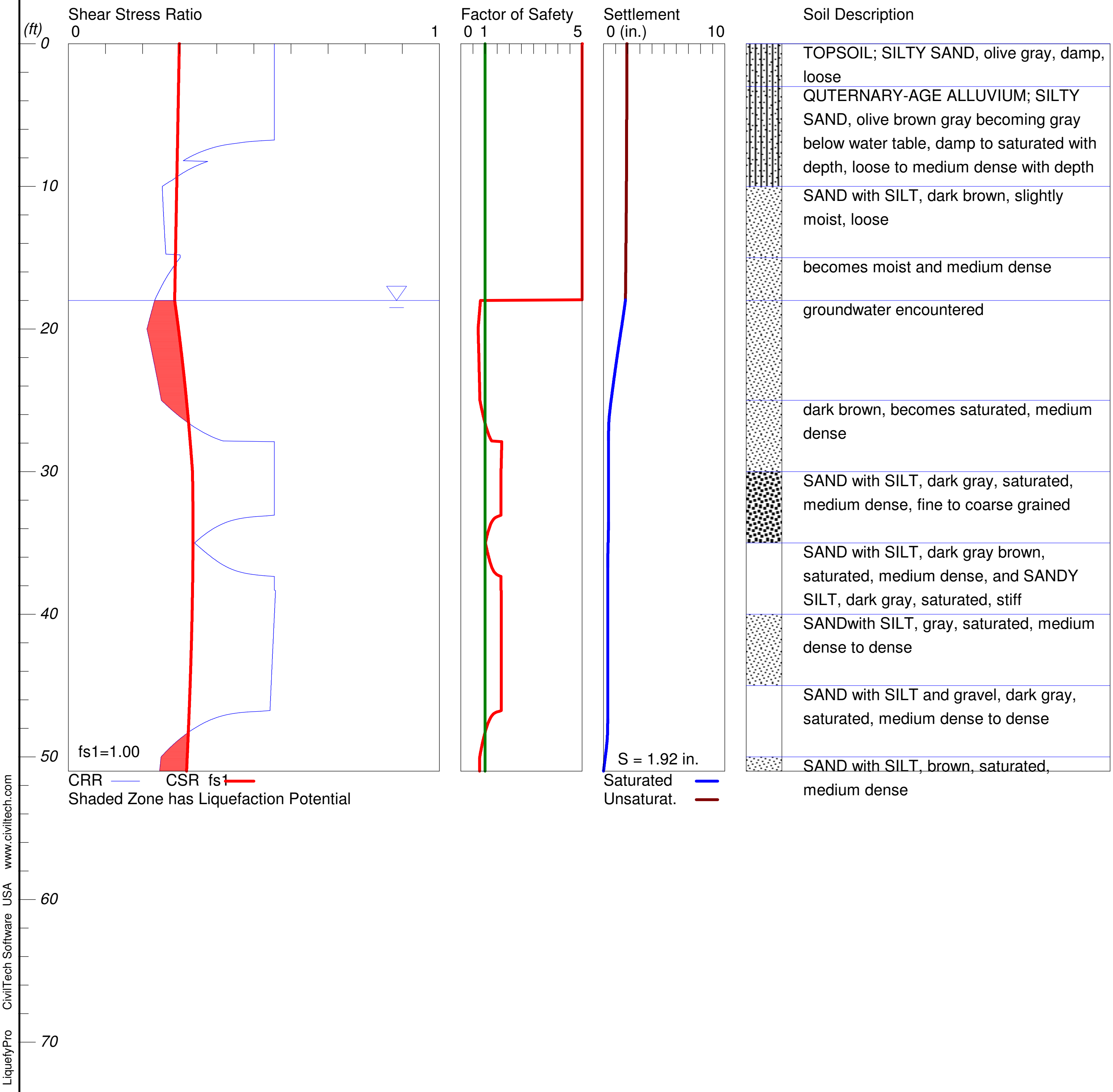
6960-A-SC, Ocean Breeze Ranch, FOS = 1.0

Hole No.=HSA-5 Water Depth=18 ft Surface Elev.=197

Ground Improvement of Fill=8 ft

Magnitude=7.2

Acceleration=0.46g



TOPSOIL; SILTY SAND, olive gray, damp, loose

QUTERNARY-AGE ALLUVIUM; SILTY SAND, olive brown gray becoming gray below water table, damp to saturated with depth, loose to medium dense with depth

SAND with SILT, dark brown, slightly moist, loose

becomes moist and medium dense

groundwater encountered

dark brown, becomes saturated, medium dense

SAND with SILT, dark gray, saturated, medium dense, fine to coarse grained

SAND with SILT, dark gray brown, saturated, medium dense, and SANDY SILT, dark gray, saturated, stiff

SANDwith SILT, gray, saturated, medium dense to dense

SAND with SILT and gravel, dark gray, saturated, medium dense to dense

SAND with SILT, brown, saturated, medium dense

SEISMIC VERTICAL DEFORMATION ANALYSIS

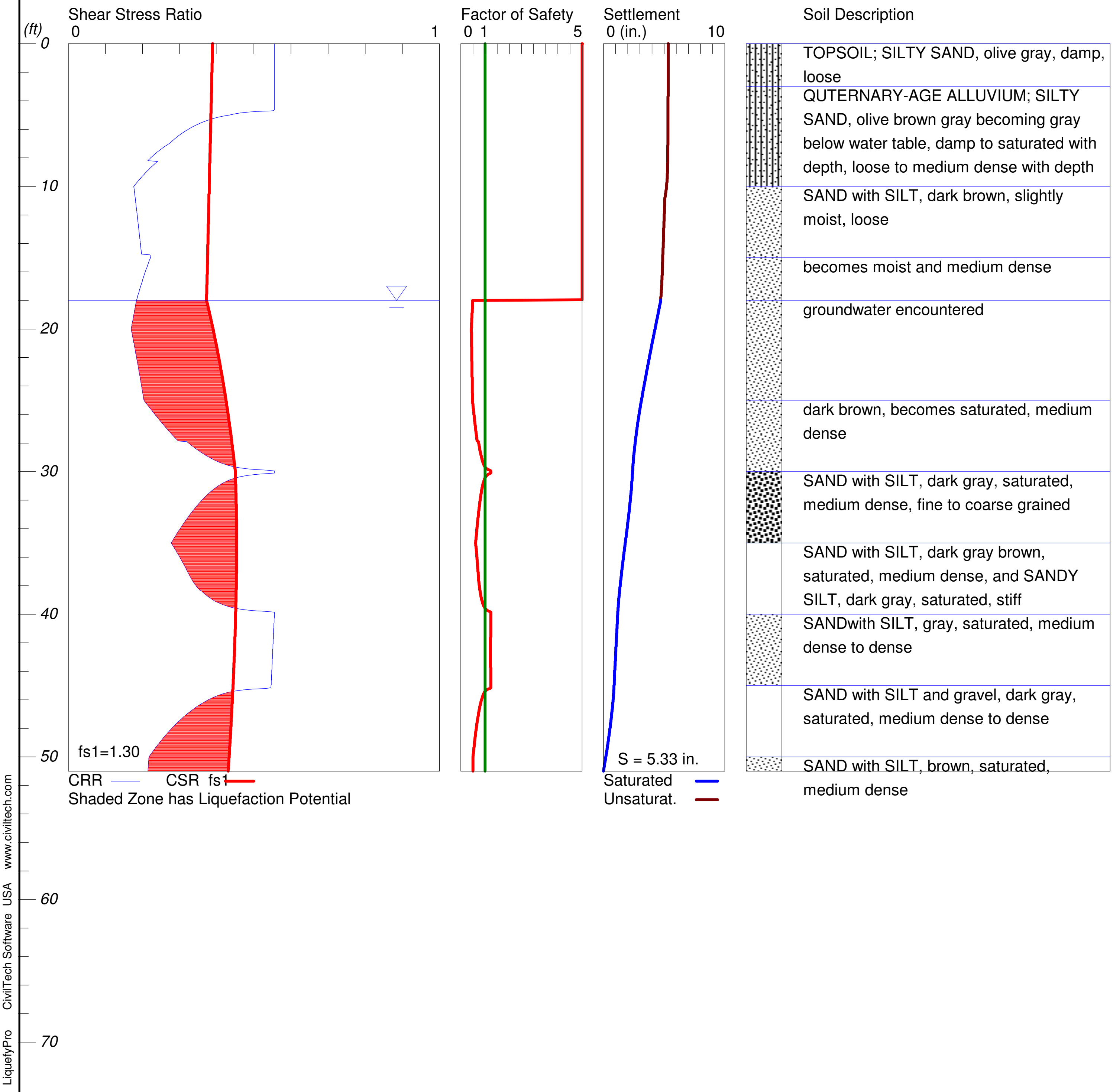
6960-A-SC, Ocean Breeze Ranch, FOS = 1.3

Hole No.=HSA-5   Water Depth=18 ft   Surface Elev.=197

Ground Improvement of Fill=3 ft

Magnitude=7.2

Acceleration=0.46g



SEISMIC VERTICAL DEFORMATION ANALYSIS

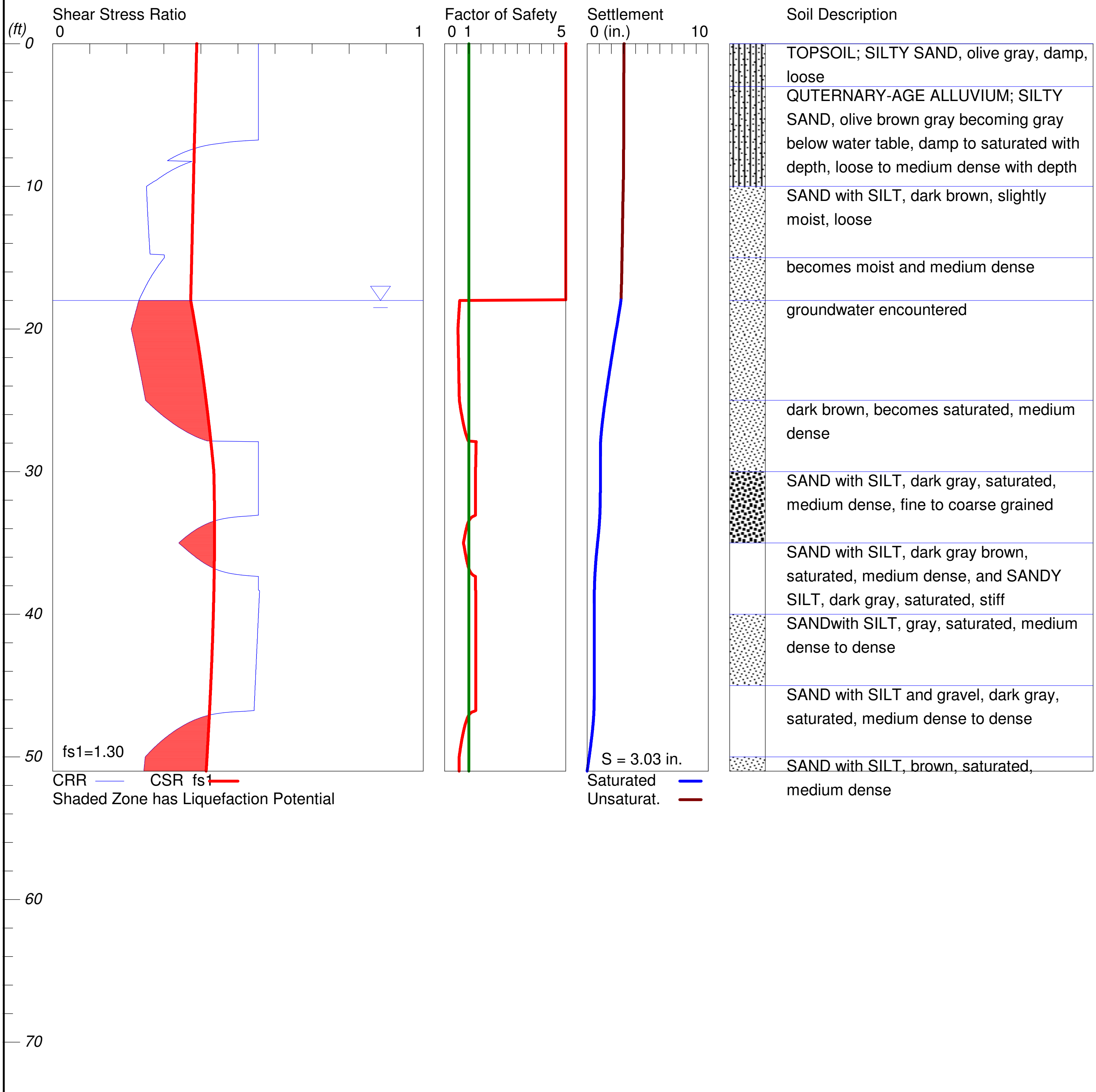
6960-A-SC, Ocean Breeze Ranch, FOS = 1.3

Hole No.=HSA-5   Water Depth=18 ft   Surface Elev.=197

Ground Improvement of Fill=8 ft

Magnitude=7.2

Acceleration=0.46g



SEISMIC VERTICAL DEFORMATION ANALYSIS

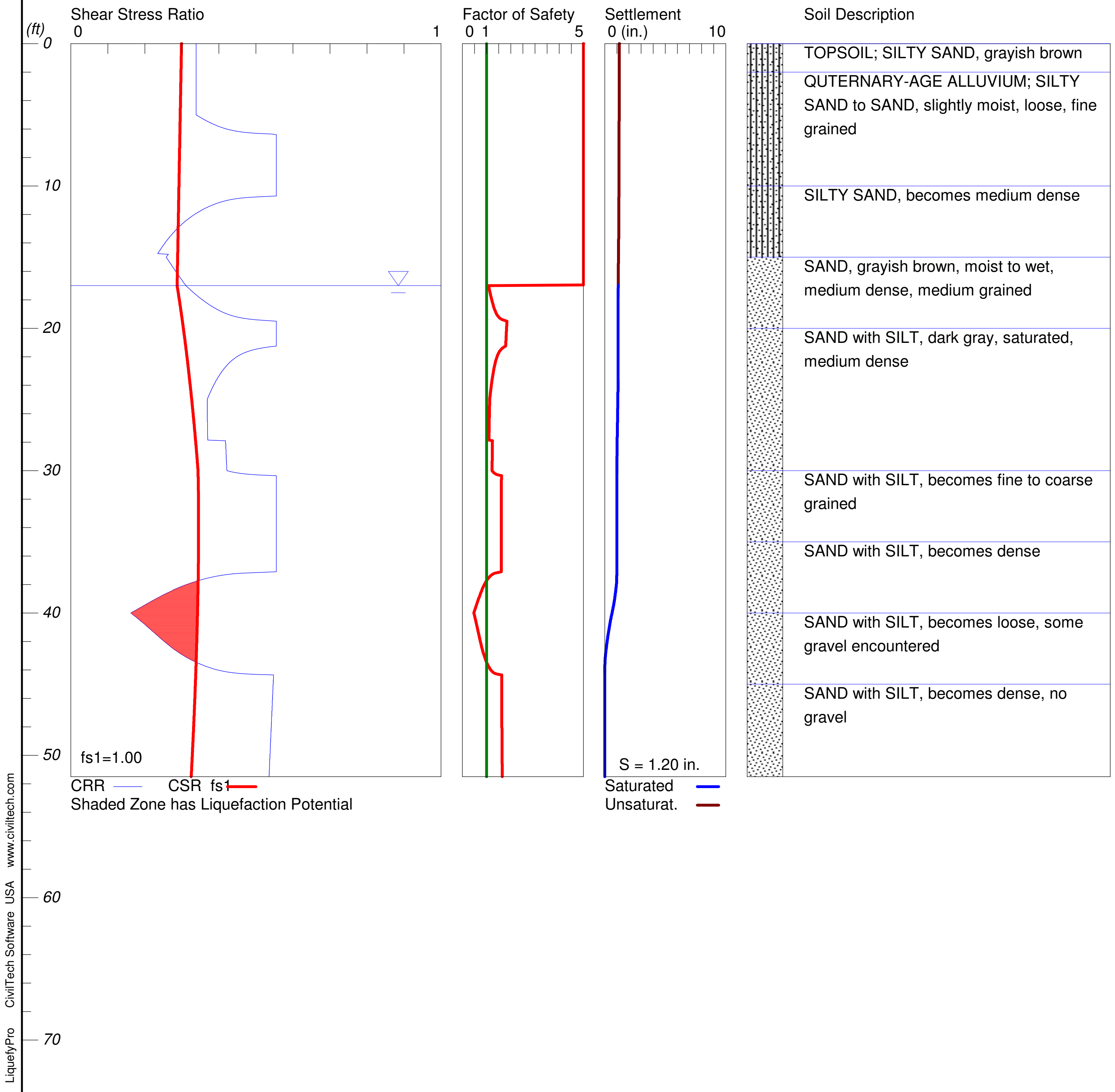
6960-A-SC, Ocean Breeze Ranch, FOS = 1.0

Hole No.=HSA-6   Water Depth=17 ft   Surface Elev.=195

Ground Improvement of Fill=6 ft

Magnitude=7.2

Acceleration=0.46g



Liquefaction Analysis

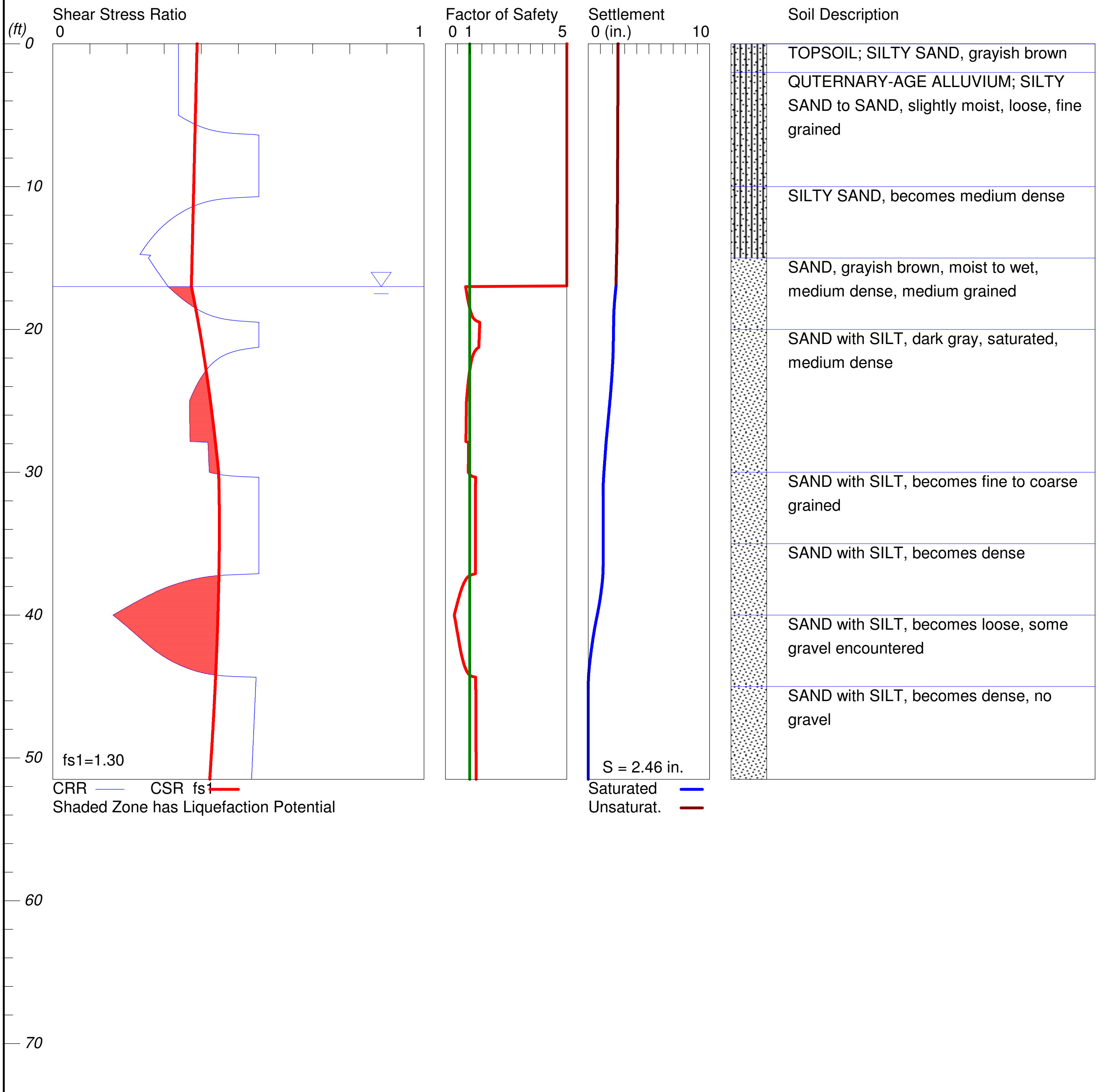
SEISMIC VERTICAL DEFORMATION ANALYSIS

6960-A-SC, Ocean Breeze Ranch, FOS = 1.3

Hole No.=HSA-6   Water Depth=17 ft   Surface Elev.=195

Ground Improvement of Fill=6 ft

Magnitude=7.2
Acceleration=0.46g



LATERAL SPREAD ANALYSIS FOR FREE FACE CONDITION

OCEAN BREEZE RANCHE 6960-A-SC

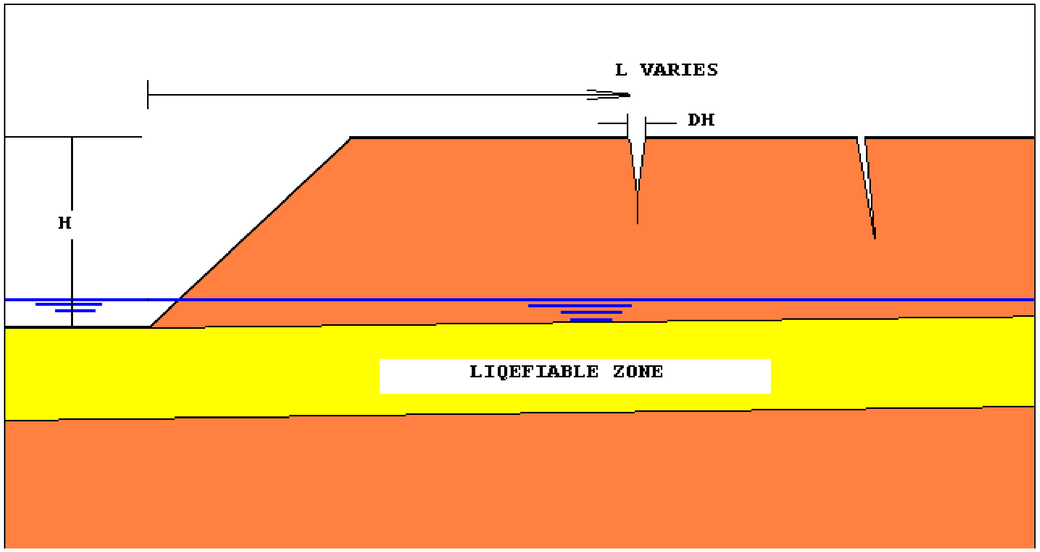
UP TO 400' AWAY FROM SLOPE BASE (REPRESENTED BY HSA-6)

M	EARTHQUAKE MOMENT MAGNITUDE	7.2
R	HORIZONTAL OR MAPPED DISTANCE TO THE NEAREST BOUND OF SEISMIC ENERGY SOURCE (KM)	18.9
H	FREE FACE HEIGHT (FEET)	35
L	DISTANCE FROM THE BASE OF FREE FACE TO THE POINT IN QUESTION IN SAME UNITS AS H	400
T_{15}	CUMULATIVE THICKNESS OF SATURATED GRANULAR LAYERS WITH N1(60)<=15 (METERS)	6
F_{15}	AVERAGE FINES CONTENT (<#200) FOR GRANULAR MATERIALS INCLUDED WITHIN T_{15} (%)	12
$D50_{15}$	AVERAGE MEAN GRAIN SIZE, D50 , FOR GRANULAR MATERIALS WITHIN T_{15} (mm)	0.35

OUTPUT

Ro	DISTANCE TERM THAT IS A FUNCTION OF MAGNITUDE (KM)	5.86
R*	MODIFIED SOURCE DISTANCE (KM)	24.76
W	FREE FACE RATIO- H/L %	8.75
LOG D_H	0.021038381	
D_H	ESTIMATED LATERAL GROUND DISPLACEMENT IN (METERS)	1.05
D_H	ESTIMATED LATERAL GROUND DISPLACEMENT IN (FEET)	3.51
RESULT:	***SINCE DH>1 FEET DAMAGE TO IMPROVEMENTS FROM LATERAL SPREADING IS LIKELY***	

ESTIMATION OF LATERAL DISPLACEMENT MAGNITUDE IN ACCORDANCE WITH "UPDATED YOUD & BARTLET" METHODOLOGY FOR FREE FACE CONDITION, AS RECOMMENDED BY CDMG SPECIAL PUBLICATION 117



APPENDIX G

INFILTRATION
(GSI, 2016)

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

Categorization of Infiltration Condition, OBR Basins Z, BB, EE, MMM, HHH (Alluvium Substrate)		**Worksheet 3.4-1 (Also Form I-8)**	
Part 1 - Full Infiltration Feasibility Screening Criteria **Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?**			
Criteria	**Screening Question**	**Yes**	**No**
1	**Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	X	
Provide basis: Infiltration rates ranging from 6 to 7.8 inches per hour were evaluated. It should also be noted that any artificial fill, created through removal/recompaction of onsite soils, would likely be less. See GSI report dated September 9, 2016 for other related discussions and references. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
2	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.	X	
Provide basis: These basins are situated in areas of relatively low relief. Groundwater levels monitored over a two-year period indicates a typical depth to groundwater ranging from about 11½ to 18 feet, or near an elevation of about 178 to 179½ feet MSL within Planning Area PA-3, to a depth of about 15½ to 21 feet within PA-5 (elevation of about 189 ½ to 213 feet MSL). Due to the relatively high infiltration, significant, long term mounding is not anticipated. See GSI report dated September 9, 2016 for other related discussions and references. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			

<table>
<tr><th colspan="4">Worksheet C.4.1 Page 2 of 4 (Form I-8)</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>3</td><td>Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensible evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:
The groundwater table in alluval areas appears to be no closer than about 11½ to 21 feet from existing surface grades locally, and should be considered in BMP design.
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>4</td><td>Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as a change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>Part 1 Result*</td><td colspan="2">In the answers to rows 1-4 are "Yes" a full infiltration design is potentially feasible. The feasibility screening category is Full Infiltration
If any answer from row 1-4 is "No", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design.
Proceed to Part 2</td><td>FULL INFIL-TRATION</td></tr>
</table>

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

<table>
<tr><th colspan="4">Worksheet C.4.1 Page 3 of 4 (Form I-8)</th></tr>
<tr><td colspan="4">Part 2 - Partial Infiltration vs. No Infiltration Feasibility Screening Criteria

Would infiltration of water in an appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?</td></tr>
<tr><td>Criteria</td><td>Screening Question</td><td>Yes</td><td>No</td></tr>
<tr><td>5</td><td>Do soil and geologic conditions allow for infiltration in any appreciable rate or volume? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.</td><td></td><td></td></tr>
<tr><td colspan="4">Provide basis:

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>6</td><td>Can infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.</td><td></td><td></td></tr>
<tr><td colspan="4">Provide basis:

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
</table>

Worksheet C.4.1 (Form I-8) Page 4 of 4

Criteria	**Screening Question**	**Yes**	**No**
7	**Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.		
Provide basis: Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
8	**Can infiltration be allowed without violating downstream water rights?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.		
Provide basis: Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
Part 2 Result*	If all answers from row 5-8 are yes then partial infiltration design is potentially feasible. The feasibility screening category is **Partial Infiltration.** If any answer from row 5-8 is no, then infiltration of any volume is considered to be **infeasible** within the drainage area. The feasibility screening category is **No Infiltration.**		

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

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

Categorization of Infiltration Condition, Basins Lot B, Lot EEE, Lot PPP, Lot NNN (OLDER ALLUVIUM)		**Worksheet 3.4-1 (FORM I-8)**	
Part 1 - Full Infiltration Feasibility Screening Criteria **Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?**			
Criteria	**Screening Question**	**Yes**	**No**
1	**Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	X	
Provide basis: Onsite testing using the inverse auger hole, or "Porchet" method evaluated infiltration rates of about 1.0 inch per hour for native site soil. It should also be noted that any artificial fill, created through removal/recompaction of onsite soils, or infiltration within deeper levels of bedrock exposed in cut areas, would likely possess an infiltration rate below the 0.5 inch/hour threshold. See GSI report dated September 9, 2016 for other related discussions and references. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
2	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.	X	
Provide basis: Yes, with exceptions. Basins located within 10 feet of any residential structure or settlement sensitive improvement (walls, pavements, etc.) can adversely affect the performance of the improvement by: 1.) facilitating heave of expansive soil; 2.) Increasing soil moisture transmission rates through concrete flooring; and 3.) Increase the potential for a loss in bearing strength of soil, due to saturation. Mitigative grading for the support of structures generally involves the removal and recompaction of near surface soils. This is anticipated to create a permeability contrast, and the potential for the development of a shallow "perched" water table, which can be anticipated to migrate laterally, beneath the structure(s). Planned utilities in the vicinity would act as "french drains" and also be adversely affected. Graded slopes would be subject to an increased potential for instability due to the lateral migration of water from a potential infiltration area located up gradient from, or near the slope. See GSI report dated September 9, 2016 for other related discussions and references. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			

GSI Appendix B, W.O. 6960-A-SC, dated September 9, 2016

From "Model BMP Design Manual, San Diego Region: Appendices, dated February 2016

<table>
<tr><th colspan="4">Worksheet C.4.1 Page 2 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>3</td><td>Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensible evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:
A groundwater table was encountered locally, at a depth of about 21 feet below the existing ground surface, and should be considered in BMP design. See GSI report dated September 9, 2016 for other related discussions and references.
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>4</td><td>Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as a change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:
This portion of planned construction is considered hillside development. Perched groundwater was evaluated at a depth as shallow as 21 feet below existing grade locally, in the vicinity of basins with the planned lowest elevations. See GSI report dated September 9, 2016 for other related discussions and references.
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>Part 1 Result*</td><td colspan="2">In the answers to rows 1-4 are "Yes" a full infiltration design is potentially feasible. The feasibility screening category is Full Infiltration
If any answer from row 1-4 is "No", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design.
Proceed to Part 2</td><td>Full Infiltration</td></tr>
</table>

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

Worksheet C.4.1 Page 3 of 4

Part 2 - Partial Infiltration vs. No Infiltration Feasibility Screening Criteria

Would infiltration of water in an appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
5	**Do soil and geologic conditions allow for infiltration in any appreciable rate or volume?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	X	
Provide basis: Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
6	**Can infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.	X	
Provide basis: Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			

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

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

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

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

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

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

<table>
<tr><td colspan="2">Categorization of Infiltration Condition, Basins Lot P, Lot YY, Lot 368, Lot 373, Lot 397 (GRANITIC SUBSTRATE)</td><td colspan="2">Worksheet 3.4-1 (Form I-8)</td></tr>
<tr><td colspan="4"><u>Part 1 - Full Infiltration Feasibility Screening Criteria</u>

Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?</td></tr>
<tr><td>Criteria</td><td>Screening Question</td><td>Yes</td><td>No</td></tr>
<tr><td>1</td><td>Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis:

Onsite testing using the inverse auger hole, or “Porchet” method evaluated infiltration rates ranging between 0.3 to 0.4 inches per hour for native site soil. It should also be noted that any artificial fill, created through removal/recompaction of onsite soils, or infiltration within deeper levels of bedrock exposed in cut areas, would likely possess an infiltration rate below the 0.5 inch/hour threshold. See GSI report dated September 9, 2016 for other related discussions and references.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>2</td><td>Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis:

Infiltration is less than 0.5 inches per hour.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
</table>

<table>
<tr><th colspan="4">Worksheet C.4.1 Page 2 of 4 (Form I-8)</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>3</td><td>Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensible evaluation of the factors presented in Appendix C.3.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis:
Infiltration is less than 0.5 inches per hour.
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>4</td><td>Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as a change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis:
Infiltration is less than 0.5 inches per hour.
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>Part 1 Result*</td><td colspan="2">In the answers to rows 1-4 are "Yes" a full infiltration design is potentially feasible. The feasibility screening category is Full Infiltration
If any answer from row 1-4 is "No", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design.
Proceed to Part 2</td><td>proceed to part 2</td></tr>
</table>

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

Worksheet C.4.1 Page 3 of 4 (Form I-8)

Part 2 - Partial Infiltration vs. No Infiltration Feasibility Screening Criteria

Would infiltration of water in an appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
5	**Do soil and geologic conditions allow for infiltration in any appreciable rate or volume?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	X	

Provide basis:

Site specific infiltration testing evaluated infiltration rates ranging between 0.3 and 0.4 inches per hour for onsite native soils. However, it should be noted that any artificial fill, created through removal/recompaction of onsite soils would likely possess a further reduced infiltration rate, and basins located within 10 feet of a residential structure, utility trench, or other improvement, would likely be adversely affected. See GSI report dated September 9, 2016 for other related discussions and references.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.

Criteria	Screening Question	Yes	No
6	**Can infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.	X	

Provide basis:

Yes with exceptions and prescribed setbacks (see Report). Basins located within 10 feet of any residential structure can adversely affect the performance of the structures foundation system by: 1.) Increasing soil moisture transmission rates through concrete flooring; and 2.) Increase the potential for a loss in bearing strength of soil, due to saturation. Mitigative grading for the support of structures generally involves the removal and recompaction of near surface soils. This is anticipated to create a permeability contrast, and the potential for the development of a shallow "perched" water table, which can be anticipated to migrate laterally, beneath the structure(s), or offsite onsite adjacent property. Planned utilities in the vicinity would potentially act as "french drains" and also be adversely affected. Adjacent, offsite slopes are generally steeper than 3:1 (horizontal to vertical) and would be subject to an increased potential for instability due to the lateral migration of water from a potential infiltration area located up gradient. See GSI report dated September 9, 2016 for other related discussions and references.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.

<table>
<tr><th colspan="4">Worksheet C.4.1 Page 4 of 4 (Form I-8)</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>7</td><td>Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:
This portion of Ocean breeze Ranch is considered a hillside development. Groundwater was evaluated at a depth of greater than 20 to 50 feet below existing grades onsite.
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>8</td><td>Can infiltration be allowed without violating downstream water rights? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:
This is a hillside development. The site currently drains offsite to the north, and no runoff appears to be retained onsite.
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>Part 2 Result*</td><td colspan="2">If all answers from row 5-8 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration.
If any answer from row 5-8 is no, then infiltration of any volume is considered to be infeasible within the drainage area. The feasibility screening category is No Infiltration.</td><td>Partial Infiltration</td></tr>
</table>

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

GeoSoils, Inc.

Geotechnical • Geologic • Environmental

5741 Palmer Way • Carlsbad, California 92008 • (760)438-3155 • FAX(760)931-0915

INVERSED AUGER HOLE (PORCHET) METHOD - DATA SHEET

PROJECT: OBR

DATE: 6-7-16

CLIENT: Ocean Breeze Ranch, LLC

WORK ORDER: 6960-A-SC

HOLE NUMBER TP-101

USCS SOIL CLASSIFICATION SM

DEPTH (D') OF TEST HOLE (in) 40

HOLE DIAMETER (in) 8

HOLE RADIUS (r) (in) 4

INITIAL WATER LEVEL (in) 38

Time	Δt (min)	t (min)	Ht (in)	ht (in) (D-Ht)	ht + ½ r
9:40	0	0	2	38	40
10:10	30	30	9	31	33
10:40	30	60	16	24	26
11:10	30	90	20.75	18.25	20.25

$K = 1.15\, r \tan \alpha$

where $\tan \alpha = [\log (h_0 + ½\, r) - \log (h_t + ½\, r)] / t\text{-}t_0$, $\alpha = 0.0036$

K = **0.98 inches per hour (Use 1.0 inches per hour). Represents Older Alluvium Substrate.**

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INVERSED AUGER HOLE (PORCHET) METHOD - DATA SHEET

PROJECT: OBR

DATE: 6-7-16

CLIENT: Ocean Breeze Ranch, LLC

WORK ORDER: 6960-A-SC

HOLE NUMBER TP-103

USCS SOIL CLASSIFICATION SM/SP

DEPTH (D') OF TEST HOLE (in) 40

HOLE DIAMETER (in) 8

HOLE RADIUS (r) (in) 4

INITIAL WATER LEVEL (in) 32

Time	Δt (min)	t (min)	Ht (in)	ht (in) (D-Ht)	ht + ½ r
9:00	0	0	2.25	37.75	39.75
9:30	30	30	5	35	37
10:00	30	60	7	33	35
10:30	30	90	9.5	30.5	32.5
11:00	30	120	10.25	28.25	30.25

$K = 1.15\ r \tan \alpha$

where $\tan \alpha = [\log (h_0 + ½ r) - \log (h_t + ½ r)] / t\text{-}t_0$, $\alpha = 0.0010$

K = **0.286 inches per hour (Use 0.3 inches per hour). Represents Granitic Substrate**

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INVERSED AUGER HOLE (PORCHET) METHOD - DATA SHEET

PROJECT: OBR

DATE: 6-7-16

CLIENT: Ocean Breeze Ranch, LLC

WORK ORDER: 6960-A-SC

HOLE NUMBER TP-104

USCS SOIL CLASSIFICATION SM/SP

DEPTH (D’) OF TEST HOLE (in) 40

HOLE DIAMETER (in) 8

HOLE RADIUS (r) (in) 4

INITIAL WATER LEVEL (in) 38

Time	Δt (min)	t (min)	Ht (in)	ht (in) (D-Ht)	ht + ½ r
11:30	0	0	2	38	40
12:00	30	30	12	28	30
12:30	30	60	19	21	23
1:00	30	90	24.5	15.5	17.5

K = 1.15 r tan α

where tan α = [log (h_0 + ½ r) - log (h_t + ½ r)] / t-t_0, α = 0.0038

K = **1.06 inches per hour (Use 1.0 inches per hour)**

Represents Basins Lot B, Lot EEE, Lot PPP, and Lot NNN (Represents Older Alluvium Substrate)

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INVERSED AUGER HOLE (PORCHET) METHOD - DATA SHEET

PROJECT: OBR

DATE: 6-7-16

CLIENT: Ocean Breeze Ranch, LLC

WORK ORDER: 6960-A-SC

HOLE NUMBER TP-107

USCS SOIL CLASSIFICATION SM/SP

DEPTH (D') OF TEST HOLE (in) 46

HOLE DIAMETER (in) 8

HOLE RADIUS (r) (in) 4

INITIAL WATER LEVEL (in) 3

Time	Δt (min)	t (min)	Ht (in)	ht (in) (D-Ht)	ht + ½ r
1:30	0	0	3	43	45
2:00	30	30	16	30	32
2:30	30	60	25	21	23
3:00	30	90	31.25	14.75	16.75

$K = 1.15\ r \tan \alpha$

where $\tan \alpha = [\log (h_0 + ½ r) - \log (h_t + ½ r)] / t-t_0$, $\alpha = 0.0047$

K = **1.3 inches per hour**

Represents Basins Lot B, Lot EEE, Lot PPP, and Lot NNN (Represents Older Alluvium Substrate)

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INVERSED AUGER HOLE (PORCHET) METHOD - DATA SHEET

PROJECT: OBR

DATE: 6-7-16

CLIENT: Ocean Breeze Ranch, LLC

WORK ORDER: 6960-A-SC

HOLE NUMBER TP-108

USCS SOIL CLASSIFICATION SM/SP

DEPTH (D') OF TEST HOLE (in) 48

HOLE DIAMETER (in) 8

HOLE RADIUS (r) (in) 4

INITIAL WATER LEVEL (in) 6

Time	Δt (min)	t (min)	Ht (in)	ht (in) (D-Ht)	ht + ½ r
2:45	0	0	6	42	44
3:15	30	30	10	38	40
3:45	30	60	13.75	34.25	36.25
4:15	30	90	17.25	30.75	32.75

$K = 1.15\ r \tan \alpha$

where $\tan \alpha = [\log (h_0 + ½ r) - \log (h_t + ½ r)] / t\text{-}t_0$, $\alpha = 0.0014$

K = **0.4 inches per hour (Represents Granitic Substrate).**

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INVERSED AUGER HOLE (PORCHET) METHOD - DATA SHEET

PROJECT: OBR

DATE: 5-19-16

CLIENT: Ocean Breeze Ranch, LLC

WORK ORDER: 6960-A-SC

HOLE NUMBER HSA-1

USCS SOIL CLASSIFICATION SM/SP

DEPTH (D’) OF TEST HOLE (in) 66

HOLE DIAMETER (in) 8

HOLE RADIUS (r) (in) 4

INITIAL WATER LEVEL (in) 32

Time	Δt (min)	t (min)	Ht (in)	ht (in) (D-Ht)	ht + ½ r
2:55	0	0	32	34	36
3:07	12	12	52	14	16
3:18	11	23	60	6	8

K = 1.15 r tan α

where tan α = [log (h_0 + ½ r) - log (h_t + ½ r)] / t-t_0, α = 0.02840

K = **7.83 inches per hour (Use 7.8 inches per hour)**

Represents Basin Lot Z (Alluvium Substrate)

GeoSoils, Inc.

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INVERSED AUGER HOLE (PORCHET) METHOD - DATA SHEET

PROJECT: OBR

DATE: 7-5-16

CLIENT: Ocean Breeze Ranch, LLC

WORK ORDER: 6960-A-SC

HOLE NUMBER HSA-9

USCS SOIL CLASSIFICATION SM/SP

DEPTH (D') OF TEST HOLE (in) 54

HOLE DIAMETER (in) 8

HOLE RADIUS (r) (in) 4

INITIAL WATER LEVEL (in) 48

Time	Δt (min)	t (min)	Ht (in)	ht (in) (D-Ht)	ht + ½ r
2:30	0	0	6	48	50
2:45	15	15	31	23	25
3:00	15	30	44.25	9.75	11.75
3:25	15	45	10	44	46
3:40	15	15	34.5	19.5	21.5

$K = 1.15\ r \tan \alpha$

where $\tan \alpha = [\log (h_0 + ½ r) - \log (h_t + ½ r)] / t\text{-}t_0$, $\alpha = 0.022$

K = **6 inches per hour**

Represents Basins Lot Z, Lot BB, Lot EE, Lot MMM, and Lot HHH (Alluvium Substrate)

APPENDIX H

GENERAL EARTHWORK, GRADING GUIDELINES
AND PRELIMINARY CRITERIA

GENERAL EARTHWORK, GRADING GUIDELINES, AND PRELIMINARY CRITERIA

General

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

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

EARTHWORK OBSERVATIONS AND TESTING

Geotechnical Consultant

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

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

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

Laboratory and Field Tests

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

Contractor's Responsibility

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

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

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

SITE PREPARATION

All major vegetation, including brush, trees, thick grasses, organic debris, and other deleterious material, should be removed and disposed of off-site. These removals must be 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

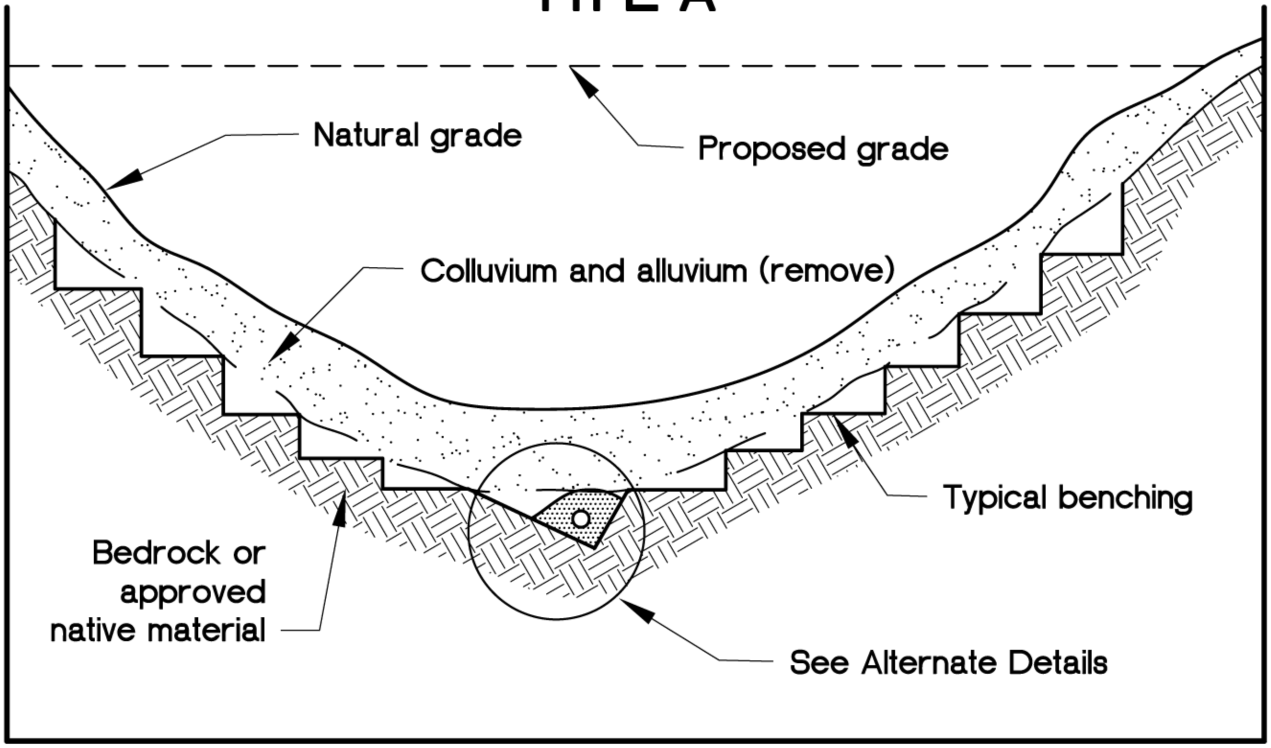
It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Our personnel are directed not to enter any excavation or vertical cut which: 1) is 5 feet or deeper unless shored or laid back; 2) displays any evidence of instability, has any loose rock or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

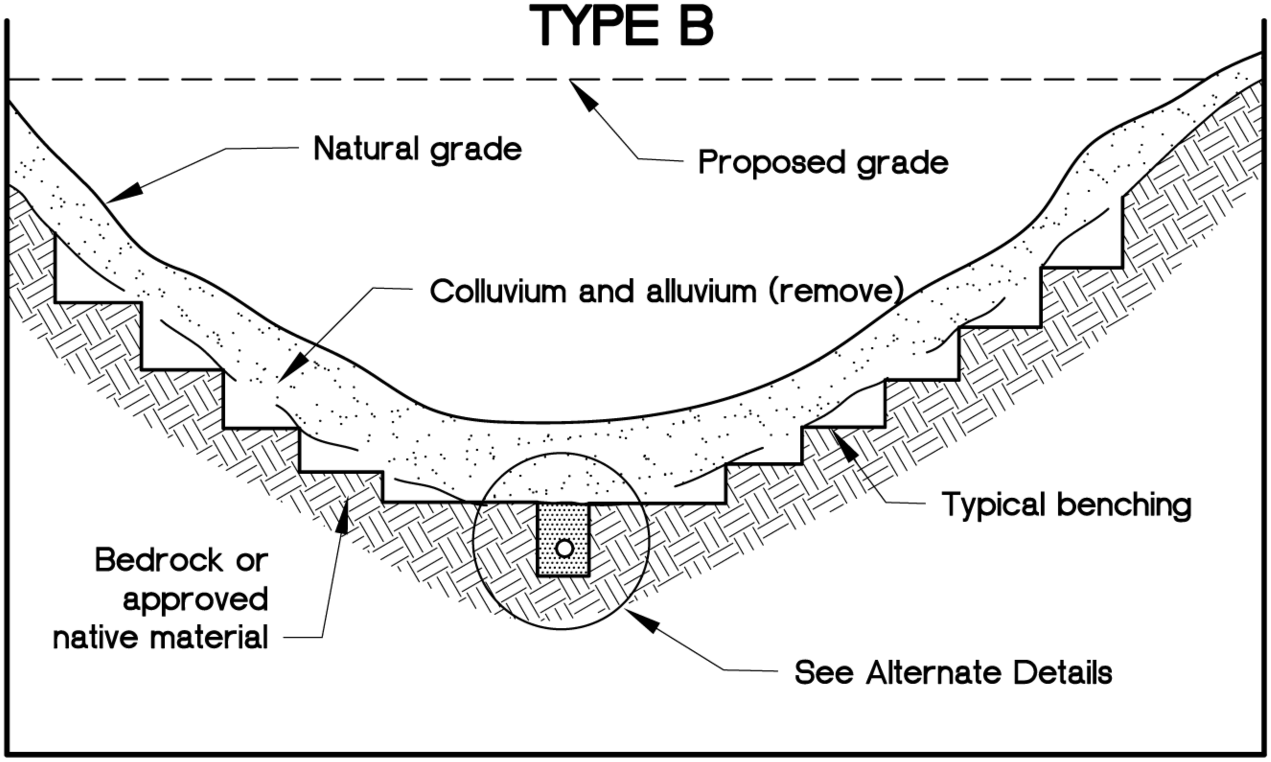
If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraw and notify his/her supervisor. The contractor’s representative will be contacted in an effort to affect a solution. All backfill not tested due to safety concerns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

TYPE A

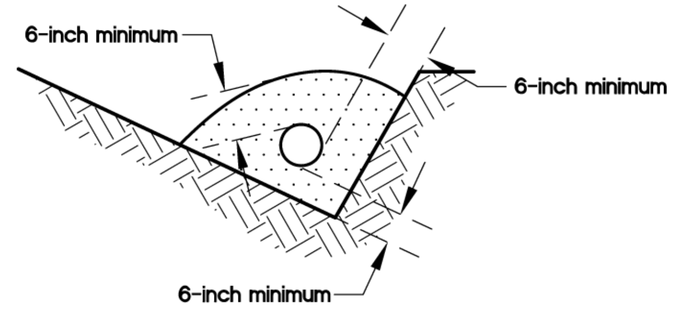


TYPE B

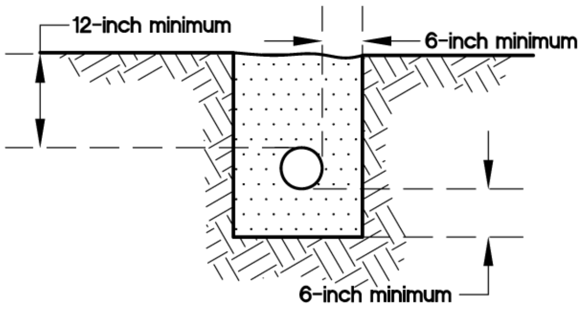


Selection of alternate subdrain details, location, and extent of subdrains should be evaluated by the geotechnical consultant during grading.

GeoSoils, Inc.	CANYON SUBDRAIN DETAIL	Plate H-1



A-1



B-1

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

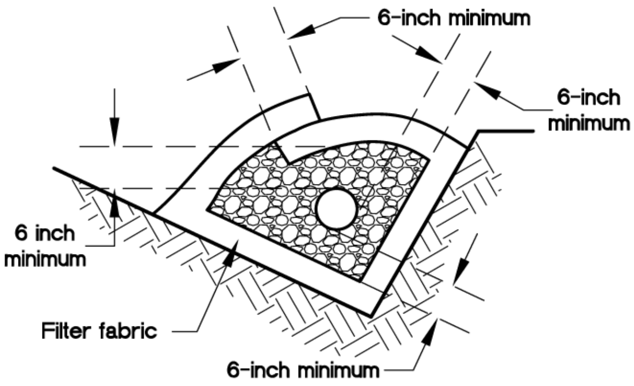
Perforated pipe: 6-inch-diameter ABS or PVC pipe or approved substitute with minimum 8 perforations (¼-inch diameter) per lineal foot in bottom half of pipe (ASTM D-2751, SDR-35, or ASTM D-1527, Schd. 40).

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

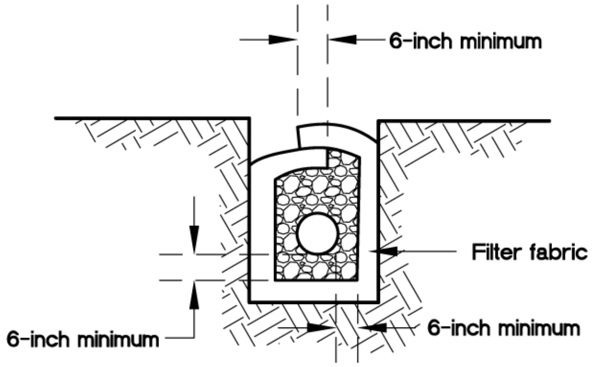
FILTER MATERIAL

Sieve Size	Percent Passing
1 inch	100
¾ inch	90-100
⅜ inch	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

ALTERNATE 1: PERFORATED PIPE AND FILTER MATERIAL



A-2

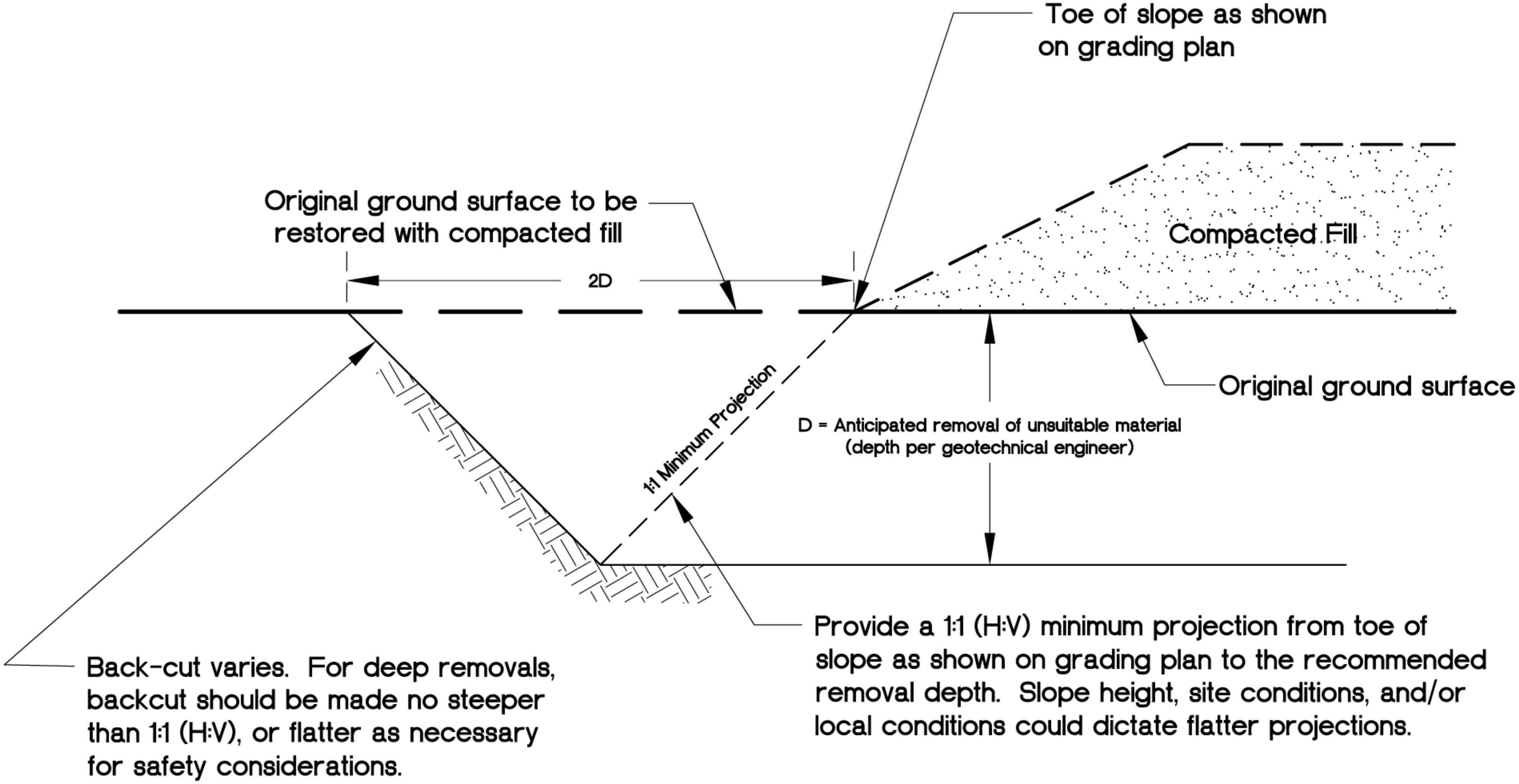


B-2

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

ALTERNATE 2: PERFORATED PIPE, GRAVEL, AND FILTER FABRIC

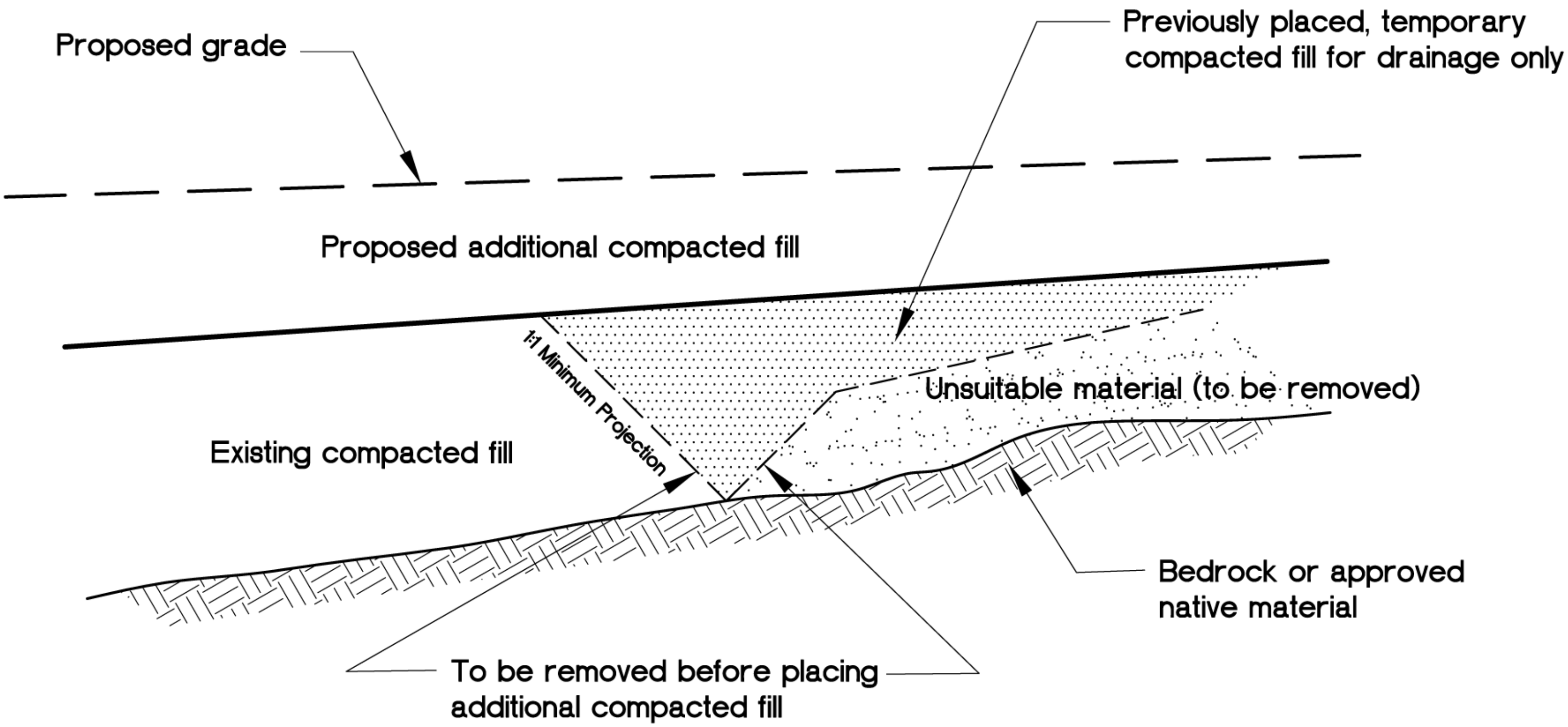
GeoSoils, Inc. | CANYON SUBDRAIN ALTERNATE DETAILS | Plate H-2



GeoSoils, Inc.

FILL SLOPE TOEING OUT ON FLAT ALLUVIATED CANYON DETAIL

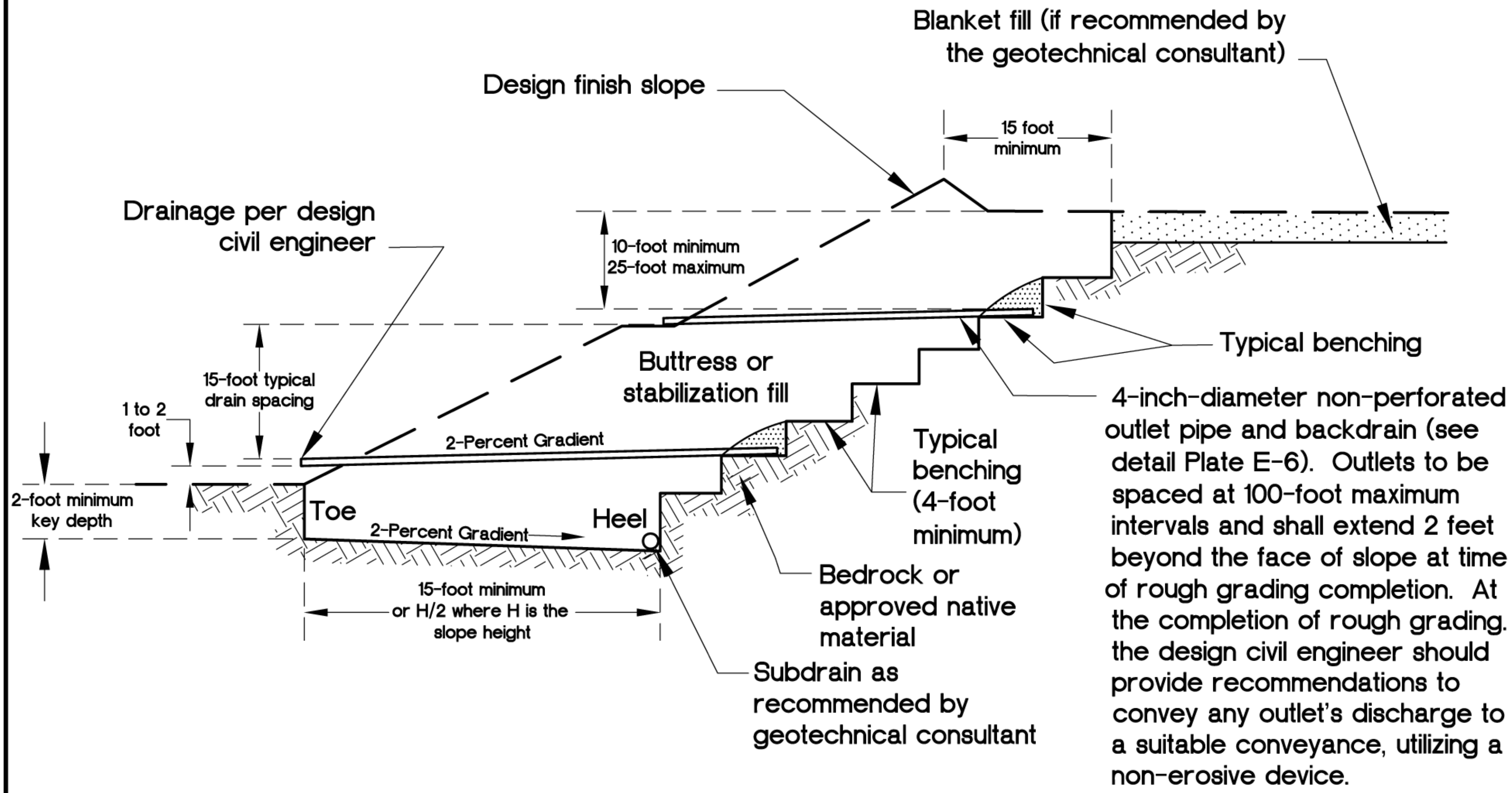
Plate H-3



GeoSoils, Inc.

REMOVAL ADJACENT TO EXISTING FILL ADJOINING CANYON FILL DETAIL

Plate H-4

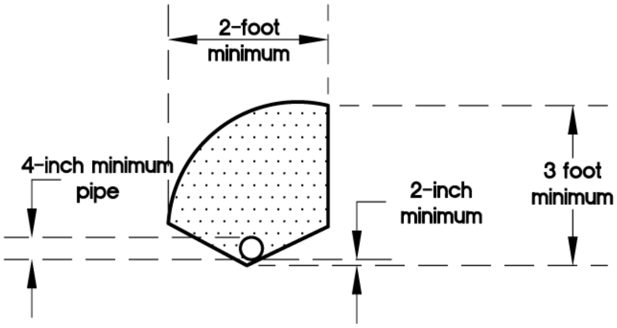


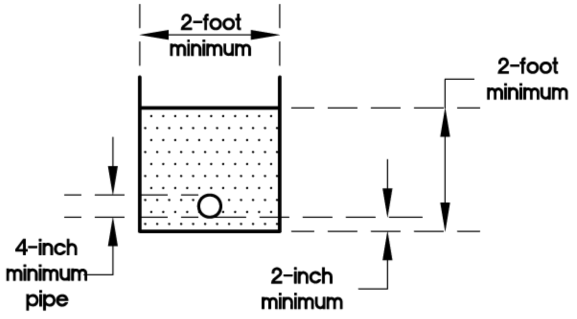
4-inch-diameter non-perforated outlet pipe and backdrain (see detail Plate E-6). Outlets to be spaced at 100-foot maximum intervals and shall extend 2 feet beyond the face of slope at time of rough grading completion. At the completion of rough grading. the design civil engineer should provide recommendations to convey any outlet's discharge to a suitable conveyance, utilizing a non-erosive device.

GeoSoils, Inc.

TYPICAL STABILIZATION / BUTTRESS FILL DETAIL

Plate H-5





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

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

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

Notes: 1. Trench for outlet pipes to be backfilled and compacted with onsite soil.

2. Backdrains and lateral drains shall be located at elevation of every bench drain. First drain located at elevation just above lower lot grade. Additional drains may be required at the discretion of the geotechnical consultant.

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

Sieve Size	Percent Passing
1 inch	100
¾ inch	90-100
⅜ inch	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

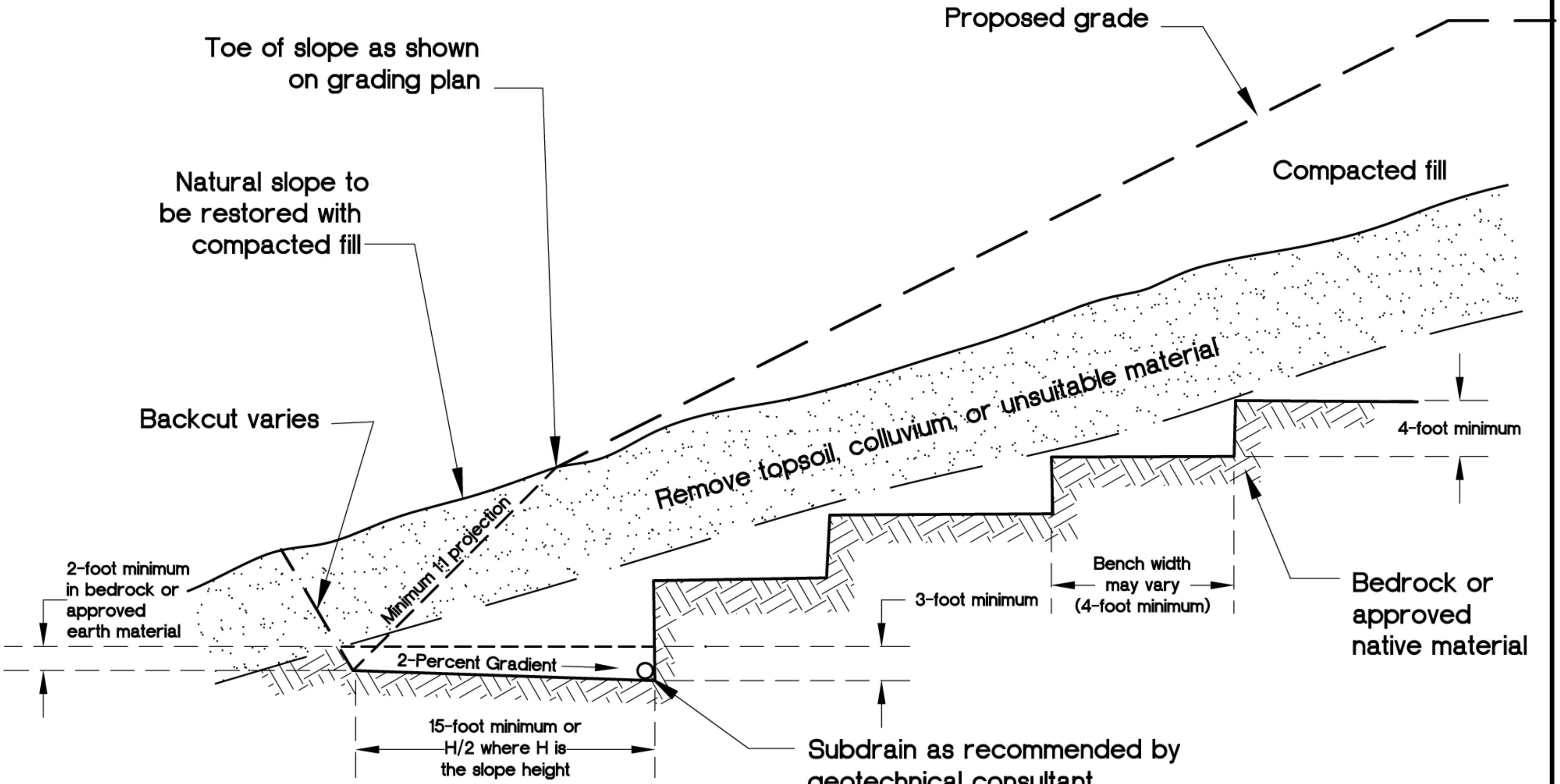
Gravel shall be of the following specification or an approved equivalent.

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

GeoSoils, Inc.

TYPICAL BUTTRESS SUBDRAIN DETAIL

Plate H-6



NOTES:

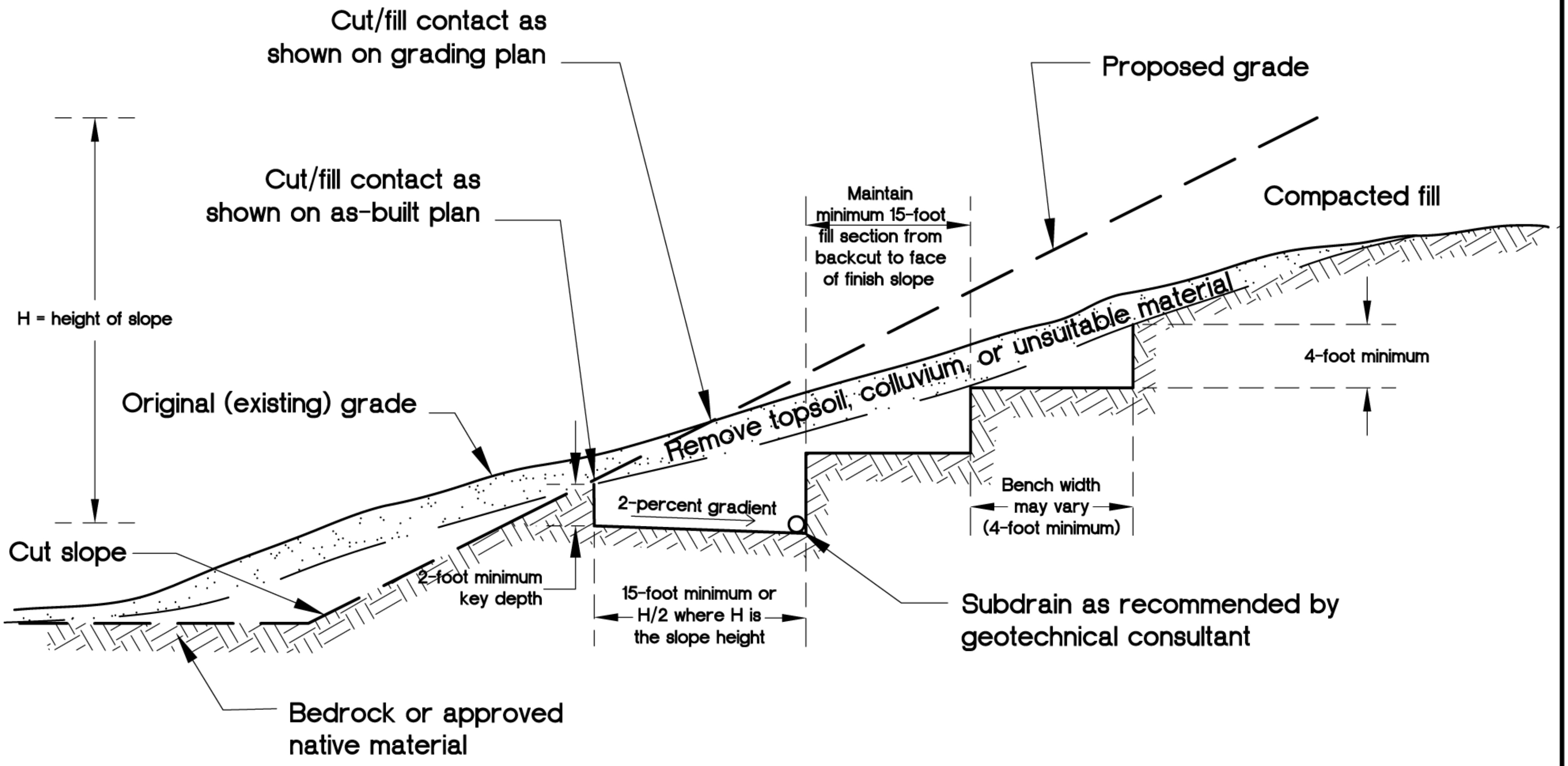
1. Where the natural slope approaches or exceeds the design slope ratio, special recommendations would be provided by the geotechnical consultant.

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 H-7

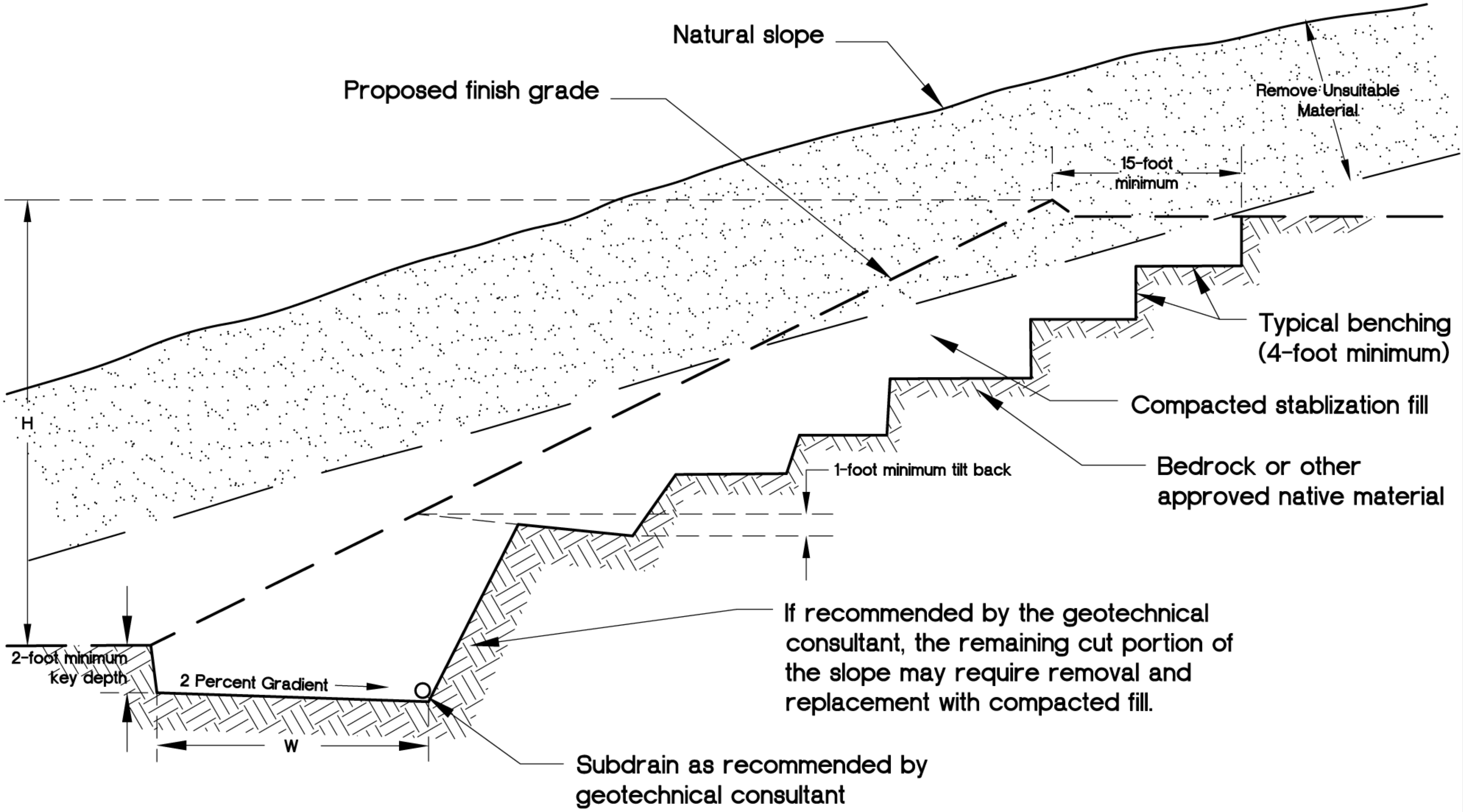


NOTE: The cut portion of the slope should be excavated and evaluated by the geotechnical consultant prior to construction of the fill portion.

GeoSoils, Inc.

FILL OVER CUT DETAIL

Plate H-8



NOTES:

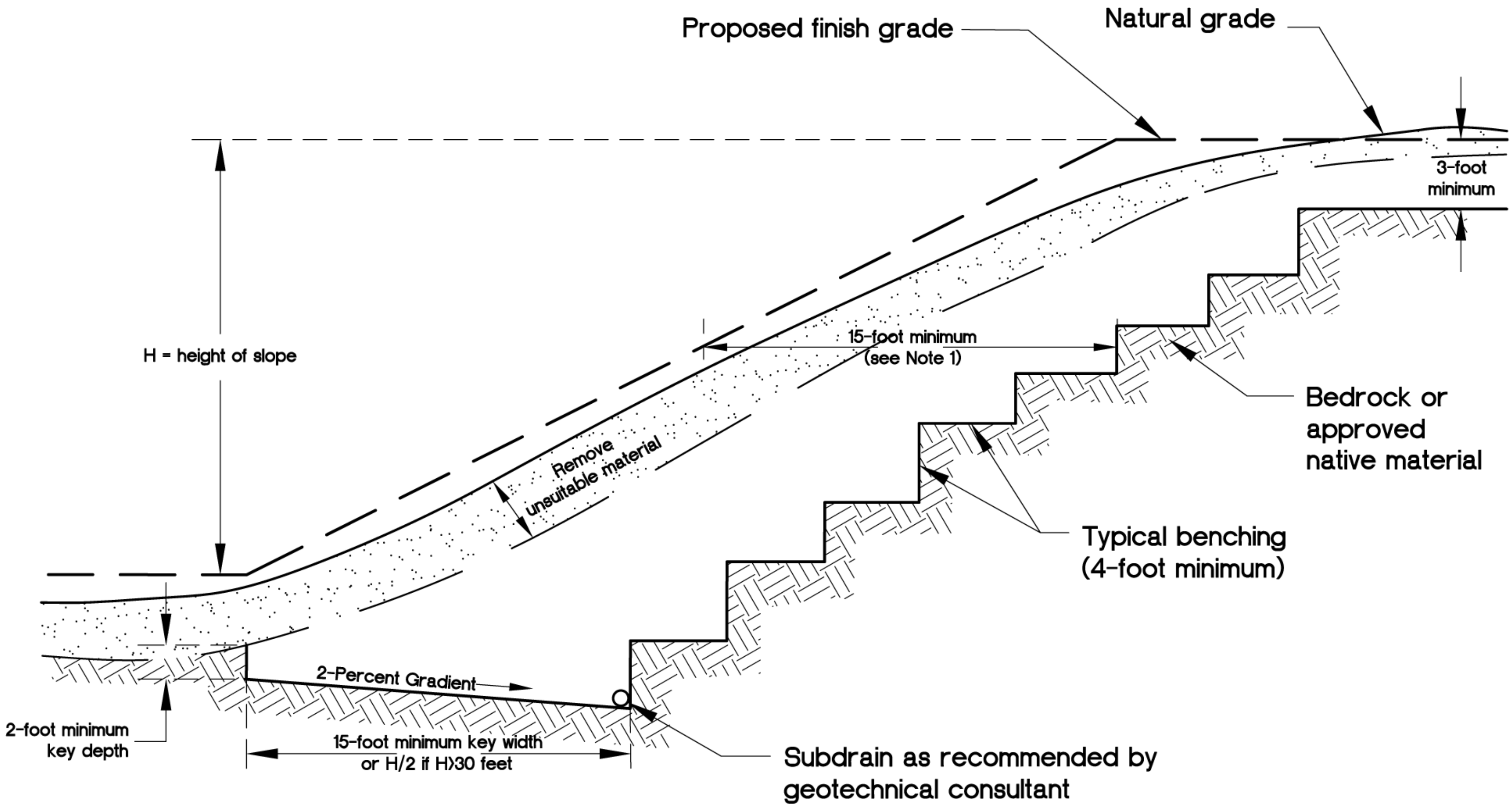
1. Subdrains may be required as specified by the geotechnical consultant.

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

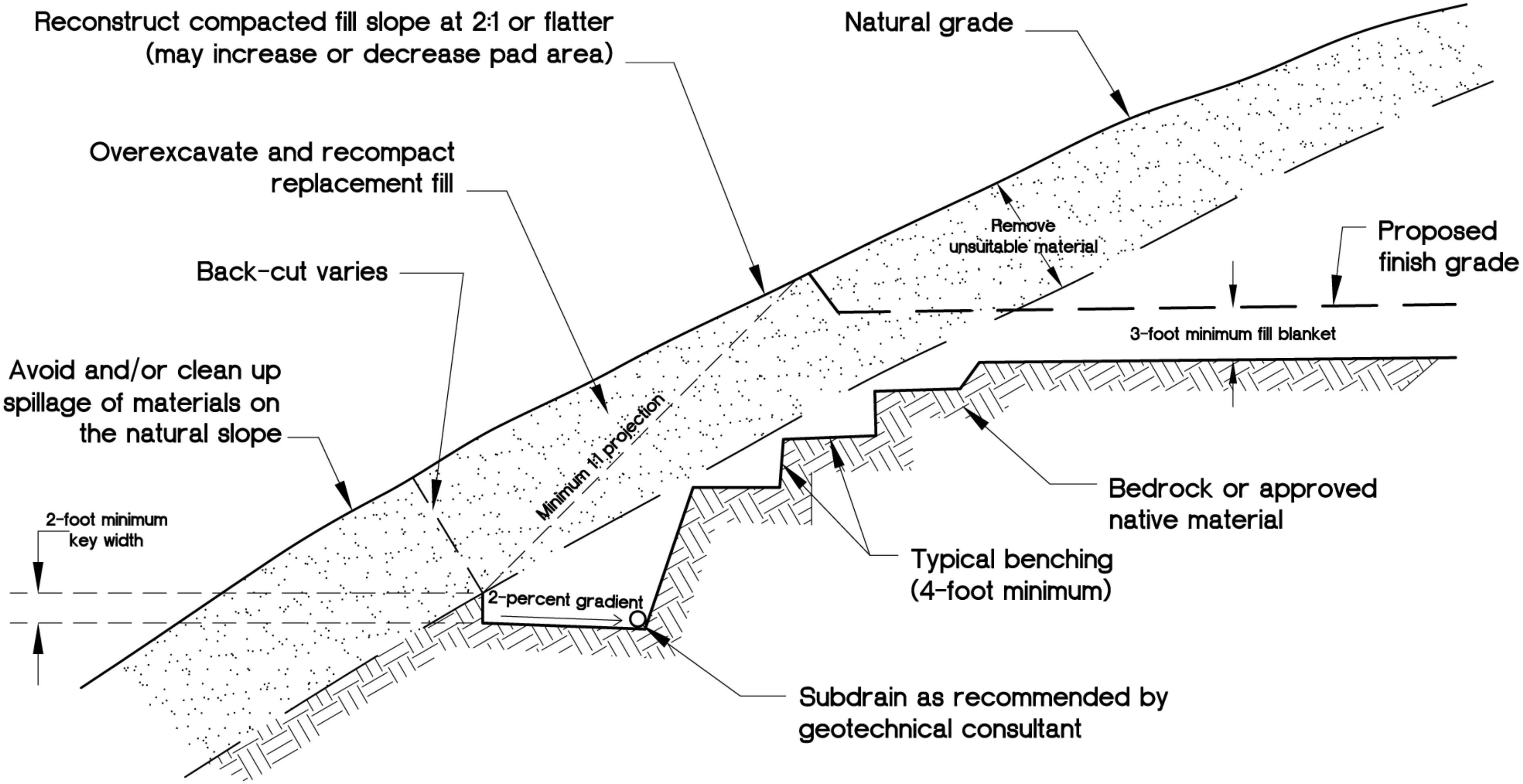
Plate H-9



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 H-10**



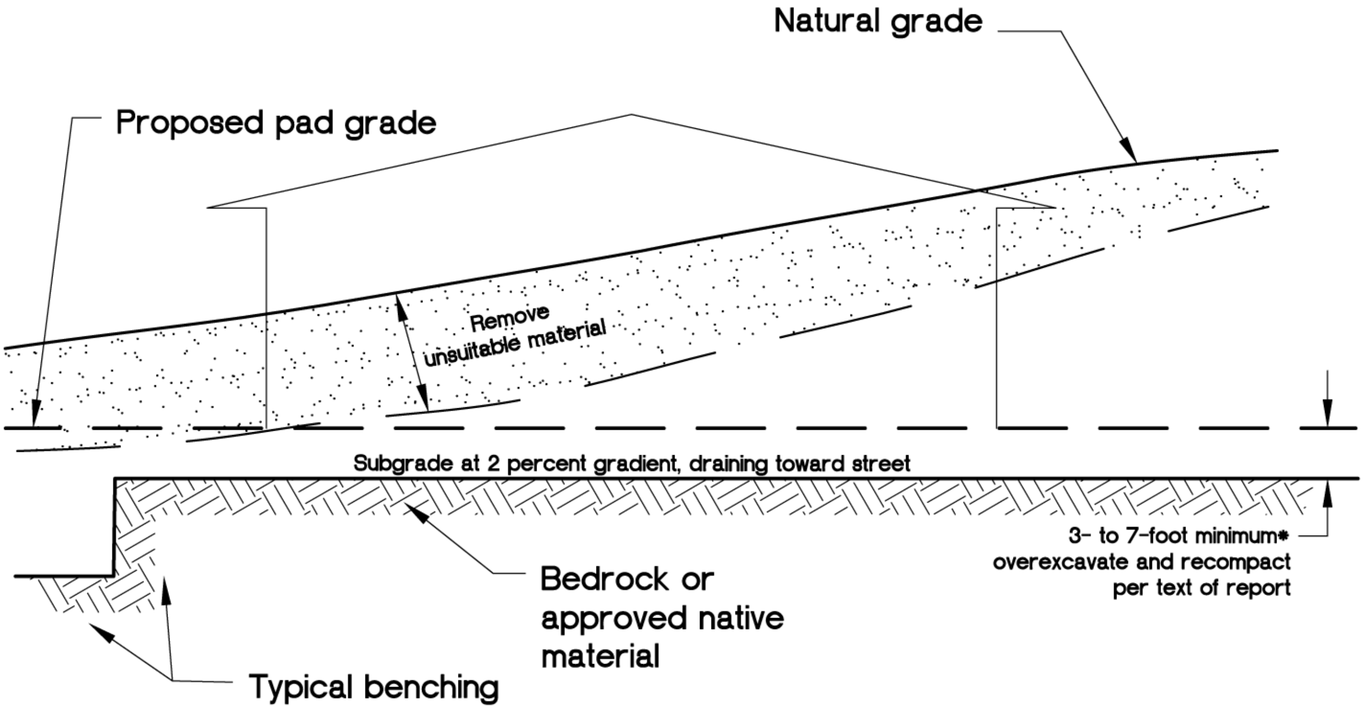
NOTES:

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.

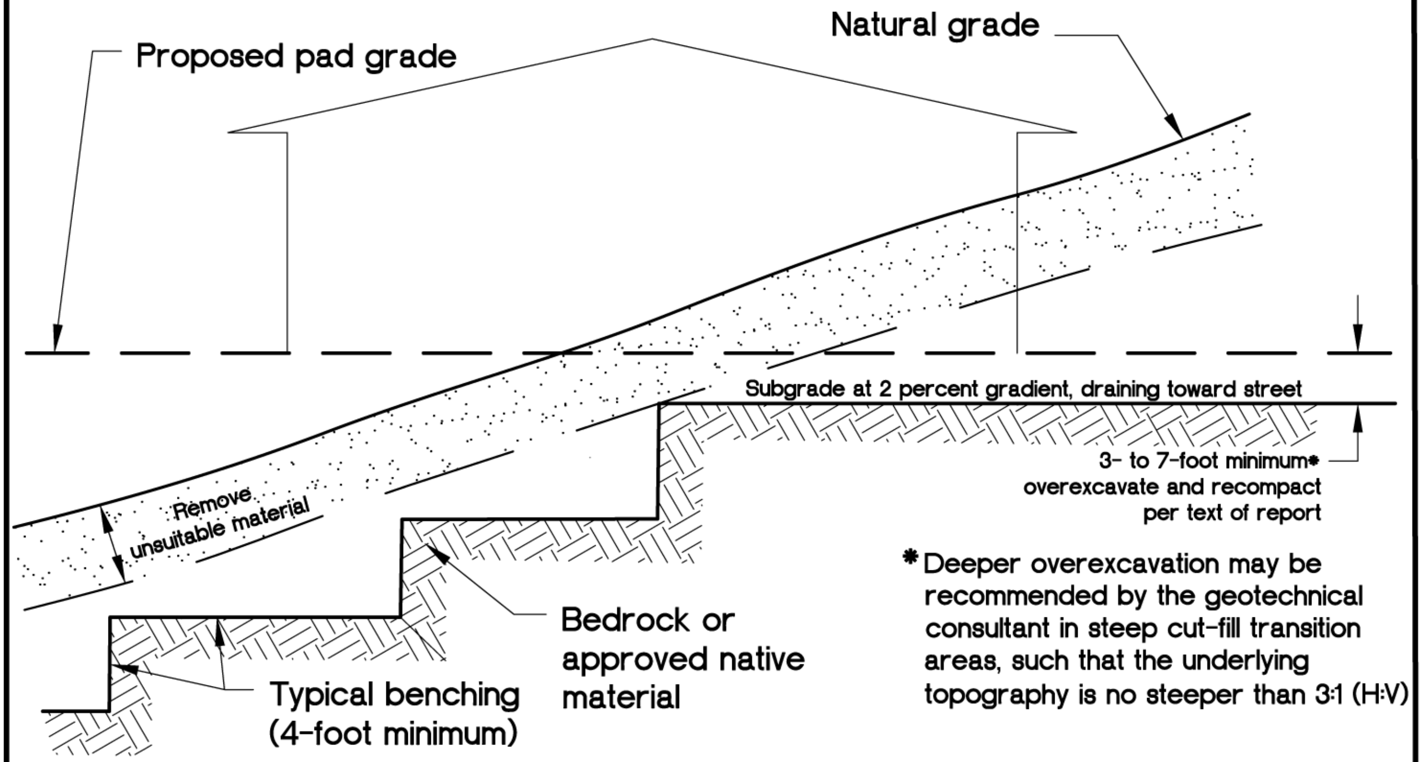
GeoSoils, Inc.

DAYLIGHT CUT LOT DETAIL

Plate H-11



CUT LOT OR MATERIAL-TYPE TRANSITION

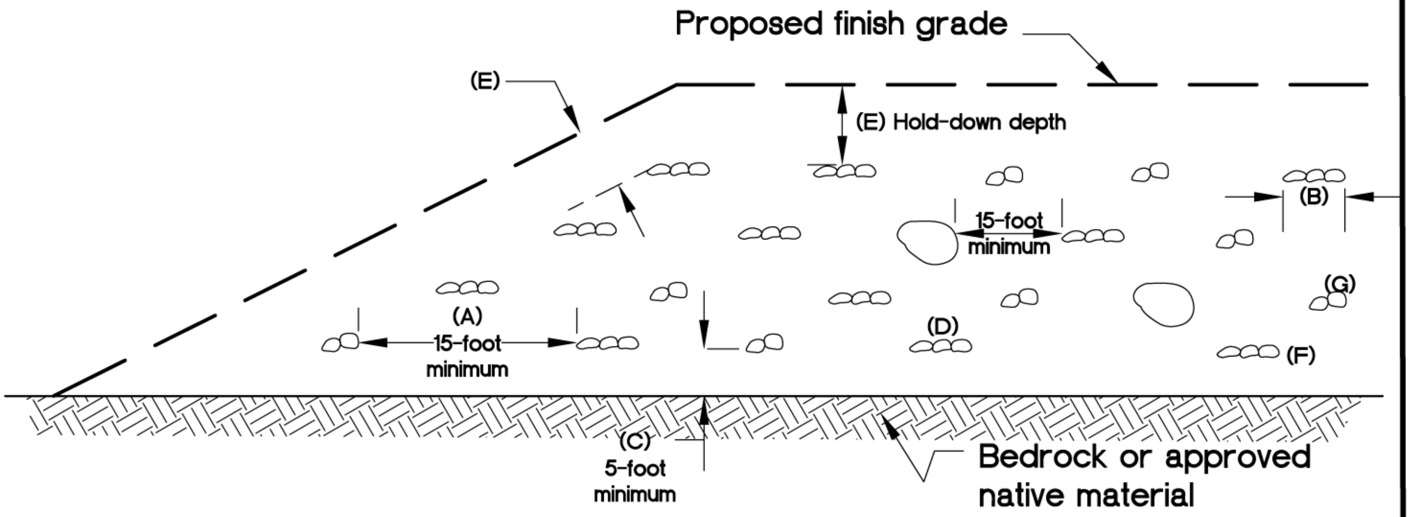


* Deeper overexcavation may be recommended by the geotechnical consultant in steep cut-fill transition areas, such that the underlying topography is no steeper than 3:1 (H:V)

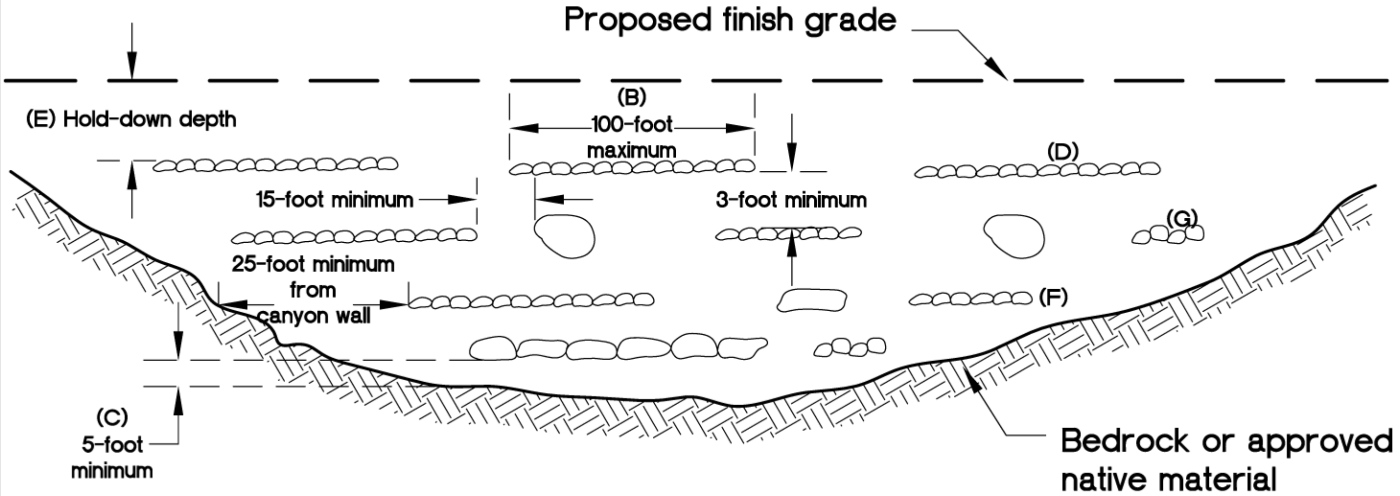
CUT-FILL LOT (DAYLIGHT TRANSITION)

GeoSoils, Inc.	TRANSITION LOT DETAILS	Plate H-12

VIEW NORMAL TO SLOPE FACE



VIEW PARALLEL TO SLOPE FACE



NOTES:

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

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

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

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

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

F. All fill over and around rock windrow shall be compacted to at leas ompaction or as recommended.

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

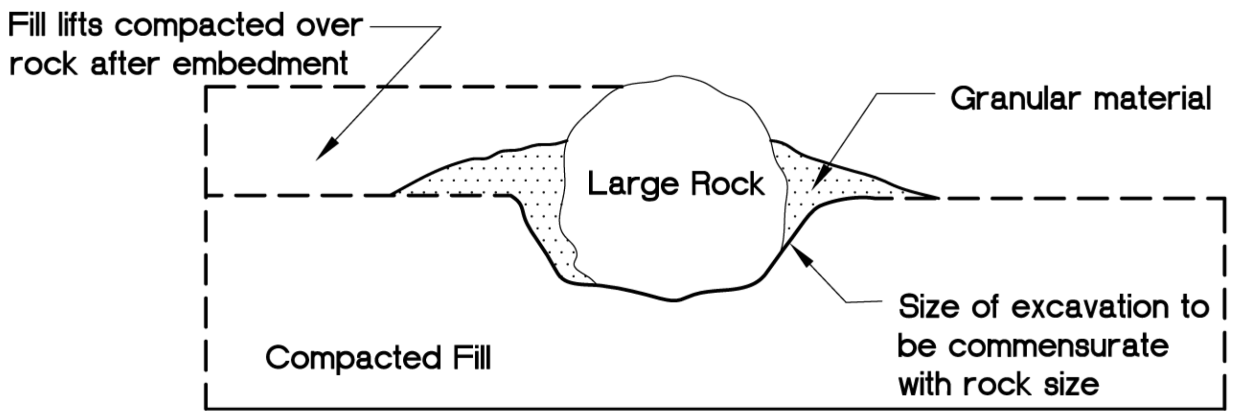
VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE
ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED

GeoSoils, Inc.

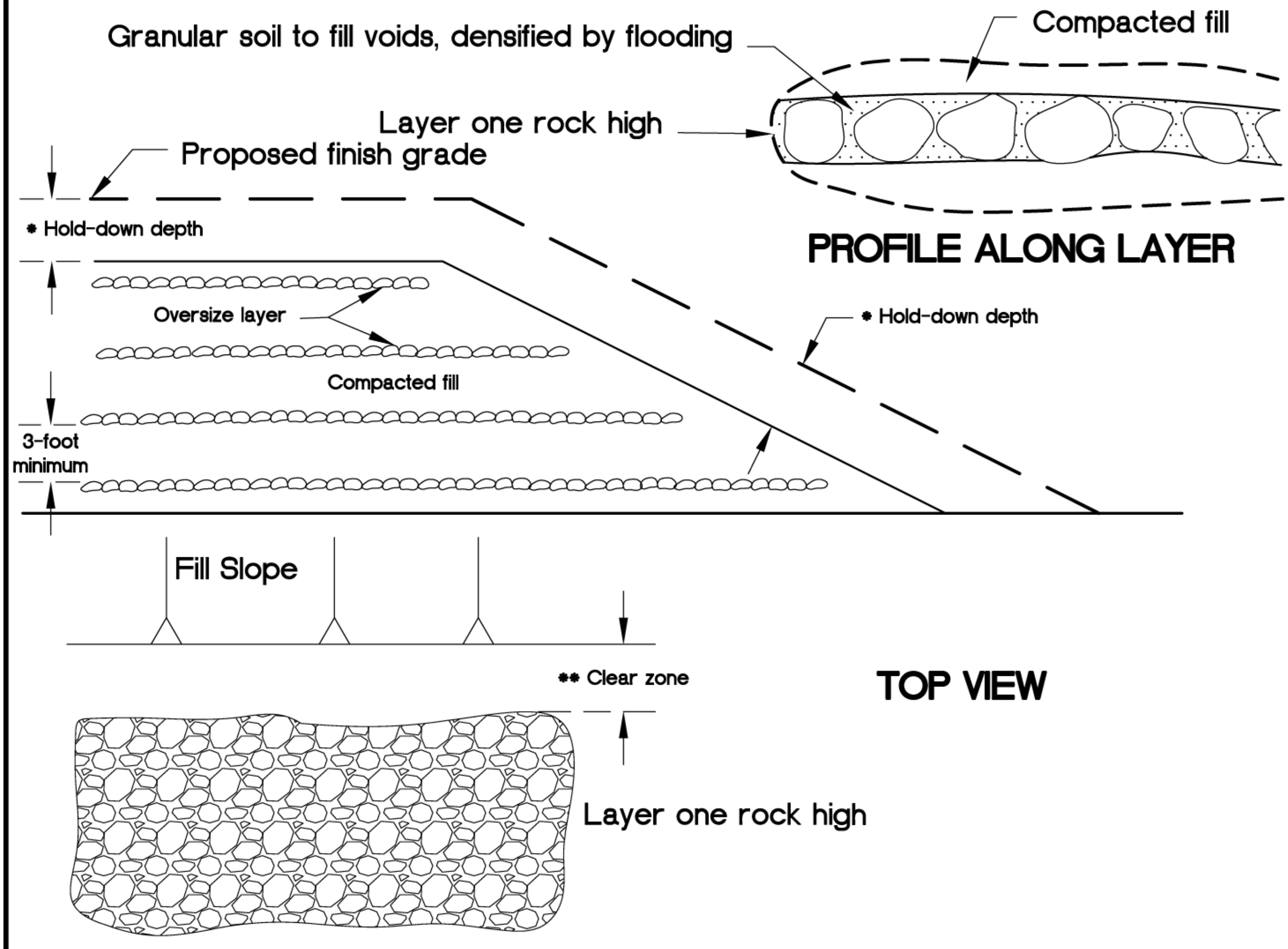
OVERSIZE ROCK DISPOSAL DETAIL

Plate H-13

ROCK DISPOSAL PITS



ROCK DISPOSAL LAYERS

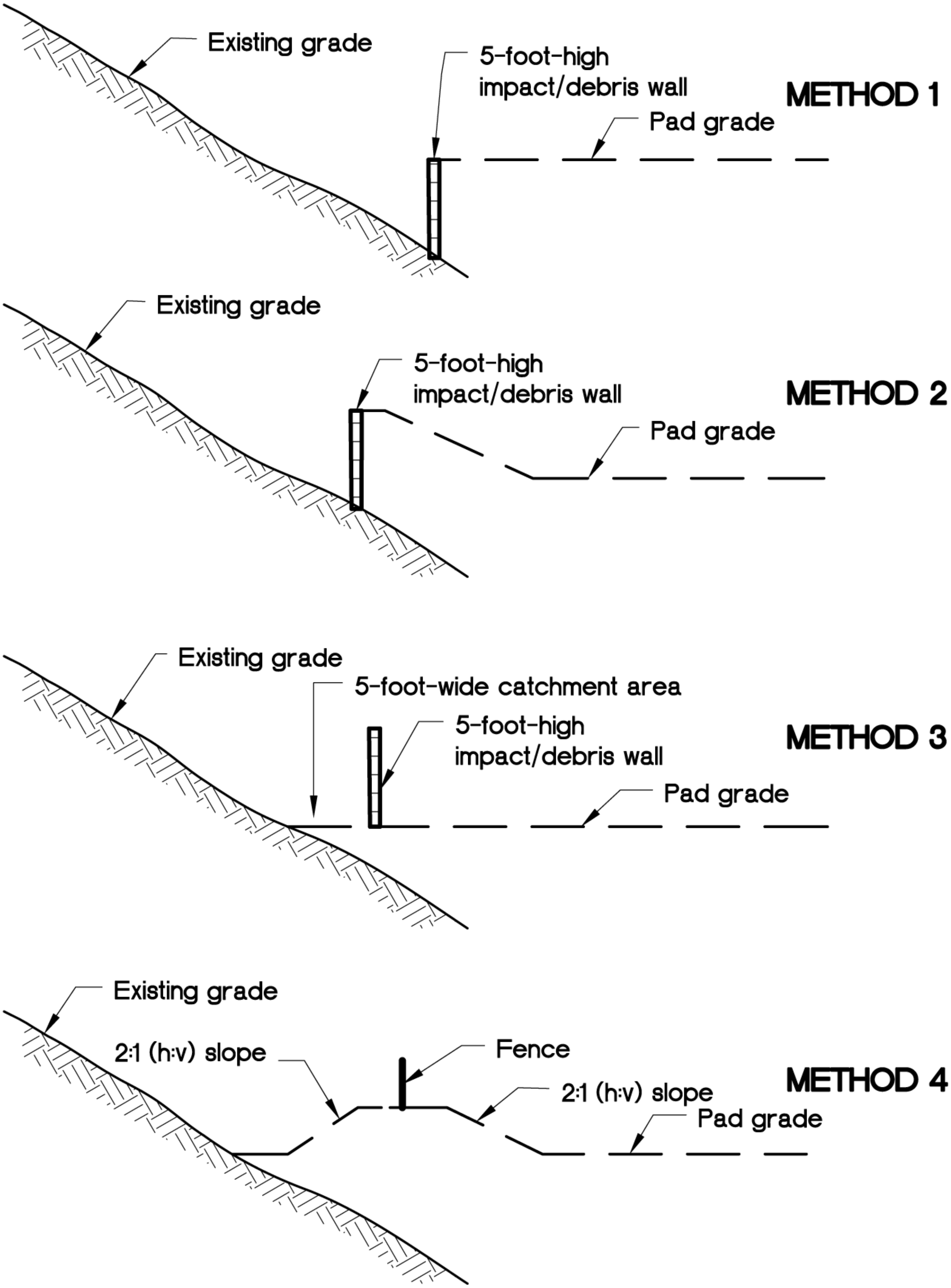


* Hold-down depth or below lowest utility as specified in text of report, subject to governing agency approval.

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

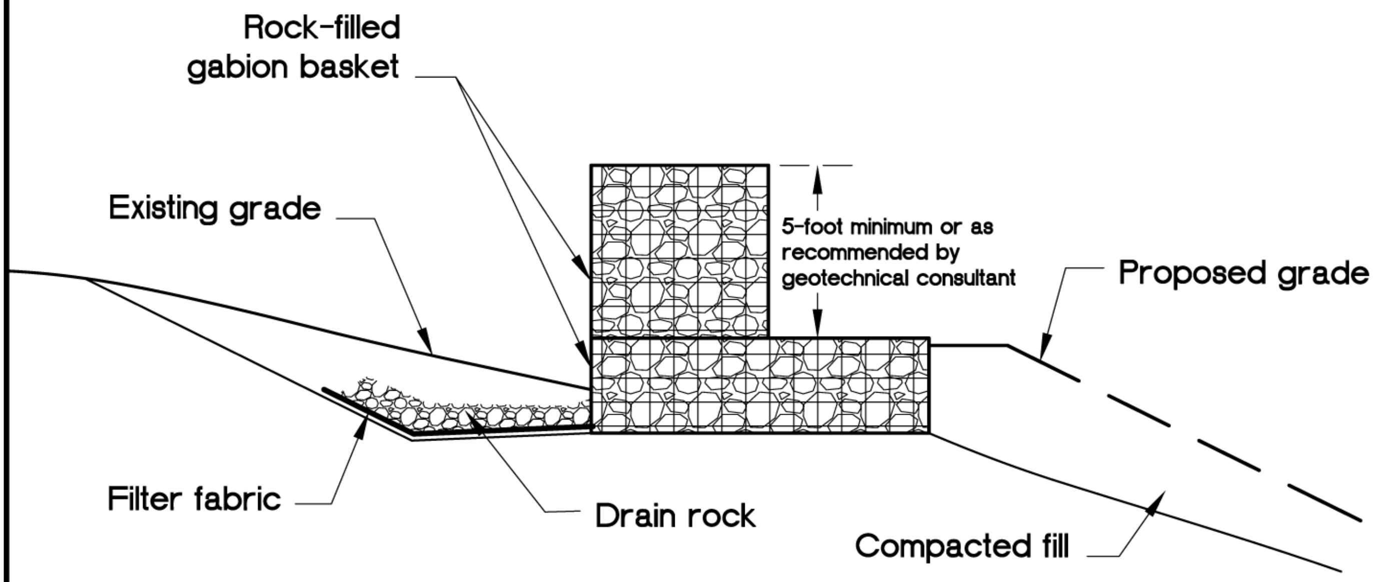
VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE
ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED IN

GeoSoils, Inc.	ROCK DISPOSAL DETAIL	Plate H-14



NOT TO SCALE

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



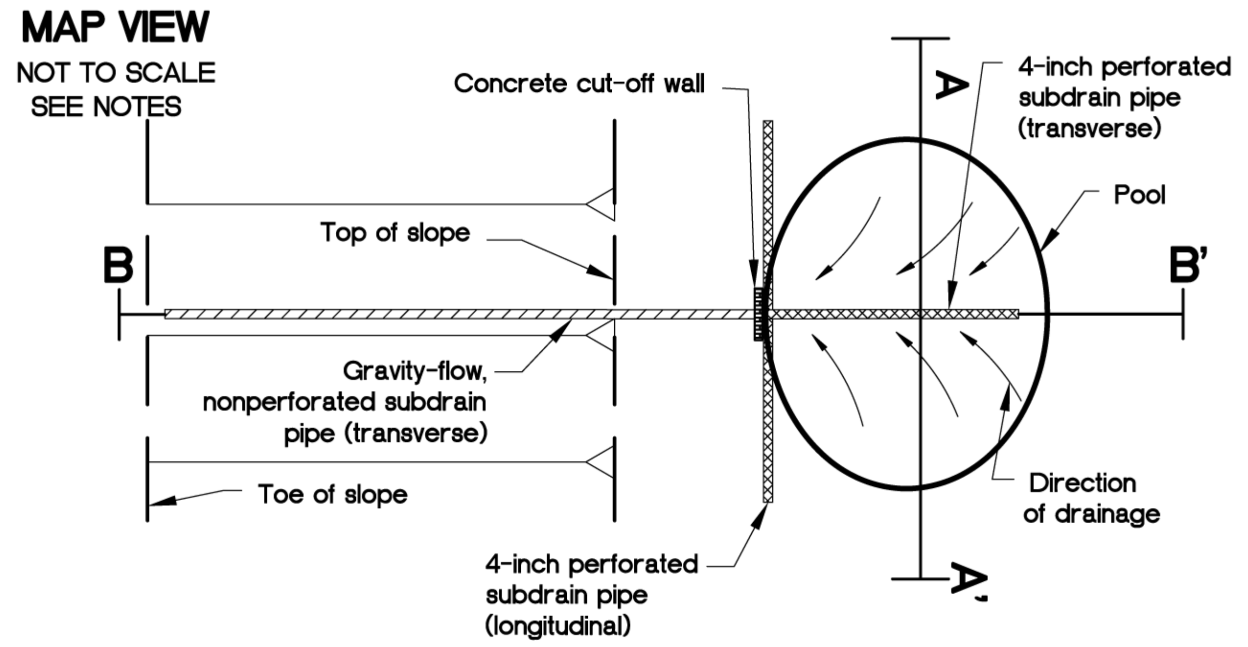
Gabion impact or diversion wall should be constructed at the base of the ascending slope subject to rock fall. Walls need to be constructed with high segments that sustain impact and mitigate potential for overtopping, and low segment that provides channelization of sediments and debris to desired depositional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

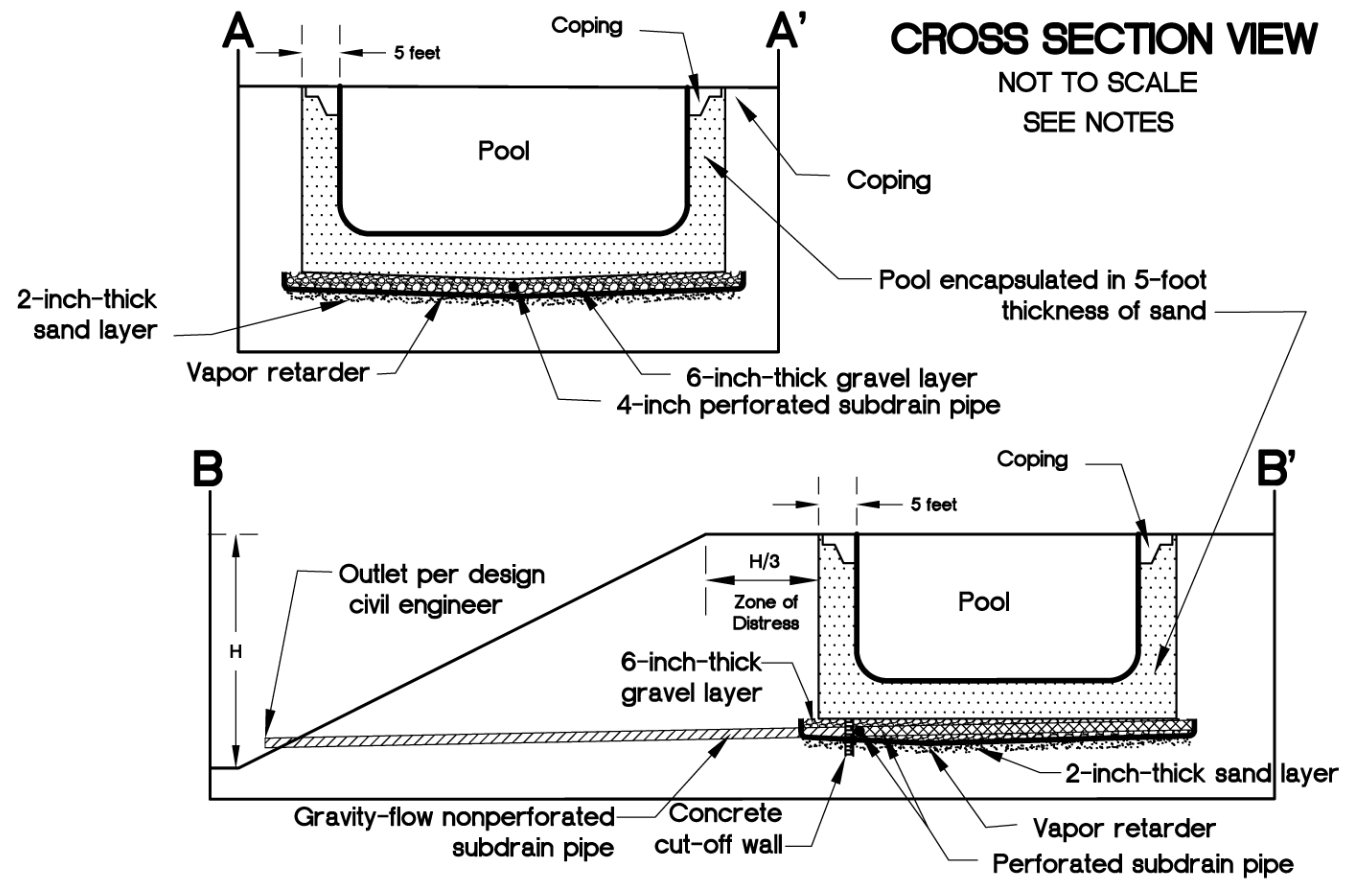
From GSA, 1987

GeoSoils, Inc.

ROCK FALL MITIGATION DETAIL

Plate H-16





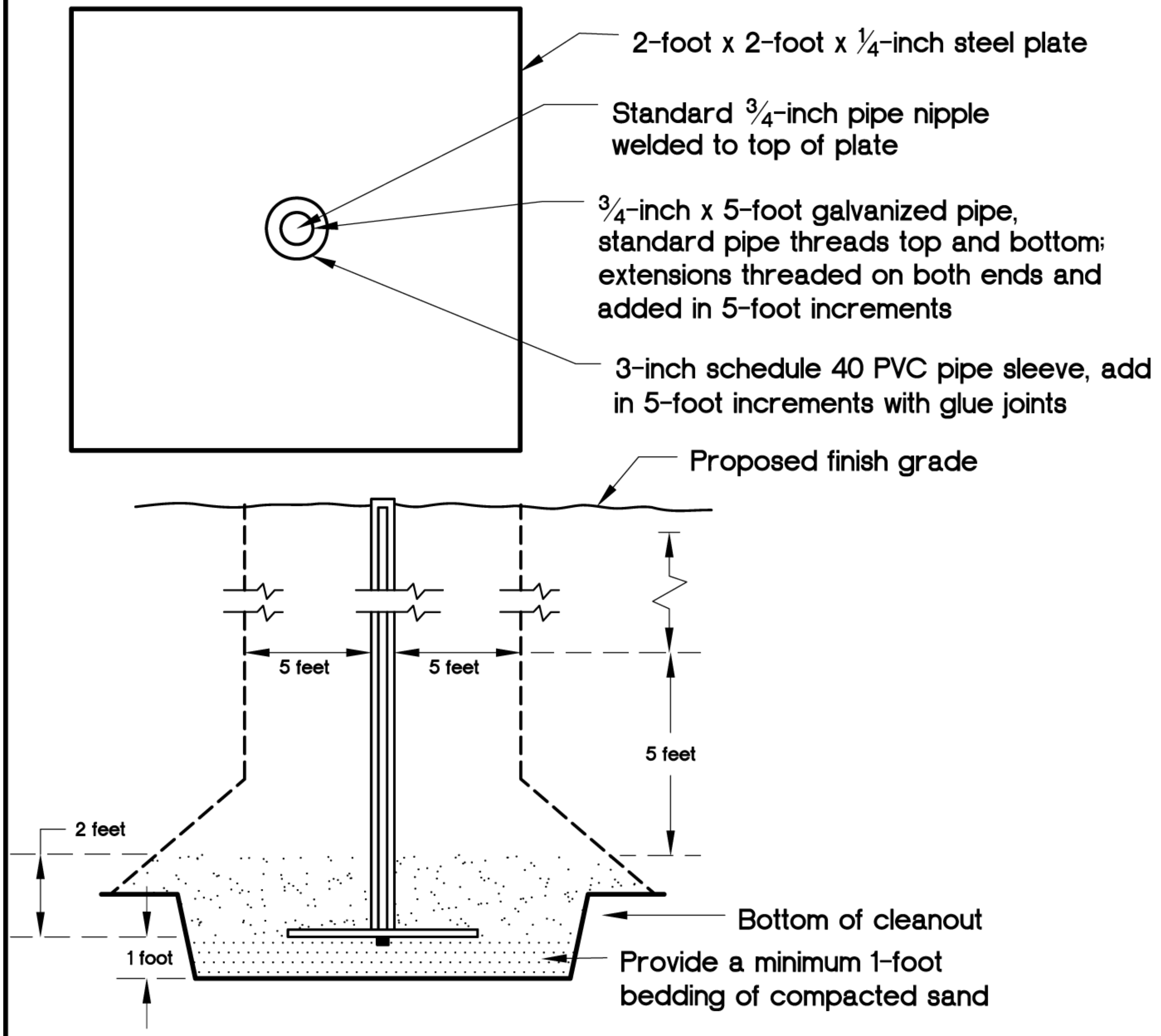
NOTES:

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

GeoSoils, Inc.

TYPICAL POOL/SPA DETAIL

Plate H-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.

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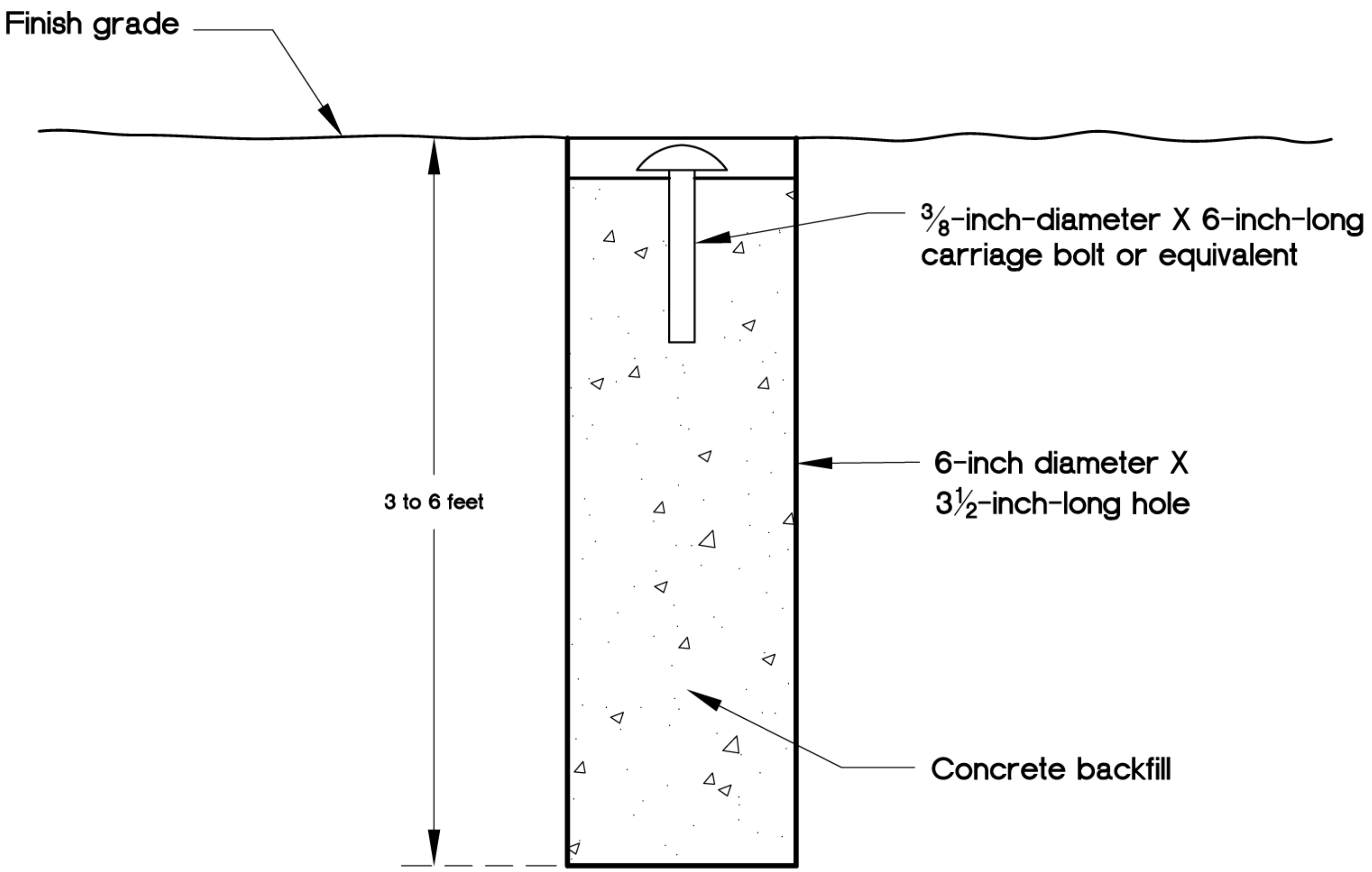
SETTLEMENT PLATE AND RISER DETAIL

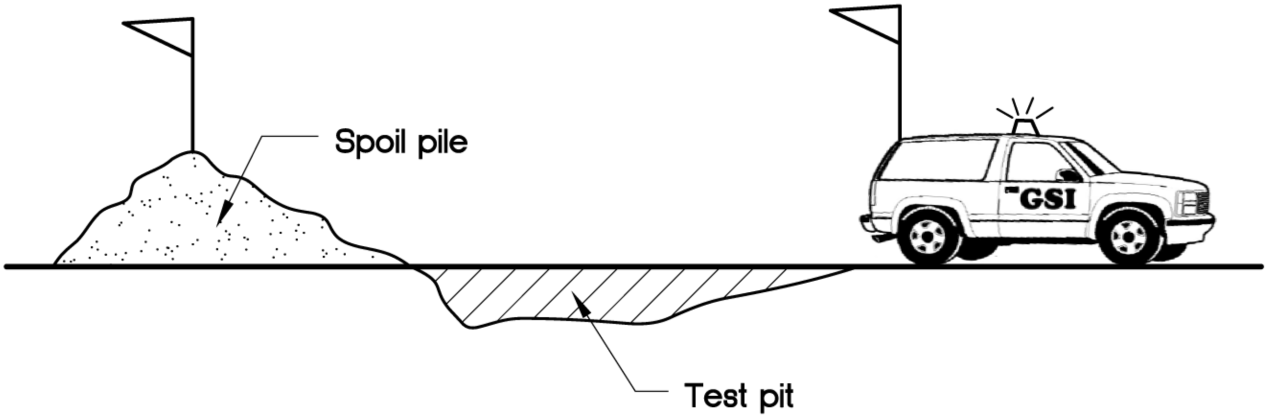
Plate H-18



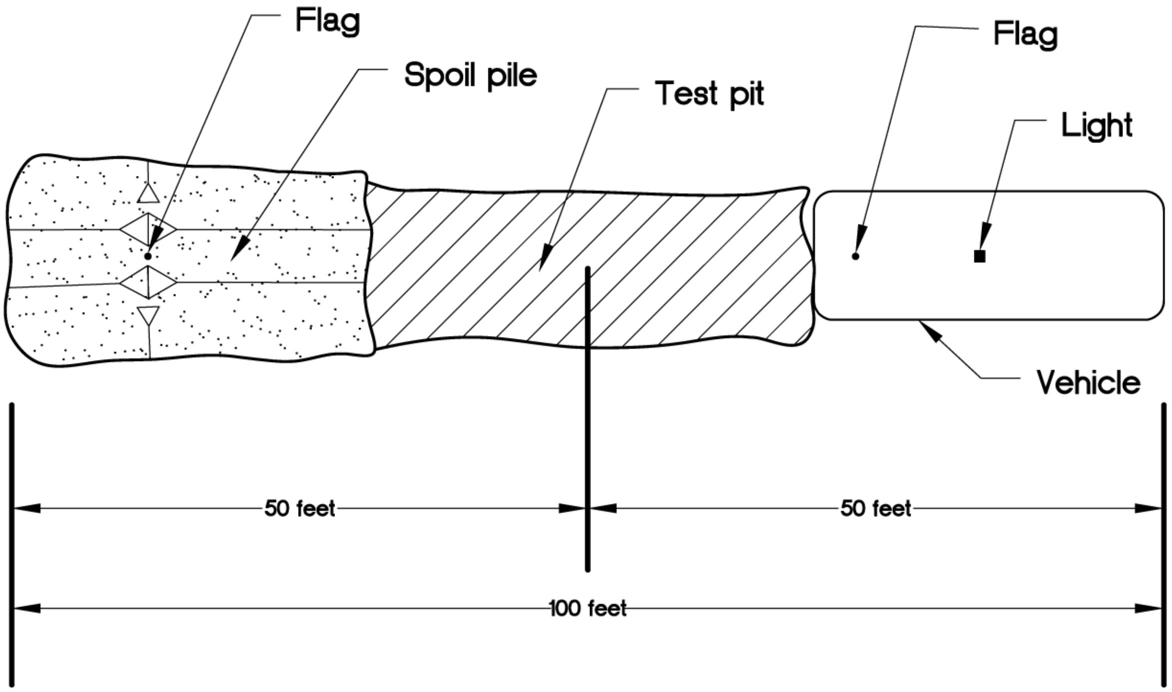
GeoSoils, Inc.

TYPICAL SURFACE SETTLEMENT MONUMENT

SIDE VIEW



TOP VIEW



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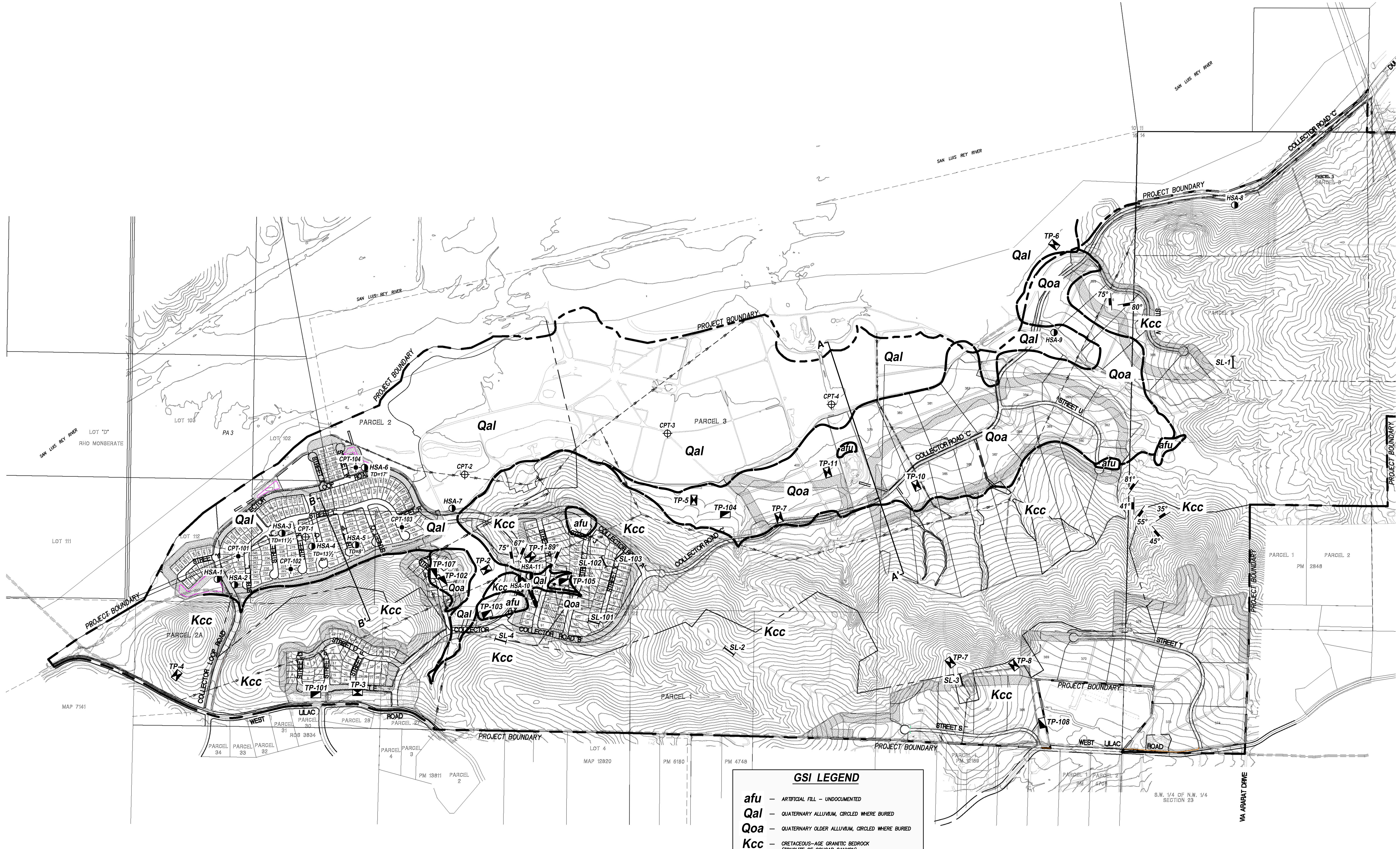
TEST PIT SAFETY DIAGRAM

Plate H-20



GSI LEGEND

afu	— ARTIFICIAL FILL – UNDOCUMENTED
Qal	— QUATERNARY ALLUVIUM, CIRCLED WHERE BURIED
Qoa	— QUATERNARY OLDER ALLUVIUM, CIRCLED WHERE BURIED
Kcc	— CRETACEOUS–AGE GRANITIC BEDROCK (TONOLITE OF COUGAR CANYON)
89°	— JOINT / FRACTURE ATTITUDE WITH DIP IN DEGREES
?	— APPROXIMATE LOCATION OF GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN
TP-108 ◩	— APPROXIMATE LOCATION OF EXPLORATORY TEST PIT (THIS STUDY)
TP-11 ☒	— APPROXIMATE LOCATION OF EXPLORATORY TEST PIT (GSI, 2015)
CPT-104 ●	— APPROXIMATE LOCATION OF CONE PENETRATION TEST WITH TOTAL DEPTH IN FEET (THIS STUDY)
CPT-4 ⊕	— APPROXIMATE LOCATION OF CONE PENETRATION TEST WITH TOTAL DEPTH IN FEET (GSI, 2015)
HSA-11 ◑ TD=51 ½'	— APPROXIMATE LOCATION OF HOLLOW–STEM AUGER BORING WITH TOTAL DEPTH IN FEET
SL-4	— APPROXIMATE LOCATION OF SEISMIC SURVEY LINE (GSI, 2015)
SL-104	— APPROXIMATE LOCATION OF SEISMIC SURVEY LINE (THIS STUDY)
B B'	— APPROXIMATE LOCATION OF GEOLOGIC CROSS SECTION

ALL LOCATIONS ARE APPROXIMATE

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Documents and should not be relied upon as being an
accurate depiction of design.

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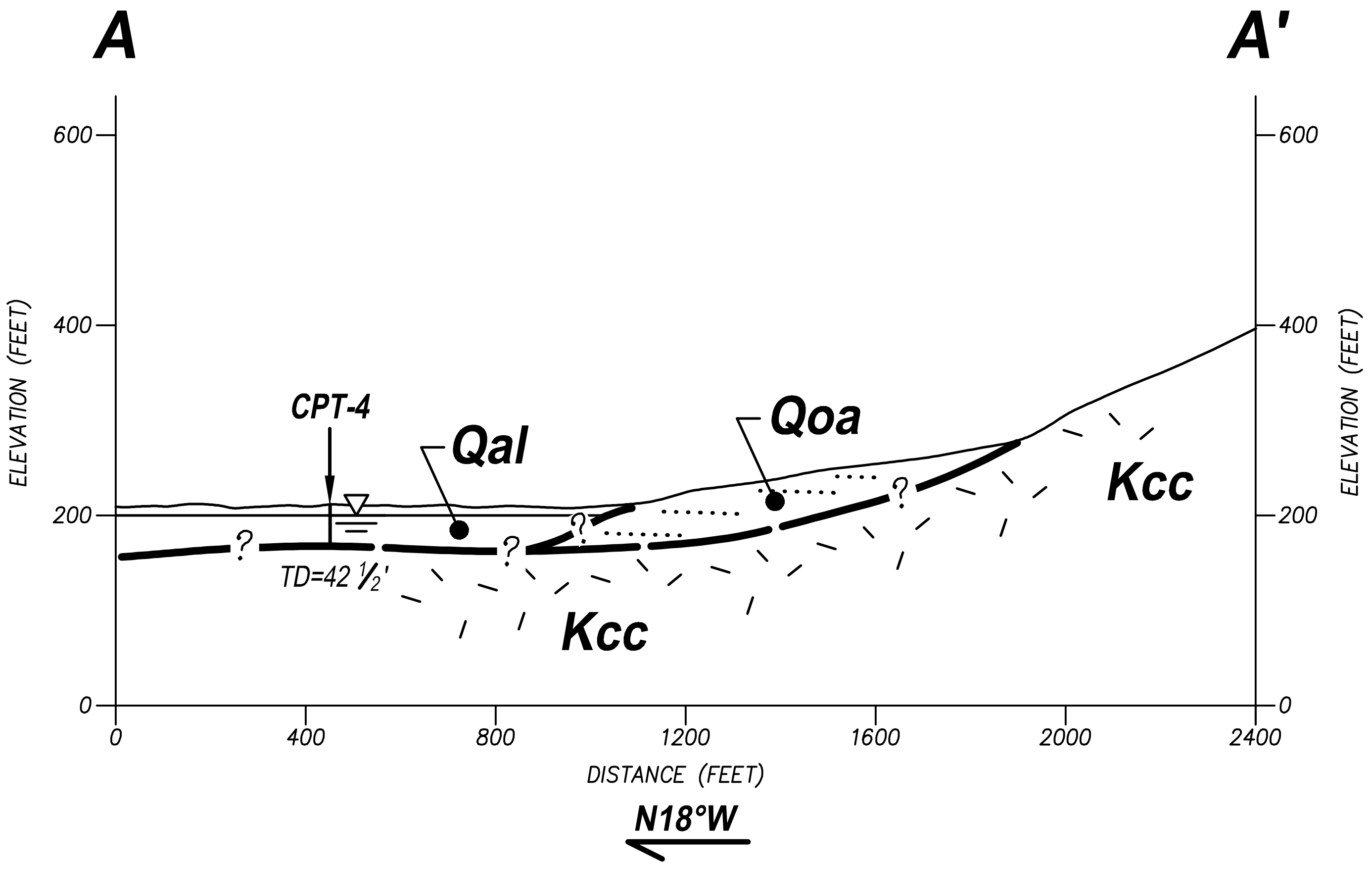
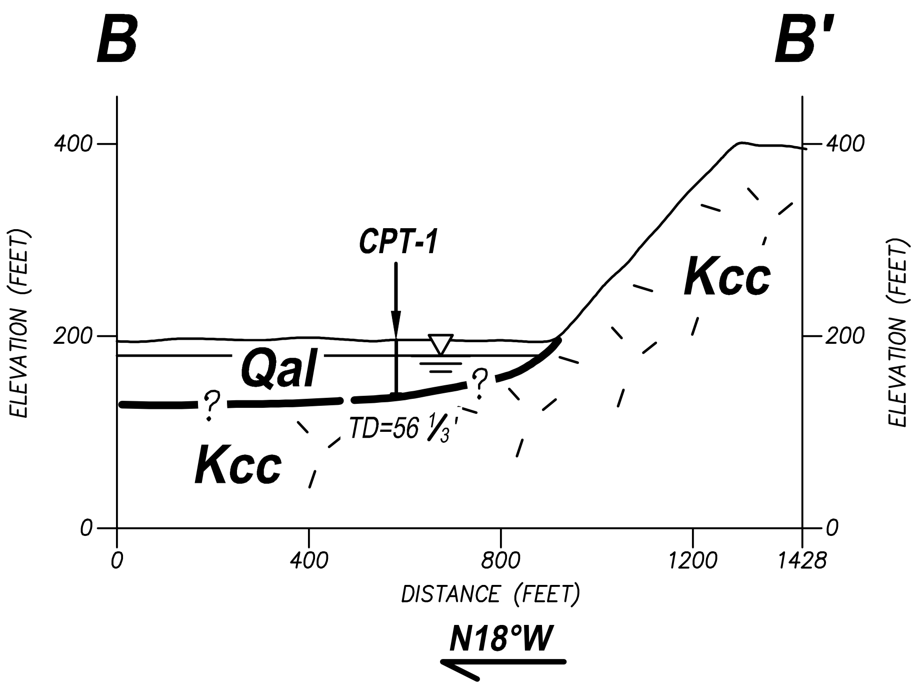
GEOTECHNICAL MAP

Plate 1

W.O. 6960-A-SC	DATE: 10/16	SCALE: 1" = 400'





GSI LEGEND

Qal — QUATERNARY ALLUVIUM

Qoa — QUATERNARY OLDER ALLUVIUM

Kcc — GRANITIC BEDROCK

—?— — APPROXIMATE LOCATION OF GEOLOGIC CONTACT (QUERIED WHERE UNCERTAIN)

▽ — APPROXIMATE ELEVATION OF GROUNDWATER SURFACE

CPT-4
TD=56 ⅓' — APPROXIMATE LOCATION (PROJECTED) OF CONE PENETRATION TEST SOUNDING WITH TOTAL DEPTH IN FEET

ALL LOCATIONS ARE APPROXIMATE

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GRAPHIC SCALES:
1" = 400' Horizontal
1" = 200' Vertical

GeoSoils, Inc.

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

Plate 2

W.O. **6960-A-SC** | DATE: **10/16** | SCALE: **As Shown**

GEOTECHNICAL FEASIBILITY EVALUATION FOR THE VESSELS STALLION RANCH BONSALL, SAN DIEGO COUNTY, CALIFORNIA

FOR

VESSELS STALLION RANCH
5772 CAMINO DEL REY
BONSALL, CALIFORNIA 92003

W.O. 6688-A-SC JANUARY 30, 2015

GeoSoils, Inc.

Geotechnical • Geologic • Coastal • Environmental

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

January 30, 2015

W.O. 6688-A-SC

Vessels Stallion Ranch
5772 Camino Del Rey
Bonsall, California 92003

Attention: Mr. William Thead

Subject: Geotechnical Feasibility Evaluation, Vessels Stallion Ranch, Bonsall, San Diego County, California

Dear Mr. Thead:

In accordance with your request and authorization, this report presents the results of GeoSoils Inc.'s (GSI's) geotechnical feasibility evaluation for the Vessels Stallion Ranch property in the community of Bonsall, San Diego County, California. The purpose of the study was to evaluate the on-site geotechnical and geologic conditions and their impacts on conceptual, mixed use site development, from a geotechnical feasibility viewpoint.

EXECUTIVE SUMMARY

Based on our review of the available data (see Appendix A), as well as field exploration (see Appendix B), seismicity analysis (see Appendix C), and geologic and engineering analysis, the proposed development of the property appears to be feasible from a geotechnical viewpoint, provided that mitigation measures presented in the text of this report are properly incorporated into design and construction of the project. The most significant elements of this study are summarized below:

- The site occupies the southern flank of a portion of the San Luis Rey River valley, consisting of a relatively flat-lying valley floor to the north, with bedrock highland to the south. Flat-lying ground in the vicinity of (primarily west of) Dulin Ranch Road, and generally within the 100-year flood plain, is underlain with Holocene alluvial sediments. Lower slopes descending to the valley floor, and flatter than about 4:1 (horizontal:vertical [h:v]) are developed on deposits of Quaternary (Pleistocene) age older alluvium (stream terrace deposits). Steeper slopes and upland areas are underlain with granitic bedrock.

- In general and based upon the available data to date, regional groundwater is not expected to be a major factor in development of the more elevated portions of the site (i.e., areas underlain with deposits of older alluvium and/or granitic bedrock). Within lower-lying areas underlain with alluvium, groundwater was encountered at depths ranging from approximately 13½ to 15½ feet below existing grade within the San Luis Rey River drainage area, and is anticipated to be a concern during development in these areas, including any deep utilities. This corresponds to elevations ranging from about 170 to 197 feet above Mean Sea Level (MSL) within the San Luis Rey River drainage. Additionally, owing to the relatively coarse-grained nature of near-surface soils, perched groundwater/sloughing should be anticipated during excavation.

- The presence of landslide deposits, slumps, or other significant forms of mass wasting were not observed within the site. Adverse geologic structures that would preclude project feasibility were not encountered.

- GSI's review and field exploration indicates no known active faults are crossing the site, and the site is not located within an Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). However, strong shaking should be anticipated should an earthquake occur on one of the nearby regional active faults, and liquefaction effects within alluvial soils should be anticipated, if not mitigated.

- The proposed structures and foundations, as well as other supporting infrastructure should be designed to resist seismic forces and deformation in accordance with the criteria contained in the 2013 California Building Code ([2013 CBC], California Building Standards Commission [CBSC], 2013). Based on our site-specific seismic hazard analysis, appropriate seismic design parameters are provided herein.

- Based on our analysis, the potential for liquefaction to adversely affect those portions of the site underlain with older alluvium and/or granitic bedrock is considered low. Regardless, some seismic induced deformation should be anticipated due to densification, and will be discussed herein. Owing to the depth to groundwater, relatively low density, grain size, young age and lack of cementation, the potential for liquefaction to adversely affect those portions of the site underlain with younger alluvium is higher, when subjected to the design level earthquake, based on the available data.

- Based upon our experience in this area, and the seismic refraction data obtained, assuming a D9L, or equivalent, bedrock within cut areas of the site appear to be rippable (i.e., seismic velocities of less than about 6,000 feet per second [fps]) at depths ranging up to ±30 feet from existing grade. Rock breakers and/or blasting should be considered during preliminary planning and budgeting for excavation depths (including foundations and utilities) greater than about ±30 feet from existing grade, on a preliminary basis.

- Using the 3,800 fps cut-off for non-rippable trenching, assuming a CAT 235 hoe, or equivalent, it is likely that some areas will require blasting (e.g., "line-shooting") for trenching of utilities onsite. Seismic velocities near, or exceeding 3,800 fps generally occur at depths ranging from depths as shallow as ±3 foot, to as deep as

±19 feet from existing grade. A conventional backhoe would likely encounter practical refusal at shallower depths.

- Excavation within bedrock areas exhibiting a seismic velocity of ≥5,000 fps will generate appreciable quantities of oversize rock >12 inches in size, requiring specialized placement techniques during grading. In addition, hard rock requiring blasting, rock breakers, etc., may not be entirely precluded from occurring near the surface, and may also generate oversize rock. Accordingly, oversize rock (<24 inches in size), may be placed in fills deeper than 10 feet from finish grade, subject to governing agency approval, or may be crushed to reduce their size for standard fill placement. Considering the thickness of proposed fills and the proximity of groundwater below existing grade, there are limited areas on the project that will accommodate the hold-down distance of 10 feet below finish grade, and that have significant volume for oversize material placement. Thus, onsite crushing of oversize materials to less than 12 inches may be necessary. This condition will need value engineering to evaluate the feasibility of either oversize rock placement and/or crushing oversize materials onsite.

- Representative samples of near surface site soils were tested for expansion potential. The Expansion Index (E.I.) test was performed in general accordance with ASTM Standard D 4829. The laboratory test results indicate that the soil expansion potentials are generally very low (E.I. 0-20). However, this does not preclude the presence of higher expansive soils onsite.

- A representative sample of site material has also been evaluated for corrosion, soluble sulfate, etc. Laboratory testing indicates that site soils generally have a negligible (not applicable) sulfate exposure to concrete, per Table 4.2.1 of ACI 318-11 (per the 2013 CBC [CBSC, 2013]), and the use of Type V cement is not required. Corrosion testing (pH/resistivity) indicates that the soils are slightly alkaline (pH of 6.99) with respect to soil acidity/alkalinity, and is mildly corrosive to ferrous metals when saturated (saturated resistivity of 1800 ohm-cm [California Highway Design Manual, 2012]). Chloride content of the soil was measured as 122 ppm, which is slightly elevated. Alternative testing methods and additional comments should be obtained from a qualified corrosion engineer with regard to foundations, piping, etc. Additional corrosion testing should be performed at the completion of site grading to further evaluate geotechnical pad characteristics.

- A settlement analysis was performed for three (3) general, as-built conditions anticipated onsite, in consideration of both static and dynamic settlement. Group 1 areas (i.e., northern portion of the proposed Equestrian/Estate area) would consist of engineered fills placed over older alluvium, Group 2 areas (i.e., PA-3, PA-4, and PA-5) would generally consist of engineered fills placed over granitic bedrock, and Group 3 areas (PA-1, PA-2, and the southern part of the Equestrian/Estate area) would be where portions of the site overly alluvium below the groundwater table. Group 3 areas may also display an increased potential to be affected by lateral spreading during a seismic event. A discussion of settlement potential for each general area is presented in the text of this report.

- It should be kept in mind that drainage reversals could occur, when considering post-construction static and seismic settlement, if relatively flat yard drainage gradients are not periodically maintained in areas underlain by alluvium. Similarly, gravity flow utilities in areas underlain by alluvium are also subject to possible drainage reversals or deflections, considering the magnitude and angular distortions of settlement reported herein.

- The treatment of existing ground prior to fill placement for specific areas of the site will vary according to each of the following two (2) general cases:

  Case I - Areas underlain with near surface, older alluvium and/or granitic bedrock.

  Case II - areas underlain with loose alluvium and a shallow groundwater table (i.e., alluvium left in place below the groundwater table).

  A discussion of specific recommendations for each case is included in the text of this report.

- All existing structures, utilities, deleterious debris, and vegetation should be removed from the site and properly disposed, should settlement-sensitive improvements be proposed within their influence. It should be noted that the 2013 CBC (CBSC, 2013) indicates that for fill placed under the purview of the grading permit, removals of unsuitable soils be performed across all areas to be graded, not just within the influence of the 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 to mitigate site perimeter conditions or existing utilities. Thus, any settlement-sensitive improvements (walls, curbs, flatwork, etc.), constructed within this zone, may require deepened foundations, reinforcement, etc., or will retain some potential for settlement and associated distress. Current conditions indicate compressible colluvium, alluvium, weathered older alluvium, and bedrock, which should be included in remedial grading efforts.

- In general, support of the new building(s) and structures may be provided entirely by engineered and compacted fill. As discussed herein, onsite soils appear to be very low, to possibly low expansive. However, the potential for medium expansive soils cannot be precluded locally.

- Based on the underlying conditions supporting engineered fills onsite, the as-built conditions will likely result in at least three (3) different foundation design/construction scenarios.

Category I - Foundations are considered to be "conventional" slab on grade foundations supported by fills overlying, very low expansive, suitable deposits of older alluvium and/or granitic bedrock (Settlement Group Areas 1 and 2). The use of a "stiffened" mat type foundation (per the Wire Reinforcement Institute [WRI, 1981; 1996]) may also be considered if grading results in areas of low expansive soil, where the E.I. is greater than 20, and the plasticity Index (PI) is greater than 15, provided that design meets 2013 CBC guidelines, and is in accordance with as-graded site soil conditions.

Category II - Foundations are considered to be post tension slab foundations and may be used for conditions applicable to Category I foundations. The use of PT foundations for lots underlain with alluvium left in place (Group 3 settlement areas) may also be considered with ground improvement.

Category III - Foundations are considered to be mat type foundations and may be used for all soil conditions. However, these foundations are best suited for lots constructed in areas underlain with left in place alluvium and shallow groundwater, with ground improvement.

- Retaining wall design and construction recommendations are provided herein. Onsite soils are generally very low expansive, to possibly low expansive, and appear suitable for wall backfill, without select import, subject to verification testing.

- Recommendations for concrete (PCC) and asphaltic concrete (AC) pavements are be provided. The majority of site soils anticipated at finish subgrade elevations are anticipated to be relatively sandy, and are considered to provide relatively good subgrade support for roadways. As such, County minimum pavement sections should be anticipated.

- Adverse geologic structures that would preclude project feasibility were not encountered. However, the potentially liquefiable and compressible deposits of alluvium will require more investigation in order to develop a program of ground mitigation and/or specialized foundation/infrastructure designs, as discussed herein.

- The project design features presented in this report should be incorporated into the design and construction considerations of the project. If the design information and/or assumptions used as a basis for the geotechnical recommendations do not reflect current design information, GSI suggests a review of the current design(s) and modification of the geotechnical recommendations as needed.

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,

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GEOTECHNICAL FEASIBILITY EVALUATION FOR THE VESSELS STALLION RANCH BONSALL, SAN DIEGO COUNTY, CALIFORNIA

SCOPE OF SERVICES

The scope of our services has included the following:

1. Review of available soils and geologic data for the site and site area, including in-house documents, and other referenced material (see Appendix A).

2. Review of the current 400-scale "structure diagram," provided by your office (VSR, 2014).

3. Geologic reconnaissance and geologic mapping of the site.

4. Subsurface exploration consisting of the excavation of 11 exploratory test pits with a rubber tire backhoe, and four (4) Cone Penetration Test (CPT) soundings for logging and sampling (Appendix B). The exploratory excavations were performed in March 2014. Samples were retrieved from the test pits for laboratory testing. The logs of the test pits, and soundings are presented in Appendix B, and the locations of the test pits, and soundings are presented on Plate 1.

5. Site-specific seismic hazard evaluation and seismicity analysis (see Appendix C).

6. Completion of four (4) seismic refraction survey profiles for the evaluation of rock hardness within areas of the site underlain with near surface granitic rock (see Plate 1 and Appendix D).

7. Obtained representative samples of site soil for laboratory testing. Testing included: moisture-density determinations; compaction standards; soil expansion; Atterberg limits; direct shear; sieve and hydrometer analyses; consolidation; R-value; and corrosion potential (see Appendix E).

8. Analysis of data, including preliminary liquefaction, and settlement analysis (Appendix F).

9. Construction of geologic cross sections depicting the subsurface data. The cross sections are provided as Plate 2. See Plate 1 for cross section locations.

10. Prepared this geotechnical engineering, and engineering geologic feasibility report that includes: descriptions of site specific and regional geology, subsurface soil characteristics, the logs and/or soundings of the explorations; laboratory test results; earthwork factors; seismic hazards; preliminary conclusions and recommendations related to project planning, preliminary foundation design, and grading guidelines (see Appendix G).

PROPERTY DESCRIPTION/PROPOSED DEVELOPMENT

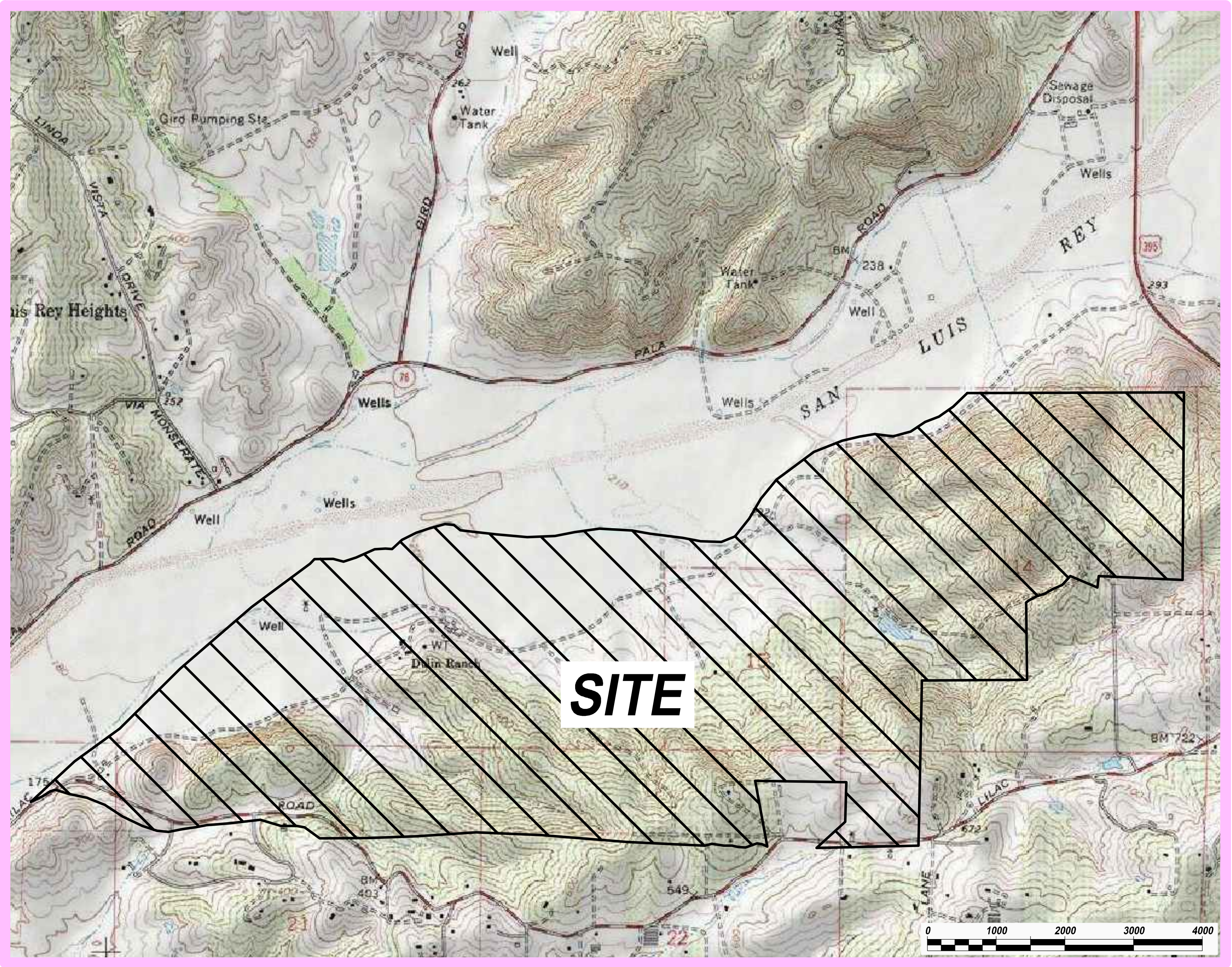
Based upon the data provided, GSI understands that the irregularly-shaped property consists of about 1,400 acres (gross), located along the southern margin of the San Luis Rey River Valley, in the vicinity of Dulin Ranch Road, including hilly and more rugged terrain generally between Dulin Ranch Road and West Lilac Road, in the community of Bonsall, San Diego County, California (see Figure 1).

Topographically, portions of the property within the San Luis Rey Valley floor area are generally flat-lying/low gradient. South of the river valley (generally south of Dulin Ranch Road), the westernmost third of the property ascends from the valley floor to somewhat more rugged, steeper terrain, with slope gradients generally steeper than about 4:1 (horizontal to vertical [h:v]) that form a roughly east-west trending ridgeline across the southern portion of the site. Within the remaining, easternmost portion of the property, the relatively flat lying river valley floor transitions to moderately sloping terrain, with north facing slopes at gradients generally on the order of 4:1 (h:v), or less. As with the western portion of the property, these low/moderate gradient slopes ascend to somewhat more rugged, steeper terrain along the southern portion of the property. Drainage is generally directed northward, from the crest of the east-west trending ridgeline, toward the San Luis Rey River, via tributary drainages incised into the north facing slope. On the backside, or south side of the ridge, drainage is generally directed offsite to the south.

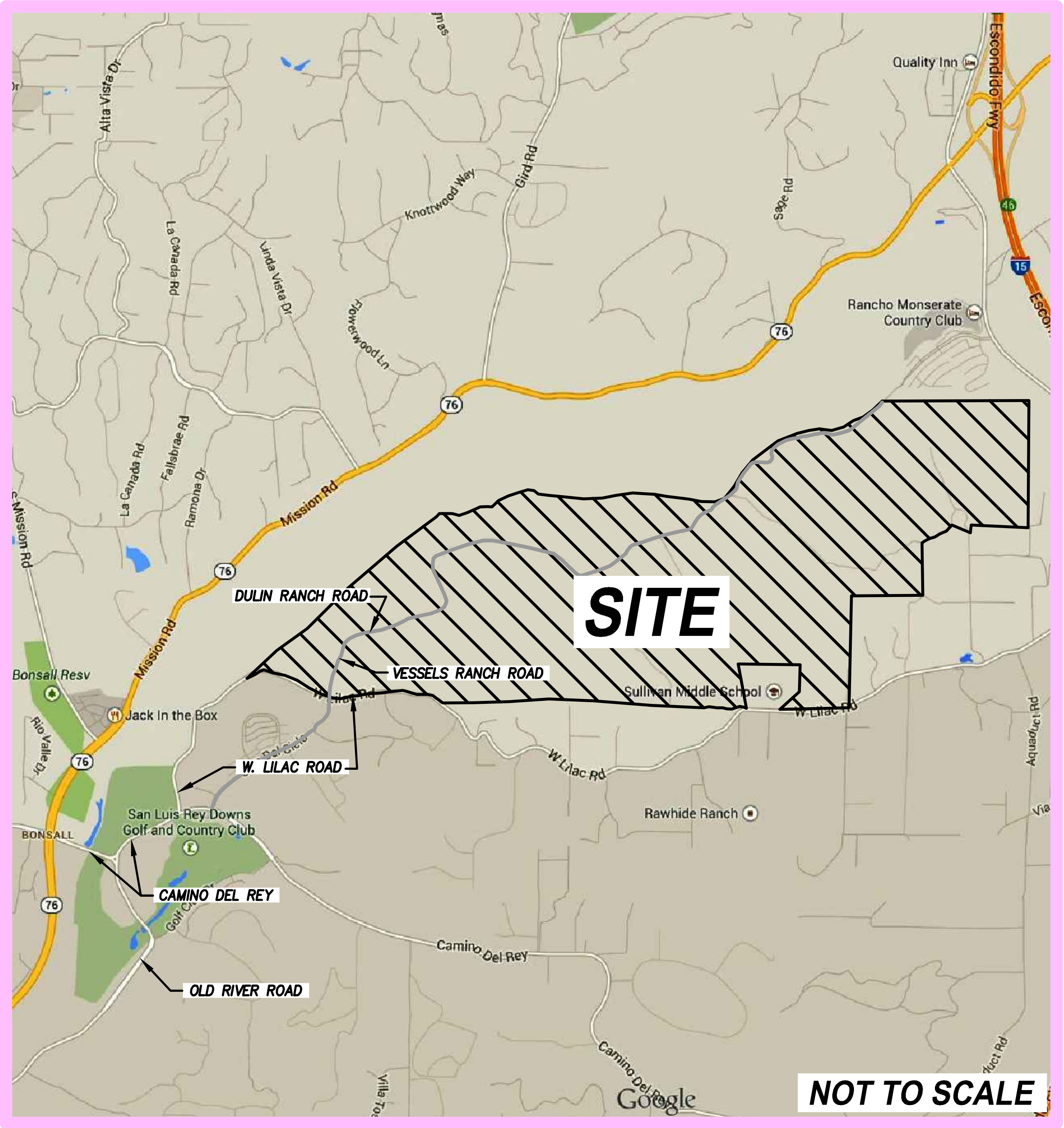
The relatively flat-lying valley floor portion of the site has elevations ranging from about 180 to 225 feet Mean Sea Level (MSL), with the area of low gradient slopes, south of the valley floor, ranging from 180 to 225 feet MSL at the valley floor/margin, up to approximately 300 feet MSL. The somewhat rugged, steeper terrain that ascends to the south, range from about 200 to as much as 747 feet MSL. Thus, overall relief across the site is on the order of about 567 feet. Portions of the site (i.e., valley floor), generally within the low/flat lying portions of the site, lie within a San Diego County 100-year flood plain.

The property is currently used for both equestrian and agricultural purposes. Existing improvements generally consist of an equestrian facility located within the low lying, northerly portions of the site, with an existing residence overlooking the equestrian facility. Scattered outbuildings were also noted throughout, and generally located in close proximity to the equestrian facility. Vegetation generally consists of some native trees, planted trees, areas of irrigated row crops, and also areas with native grasses and brush.

GSI understands that proposed development includes several Planning Areas (PA's) with different product anticipated. Current plans (Vessels, 2014) indicate at least 5 planning areas, an equestrian/estate area, the existing equestrian facility, passive parks, and trunk roadway/underground improvements. We anticipate that structures will be one- or two-story buildings utilizing typical foundations on grade, with wood frame and/or masonry block construction. Building loads are assumed to be typical for this type of relatively light construction. Sewage disposal is understood to be accommodated by tying into the regional sewage system. The need for import soils is unknown, based upon the data



Base Map: TOPO!® ©2003 National Geographic, U.S.G.S. Bonsall Quadrangle, California -- San Diego Co., 7.5 Minute, dated 1975, current, 1975.



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

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SITE LOCATION MAP

Figure 1

provided. Bio-swale/bio-retention basins are also proposed on the perimeter of the project, along the north, and southeast margins. The approximate limits of each planning area are shown on Vessels (2014), and are also shown schematically on Plate 1 included herein.

FIELD STUDIES

GSI conducted a subsurface investigation during the month of March, 2014. Our investigations consisted of 11 exploratory test pits excavated with a rubber tire backhoe, four (4) CPT soundings, four (4) seismic refraction surveys, and geologic reconnaissance mapping of the site. The approximate location of the exploratory test pits, soundings, and seismic lines, are presented on the Geotechnical Map (see Plate 1), which uses a 400-scale topographic plan, prepared by Photo Geodetic Corporation (PGC, 2013), as a base. A GSI field geologist observed the test pit excavation, and collected bulk and undisturbed samples of materials encountered for visual examination and subsequent laboratory testing. The Cone Penetration Test (CPT) soundings were directed and observed by a GSI geologist. A discussion of the seismic refraction survey is presented in a later section of this report.

REGIONAL GEOLOGY

The subject property is located within the Peninsular Ranges geomorphic province, which is characterized by steep, elongated mountain ranges and valleys that trend northwesterly (Norris and Webb, 1990). The Peninsular Ranges Geomorphic Province extends north to the base of the east-west aligned Santa Monica - San Gabriel Mountains, and south into Baja California. The province is bounded by the east-west trending Transverse Ranges Geomorphic Province to the north and northeast, by the Colorado Desert Geomorphic Province to the southeast, and by the Continental Borderlands Geomorphic Province to the west. The mountain ranges are underlain by basement rocks consisting of pre-Cretaceous metasedimentary rocks, Jurassic metavolcanic rocks, and Cretaceous plutonic (granitic) rocks, which have been uplifted, tilted, faulted, eroded and deeply incised since their formation.

In the Bonsall area during the mid to late Pleistocene (within the Quaternary-age), the granitic rocks belonging to the Peninsular Ranges Batholith have been eroded and alluvial deposits have since filled the lower valleys. Regional mapping by Tan (2000) indicates that the site is underlain by Cretaceous-age granitic rock referred to as the Couser Canyon Tonalite. Pleistocene-age older alluvium also occurs in the site vicinity (Tan, 2000).

Flat lying ground in the vicinity of (primarily west of) Dulin Ranch Road, and generally within the 100-year flood plain, is underlain with Holocene alluvium sediments. Lower slopes descending to the flood plan and flatter than about 4:1 (h:v) are developed on deposits of older alluvium (stream terrace deposits). Steeper slopes and upland areas are underlain with granitic bedrock.

SITE GEOLOGIC UNITS

General

Geologic units encountered during our current site investigation included, undocumented artificial fill, colluvium, Quaternary-age, younger alluvium, older alluvium (stream terrace deposits), and Cretaceous-age granitic bedrock. The surficial earth materials are generally described below from the youngest to oldest.

Undocumented Artificial Fill (Map Symbol - afu)

Small embankments of existing undocumented fill occur throughout the property and appear associated with the existing improvements (i.e., building pads, corrals, etc) onsite, and are likely less than approximately 10 feet in thickness. While not directly observed in any of our test pits, existing fill may be characterized as a brown silty sand to sand, dry/damp, and loose. Existing fill is considered potentially compressible in its existing state and therefore should be removed and recompacted, if settlement-sensitive improvements and/or planned fills are proposed within its influence.

Colluvium (Not Mapped)

Colluvium (topsoil) was noted to generally mantle deposits of older alluvium and granitic bedrock throughout the site. Where observed, colluvial soils generally consist of brown, and dark brown silty sand, and is typically damp, loose, and porous, with few roots. Where encountered, colluvium is on the order of approximately 1 to 7 feet in thickness. Within areas actively cultivated throughout the site, the upper 1 to 2 feet has likely been periodically reprocessed for agricultural purposes. Colluvium is considered potentially compressible in its existing state and therefore should be removed and recompacted, if settlement-sensitive improvements and/or planned fills are proposed within its influence.

Quaternary-age Alluvium (Map Symbol - Qal)

Alluvium was observed within the northern portions of the site, in areas of flat lying ground, primarily north of Dulin Ranch Road, and generally within the 100-year flood plain (i.e., PA-1, PA-2, and the northern portion of the Equestrian/Estate area).

Alluvium generally consists of light brown and very dark brown, interbedded sands, silty and sands, with silts and clays indicated at depth, based on CPT data. The thickness of this deposit generally varies from a daylight contact, adjacent to deposits of older alluvium and granitic bedrock, thickening northward to depths on the order of 42 to 62 feet below existing grades, based on field mapping, test pit, and CPT data. Within a tributary drainage located within a portion of PA-5, alluvium was encountered to a depth of at least 17 feet below existing grades.

Alluvium above the groundwater was slightly moist to moist, becoming saturated near, and below the groundwater table, and generally noted to be loose. Alluvium is considered potentially compressible in its existing state and therefore should be removed and recompacted (where possible), if settlement-sensitive improvements and/or planned fills are proposed within their influence. Alluvial soils will likely remain in place in areas of relatively high groundwater. Recommendations for the treatment of left-in-place alluvium in these areas is presented in a later section of this report.

Quaternary-age Older Alluvium (Map Symbol - Qoa)

Deposits of Quaternary (Pleistocene) age older alluvium (less than ±500,000 years old) were generally encountered at/near the surface, generally in the vicinity of the northern portion of the proposed Equestrian/Estate area, and forming the moderate slopes located between the valley floor and the southern highland ridge. Based on the distribution of these materials in plan view, and in cross section, the thickness of these sediments may be on the order of up to 30 to 50 feet locally. The older alluvium (stream terrace deposits) generally consists of interbedded silty sand, with lessor amounts of silty sand with some clay. Where observed, stream terrace deposits are light brown, brown, and yellowish brown, damp, and medium dense. Stream terrace deposits are considered suitable for the support of engineered fills, and/or structures in its existing state, provided that the recommendations presented in this report are properly incorporated into the design and construction of the project.

Cretaceous-age Granitic Bedrock (Map Symbol - Kcc)

Cretaceous-age granitic bedrock, referred to as the Couser Canyon Tonalite (Tan, 2000), was encountered near the surface, and at depth throughout the site. Where encountered, bedrock consists of fractured rock, disintegrating to sand and silty sand with brittle gravel to cobble size rock fragments in near surface excavations. Bedrock was generally observed to be brown to olive brown, brownish yellow to yellowish brown, dry to moist, and dense.

Practical refusal on hard rock with a rubber tire backhoe was encountered at depths varying from approximately 2 to 8½ feet below existing grades. Relatively unweathered bedrock is considered suitable for the support of settlement-sensitive improvements and/or planned fill in its existing state.

Structural Geology

Based on our observations and available published geologic maps of the site and surrounding area, bedding within alluvium and older alluvium appears to be relatively flat-lying. Bedrock is fractured, with fractures generally steeply inclined to the northwest, southeast, southwest, and northeast (i.e., in all four quadrants).

GROUNDWATER

The regional groundwater table was encountered in our CPT soundings at depths on the order of 13½ to 15½ feet below existing grades within the relatively flat lying, alluviated areas overlying PA-1, PA-2, and the northern portion of the equestrian/estate lots. These depths generally correspond to approximate elevations ranging from about 197 above mean sea level (MSL), up gradient, near the eastern end of the property, to approximately 170 feet MSL, down gradient, toward the western end of the property.

Perched groundwater may occur in or along zones of contrasting permeability (i.e., between contrasting soil types in the underlying deposits/bedrock or discontinuities) due to migration from adjacent drainage areas, and during and after periods of above normal or heavy precipitation or irrigation. Thus, perched groundwater conditions may occur in the future, after construction, and should be anticipated. Groundwater observations reflect site conditions at the time of this report and do not preclude changes in local groundwater conditions in the future.

FAULTING AND REGIONAL SEISMICITY

Regional Faults

Our review indicates that there are no known active faults crossing this site (Jennings and Bryant, 2010), 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 Temecula segmentf the Elsinore fault is closest known active fault to the site (located at a distance of approximately 11.1 miles [17.8 kilometers]). However, the Julian segment of the Elsinore fault (located at a distance of approximately 11.7 miles [18.9 kilometers]) should have the greatest effect on the site in the form of strong ground shaking, should the design earthquake occur. A list and the location of the Elsinore fault and other major faults relative to the site is provided 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 Elsinore fault may be on the order of 0.306g, for portions of the site underlain with alluvial soil, and 0.34g for portions of the site underlain with granitic rock. 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 July 2013). 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 July 2013. Based on the selected acceleration-attenuation relationship, a peak horizontal ground acceleration is estimated, which may have affected the site during the specific event listed. Based on the available data and the attenuation relationship used, the estimated maximum (peak) site acceleration during the period 1800 through July 2013 was about 0.077g to 0.09g, for alluvial, and rock areas, respectively. A historic earthquake epicenter map and a seismic recurrence curve are also estimated/generated from the historical data. Computer printouts of the EQSEARCH program are presented in Appendix C.

For the evaluation of liquefaction potential onsite, and in general accordance with California Department of Conservation (2008), a probabilistic seismic hazards analysis was performed using a PSHA Interactive Deaggregation computer program provided by the USGS (2012). Based on a review of these data, and considering the relative seismic activity of the southern California region, a probabilistic horizontal site acceleration (PHSA) of 0.29g was considered. This value was chosen as it corresponds to a 10 percent probability of exceedance in 50 years. For other design aspects of site design and construction, a probabilistic seismic hazards analysis was performed using the computer program "Seismic Design Maps," provided by the United States Geologic Survey (USGS, 2014).

Seismic Shaking Parameters

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

2013 CBC SEISMIC DESIGN PARAMETERS			
PARAMETER	ALLUVIUM/ OLDER ALLUVIUM	GRANITIC BEDROCK	2013 CBC AND/OR REFERENCE
Risk Category	I, II, or III	I, II, or III	Table 1604.5
Site Class	D	B (<10' of fill)	Section 1613.3.2/ASCE 7-10 (Chapter 20)
Spectral Response - (0.2 sec), S_s	1.149 g	1.150 g	Figure 1613.3.1(1)
Spectral Response - (1 sec), S_1	0.447 g	0.447 g	Figure 1613.3.1(2)
Site Coefficient, F_a	1.040	1.00	Table 1613.3.3(1)
Site Coefficient, F_v	1.553	1.00	Table1613.3.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec), S_{MS}	1.196 g	1.150g	Section 1613.3.3 (Eqn 16-37)
Maximum Considered Earthquake Spectral Response Acceleration (1 sec), S_{M1}	0.694 g	0.447 g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	0.797g	0.767 g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.463 g	0.298 g	Section 1613.3.4 (Eqn 16-40)
PGA_M	0.46 g	0.43 g	ASCE 7-10 (Eqn 11.8.1)

2013 CBC SEISMIC DESIGN PARAMETERS			
PARAMETER	**ALLUVIUM/ OLDER ALLUVIUM**	**GRANITIC BEDROCK**	**2013 CBC AND/OR REFERENCE**
Probabilistic Horizontal Ground Acceleration ([PHGA] 10% probability of exceedance in 50 years)	0.29 g	N/A	USGS (2012)
Seismic Design Category	D	D	Section 1613.3.5/ASCE 7-10 (Table 11.6-1 or 11.6-2)

GENERAL SEISMIC DESIGN PARAMETERS	
PARAMETER	**VALUE**
Distance to Seismic Source (Elsinore- Julian segment) "B" Fault (1)	11.7 mi (18.9 km)(2)
Upper Bound Earthquake (Elsinore-Temecula)	M_w = 7.1(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.

OTHER GEOLOGIC HAZARDS

Liquefaction

Our preliminary finding indicates that some of the alluvial materials are liquefiable and will display some settlement during the design earthquake. Groundwater is approximately 13½ to 15½ feet below existing grades. Mitigation will typically include, but not necessary include, fill surcharging, ground improvement, and/or relatively onerous foundation design.

Seismically-induced liquefaction is a phenomenon in which cyclic stresses, produced by earthquake induced ground motion, create excess pore pressures in relatively cohesionless soils. These soils may thereby acquire a high degree of mobility, which can lead to sand boils, lateral movement/sliding, volumetric consolidation and settlement of loose sediments, and other damaging deformations as pore pressures dissipate. This phenomenon occurs only below the water table, but after liquefaction has developed, it can propagate upward into overlying, non-saturated soil, as excess pore water dissipates. Thus, one of the primary factors controlling liquefaction potential is the depth to groundwater.

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

Liquefaction susceptibility is related to numerous factors and the following conditions must generally exist, or have the potential to exist, for liquefaction to occur: 1) sediments must be relatively young in age and not have developed a large amount of cementation; 2) sediments must consist mainly of medium to fine grained, relatively cohesionless sands; 3) the sediments must have low relative density; 4) free groundwater must be present in the sediment; and, 5) the site must have a potential for a design seismic event of a sufficient duration and magnitude, to induce straining of soil particles.

In general, subsurface and background data indicate that the requisite five concurrent conditions exist, or have the potential to exist, within areas of the site underlain with alluvial soils for this project. Thus, it would appear that significant layers of alluvium underlying said site is relatively susceptible to liquefaction. Given the intended development, the potential site accelerations, the relatively low density soils occurring along the margins of the site, where younger alluvium is on the order of 40 to 60 feet thick, and the elevation of groundwater conditions at the site, GSI has performed a liquefaction analysis for the proposed development, assuming current groundwater elevations.

The condition of liquefaction has two principal effects. One is the volumetric strain or "consolidation" of loose sediments with resultant settlement of the ground surface. The other effect is lateral sliding. Significant permanent lateral movement generally occurs only when there is significant differential loading, such as fill or natural ground slopes within susceptible materials. As such, any planned fill slopes constructed within existing alluvial areas present a potential for lateral spread to affect perimeter fill slopes underlain with unmitigated alluvial soils below the groundwater table, and should be further evaluated once grading plans are developed.

The evaluation of whether or not surface manifestation of liquefaction, such as sand boils, ground fissures, foundation tilt and cracking, etc., will occur at a site can be made using Special Publication 117A "Guidelines for Analyzing and Mitigating Liquefaction in California" (CGS, 2008). Based on the thickness of the potentially liquefiable layer, the thickness of the non-liquefiable soil (fill) cover, and ground acceleration for the design earthquake, an evaluation of these "liquefied" soils was made. Based on our evaluation, the potential for sand boils on the graded and mitigated site, is considered low. However, the potential for densification and settlement of any near surface alluvium left in place (unmitigated) is considered high, and is discussed in a later section of this report.

Seismic Settlement/Seismic Densification

Seismic densification is a phenomenon that typically occurs in low relative density granular soils (i.e., USCS classified as SP, SW, and SM) that are above the groundwater table and are significantly dry of optimum moisture content. During the seismic-induced ground shaking, these natural sediments deform under loading and volumetrically strain, resulting in ground surface settlements. Some contribution to seismic settlement has been incorporated in our evaluations herein. Some densification of the adjoining un-mitigated areas may influence improvements at the perimeter of the site. These unsaturated granular soils are susceptible if left in their original density (unmitigated), and are significantly drier than the optimum water content (as defined by the ASTM D 1557). Some of the layers of alluvium onsite that was encountered above the water table may be considered susceptible to seismic densification. However, due to the relatively shallow groundwater table (i.e., 14 to 16 feet) in alluvial areas, mitigation of this material is feasible using conventional removal and recompaction techniques during grading in most areas.

Other Seismic Hazards

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

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

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

Landslides

Landslide deposits were not noted during our review of Tan, et al. (2000), or Tan and Kennedy (2005). Landslide deposits, and/or geomorphology indicative of landslide deposits (i.e., humocky topography, scarps, lobate soil deposits, etc.) were not noted in the field. Given the site's relatively gentle relief (i.e., slope gradients on the order of 4:1 [h:v], or less), the absence of adverse geologic structure, and dense/resistant nature of the underlying bedrock, the potential for landslides to affect the proposed site development is considered low.

ROCK HARDNESS EVALUATION

A seismic refraction survey was performed in selected areas where the site is underlain with near surface granitic bedrock. The survey consisting of four seismic refraction lines, conducted using a Geometrics SmartSeis 12-channel exploration seismograph, with a hammer and plate energy source. The approximate seismic line locations are shown on Plate 1, and the velocity and depth interval results are graphically shown, and included in Appendix D. An example of the raw seismic data is also included in Appendix D, and illustrates a forward and split spread shot from the same line.

The first arrival information, shot point locations, geophone locations, and line geometry from each survey are utilized in the computer programs SIPwin (Rimrock Geophysics, 2002) which produces time-distance plots for each of the survey lines (see example, Appendix D). The graphic curves reflect the actual time-distance plots generated by the program, showing the shot points and phone locations. The first curve, from left to right shows the forward spread from the first shot. The second, or split spread shot point creates two curves in opposite directions from the shot in the middle of the spread. The third curve represent the reverse shot from the distant end of the spread.

The data for the surveys performed generally show a three-layer case. The uppermost layer is generally thin as would be expected, reflecting the surficial materials (i.e., documented fill). Undulations in time-distance curves can be attributed to a lack of elevation corrections to the raw data, possible minor disturbances from noise (e.g., wind or traffic), decreased energy at distant geophones, and discontinuities in the subsurface.

The velocity-depth models, or cross-sections, generated are included as Plates D-3 through D-6 for ST-1, ST-2, ST-3, and ST-4, respectively. As can be seen on these plates, the boundaries between various seismic velocity layers appear to be somewhat undulatory, typical of fractured and weathered crystalline (i.e., granitic) rocks where there is a access to the subsurface for air and water. Fracture, or joint density/frequency also contributes to the variation in depth of weathering and therefore differences in seismic velocities.

Layer boundaries tend to mimic the surface topography, although variations are common depending upon the depth of weathering, fracturing, etc. In general, the survey indicated a near surface layer (Layer 1) thickness (i.e., undocumented fill, colluvium, weathered bedrock), ranging from about ±2 to ±4 feet. The average velocity of Layer 1 material is about ±900fps, and is considered typical for such near surface material. The depth to the Layer 1/Layer 2 transition (bedrock) ranges from about ±2 to ±4 feet below existing grades, with the depth to the Layer 2/Layer 3 transition on the order of about 7 to 19 feet below existing surface grades. The average velocity of Layer 2 is about ±3,200 fps, with some variability. Layer 3 occurs at depths on the order of 4 to 19 feet, with average velocities in Layer 3 (relatively unweathered bedrock) ranging from 2,960 to 4,830 fps. At depths where velocities are greater than about 6,000 fps, rippability is ambiguous and blasting usually is required.

An evaluation has been made of the seismic refraction line data to estimate the approximate depth to non-rippable trenching (i.e., utility excavation) and to non-rippable bedrock. Approximate cut-off velocities of ±3,800 and ±6,000 fps are generally used as a basis for non-rippable trenching (assuming a Cat 235 Hoe [a large trackhoe], or equivalent), and non-rippable bedrock (assuming a D9L, or equivalent), respectively. It should be noted that a conventional rubber-tired backhoe can experience non-productive trenching at seismic velocities much less than ±2,000 to 2,500 fps.

Bedrock excavatability with respect to trenching shallower than the approximate ±3,800 fps cut-off depth is expected to vary from easy to very difficult and the necessity for localized areas requiring rock breaking, or blasting should be anticipated. Similarly, bedrock rippability shallower than the approximate ±6,000 fps cut-off depth is expected to vary from easy to very difficult, and the necessity for localized areas requiring rock breaking and/or blasting cannot be entirely precluded.

Variations should be expected. As such, bedrock excavations from the surface downward may generate oversize rock. Isolated “floaters or corestones may also be encountered. The bulk of the materials derived from the weathered portion of the bedrock (up to and including the ±3,800 to 6,000 fps cut-off) are anticipated to disintegrate to approximately 12 to 24 inches and smaller constituents. Any oversize materials ($\geq$ 12 inches) generated would require special handling for use in fills, and may not be placed within 10 feet of finish grade or used as backfill in utility trenches. Oversize materials typically become commonplace during excavation into 5,000 fps materials, usually requiring specialized placement techniques during grading.

Based upon our experience in this area, and the seismic refraction data obtained, the following table reflects our estimates of the rippability and trenchability at the locations of the seismic refraction survey lines; other interpretations are possible:

SEISMIC LINE NO.	GENERAL RIPPABILITY (ASSUMING A D9L DOZER OR CAT 235 HOE, OR EQUIVALENT)
ST-1	Rippable and trenchable to depths explored of ±30 feet. Difficult trench below depths of 7 to 12 feet. localized blasting and/or rock breaking may not be precluded below depths of 10 to 15 feet.
ST-2	Rippable and trenchable to depths explored of ±30 feet. Difficult trenching below depths of 15 to 19 feet. localized blasting and/or rock breaking may not be precluded below depths of 20 feet.
ST-3	Rippable and trenchable to depths explored of ±30 feet. Moderate to difficult trenching below depths of 15 feet. localized blasting and/or rock breaking may not be precluded below depths of 20 feet.
ST-4	Rippable depths explored of ±30 feet. Not trenchable beloe depths of 3 to 4 feet. Localized blasting and/or rock breaking may not be precluded below depths of 20 feet.

Rock Hardness Summary

In general, utilizing the seismic data, it appears that the site area in the vicinity of our seismic lines may be characterized as being underlain by a surficial soils (fill, colluvium, weathered rock) to depths ranging from about ±2 to about ±4 feet in thickness. At depths greater than approximately 4 to 19 feet, relatively fresh and very dense granitic bedrock exists. Based on all of the above, the need for overexcavation, blasting and/or line shooting would be anticipated on the site, should proposed cut grades exceed the depths indicated herein, in areas underlain with granitic bedrock (see Plate 1). It should be noted that a conventional rubber-tired backhoe will experience non-productive trenching at seismic velocities much less than ±2,000 to 2,500 fps. The seismic refraction data presented herein should be further reviewed in conjunction with final grading plans (when available). It should be noted that due to the variability of bedrock weathering, and the potential for local boulders, or less weathered bedrock, very difficult ripping, rock breaking, and/or blasting cannot be entirely precluded at shallower depths.

LABORATORY TESTING

General

Laboratory tests were performed on representative samples of the onsite earth materials collected from the subsurface geotechnical investigation, in order to evaluate their physical characteristics and engineering properties with respect to anticipated site development. The test procedures used and subsequent results are presented below:

Classification

Soils were classified with respect to the U.S.C.S. in general accordance with ASTM D 2487 and D 2488. The soil classification is presented with the Test Pit Logs (see Appendix B).

Field Moisture and Density

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

Laboratory Standard

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

LOCATION AND DEPTH	SOIL TYPE	MAXIMUM DENSITY (PCF)	OPTIMUM MOISTURE CONTENT (%)
TP-7 @ 4'	Silty SAND, gray brown	128.0	11.0

Expansion Index

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

LOCATION AND DEPTH	EXPANSION INDEX	EXPANSION POTENTIAL
TP-1 @ 1-4'	<5	Very Low
TP-2 @ 8'	<5	Very Low

Our evaluation to date indicates that site soils appear to be very low expansive. Soils derived from excavation in bedrock are anticipated to also be very low expansive

Direct Shear

Strain-controlled direct shear tests were performed on representative soil samples in general conformance with the ASTM D 3080 test method. The test results are presented in Appendix E.

Particle-Size Analysis

An evaluation was performed on selected representative soil samples in general accordance with ASTM D 422. Particle size analyses were performed on selected samples from our exploratory borings. The grain-size distribution curves are presented in Appendix E. These test results were utilized in evaluating the soil classifications in accordance with the Unified Soil Classification System.

Corrosivity Testing

Corrosivity testing, performed for a representative sample of site soil indicates a pH of 6.99 (which is considered relatively neutral); a soluble sulfate content of 0.011 percent by weight (which is considered negligible to moderate [Class S_0 and S_1, respectively] per Table 4.2.1 of ACI 318-11 (per 2013 CBC [CBSC, 2013]); a chloride content of 122 parts per million (ppm), which is considered slightly elevated; and a saturated resistivity of 1,800 ohm-cm (which is considered mildly corrosive to ferrous metals [California Highway Design Manual, 2012]). While it is our understanding that typical structural concrete (f'c $\geq$ 3,000 to 4,500 psi) with minimal design cover is generally sufficient mitigation for such conditions, GSI recommends consultation with a corrosion engineer. Test results are presented in Appendix E.

PRELIMINARY SETTLEMENT, LIQUEFACTION AND LATERAL SPREAD ANALYSIS

GSI has estimated the potential magnitudes of total settlement, differential settlement, and angular distortion for the site. The analyses were based on laboratory test results and subsurface data collected from test pits and CPT soundings completed in preparation of this study. Site specific conditions affecting settlement potential include depositional environment, grain size and lithology of sediments, cementing agents, stress history, moisture history, material shape, density, void ratio, etc. The following discussion is preliminary. Additional studies are recommended once plans are developed.

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

Due to the varied geologic conditions, and for the purposes of this evaluation, at least three (3) general, as-built conditions are anticipated, and summarized into the following groups, as follows:

- Group 1 - Areas where the complete removal of surficial deposits of alluvium, colluvium, and any unsuitable older alluvium are removed to suitable older alluvium, (i.e., the southern portion of the Equestrian/Estate area). This condition also includes over excavated cut lots exposing suitable older alluvium.

- Group 2 - Areas where the complete removal of surficial deposits of alluvium, colluvium, and any unsuitable older alluvium are removed to granitic bedrock (PA-3, PA-4, and PA-5). This condition also includes over excavated cut lots exposing suitable granitic bedrock.

- Group 3 - Areas of alluvium left in place below the regional groundwater table. This condition would generally occur within PA-1, PA-2, and the northern portion of the Equestrian/Estate area located within the existing valley floor.

Static Settlement of Fill Areas

On a preliminary basis, and for the purposes of this report, site grading is anticipated to create cuts and fill over natural soils on the order of up to about 20 to 30 feet. This estimated fill thickness will likely be increased by remedial grading.

Group 1 lots are anticipated to consist of fill over dense older alluvium, and primarily occur within the southern portion of the proposed Equestrian/Estate area. The evaluation of older alluvium exposed in test pits indicate the natural older alluvium is not prone to excessive post-construction compression. The total post-construction “static” settlement may be approximately 1½ inch, and differential settlement on the order of ¾ inch in 40 lateral feet, with overlying fills up to 20 feet in thickness, on a preliminary basis. Thicker fills will result in increased differential settlements.

Group 2 lots, primarily located within proposed Planning Areas PA-3, PA-4, and PA-5 are anticipated to be graded with fill over bedrock (including over excavation) on the order of 30 feet or less. Static, post-construction settlement on these lots is anticipated to be on the order of 1½ inch total and differential settlement on the order of ¾ inch in 40 lateral feet, with overlying fills up to 30 feet in thickness, on a preliminary basis. Thicker fills will result in increased differential settlements. Subdrainage and slope of overexcavation cuts is important to the reduction of potential perched water and subsequent compression.

Group 3 lots (PA-1, PA-2, and the northern portion of the Equestrian/Estate area) are likely to have left in place younger alluvium over bedrock beneath these areas at the conclusion of grading. Remedial grading has been estimated as approximately 11½ to 13½ feet below existing grades. That is, from the existing ground surface approximately 11½ to 13½ feet of wet, soft/loose alluvium will be removed up to and including the material at about 2 feet from the water table. The static settlement evaluation was based on the compression of up to 30 feet of planned and remedial grading (13 to 14 feet of remedial plus 15 feet of fill above grade. This may result in immediate compressions of the loose/soft layers of alluvium to long-term (10- to 30-year) compressions with the potential to induce angular distortions in excess of 1/480 in this area. Due to the anticipated settlements in this area, multiple remedial measures are suggested in a later section of this report to bring building lots to approximately 1/480 of angular distortion for structural foundation design. Due to the estimated magnitude of settlement, additional geotechnical review and analysis of this area and remedial measures should be performed.

These static settlement estimates do not include the effects of expansive soils (shrink and swell) and the loading of soils under foundations, as well as top-of-slope creep effects, which are described in a later section of this report.

Seismic-Induced Settlement, Liquefaction and Densification

Following a review of the CPT data, and recent laboratory testing, the CPT data were evaluated for liquefaction potential within the alluvial areas. The liquefaction analyses were performed using the LiquefyPro computer program (Civiltech Software, 2006 [version 5a]), field boring/laboratory data, and the data from the recent CPTs. Computer printouts from the analysis are presented in Appendix F. The analysis was conducted in general accordance with Special Publication 117A "Guidelines for Evaluating and Mitigating Seismic Hazards in California" (California Department of Conservation, California Geological Survey [CGS], 2008). For the analyses, GSI utilized a groundwater depth of 14 to 16 feet below the existing grade in alluvial areas, to account for an anticipated groundwater level at the time of the design seismic event. For ground acceleration, GSI used the 10 percent in 50 years probabilistic horizontal ground acceleration (PHGA), and the value of PGAm, as previously discussed herein. GSI reviewed the PHGA with a spectral period ranging between 0.2 and 1 second. A PHGA value based on the value of S_{DS} value divided by 2.5, in accordance with Section 11.8.3 of the American Society of Civil Engineers (ASCE) Manual 7-10 (ASCE, 2010), was also considered, but not used for this feasibility level evaluation. The PHGA used in the liquefaction analysis ranged from 0.29g to 0.46g. Lastly, the design earthquake magnitude of 7.1 on the Julian strand of the Elsinore fault (Cao, 2003) was also used. A review of the CPT data generally indicate that alluvium generally consists of interlayered sands, silts, and clays.

The results of the liquefaction analyses indicate that liquefaction may occur within areas underlain by younger alluvium occurring below the groundwater table. We have evaluated this potential for seismic induced deformation using fill thicknesses of up to approximately 15 feet above existing grades. Therefore, it is the opinion of GSI, that liquefaction and its corresponding secondary effects including seismic settlement and lateral spreading, are considered potential secondary seismic hazards in alluvial areas. As a result, mitigation measures will be necessary to reduce the impact of earthquake induced liquefaction. Liquefaction mitigation at the site requires either special foundation design or ground improvements or both. Due to the presence of one free-face condition (i.e. perimeter fill slopes) along the northern part of Planning Areas PA-1, PA-2, and the northern equestrian/Estate area (i.e., Group 3 settlement areas), the potential for lateral spreading exists with respect to the performance of perimeter fill slopes that "toe out" into the adjacent river valley floor.

The magnitude of potential seismic settlement for both "free-field" as well as under the anticipated fill loads were evaluated using various methods within the LiquefyPro program in general accordance with Special Publication 117A "Guidelines for Evaluating and Mitigating Seismic Hazards in California" (CGS, 2008) and ASCE 7-10, Section 11.8.3 (ASCE, 2010). GSI has provided these analyses with a consideration for a factor-of-safety (FOS) of 1.0 to 1.3. Based upon the assumed, current design configuration and the results of our seismic deformation analysis, the total free-field ground settlement in alluvial areas during the design seismic event for Group 3 lots is estimated to be on the order of 1 to 5¾ inches with a potential differential settlement of up to approximately 2½ inches over 100 feet (angular distortion of 1/266), using a factor of safety (FOS) ranging from 1.0 to 1.3. Our evaluation generally indicates that seismically induced settlements decrease with an increase in fill loading. As evaluated in this study, settlements were generally reduced to on the order of ¼ to 1¾ inches for a minimum surcharge of 15 feet. This potential seismic deformation should be considered in foundation and improvement design, and should be reevaluated once planned grades are known in this area.

Elsewhere on the project, in Groups (Areas) 1 and 2, seismic settlement is not anticipated to be more than ½ inch total, and ¼ inch differential over a lateral distance of 40 feet. Computer printouts of the seismic settlement analysis within Group 3 are provided in Appendix F. These seismic deformations are for conditions under the pads and do not include edge lateral spread effects.

Foundation Settlement Due to Structural Loads (All Areas)

The settlement of the structures supported on strip and/or spread footings founded on compacted fill will depend on the actual footing dimensions, the thickness and compressibility of fill below the bottom of the footing, and the imposed structural loads. Provided the thickness of fill below the bottom of the footing is at least equal to the width of the footing, and based on a maximum allowable bearing pressure of 2,500 pounds per square foot (psf) or less, provided in this report, post-construction total settlement of less than 1 inch should be anticipated; however, this assumes all fill is properly compacted. Given this condition, the majority of the foundation settlement should occur as the building loads are applied during construction. Post construction differential settlement between the lightest and heaviest loaded footings, due to applied loads, may occur if the foundation is of the conventional type, and is anticipated to minimally be on the order of ¼ to ½ inch. Further review will be needed once draft foundation plans and building loads are provided.

Lateral Spreading

Lateral spread phenomenon is described as the lateral movement of stiff, surficial, mostly intact blocks of sediment or compacted fill displaced downslope towards a free face along a shear zone that has formed within the liquefied sediment. The resulting ground deformation typically has extensional fissures at the head of the failure, shear deformations along the side margins, and compression or buckling of the soil at the toe. The extent of lateral displacement typically ranges from less than an inch to several feet. Two types of lateral spread can occur: 1) lateral spread towards a free face (e.g., river channel or embankment); and 2) lateral spread down a gentle ground slope where a free face is absent. Factors such as earthquake magnitude, distance from the seismic energy source, thickness of the liquefiable layers, the slope of the underlying bedrock surface (Group 2 lots) and the fines content and particle size of those sediments also correlate with ground displacement.

Areas underlain with alluvium along the eastern side of the project (Northern edge of PA-1, PA-2, and the northern edge of the equestrian/estate area) will likely have one “free face” (north facing fill slope), where this slope “toe’s out”into the adjacent flood plain area where groundwater was observed to be as shallow as 13½ to 15½ feet below existing grades. On a preliminary basis, the outer 10 to 15 feet of the pads adjacent to unmitigated flood plain areas may be subject to lateral spread and may significantly affect improvements, drainage, and top-of-slope stability in these areas, if no mitigation is used. This phenomena should be further evaluated once preliminary plans have been developed.

Subsidence

The effects of areal subsidence generally occur at the transition or boundaries between low-lying areas and adjacent hillside terrain, where materials of substantially different engineering properties (i.e., alluvium/older alluvium - bedrock [granitic]) are present. Subsidence may occur at any time when site conditions change, including groundwater or fluid withdrawal, loading or heavy vibrations, etc., but is most noticeable during large-scale seismic events.

Provided the guideline presented in this report are properly incorporated into the design and construction of the project, the potential for significant areal subsidence is considered low.

PRELIMINARY SLOPE STABILITY EVALUATION

Gross Stability

Graded slopes are generally considered to be stable, up to gradients of 2:1 (h:v) or flatter, and bedrock slopes may be suitable to gradients of 1.5:1, or flatter. However, mapping indicates some potential for dip slope oriented fractures/joints in bedrock that may require stabilization, and slope gradients of 2:1, or flatter. Natural slopes appear to be performing adequately. Additional geotechnical review of the seismic stability of those fill slopes is warranted.

All graded slope construction will require observation during grading in order to evaluate the findings and conclusions presented herein and in subsequent reports. Our analysis assumes that graded slopes are designed and constructed in accordance with guidelines provided by the County, the 2013 CBC (CBSC, 2013), the current edition of the "Greenbook," and recommendations provided by this office. These slopes are generally anticipated to be stable, assuming proper construction, maintenance, and normal climatic conditions.

If liquefaction occurs in unmitigated soils at the limit of fill slopes constructed within Group 3 settlement areas, the seismic FOS may be less than 1.1. Additional geotechnical review of the seismic stability of those fill slopes is warranted.

Temporary backcuts for construction slopes and keyways, are anticipated to be 1:1 (h:v) or flatter, and are anticipated to have a static FOS of 1.2. Should perched groundwater or other unexpected conditions be exposed during excavation, the project geotechnical consultant should review the conditions and revise recommendations as needed.

Surficial Stability

Surficial stability was evaluated for graded slopes constructed of compacted fills and/or formational soil. On a preliminary basis, our evaluation indicates that slopes should perform adequately against surficial failure, provided that the slopes are properly constructed and maintained, under normal rainfall.

Onsite soils are granular, sandy soils. If sandy soils with a cohesion of less than 200 psf are used on slope faces, the slopes may have surficial stability/erosion issues and perhaps a FOS against surficial instability of less than 1.5. Planting and management of surficial drainage is imperative to the surficial performance of slopes. Typically, similar to coastal bluff retreat, a surficial erosion rate (average) of up to about 1¼ inches/year for natural and unprotected sandy slopes may be assumed. Foot traffic and other activities that exacerbate surficial erosion should not be allowed to occur on slopes. Failure to adhere to these conditions may drastically increase and localize surficial erosion, requiring mitigation, so that headward erosion does not result, and impact roadways, pads, and other improvements.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

General

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

- Depth to competent bearing material below existing pad grade.
- Expansion and corrosion potential of onsite soils.
- Perimeter conditions and the influence of onsite and offsite unmitigated soils.
- Seepage, drainage, and moisture transmission through foundations.
- Settlement potential (static and seismic).
- Groundwater.
- Lateral spreading potential.
- Regional seismic activity.
- Rock hardness and utility installation/foundation construction.
- Potential for oversize rock exceeding 12 inches in long dimension.

The recommendations presented herein consider these as well as other aspects of the site. The engineering analyses, performed, concerning site preparation and the recommendations presented herein have been completed using the information provided and obtained during our field work. In the event that any significant changes are made to proposed site development, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the recommendations of this report are evaluated or modified in writing by this office. Foundation design parameters are considered preliminary until the foundation design, layout, and structural loads are provided to this office for review.

- Removals should consist of all surficial deposits of existing fill, colluvium, and near surface, weathered natural soils. Conventional removals of alluvium will be limited locally, due to the presence of a shallow groundwater table.

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

- Geologic observations should be performed during grading to observe and/or further evaluate site geologic conditions. Although unlikely, if adverse geologic structures are encountered, supplemental recommendations and earthwork may be warranted.

- A shallow groundwater table will be encountered during removals/excavation within alluvium, primarily along the northern margin of the property (PA-1, PA-2, and the northern portion of the proposed Equestrian/Estate area). Within other areas of the site, regional groundwater is not expected to be encountered during excavation. However, seepage between layers of fill, fill/bedrock contacts, and in discontinuities within bedrock, cannot be precluded in all areas during, or after grading.

- Our laboratory test results and experience on this site indicate that soils with a very low, and possibly low expansion potential generally underlie the site. This should be considered during project design and construction. Preliminary foundation design and construction recommendations are provided herein for these soil conditions.

- Building foundations will need to be designed to accommodate the expansive soil conditions, corrosive soils, and potential settlements. Foundation alternatives including stiffened slabs, mat slabs, and post tensioned slabs, are provided.

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

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

Based on the findings of this study, the site is suitable for the proposed development from a geotechnical engineering and geologic viewpoint, provided the recommendations presented herein are properly incorporated into the design and construction phases of development. Preliminary remedial earthwork and foundation recommendations are provided in the following sections.

EARTHWORK CONSTRUCTION RECOMMENDATIONS

General

Remedial earthwork will likely be necessary for the support of the proposed settlement-sensitive improvements. Remedial grading should conform to the guidelines presented in Appendix J of the 2013 CBC (CBSC, 2013), the requirements of the County, and the Grading Guidelines presented in Appendix G, except where specifically superceded in the text of this report. In case of conflict, the more onerous code or recommendations should govern. Prior to grading, a GSI representative should be present at the pre-construction meeting to provide additional grading guidelines, if needed, and review the earthwork schedule.

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

Demolition/Grubbing

1. Vegetation, and any miscellaneous deleterious debris generated from the demolition of existing site improvements should be removed from the areas of proposed grading/earthwork.

2. Cavities or loose soils remaining after demolition and site clearance should be cleaned out and observed by the geotechnical consultant. 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.

3. Any septic systems encountered should be removed and disposed of per County guidelines.

Treatment of Existing Ground

The treatment of existing ground will vary by area/geologic conditions onsite, and may be subdivided into at least three (3) general cases, as follows:

Case I - Areas underlain with near surface, older alluvium and/or granitic bedrock.

Case II - Areas underlain with alluvium below a shallow groundwater table.

A discussion of existing ground treatment is presented for each case as follows:

Case I, Areas Underlain With Near Surface, Older Alluvium and/or Granitic Bedrock (Settlement Group Areas 1 and 2)

1. Areas underlain with near surface, older alluvium and/or granitic rock generally occur in the vicinity of PA-3, PA-4, PA-5, and the northern portion of the Equestrian/Estate Area.

2. Where not removed by the planned excavations, all undocumented fill, colluvium, alluvium, and weathered older alluvium/bedrock should be removed to competent older alluvium/bedrock, cleaned of deleterious materials, moisture conditioned, and recompacted within areas proposed for settlement-sensitive improvements. In general, the remedial removal excavations are anticipated to be on the order of 1½ to 5½ feet, to as much as 17 feet locally, where observed in our test pits. However, local deeper removal excavations cannot be precluded and should be anticipated. Actual depths of removals will be evaluated in the field during grading by the soil engineer.

3. Subsequent to the above removals, the upper 8 inches of the exposed subsoils/bedrock should be scarified, brought to at least optimum moisture content,

and recompacted to a minimum relative compaction of 90 percent of the laboratory standard (ASTM D 1557), prior to any fill placement.

4. Localized deeper removals may be necessary due to buried drainage channel meanders or dry porous materials. The project soils engineer/geologist should observe all removal areas during the grading.

Case II, Areas Underlain with Loose Alluvium and a Shallow Groundwater Table (Settlement Group Area 3):

1. Areas underlain with loose, surficial deposits of alluvium and a shallow groundwater table, generally occur in the vicinity of PA-1, PA-2, and the northern portion of the Equestrian/Estate area.

2. Alluvium should be removed to near the existing groundwater table, cleaned of deleterious materials, moisture conditioned, and recompacted within areas proposed for settlement-sensitive improvements. In general, the remedial removal excavations are anticipated to near the groundwater table, at depths on the order of 11½ to 13½ 14 feet, and be completed to at least 15 feet outside the improvement. Excavations may generate wet materials that will require "drying back" to a workable moisture content prior to placement as compacted fill.

3. In order to mitigate the potential for adverse settlement/lateral spreading due to earthquake shaking, ground treatment options for alluvial soils are presented in the following table.

GROUND TREATMENT	DESCRIPTION	COMPATIBLE FOUNDATION TYPES	QUALITY AND COST
Partial Removal/ Recompaction (R&R)	R&R completed to near the groundwater table.	Structural mat*	Treats surficial, unsaturated soils. Foundation design must accommodate potential settlements due to differential settlement and liquefaction. Structural mats could potentially require re-leveling after event or after significant time.
Partial Removal/ Recompaction (R&R) with geotextile reinforcement	R&R completed to near the groundwater table. Placement of geotextile fabrics (Mirafi HP 540, or equivalent) along removal bottom. The use of geotextiles in slope construction potentially mitigates lateral spreading.	Structural mat Post-tension slab	Treats surficial, unsaturated soils. Geotextile reinforces fill embankment, further minimizing differential settlements. Foundation design must accommodate potential settlements due to differential settlement and Liquefaction. Potential for foundation re-leveling after event.
Complete R&R	Complete R&R to suitable formation. Dewatering and perimeter shoring required	Structural mat Post-tension slab	Treats loose, near surface unsaturated and saturated soils below the groundwater table. Dewatering and shoring may be cost, or time prohibitive.

GROUND TREATMENT	DESCRIPTION	COMPATIBLE FOUNDATION TYPES	QUALITY AND COST
R&R with stone columns	R&R completed to near the groundwater table. Stone columns are vibrated stone columns, which are continuous vertical columns of dense interlocking aggregate, free of non-granular inclusions.	Structural mat Post-tension slab	Treats loose, near surface unsaturated soil. Stone columns reinforce cohesive soils and densify granular soils in order to increase bearing capacity, decrease total and differential settlement, provide vertical drainage pathways to increase the time-rate of consolidation settlement, and reduce the potential for liquefaction. A Cost/benefit evaluation vs. other methods will be needed.
R&R with Deep Soil Mixing	R&R completed to near the groundwater table. Deep soil mixing, or DSM is a process of mechanically blending the in situ soil with cementitious materials that are referred to as binders using a hollow stem auger and paddle arrangement. The intent of the soil mixing method is to achieve improved soil properties.	Structural mat Post-tension slab	Treats loose, near surface unsaturated soil. Deep soil mixing provides similar benefits as stone columns. A Cost/benefit evaluation vs. other methods will be needed.
R&R with compaction grouting.	R&R completed to near the groundwater table. Compaction grouting is a method of ground treatment that involves injecting a very stiff homogeneous grout mix in order to displace and compact soils. The injected grout pushes the soils to the side as it forms a grout column or bulb.	Structural mat Post-tension slab	Treats loose, near surface unsaturated soil. Compaction grouting provides similar benefits as stone columns. A Cost/benefit evaluation vs. other methods will be needed.

* Deep foundations may be considered, but will not mitigate pad settlement in this condition.

Ground Improvement

In order to improve the static and seismic performance of the natural alluvial sediments during the life of the proposed improvements, either specialized foundations (mat or pile supported) or ground improvements (see table above) should be considered. Deep foundations would potentially mitigate residential foundations, but not reduce static/seismic pad settlement. The static and seismic differential angular distortion for both the pad and foundation loaded areas of the "Group 3" areas of the project site may be more than 0.008 (δ/L) (1/120). This may exceed the tolerance of the proposed improvements in this area.

Given the typical shallow/residential foundations, the anticipated loading, as well as the thickness of the potentially compressible/liquefiable sediments as discussed herein, consideration should be given to ground improvement for Group 3 lots. The depth of the groundwater will likely impact the extent (vertical) of remedial grading that may be used without shoring and dewatering efforts. Dynamic deep compaction using a heavy (5 to 25 ton) weight falling 40 to 75 feet may not be effective due to the depth to the groundwater. Other ground improvement methods considered were chemical grouting and surcharging. The latter may be cost effective due to the granular nature and depth of the natural sediments, but will not eliminate seismic effects. The depth of treatment will be on the order of two-thirds to all of the saturated thickness of the in-place sediments, on the order of 25 to 40 feet (i.e., compressible and/or liquefiable deposits) underlying Group 3 areas. That is to say, the extent of the ground improvement should be sufficient to remove approximately one-half or more of the potential deformation and bring the proposed improvement sites within the deformation tolerances of 0.002 δ/L (1/480).

Miscellaneous

It should be noted, that the 2013 CBC (CBSC, 2013) indicates that removals of unsuitable soils be performed across all areas under the purview of the grading permit, not just within the influence of the structure. Relatively deep removals may also necessitate a special zone of consideration, on perimeter/confining areas or near existing utilities. This zone would be approximately equal to the depth of removals, if removals cannot 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 require proper disclosure to all future owners and interested/affected parties. Utilities that cross this zone between mitigated and unmitigated ground may require special details to reduce the potential for rupture during a seismic event.

Transitions/Overexcavation

In order to reduce the potential for differential settlement and facilitate trenching for foundations underground utilities, etc., the entire cut portion of the building pad(s), areas with planned fills less than 4 feet thick, and areas where the as-built fill thickness would be less than 4 feet after remedial removals have been performed should be overexcavated to a minimum depth of 4 feet below finish grade or 2 feet below the lowest foundation element (whichever is greater) and be replaced with compacted fill. The overexcavation subgrade bottom should be inclined to drain away from the structure(s), and into the street. Prior to fill placement, the overexcavation subgrade should be scarified at least 8 inches in depth, moisture conditioned as necessary, and be recompacted to at least 90 percent of the laboratory standard (ASTM D 1557). Overexcavation should be completed to a minimum lateral distance of 5 feet outside the outermost exterior foundation. Overexcavation for underground utilities may be completed to at least 1 foot below the lowest utility invert and be replaced with compacted fill. The undercut transition should not create a minimum to maximum of fill thickness variation of more than 3:1 (maximum to minimum) across any lot.

Fill Import

If the importation of fill soil is necessary, the import material should be reviewed by this office prior to delivery. In general, import fill should be very low to low expansive (expansion index less than 50), and contain 6 inch minus material.

Engineered Fill Placement

Engineered fill should be placed in thin (±6- to 8-inch) lifts, that have been cleaned of vegetation and debris, and moisture conditioned, and mixed to minimally achieve the soil's optimum moisture content, and then be compacted to at least 90 percent of the laboratory standard (ASTM D 1557). Onsite expansive soils may be placed in thin (±6- to 8-inch) lifts that have been cleaned of vegetation and debris, brought to at least 120 percent of (1.2 times) the soil's optimum moisture content, and compacted to achieve a minimum relative compaction of 90 percent of the laboratory standard (ASTM D 1557). Engineered fill placement should be observed and selectively tested for moisture content and compaction by the geotechnical consultant.

Fill Quality

Fill material produced from excavations within onsite soils (i.e., existing fills, colluvium, alluvium, and older alluvium) will generally generate mixtures of silty sand, sand and gravelly sand, with minor amounts of clayey sand, and produce good to fair quality fill material.

Excavations within the underlying granitic bedrock will generally produce good quality material near the surface, with poor quality fill material consisting of angular gravel to cobble to boulder size rock fragments becoming more abundant with depth of excavation.

Onsite soils may be reused as compacted fill provided that major concentrations of vegetation, miscellaneous debris, and oversize material (see below) are removed from the fill, prior to or during fill placement. General recommendations for the treatment of rock onsite is presented in a following section.

Slope Considerations and Slope Design

Graded Slopes

Graded slopes are generally considered to be stable, up to gradients of 2:1 (h:v) or flatter, and bedrock slopes may be suitable to gradients of 1.5:1, or flatter. However, mapping indicates some potential for dip slope oriented fractures/joints in bedrock that may require stabilization, and slope gradients of 2:1, or flatter. Natural slopes appear to be performing adequately.

All slopes should be designed and constructed in accordance with the minimum requirements of City/County, the 2013 CBC (CBSC, 2013), the current "Greenbook," and the recommendations in Appendix G. Due to the predominantly granular nature of site soils, slopes are anticipated to have erosion and surficial instability issues if left unplanted, and without engineered surface drainage control, and as such, will require periodic and regular maintenance above and beyond what is normally performed for slopes in general.

Cut Slopes

Cut slopes are generally considered to be grossly stable. However, the dense nature of cut slopes constructed in granitic bedrock may present difficulties with respect to landscaping and planting. In order to enhance the plantability of these slopes, consideration may be given to reconstructing cut slopes as stability fill slopes, if desired. General stabilization fill slope design and construction is presented in Appendix G.

Planned Fill Slopes

Planned fill slopes are generally considered to be grossly stable to the anticipated heights and gradients shown on the plans. Fill slopes should performed adequately assuming that the slope are properly constructed, and maintained, under conditions of normal rain fall and climate.

Subdrains

The need for subdrainge within perimeter fill slope keyways will be evaluated during grading. Subdrains will be recommended at the base of any canyon fill. Subdrains will also be recommended within stabilization fill keyways, if constructed. If encountered, local seepage along the contact between the bedrock and overburden materials, or along jointing patterns of the bedrock may require a subdrain system. Typical subdrain design and construction details are presented in Appendix G.

Toe Drains

In order to mitigate perched water conditions associated with permeability contrast between fill and bedrock, and due to the potential for significant storm water runoff rom cut slopes, cut slopes in granitic bedrock should be provided with a toe of slope subdrain, or "toe drain" as discussed in the "Development Criteria" section of this report. Toe drains may be warranted at other locations as well.

Temporary Slopes

Temporary slopes completed in non-saturated, medium dense to dense, granular soils 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 is not encountered. Construction materials or soil stockpiles should not be placed within 'H' of any temporary slope where 'H' equals the height of the temporary slope.

For saturated soils encountered near the groundwater table, temporary slopes should conform to CAL-OSHA and/or OSHA requirements for Type "C" soils. Local dewatering may also be required.

All temporary slopes should be observed by a licensed engineering geologist and/or geotechnical engineer prior to worker entry into the excavation. Based on the exposed field conditions, inclining temporary slopes to flatter gradients or the use of shoring may be necessary if adverse conditions are observed.

Embankment Factors

Embankment factors (shrinkage/bulking) for the site have been estimated based upon our experience with other sites in the general vicinity, and limited field density testing of near-surface soils. It is apparent that shrinkage would vary with depth and with areal extent over the site, based on previous site use. Variables include vegetation, weed control, discing, and previous filling or exporting. However, all these factors are difficult to define in a three-dimensional fashion, and the contractors compactive efforts may also contribute some variance. Therefore, the information presented below represents average shrinkage and bulking values, using the following assumptions.

Colluvium	10-15% Shrinkage
Alluvium	10-15% Shrinkage
Older Alluvium	2-5% Shrinkage
Existing Fill	2-5% Shrinkage
Bedrock (from Church, 1981)	
25% Rock/75% Earth	8% Shrinkage
75% Rock/50% Earth	5% Shrinkage
75% Rock/25% Earth	12% Bulk
100% Rock	33% Bulk

Rock Crushing and/or Placement Guidelines

Crushing/Rock Disposal

GSI anticipates that some of the onsite soils to be utilized as fill material for the subject project may contain some rock, especially during grading operations in the vicinity of Planning areas PA-3, PA-4, and PA-5. Appropriately, the need for rock crushing and/or disposal may be necessary during grading operations on the site. The option for crushing rocks or oversize disposal should be value engineered. 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 rock 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 in occurring in cut or natural areas, would need to be disclosed to all interested/affected parties. Once approved by the governing agency, the hold-down depth for oversized rock (i.e., greater than 12 inches) in fills on this project is provided as 10 feet. The re-use of oversized materials around pools (next to or below) is not recommended.

General

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

Materials 8 Inches in Diameter or Less

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

1. Fines and rock fragments 8 inches or less in diameter may be placed as compacted fill cap materials within the building pads, slopes, and driveway areas as described below. The rock fragments and fines should be brought to <u>at least</u> optimum moisture content and compacted to a minimum relative compaction of 90 percent of the laboratory standard.

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

3. Fill materials 8 inches or less in diameter should be placed (but not limited to) within the hold-down depth on proposed fill pads, the upper 5 feet of overexcavated cut areas of cut/fill transition pads, and the entire street right-of-way width, including the proposed overexcavated areas and replacement fill areas, from the depth of the lowest utility (within the street and lot), to subgrade, or to the hold-down depth below finish grade. Overexcavation is discussed later in this report.

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

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

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

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

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

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

6. Minor amounts of rock material in this size range may also be placed a rock windrows (see below).

Substructures Placed in the Hold-down Depth Zone

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

RECOMMENDATIONS - FOUNDATIONS

Typical foundation design for very low to low expansive soil conditions is anticipated where support is provided by engineered fill overlying older alluvium or bedrock. Building areas underlain with alluvial deposits and shallow groundwater will require relatively more onerous foundation design, in addition to mitigative earthwork such as, but not necessarily limited to fill surcharging, and/or other ground improvement.

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 structure are made, the conclusions and recommendations contained in this report are for the subject site only and 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 considered minimums and are not meant to supercede design(s) by the project structural engineer or civil engineer specializing in structural design. Upon request, GSI could provide additional consultation regarding soil parameters, as related to foundation design. They are considered preliminary recommendations for proposed construction, in consideration of our field investigation, laboratory testing, and engineering analysis. We anticipate that the wall loads of 1.5 to 3.0 kips/foot, and column loads of 5 to 50 kips will be utilized.

As previously indicated, foundation systems will be supported by engineered fill bearing on older alluvium and/or granitic bedrock, left in place alluvium below the groundwater table, or left in place alluvium that has been improved by methods such as stone columns, grouting, deep mixing, etc. Based on the as-built conditions, including area geology, soil expansion, treatment of existing ground, and/or ground improvement, etc., GSI recommends foundation design in accordance with the following categories:

Category I - Conventional slabs. Limited to very low to low expansive soil conditions. Best suited for settlement Group Areas 1 and 2 (PA-3, PA-4, PA-5, and the north Equestrian/Estate area).

Category II - Post tension [PT] slab foundations. May be used for all expansive soil conditions onsite, and may be used for settlement Group Areas 1, and 2. May be used for structures within settlement Group Area 3, dependant upon method or extent of ground improvement.

Category III - Structural Mat slabs and/or stiffened slabs per WRI (1981, 1986). May be used for all expansive soil conditions onsite. May be used for settlement Group Areas 1 and 2. May be used for Group 2 areas, dependant upon method or extent of ground improvement.

Ancillary structures (benches, light poles, utility boxes) may use either these types, or conventional spread footings for support.

Foundation Design Parameters

General

1. The foundation systems should be designed and constructed in accordance with guidelines presented in the 2013 CBC (CBSC, 2013). All foundations should be embedded entirely into newly compacted or mitigated fill (90 percent of ASTM D 1557).

2. An allowable bearing value of 2,000 pounds per square foot (psf) may be used for design of footings that maintain a minimum width of 12 inches and a minimum depth of 12 inches, and founded in compacted fill. This value may be increased by 20 percent for each additional 12 inches in depth to a maximum value of 2,500 psf.

In addition, this value may be increased by one-third when considering short duration wind or seismic loads. Isolated pad footings should have a minimum dimension of at least 24 inches square and minimum depth of 24 inches, and be connected in two directions back to the main portion of the foundation. The depth of embedment shall not include the slab thickness nor underlayment, and shall be below the lowest adjacent grade.

3. Passive earth pressure may be computed as an equivalent fluid having a density of 250 pounds per cubic foot (pcf), with a maximum lateral earth pressure of 1,500 psf. Lateral passive pressures for shallow foundations within 2013 CBC setback zones should be reduced following a review by the geotechnical engineer unless proper setback can be established.

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

5. For the evaluation of total lateral resistance on the foundation and combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third. For effect of shrink-swell soils on hillside foundations, the geotechnical consultant should review foundation designs when available. The addition of creep loads on top-of-slope or mid-slope foundations should be considered.

6. Seismic design parameter are presented in a previous section of this report.

Settlement Summary

For preliminary design purposes, a summary of potential foundation settlement is presented in the following table.

SETTLEMENT SUMMARY ESTIMATES*			
SETTLEMENT GROUP AREA.	**STATIC**	**SEISMIC**	**STATIC PLUS SEISMIC DIFFERENTIAL SETTLEMENT**
Group 1 - Fill over Older Alluvium (North Equestrian/ Estate area	1½ inch total, ¾ in differential in 40 feet for fills up to 20 feet	½ inch total, and ¼ inch differential in 40 feet	1 inch in 40 feet. (Thicker fills will result in greater differential settlements)
Group 2 - Fill over Granitic Bedrock	1½ inch total, ¾ in differential in 40 feet for fills up to 30 feet	½ inch total, and ¼ inch differential in 40 feet	1 inch in 40 feet (Fills thicker than 30 feet will result in greater differential settlements)

SETTLEMENT SUMMARY ESTIMATES*			
SETTLEMENT GROUP AREA.	**STATIC**	**SEISMIC**	**STATIC PLUS SEISMIC DIFFERENTIAL SETTLEMENT**
Group 3, fill over alluvium below the groundwater table. PA-1, PA-2, North Equestrian/Estate area	Angular distortions of greater than 1/480. With ground improvement, angular distortions could be reduced to 1/480.	Up to ±6 inches total, and up to 2½ inches differential over 100 feet. Seismic settlement reduced with increased fill surcharge (i.e., fill placed above existing grade)	1 inch in 40 feet (with ground improvement)

* Does not include foundation settlement due to applied footing loads.

It should be kept in mind that drainage reversals could occur in areas underlain with alluvium left in place below the groundwater table, when considering post-construction static and seismic settlement, if relatively flat yard drainage gradients are not periodically maintained by the maintenance department, owners, and/or other interested/affected parties. Similarly, gravity flow utilities in areas underlain by alluvium are also subject to possible drainage reversals or deflections, considering the magnitude and angular distortions of settlement reported herein.

Category I (i.e., Very Low Expansive Soils, Settlement Group Areas 1 and 2)

Conventional Slabs

The following foundation construction recommendations are presented as a minimum criteria from a soils engineering viewpoint for very low expansive soils consisting of engineered fill over older alluvium, or granitic bedrock only. Recommendations by the project's design/structural engineer or architect, which may exceed the soils engineer's recommendations, should take precedence over the following minimum requirements. These are for conventional foundations of ancillary structures (other than buildings) that need not comply with criteria for foundations on expansive soils per Code.

1. Continuous footings should be founded at a minimum depth of 12 and 18 inches below the lowest adjacent ground surface bearing properly compacted fill, for one- or two-story floor loads, respectively. All footings should be reinforced with a minimum of two No. 4 reinforcing bars at the top and two No. 4 reinforcing bars at the bottom (four bars total). Reinforcement of Isolated footings should be provided by the structural engineer. The depth of embedment is measured from the lowest adjacent grade, and does not include slab underlayment or the landscape zone.

2. A grade beam, reinforced as above, and at least 12 inches square, should be provided across any large entrance (garage, etc.). The base of the reinforced grade beam should be at the same elevation as the adjoining footings.

3. Concrete slabs should be a minimum of 5 inches. Recommendations for floor slab construction and the mitigation of moisture vapor transmission are presented in a later section of this report.

4. Concrete slabs, including large building entrance areas, should be minimally reinforced with No. 3 reinforcement bars placed on 18-inch centers, in two horizontally perpendicular directions (i.e., long axis and short axis). 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.

5. The slab and footing subgrade should be free of loose and uncompacted material prior to placing concrete.

6. Soils generated from footing excavations to be used onsite should be compacted to a minimum relative compaction 90 percent of the laboratory standard (ASTM D 1557), whether it is 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.

7. Footings should maintain a horizontal distance, X, between any adjacent descending slope face and the bottom outer edge of the footing. The horizontal distance, X, may be calculated by using X = H/3, where “H” is the height of the slope. X should not be less than 7 feet, nor need not be greater than 40 feet. X may be maintained by deepening the footings. Setbacks should minimally conform to Section 1808.7.2, and 1808.7.3 of the 2013 CBC (CBSC, 2013) guidelines as applicable, unless specifically superceded herein.

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, including the above recommendations for conventional slabs.

For a typical slab designed with interior ribs, or stiffeners, the slab should minimally be at least 5 inches thick. The ribs should be provided in both transverse and longitudinal directions. The interior rib spacing and depth should be provided by the project structural engineer. The perimeter beams, however, should be embedded as specified in the post-tension slabs section of this report, 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. Please note that stiffener beams will tend to make water vapor retarder installation more complex.

Category II - Post-tension Slab Foundations, Settlement Group Areas 1 and 2. Group 3 with Ground Improvement

Post-tension (PT) slab foundation may also be used to support the structure. 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). 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).

TABLE - POST-TENSION FOUNDATION DESIGN(3)

DESIGN PARAMETER(3)	VERY LOW TO LOW EXPANSION POTENTIAL
e_m center lift	9.0 feet
e_m edge lift	5.2 feet
y_m center lift	0.3 inches
y_m edge lift	0.7 inch
Bearing Value (1)	1,000 psf
Lateral Pressure	250 psf
Subgrade Modulus (k)	100 pci/inch
Minimum Perimeter Footing Embedment (2)	12 inches

(1) Internal bearing values within the perimeter of the post-tension slab may be increased to 1,500 psf for a minimum embedment of 18 inches, then by 20 percent for each additional foot of embedment to a maximum of 2,000 psf.
(2) As measured below the lowest adjacent compacted subgrade surface.
(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 maintenance staff, owners, affected/interested parties. The values tabulated above may not be appropriate to account for possible differential settlement of the slab due to other factors, such as excessive settlements. If a stiffer slab is desired, alternative Post-Tensioning Institute ([PTI] third edition) parameters may be recommended.

Category III - Structural Mat Foundations, Settlement Group Areas 1 and 2. Group 3 with Ground Improvement

As previously, indicated soils within the influence of the proposed structures are generally considered to be very low to possibly low expansive. However, settlement potentials due to the presence of left in place alluvium in settlement area 3 (PA-1, PA-2, and south Equestrian/Estate areas) generally exceed the tolerance of a typical slab on grade foundation system. As such, a mat slab foundation may be considered in these areas.

A structural mat-type foundation slabs should be a minimum of 10 inches in thickness, and in accordance with the structural engineer, and also be reinforced with a double mat of rebars at the spacing recommended by the structural engineer. Footings should be embedded as indicated herein, below the lowest adjacent grade into properly compacted fill, unless expansive soil conditions dictate deeper embedments as discussed in a following section. The need and arrangement of grade beams will be in accordance with the structural consultant's recommendations. Alternative uniform thickness mat slabs may be used in the design if the structural consultant can demonstrate that the alternative is equivalent to the recommended mat slab/footing. All mat-type designs should resist expansive soil conditions as explained herein.

Recommended design parameters used in the design of WRI foundations (WRI, 1996) and slabs-on-grade are provided in the following table.

WRI DESIGN PARAMETERS	
Effective Plasticity Index*	20
Unconfined Compressive Strength*	1,000 psf (0.5 tsf)
Modulus of Subgrade Reaction	100 pci
Settlement Potential	see Text
Resistance Value (R-value)*	38
Minimum Slab Thickness	6 inches
Minimum Steel Reinforcement per Structural Engineer	Double Mat of Steel Reinforcement Bars per Structural Consultant

* To be re-evaluated upon completion of grading.

For this method, either a uniform thickness foundation (UTF) or mat may be used. Alternatively, the slab (in plan view) may be divided up into at least quarters and grade beams should be used to enhance the strength of the slab to resist the expansive soil forces. The foundation bearing capacity and other geotechnical parameters previously provided in this report are still applicable.

Perimeter cut-off walls may be incorporated into the UTF design and should be 18 inches deep for the medium to highly expansive soil conditions evaluated onsite. The cut-off walls may be integrated into the slab design or independent of the slab. The cut-off walls should be a minimum of 6 inches thick. The bottom of the perimeter cut-off wall should be designed to resist tension, using reinforcement per the structural engineer.

Slab Subgrade Pre-Soaking

Pre-moistening of the slab subgrade soil is recommended for these soil conditions. The moisture content of the subgrade soils should be equal to or greater than optimum moisture to a depth equivalent to the exterior footing depth in the slab areas (typically 12, for very low to low expansive soils). Pre-moistening and/or pre-soaking should be evaluated by the soils engineer 72 hours prior to vapor retarder placement. In summary:

EXPANSION INDEX	PAD SOIL MOISTURE	CONSTRUCTION METHOD	SOIL MOISTURE RETENTION
Very Low (0-20)	Upper 12 inches of pad at or above soil optimum moisture	Wetting and/or reprocessing	Periodically wet or cover with plastic after trenching. Evaluation 72 hours prior to placement of concrete.
Low (21-50)	Upper 12 inches of pad soil moisture 2 percent over optimum	Wetting and/or reprocessing	Periodically wet or cover with plastic after trenching. Evaluation 72 hours prior to placement of concrete.

SOIL MOISTURE CONSIDERATIONS

GSI has evaluated the potential for vapor or water transmission through the slabs, in light of typical floor coverings and improvements. Generally, slab moisture emission rates range from about 2 to 27 lbs./1,000 square feet from a typical slab (Kanare, 2005), while most floor covering manufacturers recommend about 3 lbs./24 hours as an upper limit. Thus, the client will need to evaluate the following in light of a cost versus benefit analysis (tenant complaints and repairs/replacement), along with disclosure to owners.

Considering the proximity of groundwater, potential for perched groundwater to occur, E.I. test results, anticipated typical water vapor transmission rates, and floor coverings and improvements (to be chosen by the client) that can tolerate those rates without distress, the following alternatives are provided:

- Concrete slabs should be a minimum of 5 inches thick.

- Concrete slab underlayment should consist of a 15-mil vapor retarder, or equivalent, with all laps sealed per the 2013 CBC (CBSC, 2013) 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 D 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.).

- The vapor retarder should be underlain with 2 inches of washed sand, and should be overlain by a 2-inch thick layer of washed sand (SE>30).

- Concrete should have a maximum water/cement ratio of 0.50. This does not supercede the 2013 CBC (CBSC, 2013) 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 above, 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.

- Owner(s) should be specifically advised which areas are suitable for tile flooring, wood 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, and areas potentially using moisture sensitive floor coverings and/or moisture sensitive storage, should be identified 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.

Corrosion and Concrete Mix

Upon completion of grading, laboratory testing should be performed of site materials for corrosion to concrete and corrosion to steel. Soils with negligible to moderate levels of sulfate content are present near the surface. As such, the use of Type V concrete is not required per 2013 CBC, as well as ACI 318-11, on a preliminary basis. Additional comments may be obtained from a qualified corrosion engineer.

WALL DESIGN PARAMETERS

Conventional Retaining Walls

The design parameters provided below assume that either very low expansive soils (typically Class 2 permeable filter material or Class 3 aggregate base) or native onsite materials with an expansion index up to 20 are used to backfill any retaining wall. The type of backfill (i.e., select or native), should be specified by the wall designer, and clearly shown on the plans. Building walls, below grade, should be water-proofed. Waterproofing may also be provided for site retaining walls in order to reduce the potential for efflorescence staining. Recommendations for specialty walls, or mechanically stabilized earth (MSE) walls (i.e., crib, earthstone, geogrid, etc.) can be provided on request.

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 any topsoil/colluvium, or landscape layer [upper 6 inches]), into suitable bedrock.

Minimum Footing Width - 24 inches

Allowable Bearing Pressure - An allowable bearing pressure of 3,000 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 bedrock. This pressure may be increased by one-third for short-term wind and/or seismic loads.

Passive Earth Pressure - A passive earth pressure of 300 pcf with a maximum earth pressure of 3,000 psf may be used in the preliminary design of retaining wall foundations 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 120 pcf and 125 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 earth material 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 (level backfill). The design should include any applicable surcharge loading. For areas of male or re-entrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall (2H) laterally from the corner.

Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls up to 10 feet high. Design parameters for 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. (SELECT NATIVE BACKFILL)(3)
Level(1)	38	50
2 to 1	55	65

(1) Level backfill behind a retaining wall is defined as compacted earth materials, properly drained, without a slope for a distance of 2H behind the wall, where H is the height of the wall.
(2) SE $\geq$ 30, P.I. < 15, E.I. < 21, and $\leq$ 10% passing No. 200 sieve.
(3) E.I. = 0 to 30, SE $\geq$ 20, P.I. < 20, and $\leq$ 20% passing No. 200 sieve; confirmation testing required.
(4) E.I. = 30 to 50. E.I. > 50 material should not be used.

Earthquake Loads (Seismic Surcharge)

For engineered retaining walls, GSI recommends that the walls be evaluated for a seismic surcharge (in general accordance with 2013 CBC requirements). The site walls in this category should maintain an overturning Factor-of-Safety (FOS) of approximately 1.25 when the seismic surcharge (increment), is applied. For restrained walls, the seismic surcharge should be applied as a uniform surcharge load from the bottom of the footing (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. Reference for the seismic surcharge is Section 1802.2 of the 2013 CBC. 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:

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

Where: P_h = Seismic increment
a_h = Probabilistic horizontal site acceleration with a percentage of "g"
γ_t = Total unit weight (125 to 130 pcf for site soils at 90 percent relative compaction).
H = Height of the wall from the bottom of the footing or point of pile fixity.

Retaining Wall Backfill and Drainage

Positive drainage must be provided behind all retaining walls in the form of gravel wrapped in geofabric and outlets. A backdrain system is considered necessary for retaining walls that are 2 feet or greater in height. Details 1, 2, and 3, present the backdrainage options discussed below. Backdrains should consist of a 4-inch diameter perforated PVC or ABS pipe encased in either Class 2 permeable filter material or ¾-inch to 1½-inch gravel wrapped in approved filter fabric (Mirafi 140 or equivalent). For low expansive backfill, the filter material should extend a minimum of 1 horizontal foot behind the base of the walls and upward at least 1 foot. This material should be continuous (i.e., full height) behind the wall, and it should be constructed in accordance with the enclosed Detail 1 (Typical Retaining Wall Backfill and Drainage Detail). For limited access and confined areas, (panel) drainage behind the wall may be constructed in accordance with Detail 2 (Retaining Wall Backfill and Subdrain Detail Geotextile Drain). Any materials (if encountered) with an expansion index (E.I.) potential of greater than 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).

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

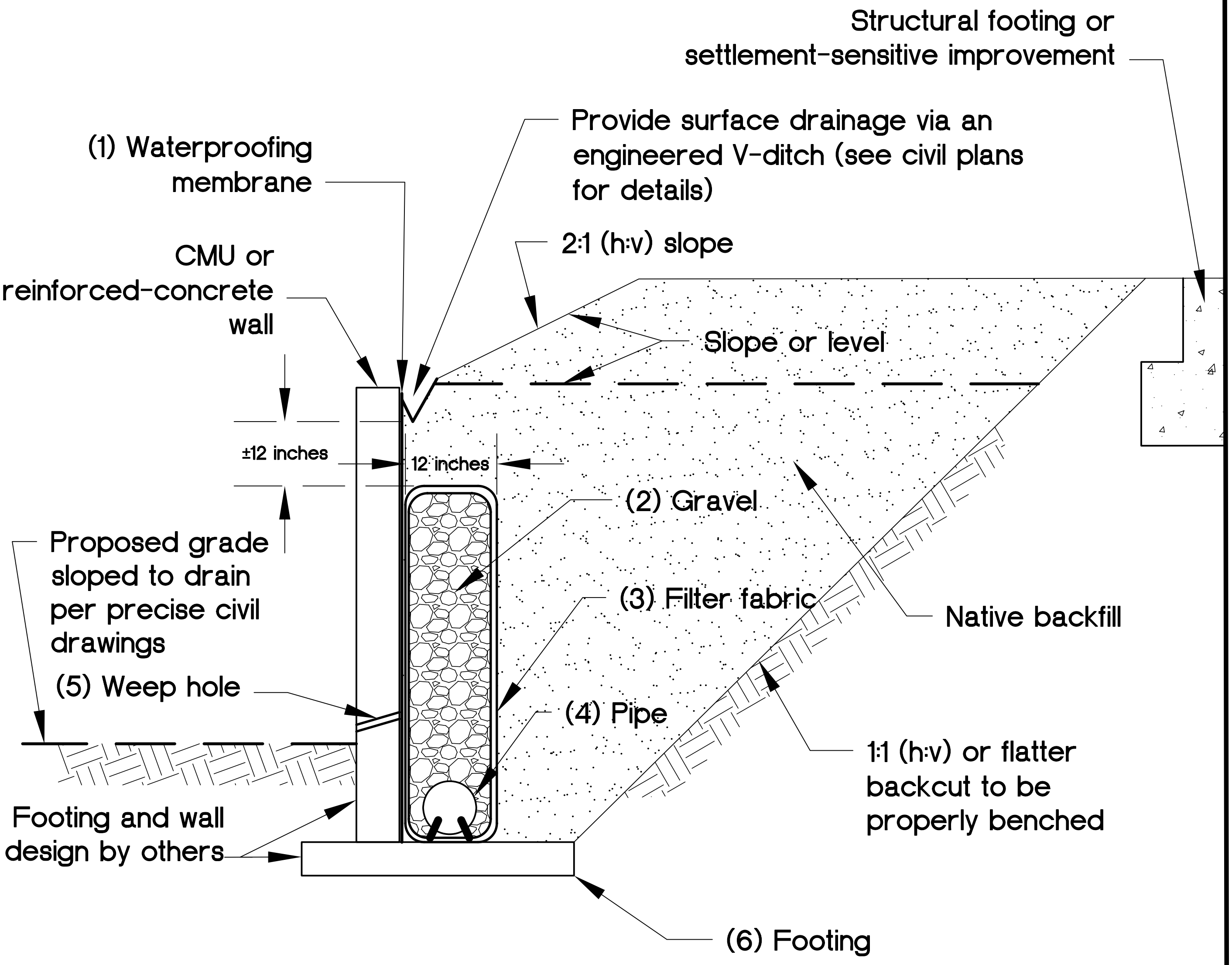
Wall/Retaining Wall Footing Transitions

Site walls are anticipated to be founded on footings designed in accordance with the recommendations in this report. Should wall footings transition from 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 an angular distortion of 1/360 for a distance of 2H on either side of the transition may be accommodated. Expansion joints should be placed no greater than 20 feet on-center, in accordance with the structural engineer's/wall designer's recommendations, regardless of whether or not transition conditions exist. Expansion joints should be sealed with a flexible, non-shrink grout.

c) Embed the footings entirely into native formational material (i.e., deepened footings).



(1) Waterproofing membrane.

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

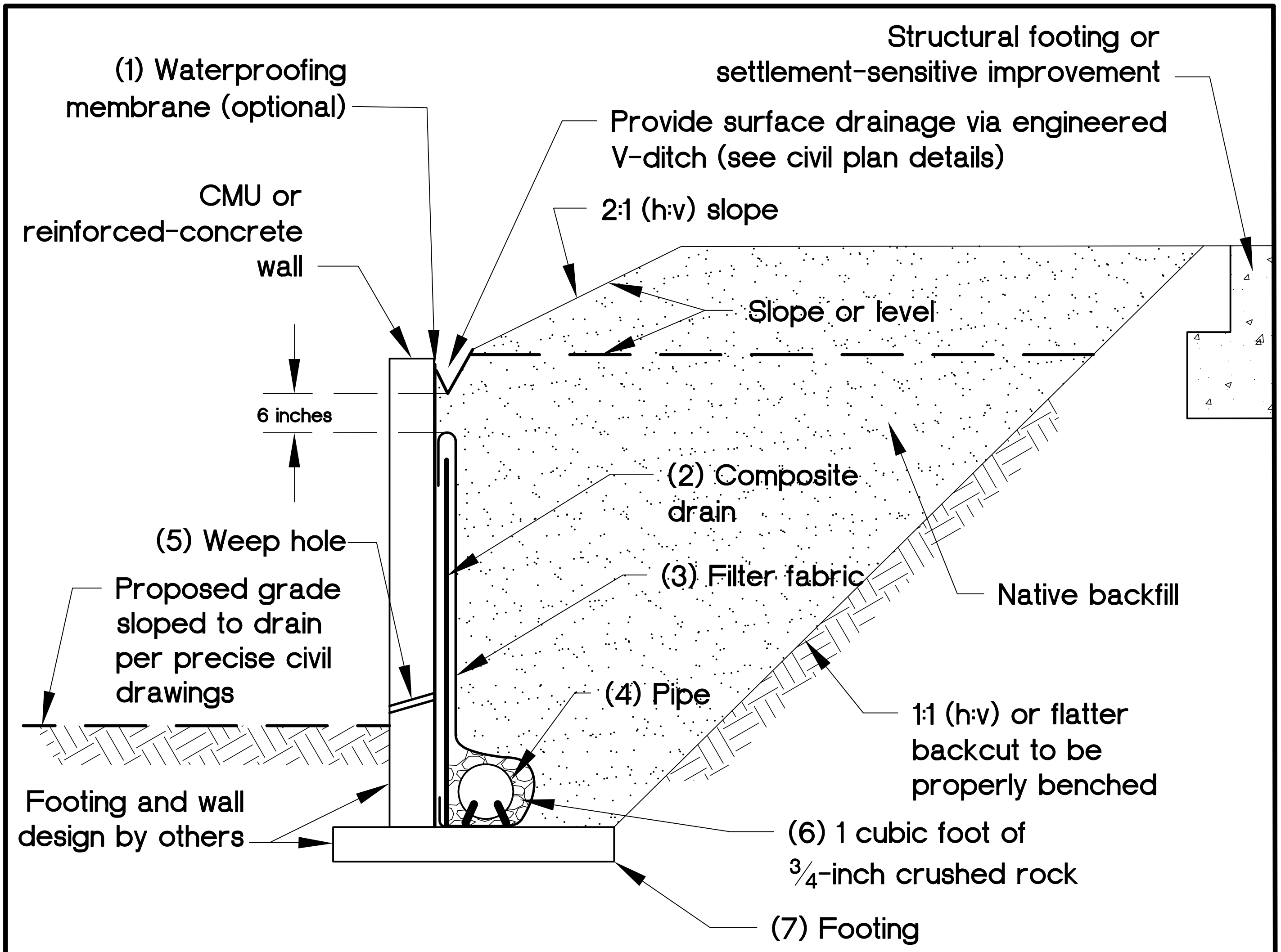
(3) Filter fabric: Mirafi 140N or approved equivalent.

(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.

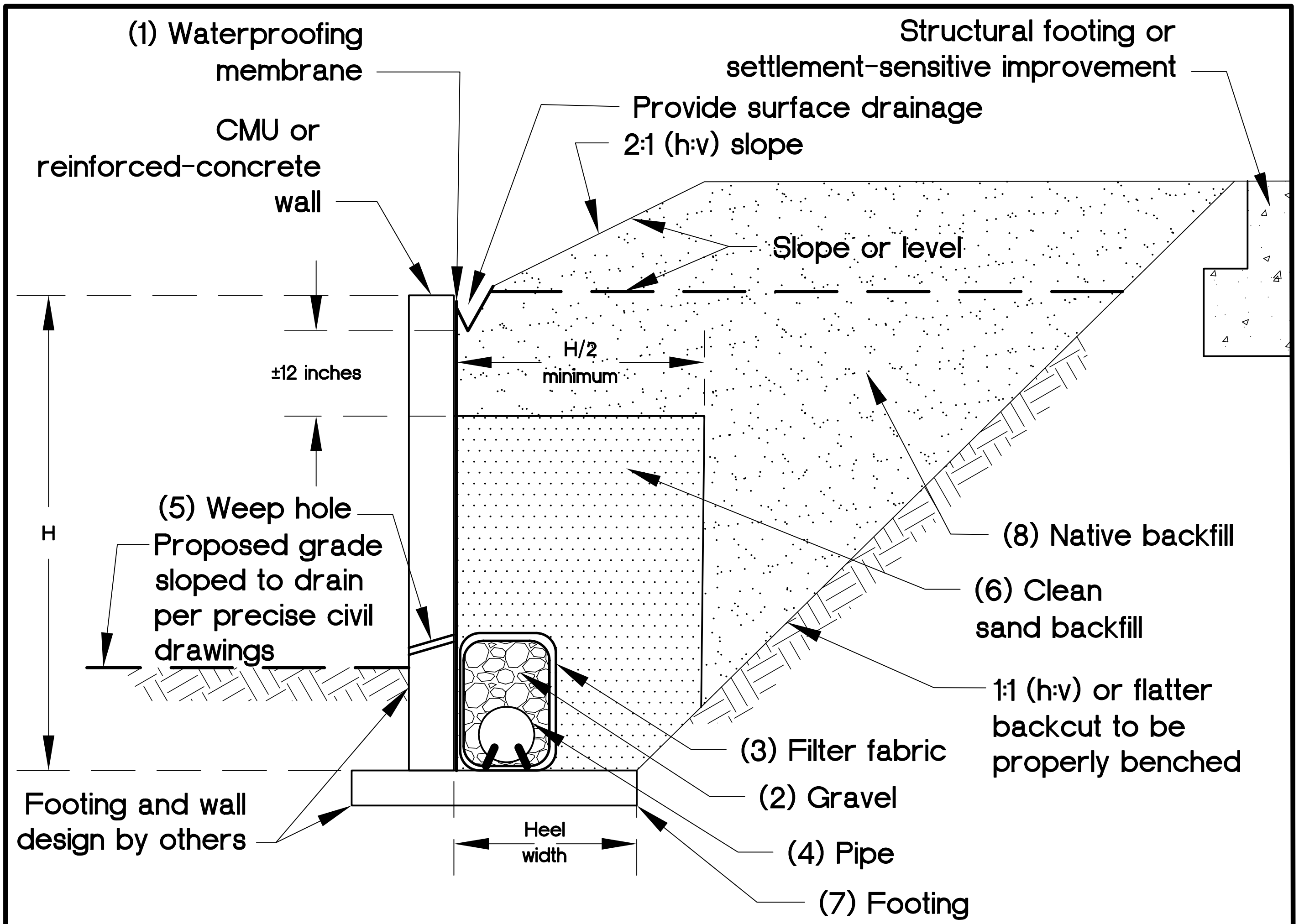
(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

(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, $^3\!/_4$ to $1^1\!/_2$ inch.

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

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

(5) Weep hole: Minimum 2-inch diameter placed at 20-foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

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

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

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

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

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.

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

Expansive Soils and Slope Creep

Soils at the site are likely to be expansive (i.e., E.I. > 0) and therefore, become desiccated when allowed to dry. Such soils are susceptible to surficial slope creep, especially with seasonal changes in moisture content. Typically in southern California, during the hot and dry summer period, these soils become desiccated and shrink, thereby developing surface cracks. The extent and depth of these shrinkage cracks depend on many factors such as the nature and expansivity of the soils, temperature and humidity, and extraction of moisture from surface soils by plants and roots. When seasonal rains occur, water percolates into the cracks and fissures, causing slope surfaces to expand, with a corresponding loss in soil density and shear strength near the slope surface. With the passage of time and several moisture cycles, the outer 3 to 5 feet of slope materials experience a very slow, but progressive, outward and downward movement, known as slope creep. For slope heights greater than 10 feet, this creep related soil movement will typically impact all rear yard flatwork and other secondary improvements that are located within about 15 feet from the top of slopes, such as concrete flatwork, etc., and in particular top of slope fences/walls. This influence is normally in the form of detrimental settlement, and tilting of the proposed improvements. The dessication/swelling and creep discussed above continues over the life of the improvements, and generally becomes progressively worse. Accordingly, the developer should provide this information to all interested/affected parties.

In addition, surficial slope failures occurring along the slope descending from the subject site have the potential to affect improvements (walls, flatwork, etc.) constructed within about 5 feet from the top of this slope. To that end, improvements located within this zone should be supported by CIDH piles (caissons).

Top of Slope Walls/Fences

Due to the potential presence of loose/soft bearing soils along property lines, some settlement and tilting of the walls/fence with the corresponding distresses, should be expected. Furthermore, due to the potential for slope creep for slopes higher than about 10 feet, some settlement and tilting of the walls/fence with corresponding distress, should be expected. To mitigate the tilting of top of slope walls/fences, we recommend that the walls/fences be constructed on a combination of grade beam and caisson foundations. The grade beam should be at a minimum of 12 inches by 12 inches in cross section, supported by drilled caissons, 12 inches minimum in diameter, placed at a maximum spacing of 6 feet on center, and with a minimum embedment length of 7 feet below the bottom of the grade beam. The strength of the concrete and grout should be evaluated by the structural engineer of record. The proper ASTM tests for the concrete and mortar should be provided along with the slump quantities. The concrete used should be appropriate to mitigate severe sulfate exposure. The design of the grade beam and caissons should be in accordance with the recommendations of the project structural engineer, and include the utilization of the following geotechnical parameters:

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

Creep Load: The creep load projected on the area of the grade beam should be taken as an equivalent fluid approach, having a density of 60 pcf. For the caisson, embedded into low to highly expansive soil, it should be taken as a uniform 900 pounds per linear foot of caisson's depth, located above the creep zone.

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

Passive Resistance: Passive earth pressure of 300 psf per foot of depth per foot of caisson diameter, to a maximum value of 4,000 psf may be used to determine caisson depth and spacing, provided that they meet or exceed the minimum requirements stated above. To determine the total lateral resistance, the contribution of the creep prone zone above the point of fixity, to passive resistance, should be disregarded.

Allowable Axial Capacity:

Shaft capacity : 350 psf applied below the point of fixity (in formational soil) over the surface area of the shaft.

Tip capacity: 4,000 psf (clear of loose soil, bearing into dense formational soil).

CONCRETE FLATWORK, AND OTHER IMPROVEMENTS

The soil materials on site are expansive (i.e., E.I. > 0). 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 recommended that the developer should notify any interested/affected parties of this long-term potential for distress. To reduce the likelihood of distress, the following recommendations are presented for all exterior flatwork:

1. Concrete slabs should be founded entirely on properly compacted fill. The subgrade area for concrete slabs should be compacted to achieve a minimum 90 percent relative compaction, and then be presoaked to 2 to 3 percentage points above (or 130 percent of) the soils' optimum moisture content, to a depth of 18 inches below subgrade elevation. Refer to slab subgrade pre-soaking recommendation a previous section of this report. This moisture content should be maintained in the subgrade soils during concrete placement to promote uniform curing of the concrete and minimize the development of unsightly shrinkage cracks. If very low expansive soils are present, only optimum moisture content, or greater, is required and specific presoaking is not warranted. The moisture content of the subgrade should be proof tested within 72 hours prior to pouring concrete.

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

3. Exterior slabs supporting pedestrian traffic only should be a minimum of 4 inches thick.

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

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

   In order to reduce the potential for unsightly cracks, slabs should be reinforced at mid-height with a minimum of No. 3 bars placed at 18 inches on center, in each direction. The exterior slabs should be scored or saw cut, ½ to ⅜ inches deep, often enough so that no section is greater than 10 feet by 10 feet. For sidewalks or narrow slabs, control joints should be provided at intervals of every 6 feet. The slabs should be separated from the foundations and sidewalks with expansion joint filler material.

6. Surface and shrinkage cracking of the finish slabs may be reduced if a low slump and water-cement ratio are maintained during concrete placement. Excessive water

added to concrete prior to placement is likely to cause shrinkage cracking, and should be avoided. Some concrete shrinkage cracking, however, is unavoidable.

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

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

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

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

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

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

13. Positive site drainage should be maintained at all times. Finish grades should be provided with a minimum of 1 to 2 percent fall to the street, or other approved area, as indicated herein. It should be kept in mind that drainage reversals could occur, including post-construction settlement, if relatively flat yard drainage gradients are not periodically maintained by the maintenance department, school, owners, and/or other interested/affected parties.

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.

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

PRELIMINARY PAVEMENT DESIGN/CONSTRUCTION

Structural Section

Traffic Indices (TI) were assumed to range from 4.5 to 6.0 for the subject traffic areas, and should be reviewed by the project civil engineer for comment, and any revisions, as necessary. An R-value of 38 was assumed for preliminary planning purposes. The recommended preliminary pavement sections for both asphaltic concrete (A.C.) pavement over aggregate base (A.B.), and Portland concrete cement pavement (PCCP), are provided in the following tables:

APPROXIMATE TRAFFIC AREA	TRAFFIC INDEX(1)	SUBGRADE R-VALUE(2)	A.C. THICKNESS (INCHES)	A.B. THICKNESS(3) (INCHES)
Cul du Sac	4.5	38	3.0	6.0
Residential Street	5.0	38	3.0	6.0
Residential Street	5.5	38	3.0	6.0
Collector Street	6.0	38	4.0	8.0

(1) The TI is an estimation based on the intended use. The TI should be review for comment by the project civil engineer. Trash disposal areas, entry areas, fire vehicle access may require special design detailing.
(2) Estimate, to be verified by the project civil engineer.
(3) Denotes Class 2 Aggregate Base R ≥78, SE ≥25)
(4) Designs should follow city of san diego guidelines for PCCP aprons in front of trash enclosures.

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 4-inch thick leveling course of compacted aggregate base, or crushed rock may be considered to improve performance. All PCCP should be properly detailed (jointing, etc.) per the industry standard. Pavements may be additionally reinforced with #4 reinforcing bars, placed 12 inches on center, each way, for improved performance. Trash truck loading pads shall be 8 inches per the City standard reinforced accordingly.

All pavement installation, including preparation and compaction of subgrade, compaction of base material, and placement and rolling of asphaltic concrete, etc., shall be done in accordance with the County guidelines, and under the observation and testing of the project geotechnical engineer and/or the County.

The recommended pavement sections are meant as minimums. If thinner or highly variable pavement sections are constructed, increased maintenance and repair may be needed. The recommended pavement sections provided above are 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 TI 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.

Pervious Pavements

Manufacturer's guidelines for paver installation should be strictly adhered to. GSI should review such guidelines for comment, prior to construction. Pervious asphaltic concrete (A.C.) or Portland Cement Concrete (PCC) pavements should be reviewed for location and anticipated vehicle loading. Use of the AC or PCC pavement sections for said porous pavements should not use the sections herein without additional review and analysis by GSI.

Aggregate Base Rock

Compaction tests are required for the recommended base section. Minimum relative compaction required will be 95 percent of the laboratory maximum density as evaluated by ASTM Test Designation D 1557. Base aggregate should be in accordance to the Caltrans Class 2 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 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 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 base course and paving and the time between completion of base and paving is reduced to three days, provided the base is free of loose soil or debris. Where prime coat has been omitted and rain occurs, traffic is routed over base course, or paving is delayed, measures shall be taken to restore base course, and subgrade to conditions that will meet specifications as directed by the County and/or geotechnical consultant.

Onsite Infiltration-Runoff Retention Systems

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: terrace deposits, claystone, siltstone, cemented sandstone, igneous and metamorphic bedrock, as well as expansive fill soils.

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.

- Where possible, infiltration system design should be based on actual infiltration testing results/data, preferably utilizing double-ring infiltrometer testing (ASTM D 3385) to determine the infiltration rate of the earth materials being contemplated for infiltration. On a preliminary basis, infiltration may range from Hydrologic subgroup D, for compacted fills and bedrock, to Hydrologic subgroup A, for unmitigated alluvium within the valley floor area.

- 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.

- Infiltrations systems should not be installed within ±8 feet of building foundations utility trenches, and walls, or a 1:1 (h:v) slope (down and away) from the bottom elements of these improvements. Alternatively, deepened foundations and/or pile/pier supported improvements may be used.

- Infiltrations systems should not be installed adjacent to pavement and/or hardscape improvements. Alternatively, deepened/thickened edges and curbs and/or impermeable liners may be utilized in areas adjoining the OIRRS.

- As with any OIRRS, localized ponding and groundwater seepage should be anticipated. The potential for seepage and/or perched groundwater to occur after site development should be disclosed to all interested/affected parties.

- Installation of infiltrations systems should avoid expansive soils (Expansion Index [E.I.] ≥51) or soils with a relatively high plasticity index (P.I. > 20).

- Infiltration systems should not be installed where the vertical separation of the groundwater level is less than ±10 feet from the base of the system.

- Where permeable pavements are planned as part of the system, the site Traffic Index (T.I.) Should be less than 25,000 Average Daily Traffic (ADT), as recommended in Allen, et al. (2011).

- Infiltration systems should be designed using a suitable factor of safety (FOS) to account for uncertainties in the known infiltration rates (as generally required by the controlling authorities), and reduction in performance over time.

- As with any OIRRS, proper care will need to provided. Best management practices should be followed at all times, especially during inclement weather. Provisions for the management of any siltation, debris within the OIRRS, and/or overgrown vegetation (including root systems) should be considered. An appropriate inspection schedule will need to adopted and provided to all interested/affected parties.

- Any designed system will require regular and periodic maintenance, which may include rehabilitation and/or complete replacement of the filter media (e.g., sand, gravel, filter fabrics, topsoils, mulch, etc.) or other components utilized in construction, so that the design life exceeds 15 years. Due to the potential for piping and adverse seepage conditions, a burrowing rodent control program should also be implemented onsite.

- All or portions of these systems may be considered attractive nuisances. Thus, consideration of the effects of, or potential for, vandalism should be addressed.

- Newly established vegetation/landscaping (including phreatophytes) may have root systems that will influence the performance of the OIRRS or nearby LID systems.

- The potential for surface flooding, in the case of system blockage, should be evaluated by the design engineer.

- Any proposed utility backfill materials (i.e., inlet/outlet piping and/or other subsurface utilities) located within or near the proposed area of the OIRRS may become saturated. This is due to the potential for piping, water migration, and/or seepage along the utility trench line backfill. If utility trenches cross and/or are proposed near the OIRRS, cut-off walls or other water barriers will need to be installed to mitigate the potential for piping and excess water entering the utility backfill materials. Planned or existing utilities may also be subject to piping of finesinto open-graded gravel backfill layers unless separated from overlying or adjoining OIRRS by geotextiles and/or slurry backfill.

- The use of OIRRS above existing utilities that might degrade/corrode with the introduction of water/seepage should be avoided.

DEVELOPMENT CRITERIA

Slope Maintenance and Planting

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

Drainage

Adequate lot surface drainage is a very important factor in reducing the likelihood of adverse performance of foundations, hardscape, and slopes. Surface drainage should be sufficient to prevent ponding of water anywhere on a pad, and especially near structures and tops of slopes. Pad surface drainage should be carefully taken into consideration during fine grading, landscaping, and building construction. Therefore, care should be taken that future landscaping or construction activities do not create adverse drainage conditions. Positive site drainage within lots and common areas should be provided and maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond and/or seep into the ground. In general, the area within 5 feet around a structure should slope away from the structure. We recommend that unpaved lawn and landscape areas have a minimum gradient of 1 percent sloping away from structures, and whenever possible, should be above adjacent paved areas. Consideration should be given to avoiding construction of planters adjacent to structures (buildings, ancillary slabs, etc.). Pad drainage should be directed toward the street or other approved area(s). Although not a geotechnical requirement, roof gutters, down spouts, or other appropriate means may be utilized to control roof drainage. Down spouts, or drainage devices should outlet a minimum of 5 feet from structures or into a subsurface drainage system. Areas of seepage may develop due to irrigation or heavy rainfall, and should be anticipated. Minimizing irrigation will lessen this potential. If areas of seepage develop, recommendations for minimizing this effect could be provided upon request.

Toe of Slope Drains/Toe Drains

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

- Is there a source of irrigation above or on the slope that could contribute to saturation of soil at the base of the slope?

- Are the slopes hard rock and/or impermeable, or relatively permeable, or; do the slopes already have or are they proposed to have subdrains (i.e., stabilization fills, etc.)?

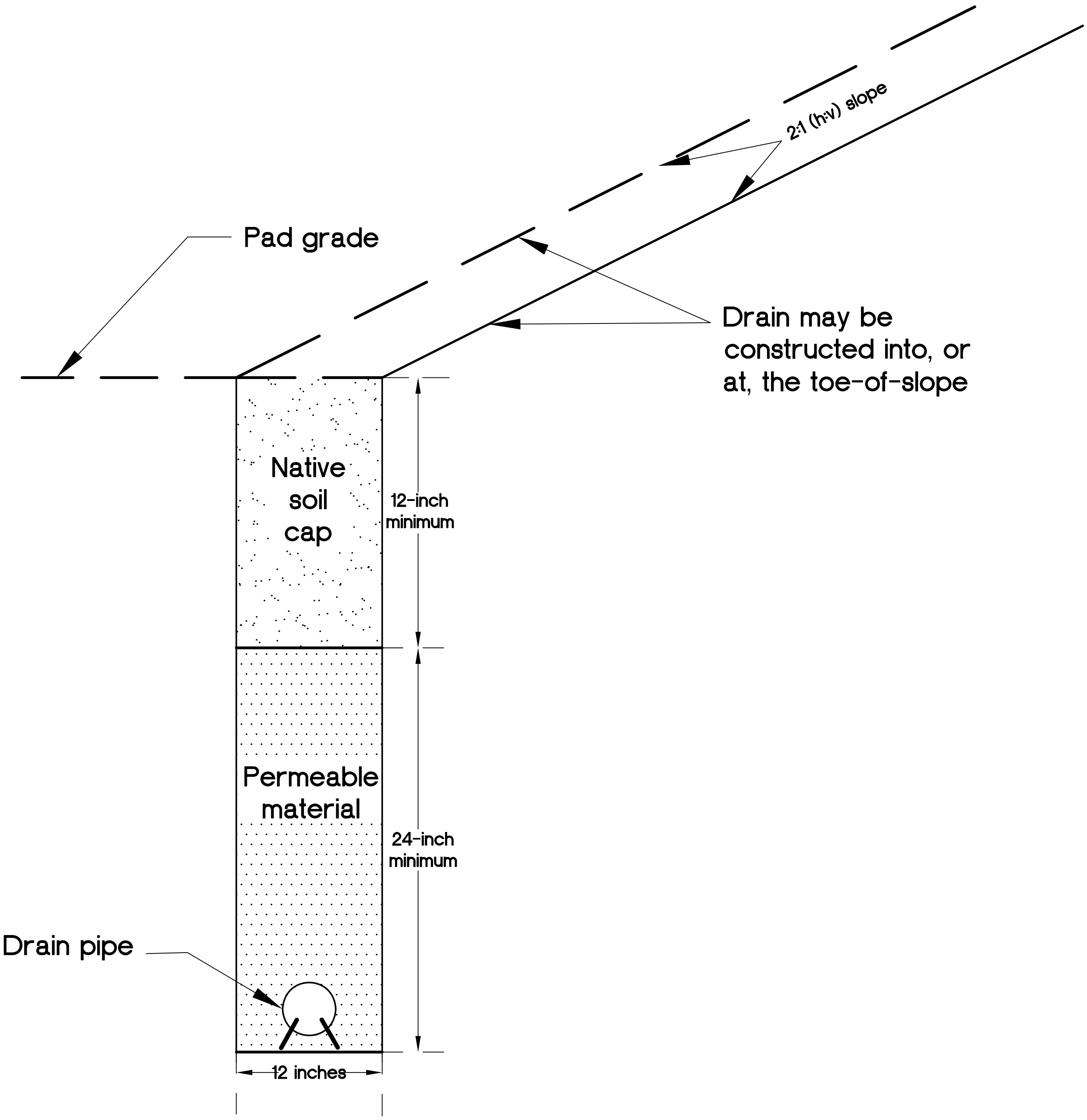
- Was the lot at the base of the slope overexcavated or is it proposed to be overexcavated? Overexcavated lots located at the base of a slope could accumulate subsurface water along the base of the fill cap.

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

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

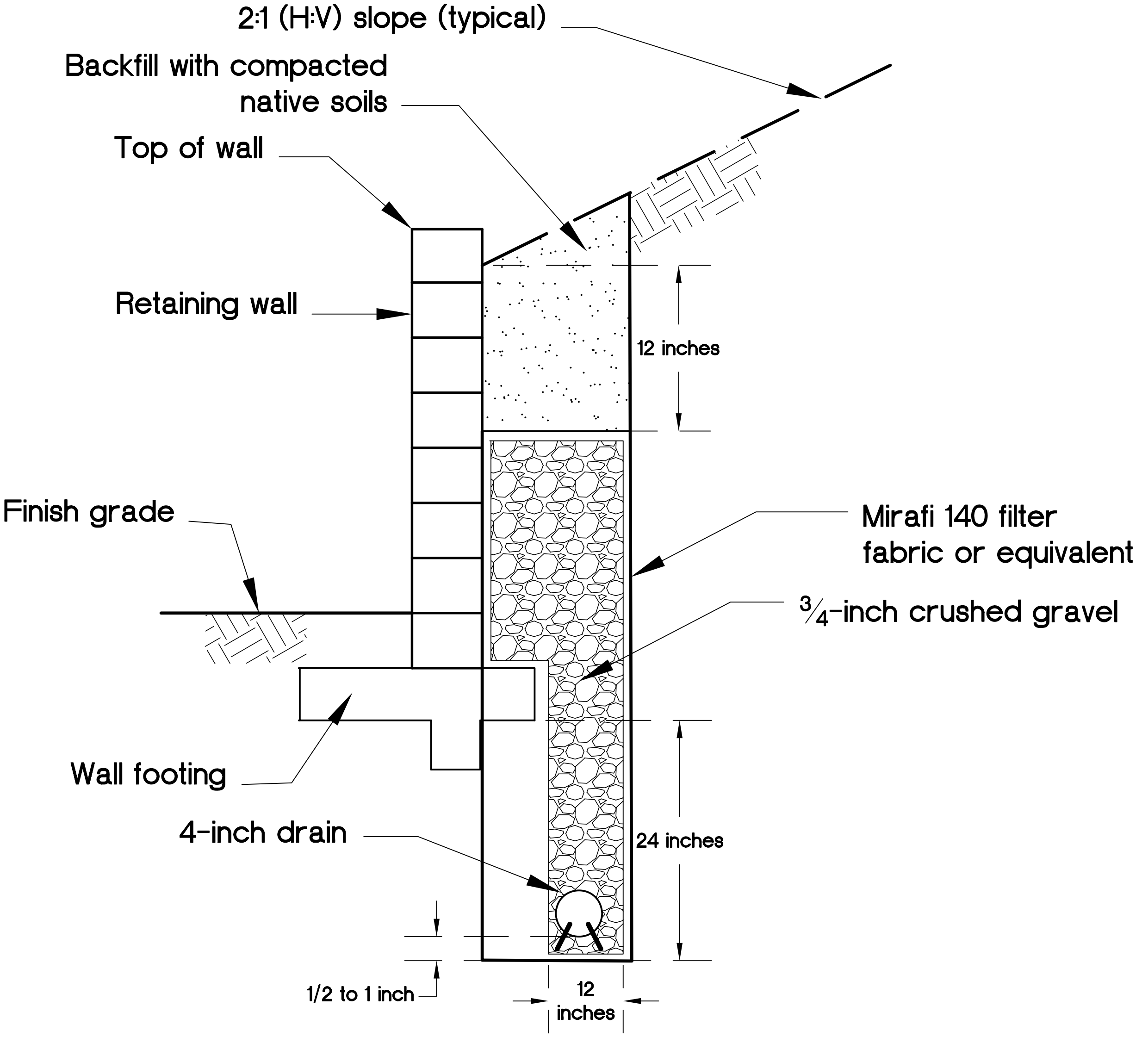
- Do the slopes “toe out” into a residential lot or a lot where perched or ponded water may adversely impact its proposed use?

Based on these general criteria, the construction of toe drains may be considered by the design engineer along the toe of slopes, or at retaining walls in slopes, descending to the rear of such lots. Following are Detail 4 (Schematic Toe Drain Detail) and Detail 5 (Subdrain Along Retaining Wall Detail). Other drains may be warranted due to unforeseen conditions, homeowner irrigation, or other circumstances. Where drains are constructed during grading, including subdrains, the locations/elevations of such drains should be



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

GeoSoils, Inc.	SCHEMATIC TOE DRAIN DETAIL	Detail 4



NOTES:

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

GeoSoils, Inc.	SUBDRAIN ALONG RETAINING WALL DETAIL	Detail 5

surveyed, and recorded on the final as-built grading plans by the design engineer. It is recommended that the above be disclosed to all interested parties, including homeowners and any homeowners association.

Erosion Control

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

Landscape Maintenance

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.

Subsurface and Surface Water

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

Site Improvements

If in the future, any additional improvements (e.g., wall, enclosures, 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. 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.

Additional Grading

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

Footing Trench Excavation

All footing excavations should be observed by a representative of this firm subsequent to trenching and prior to concrete form and reinforcement placement. The purpose of the observations is to evaluate that the excavations have been made into the recommended bearing material and to the minimum widths and depths recommended for construction. If loose or compressible materials are exposed within the footing excavation, a deeper footing or removal and recompaction of the subgrade materials would be recommended at that time. Footing trench spoil and any excess soils generated from utility trench excavations should be compacted to a minimum relative compaction of 90 percent (ASTM D 1557), if not removed from the site.

Trenching/Temporary Construction Backcuts

Considering the nature of the onsite earth materials, it should be anticipated that caving or sloughing could be a factor in subsurface excavations and trenching into onsite granular soils. Shoring or excavating the trench walls/backcuts at a maximum angle of 45 degrees (except as specifically superceded within the text of this report), should be anticipated. All excavations should meet a minimum FOS for temporary slope, backcut, shoring conditions of at least 1.25, and be observed by a geologist or 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 (such as groundwater) exist, appropriate recommendations would be offered at that time. The above recommendations should be provided to any contractors and/or subcontractors, etc., that may perform such work. If water is present or exposed during the trench excavation trench shields, shoring and dewatering should be used to complete excavations. Depending on the height of the groundwater above the trench shoring on trench shield bottom heave of sands may occur.

Utility Trench Backfill

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

2. Exterior trenches adjacent to, and within areas extending below a 1:1 plane projected from the outside bottom edge of the footing, and all trenches beneath hardscape features and in slopes, should be compacted to at least 90 percent of the laboratory standard. Sand backfill, unless excavated from the trench, should not be used in these backfill areas. Compaction testing and observations, along with probing, should be accomplished to evaluate the desired results.

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

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

Monitoring of Structures

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

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

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

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

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

approval by the Building Official. Monitoring should be performed by a licensed surveyor.

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

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

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

SUMMARY OF RECOMMENDATIONS REGARDING GEOTECHNICAL OBSERVATION AND TESTING

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

- During grading/recertification.
- During excavation.
- During the excavation and placement of drilled piers (CIDH piles).
- During placement of subdrains, toe drains, or other subdrainage devices, prior to placing fill and/or backfill.
- After excavation of retaining wall footings/foundations, and free standing walls footings, prior to the placement of reinforcing steel or concrete.
- Prior to pouring any slabs or flatwork, after presoaking/presaturation of building pads and other flatwork subgrade, before the placement of concrete, reinforcing steel, capillary break (i.e., sand, pea-gravel, etc.), or vapor retarders (i.e., visqueen, etc.).
- During retaining wall subdrain installation, prior to backfill placement.
- During placement of backfill for area drain, interior plumbing, utility line trenches, and retaining wall backfill.

- During slope construction/repair.

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

- When any improvements, such as flatwork, walls, etc., are constructed, prior to construction. GSI should review and approve such plans prior to construction.

- A report of geotechnical observation and testing should be provided at the conclusion of each of the above stages, in order to provide concise and 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 any foundation. The structural engineer/foundation and/or slab designer should provide recommendations to not allow water or vapor to enter into the structure so as to cause damage to another building component, or so as to limit the installation of the type of flooring materials typically used for the particular application, as appropriate.

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

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

PLAN REVIEW

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

LIMITATIONS

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

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

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

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APPENDIX B

TEST PIT AND CPT 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.

PLATE B-1

GeoSoils, Inc.

W.O. 6688-A-SC
Vessels Stallion Ranch
Vessels Stallion Ranch, Bonsall
Logged By: RGC
March 7, 2014, March 26, 2014

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-1	±232' MSL	0-2	SM	1-2			**COLLUVIUM:** SILTY SAND, very dark brown, dry, loose.
		2-3½	SM/SC	2-3½			SILTY SAND with some CLAY, brown, moist, loose; porous.
		3½-5	SM	3½-4			**TERRACE DEPOSITS:** SILTY SAND, brown, damp, medium dense; slightly porous, weakly cemented.
							Total Depth = 5' No Groundwater/Caving Encountered Backfilled 3-7-2014
TP-2	±225' MSL	0-1½	SW				**ALLUVIUM:** SAND, light brown, damp, loose; few roots in upper 2".
		1½-3½	SM	Ring @ 3	108.7	5.5	SILTY SAND, very dark brown, moist, loose to medium dense.
		3½-17	SM				SILTY SAND, dark brown, moist, loose.
							Total Depth = 17' No Groundwater/Caving Encountered Backfilled 3-26-2014

GeoSoils, Inc.

W.O. 6688-A-SC
Vessels Stallion Ranch
Vessels Stallion Ranch, Bonsall
Logged By: RGC
March 7, 2014, March 26, 2014

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-3	±292' MSL	0-½	SM				**COLLUVIUM:** SILTY SAND, grayish brown, damp, loose; many roots, porous.
		½-4	SM	2	103.1	5.7	SILTY SAND, brown, dry, loose; very porous (pores to 1/8").
		4-7	SM				SILTY SAND, brown, damp, loose; few pores.
		7-8	SM				**BEDROCK:** GRANITIC ROCK breaking to SILTY SAND upon excavation, olive brown to dark brown, damp, medium dense to dense @ 8'.
							Total Depth = 8' No Groundwater/Caving Encountered Backfilled 3-26-2014
TP-4	±315' MSL	0-1	SM				**COLLUVIUM:** SILTY SAND, grayish brown, dry, loose; few roots, porous.
		1-2	SW				**HIGHLY WEATHERED BEDROCK:** GRANITIC ROCK breaking to SAND upon excavation, brown, damp, loose.
		2-5	SW				**BEDROCK:** GRANITIC ROCK breaking to SAND, damp/dry, medium dense becoming dense at 4'; joint sets: N40°W, 65°NE.
							Total Depth = 5' (Practical Refusal) No Groundwater/Caving Encountered Backfilled 3-26-2014

GeoSoils, Inc.

W.O. 6688-A-SC
Vessels Stallion Ranch
Vessels Stallion Ranch, Bonsall
Logged By: RGC
March 7, 2014, March 26, 2014

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-5	±230' MSL	0-1	SM				**COLLUVIUM:** SILTY SAND, brown, dry, loose; few roots, porous.
		1-2	SM				SILTY SAND, dark brown, damp, loose; very porous.
		2-3	SM	2	102.4	5.1	SILTY SAND, brown, dry, loose to medium dense; slightly porous, disseminated, carbonates, weakly cemented.
		3-5	SM				**TERRACE DEPOSITS:** SILTY SAND, light brown, to yellowish brown, damp to dry, medium dense; slightly porous, disseminated carbonates, moderately cemented.
		5-6	SM				As per 3', no visible pores, to few pinhole pores.
							Total Depth = 6' No Groundwater/Caving Encountered Backfilled 3-26-2014
TP-6	±225' MSL	0-2	SM				**COLLUVIUM:** SILTY SAND, brown, damp, loose; few roots.
		2-14	SP	Ring 3'	93.4	4.6	**ALLUVIUM:** SAND, brownish gray, damp, loose; fine grained.
		13½-14					**BEDROCK:** GRANITIC ROCK breaking to SILTY SAND and brittle gravel to cobble-size rock fragments upon excavation, yellowish brown-brownish yellow, moist, dense.
							Total Depth = 14' No Groundwater/Caving Encountered Backfilled 3-26-2014

GeoSoils, Inc.

W.O. 6688-A-SC
Vessels Stallion Ranch
Vessels Stallion Ranch, Bonsall
Logged By: RGC
March 7, 2014, March 26, 2014

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-7	±700' MSL	0-1½	SM				**COLLUVIUM:** SILTY SAND, dark brown, damp, loose; few roots.
		1½-7	SW				**BEDROCK:** GRANITIC ROCK breaking to SAND and brittle ground to cobble-size rock fragments upon excavation, yellowish brown to light grayish brown, dry, dense; practical refusal at 7'.
							Total Depth = 7' (practical refusal) No Groundwater/Caving Encountered Backfilled 3-26-2014
TP-8	±650' MSL	0-2	SM				**COLLUVIUM:** SILTY SAND with angular cobble to small boulder-size rock fragment, dark brown, damp, loose; porous, few roots in upper; 6".
		2-6	SW				**BEDROCK:** GRANITIC ROCK breaking to SAND and brittle cobble-size rock fragments upon excavation, yellowish brown, dry, dense; practical refusal at 6' on hard rock.
							Total Depth = 6' (Practical Refusal) No Groundwater/Caving Encountered Backfilled 3-26-2014

GeoSoils, Inc.

W.O. 6688-A-SC
Vessels Stallion Ranch
Vessels Stallion Ranch, Bonsall
Logged By: RGC
March 7, 2014, March 26, 2014

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-9	±260' MSL	0-3	SM				**COLLUVIUM:** SILTY SAND, brown, damp, loose; porous, some angular rock fragments.
		3-4	SM				**HIGHLY WEATHERED BEDROCK:** GRANITIC ROCK breaking to SILTY SAND, yellowish brown, damp, loose to medium dense; highly weathered, relict bedrock structure.
		4-12					**BEDROCK:** GRANITIC ROCK breaking to SILTY SAND upon excavation, yellowish brown to brownish yellow, damp, medium dense; fractured and brittle gravel to cobble-size rock fragments.
							Total Depth = 12' No Groundwater/Caving Encountered Backfilled 3-26-2014
TP-10	±225' MSL	0-1	SW				**FILL:** SAND, gray brown, dry, loose.
		1-2	SM				**COLLUVIUM:** SILTY SAND, dark brown, damp, loose; porous.
		2-3½	SC				CLAYEY SAND to SAND with CLAY, brown, damp, loose; porous, blocky.
		3½-4	SM				**TERRACE DEPOSITS:** SILTY SAND, brown, damp, medium dense.
							Total Depth = 4' No Groundwater/Caving Encountered Backfilled 3-7-2014

GeoSoils, Inc.

W.O. 6688-A-SC
Vessels Stallion Ranch
Vessels Stallion Ranch, Bonsall
Logged By: RGC
March 7, 2014, March 26, 2014

LOG OF EXPLORATORY TEST PITS

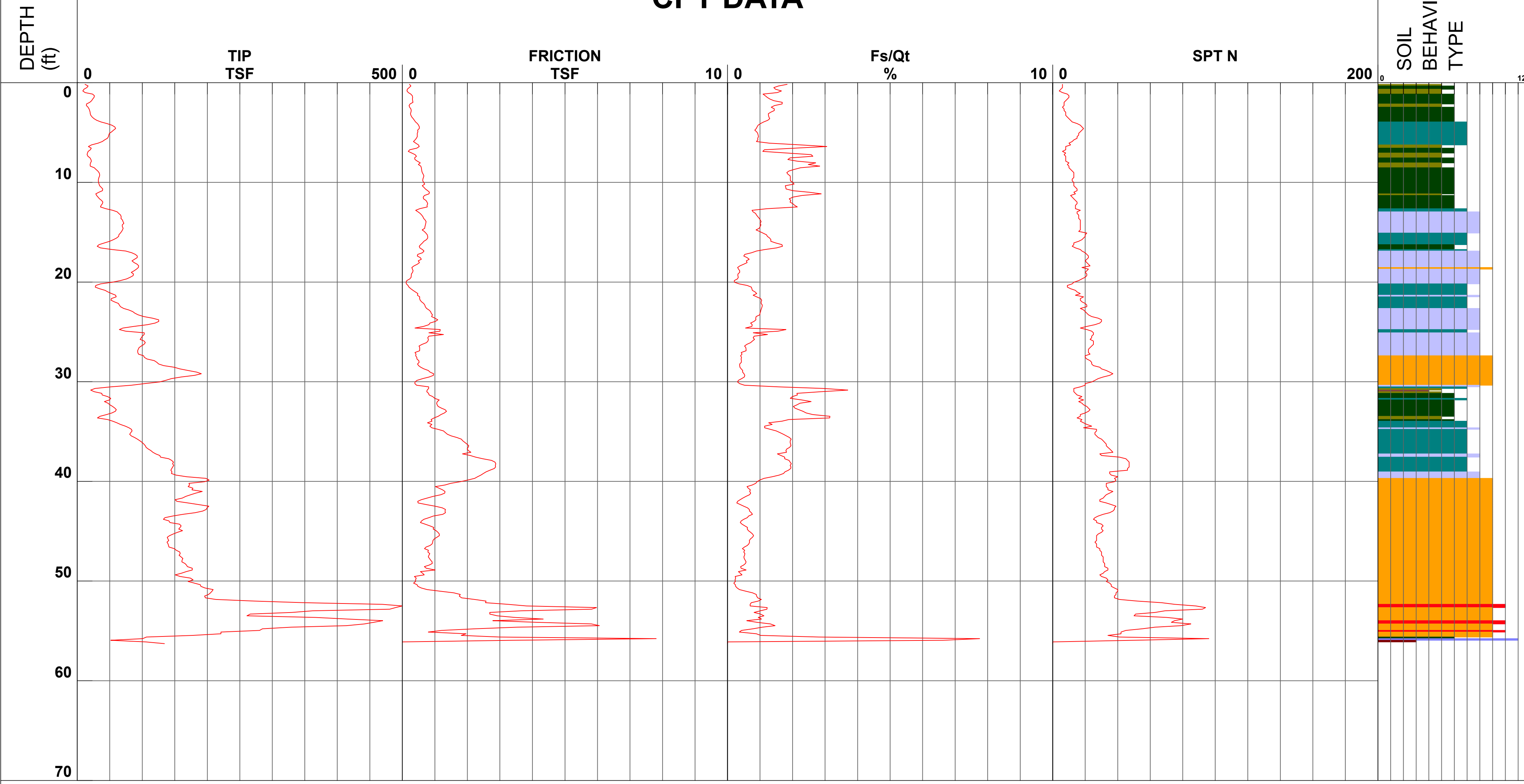
TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-11	±265' MSL	0-2					**COLLUVIUM:** SILTY SAND, very dark brown, damp, loose.
		2-3					SILTY SAND with CLAY, brown, damp, loose; porous.
		3-5					**TERRACE DEPOSITS:** SILTY SAND, brown, damp, medium dense.
		5-6					**BEDROCK:** GRANITIC ROCK breaking to SILTY SAND and SAND with trace CLAY, olive brown, moist, medium dense.
							Total Depth = 5' No Groundwater/Caving Encountered Backfilled 3-7-2014

Geosoils Inc

Project	Vessels	Operator	BH-MM	Filename	SDF(580).cpt
Job Number	6688-A	Cone Number	DDG1281	GPS	
Hole Number	CPT-01	Date and Time	3/7/2014 8:45:55 AM	Maximum Depth	56.27 ft
EST GW Depth During Test		14.00 ft			

Net Area Ratio .8

CPT DATA



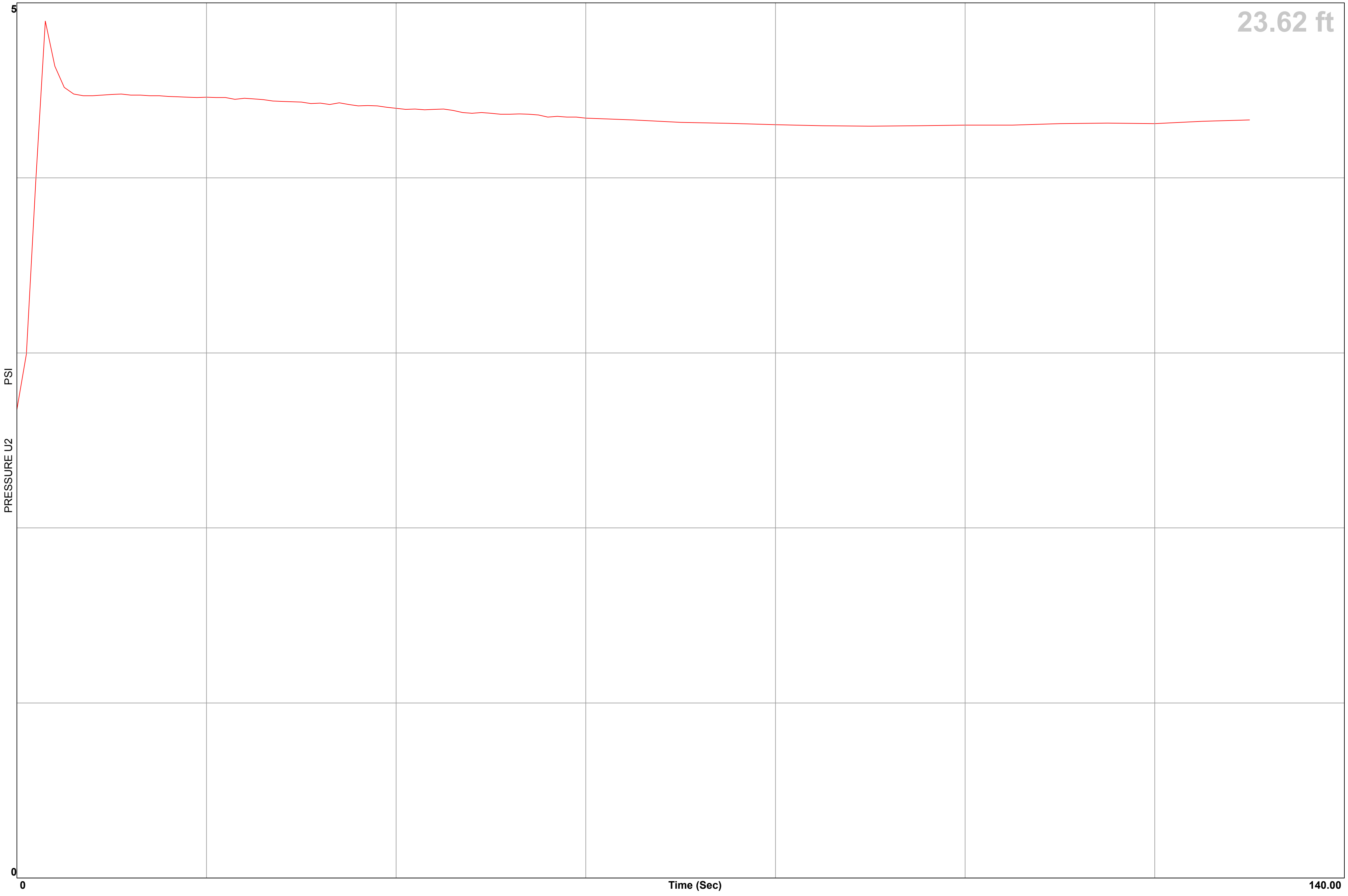
■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

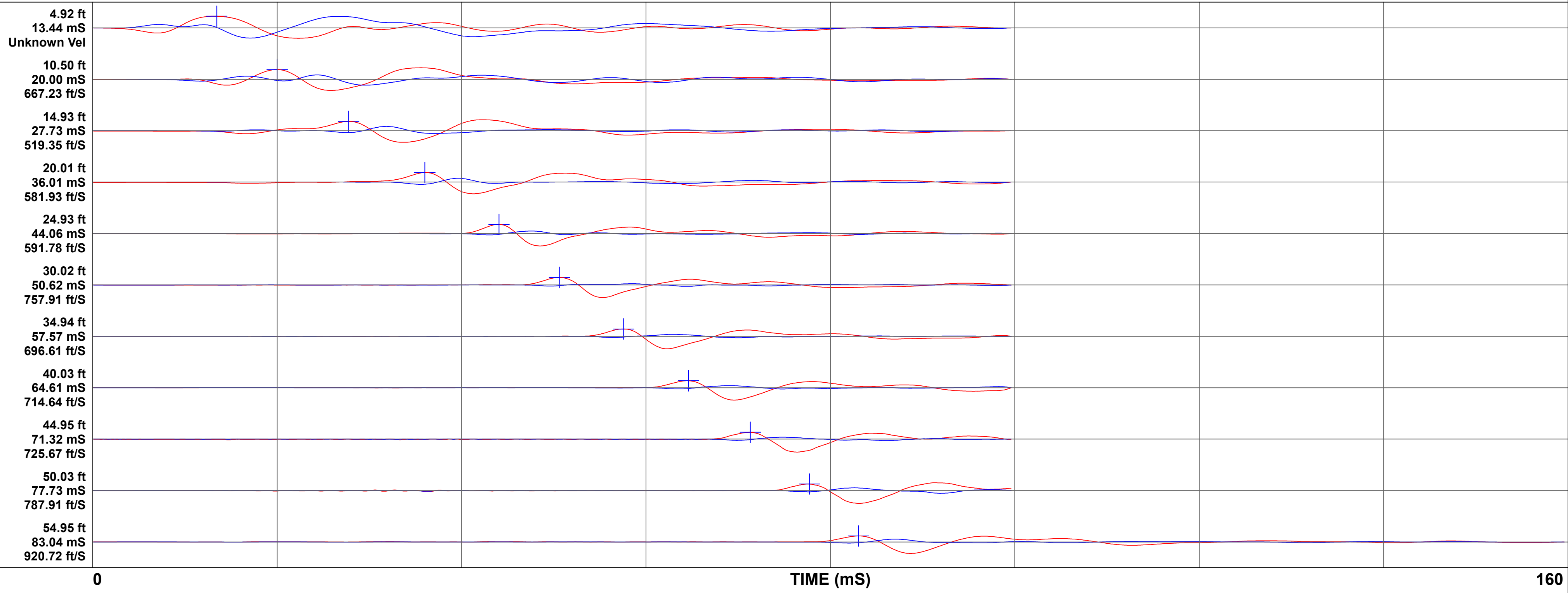
Geosoils Inc

Location	Vessels	Operator	BH-MM		
Job Number	6688-A	Cone Number	DDG1281	GPS	
Hole Number	CPT-01	Date and Time	3/7/2014 8:45:55 AM		
Equilized Pressure	4.3	EST GW Depth During Test	13.6		



Geosoils Inc

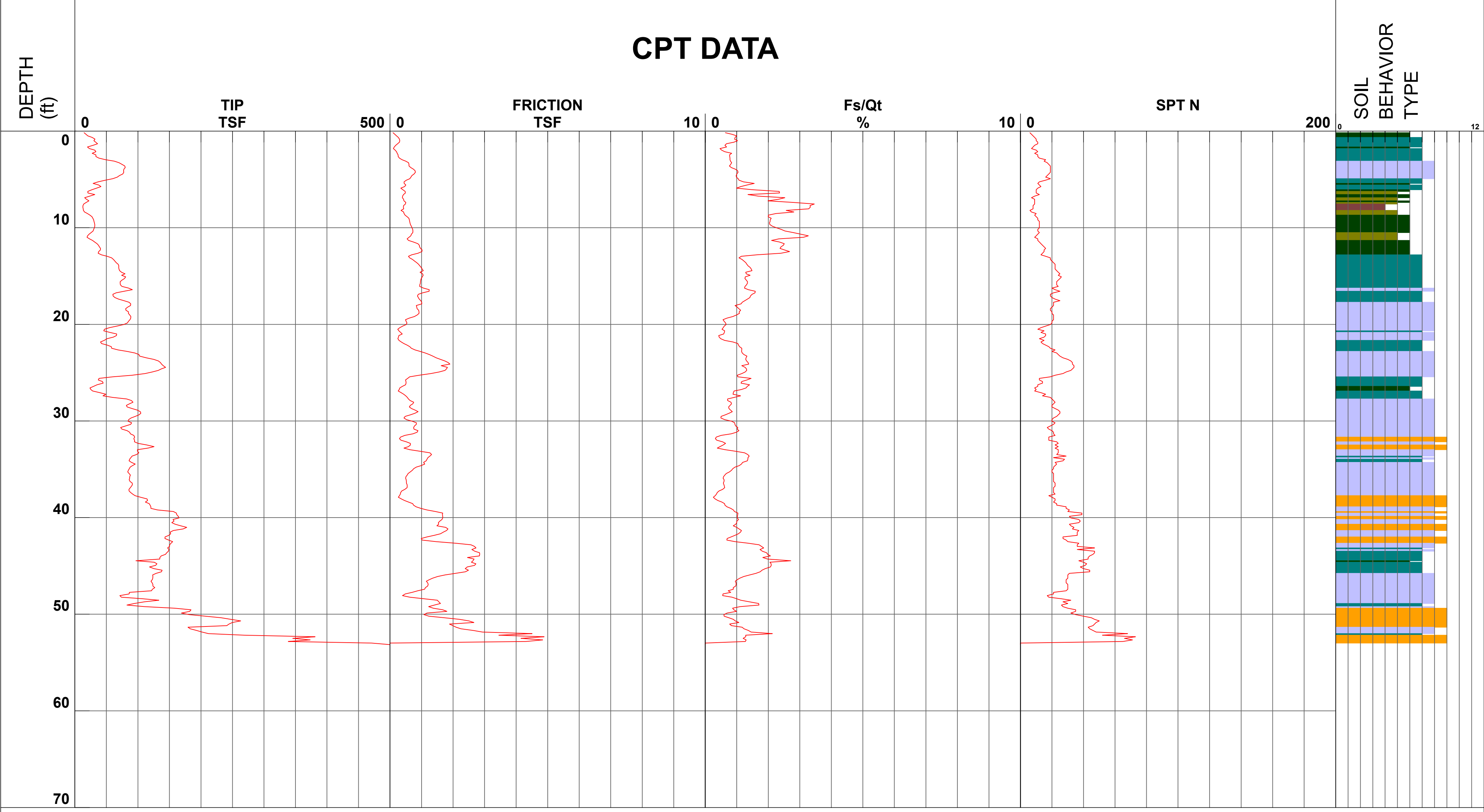
Location Vessels
Job Number 6688-A
Hole Number CPT-01
Operator BH-MM
Cone Number DDG1281
Date and Time 3/7/2014 8:45:55 AM
GPS



Geosoils Inc

Project Vessels
Job Number 6688-A
Hole Number CPT-01A
EST GW Depth During Test 14.00 ft
Operator BH-MM
Cone Number DDG1281
Date and Time 3/7/2014 9:41:55 AM
Filename SDF(581).cpt
GPS
Maximum Depth 53.15 ft

Net Area Ratio .8



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

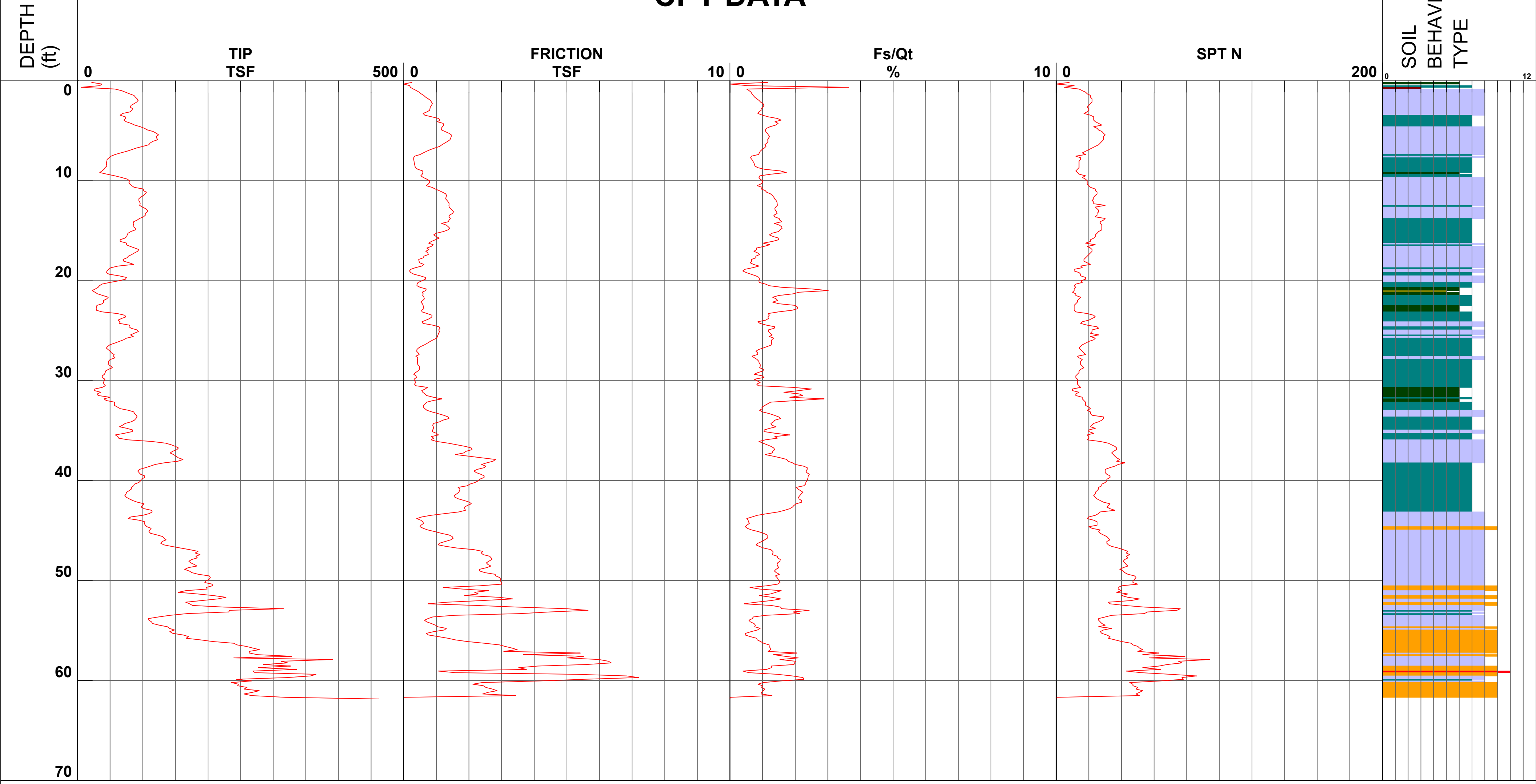
S*Soil behavior type and SPT based on data from UBC-1983

Geosoils Inc

Project Vessels
Job Number 6688-A
Hole Number CPT-02
EST GW Depth During Test 14.00 ft
Operator BH-MM
Cone Number DDG1281
Date and Time 3/7/2014 10:32:26 AM
Filename SDF(582).cpt
GPS
Maximum Depth 61.84 ft

Net Area Ratio .8

CPT DATA



- ■ **1 - sensitive fine grained**
- ■ **2 - organic material**
- ■ **3 - clay**
- ■ **4 - silty clay to clay**
- ■ **5 - clayey silt to silty clay**
- ■ **6 - sandy silt to clayey silt**
- ■ **7 - silty sand to sandy silt**
- ■ **8 - sand to silty sand**
- ■ **9 - sand**
- ■ **10 - gravelly sand to sand**
- ■ **11 - very stiff fine grained (*)**
- ■ **12 - sand to clayey sand (*)**

Cone Size 10cm squared

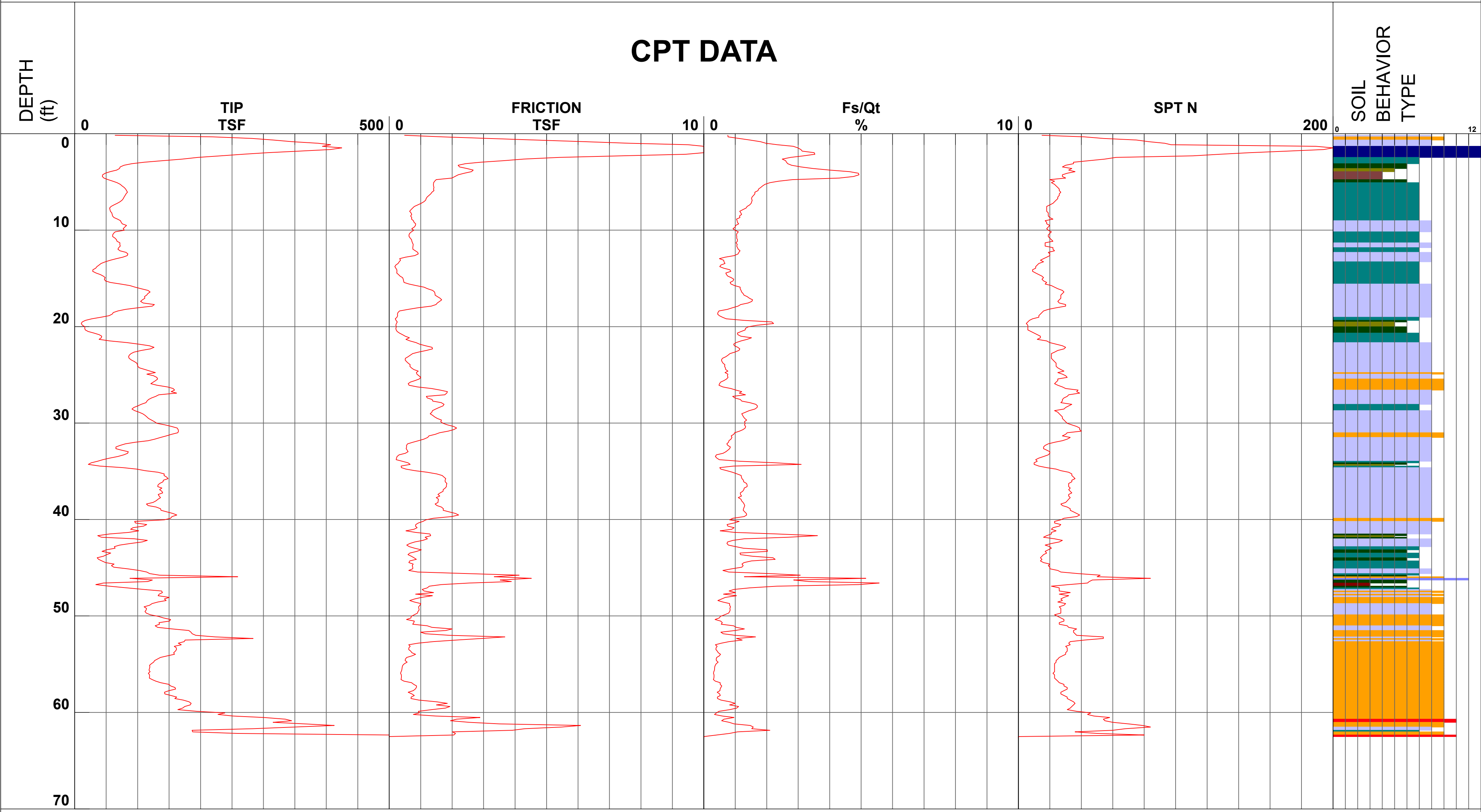
S*Soil behavior type and SPT based on data from UBC-1983

Geosoils Inc

Project: Vessels
Job Number: 6688-A
Hole Number: CPT-03
EST GW Depth During Test: 15.00 ft
Operator: BH-MM
Cone Number: DDG1281
Date and Time: 3/7/2014 11:28:59 AM
Filename: SDF(583).cpt
GPS:
Maximum Depth: 62.66 ft

Net Area Ratio .8

CPT DATA



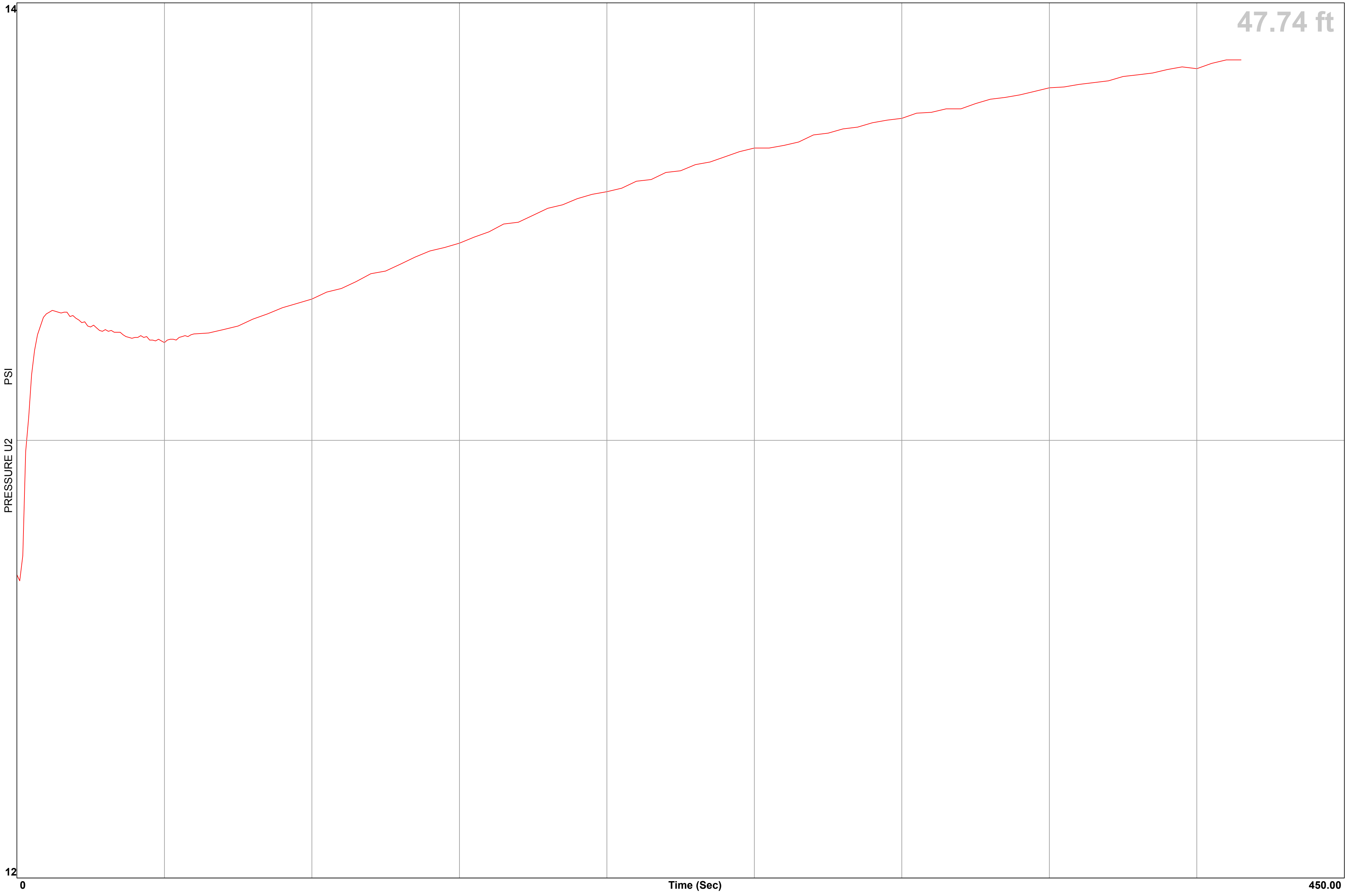
🟫 1 - sensitive fine grained
🟫 2 - organic material
🟥 3 - clay
🟫 4 - silty clay to clay
🟫 5 - clayey silt to silty clay
🟩 6 - sandy silt to clayey silt
🟦 7 - silty sand to sandy silt
🟪 8 - sand to silty sand
🟧 9 - sand
🟥 10 - gravelly sand to sand
🟦 11 - very stiff fine grained (*)
🟦 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

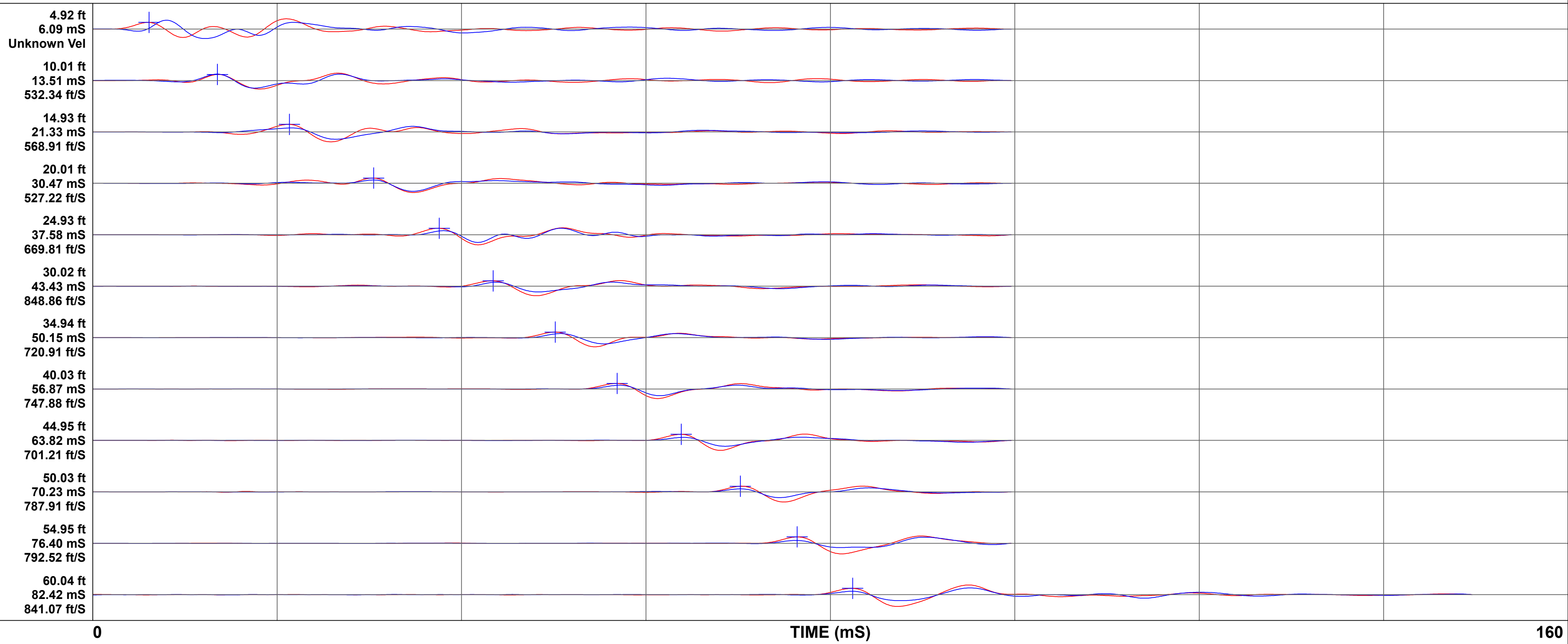
Geosoils Inc

Location	Vessels	**Operator**	BH-MM		
Job Number	6688-A	**Cone Number**	DDG1281	**GPS**	
Hole Number	CPT-03	**Date and Time**	3/7/2014 11:28:59 AM		
Equilized Pressure	13.8	**EST GW Depth During Test**	15.7		



Geosoils Inc

Location Vessels
Job Number 6688-A
Hole Number CPT-03
Operator BH-MM
Cone Number DDG1281
Date and Time 3/7/2014 11:28:59 AM
GPS

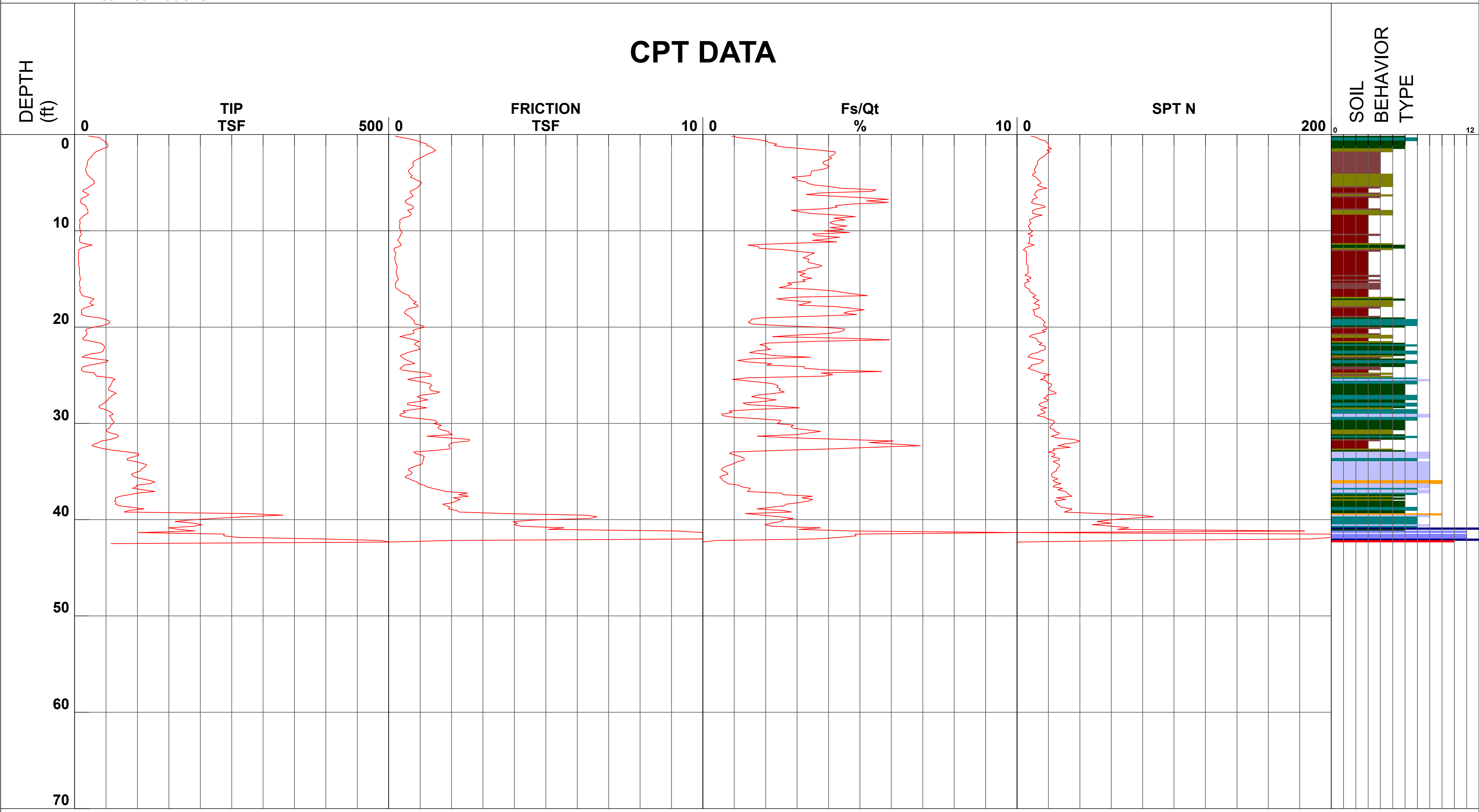


Geosoils Inc

Project: Vessels
Job Number: 6688-A
Hole Number: CPT-04
EST GW Depth During Test: 15.00 ft
Operator: BH-MM
Cone Number: DDG1281
Date and Time: 3/7/2014 12:54:37 PM
Filename: SDF(584).cpt
GPS:
Maximum Depth: 42.49 ft

Net Area Ratio .8

CPT DATA



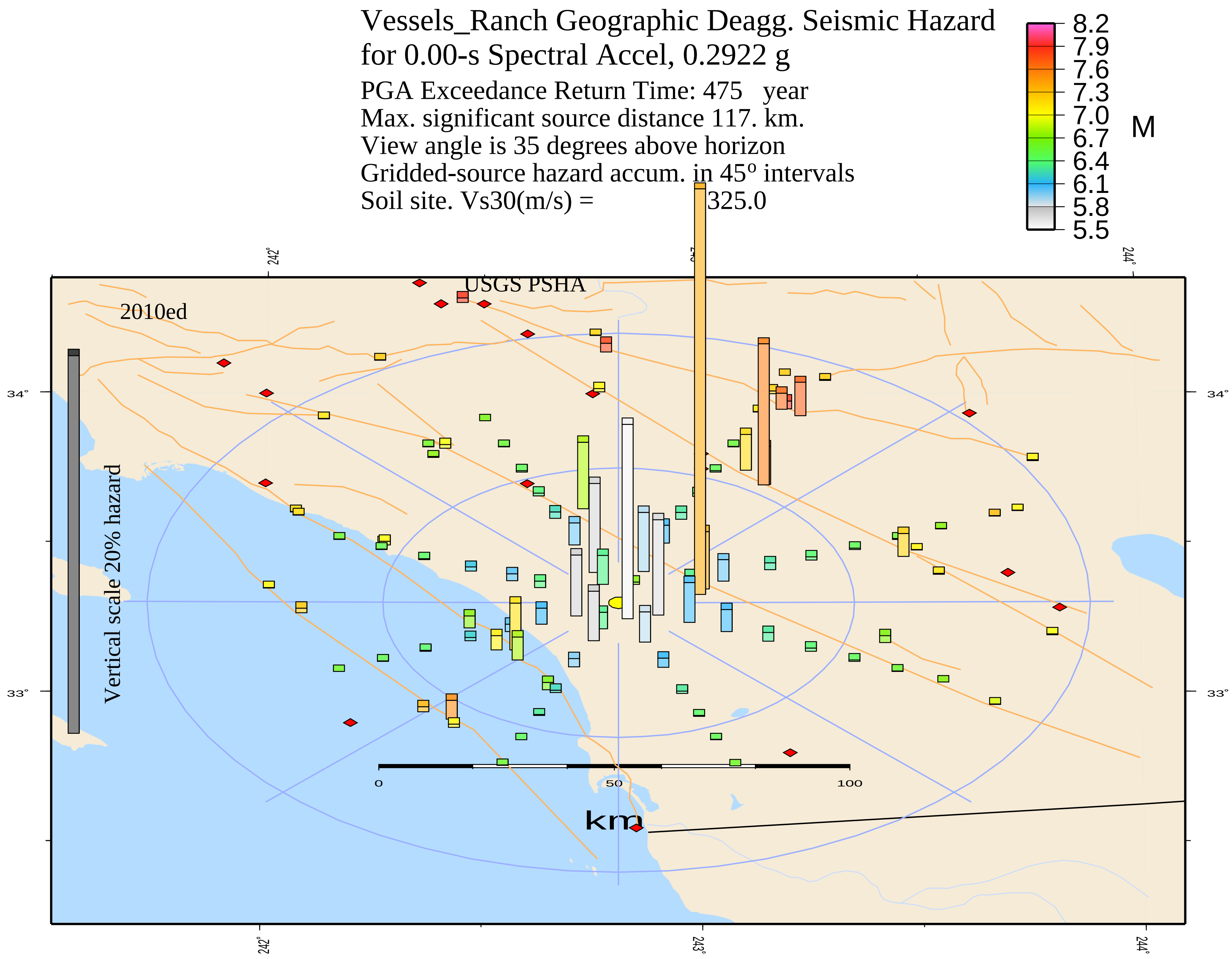
■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

APPENDIX C

SEISMICITY ANALYSIS



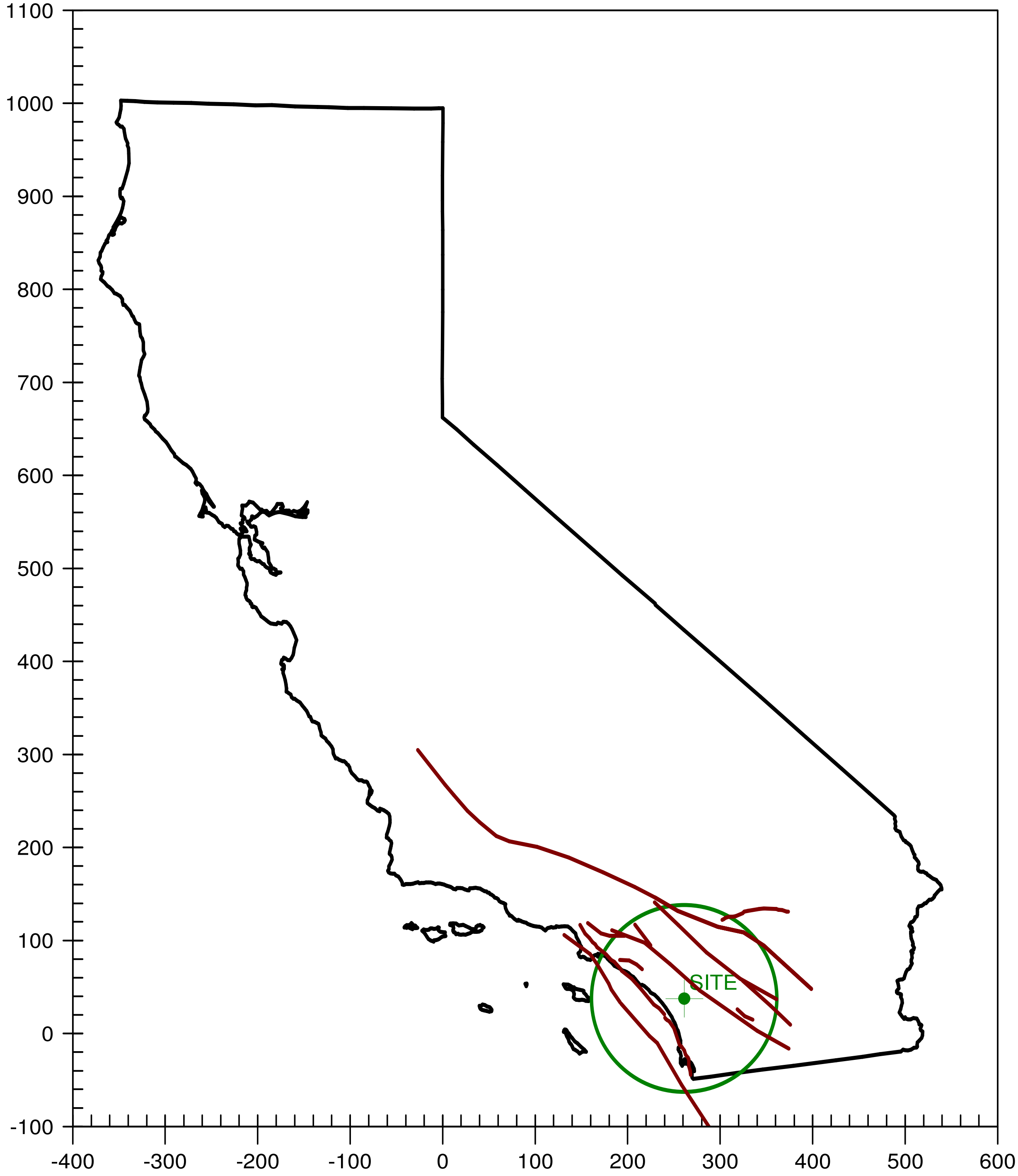
Vessels_Ranch Geographic Deagg. Seismic Hazard
for 0.00-s Spectral Accel, 0.2922 g
PGA Exceedance Return Time: 475 year
Max. significant source distance 117. km.
View angle is 35 degrees above horizon
Gridded-source hazard accum. in 45° intervals
Soil site. Vs30(m/s) = 325.0

W.O. 6688-A-SC
PLATE C-1

GMT 2014 Apr 14 19:42:37 **Site Coords:-117.193 33.3027 (yellow disk) Vs30= 325.0. Max annual ExcdRate .4527E-03 (column height prop. to ExRate). Red diamonds: historical earthquakes, M>6**

CALIFORNIA FAULT MAP

Vessels non-rock areas



W.O. 6688-A-SC
PLATE C-2

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

                 DETERMINISTIC ESTIMATION OF
           PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 6688
                                                DATE: 01-27-2015

JOB NAME: Vessels  non rock

CALCULATION NAME: Test Run Analysis

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


SITE COORDINATES:
   SITE LATITUDE:  33.3027
   SITE LONGITUDE:  117.1933

SEARCH RADIUS:   62.4  mi

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

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


MINIMUM DEPTH VALUE (km):  3.0
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                      EQFAULT SUMMARY
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                  DETERMINISTIC SITE PARAMETERS
                  -----------------------------

Page  1
-------------------------------------------------------------------------------
                                |              |ESTIMATED MAX. EARTHQUAKE EVENT
                                | APPROXIMATE  |-------------------------------
          ABBREVIATED           |   DISTANCE   | MAXIMUM  |   PEAK   |EST. SITE
          FAULT  NAME           |   mi   (km)  |EARTHQUAKE|   SITE   |INTENSITY
                                |              | MAG.(Mw) | ACCEL. g |MOD.MERC.
================================|==============|==========|==========|=========
ELSINORE (TEMECULA)             |  11.1(  17.8)|   6.8    |   0.301  |   IX
ELSINORE (JULIAN)               |  11.7(  18.9)|   7.1    |   0.340  |   IX
NEWPORT-INGLEWOOD (Offshore)    |  17.1(  27.5)|   7.1    |   0.241  |   IX
ROSE CANYON                     |  18.3(  29.5)|   7.2    |   0.240  |   IX
ELSINORE (GLEN IVY)             |  25.2(  40.6)|   6.8    |   0.134  |  VIII
SAN JACINTO-ANZA                |  33.5(  53.9)|   7.2    |   0.131  |  VIII
CORONADO BANK                   |  34.1(  54.9)|   7.6    |   0.171  |  VIII
SAN JACINTO-SAN JACINTO VALLEY  |  34.2(  55.1)|   6.9    |   0.104  |   VII
SAN JOAQUIN HILLS               |  34.7(  55.8)|   6.6    |   0.119  |   VII
EARTHQUAKE VALLEY               |  36.4(  58.5)|   6.5    |   0.075  |   VII
SAN JACINTO-COYOTE CREEK        |  41.0(  66.0)|   6.6    |   0.071  |   VI
CHINO-CENTRAL AVE. (Elsinore)   |  42.2(  67.9)|   6.7    |   0.103  |   VII
PALOS VERDES                    |  43.2(  69.5)|   7.3    |   0.108  |   VII
WHITTIER                        |  46.2(  74.4)|   6.8    |   0.071  |   VI
NEWPORT-INGLEWOOD (L.A.Basin)   |  47.2(  75.9)|   7.1    |   0.086  |   VII
SAN JACINTO-SAN BERNARDINO      |  49.5(  79.6)|   6.7    |   0.062  |   VI
SAN ANDREAS - Whole M-1a        |  53.1(  85.4)|   8.0    |   0.146  |  VIII
SAN ANDREAS - San Bernardino M-1|  53.1(  85.4)|   7.5    |   0.101  |   VII
SAN ANDREAS - SB-Coach. M-1b-2  |  53.1(  85.4)|   7.7    |   0.117  |   VII
SAN ANDREAS - SB-Coach. M-2b    |  53.1(  85.4)|   7.7    |   0.117  |   VII
ELSINORE (COYOTE MOUNTAIN)      |  53.4(  86.0)|   6.8    |   0.061  |   VI
PUENTE HILLS BLIND THRUST       |  58.2(  93.6)|   7.1    |   0.098  |   VII
SAN JACINTO - BORREGO           |  58.2(  93.6)|   6.6    |   0.049  |   VI
PINTO MOUNTAIN                  |  58.7(  94.5)|   7.2    |   0.073  |   VII
SAN ANDREAS - Coachella M-1c-5  |  59.8(  96.3)|   7.2    |   0.072  |   VI
*******************************************************************************



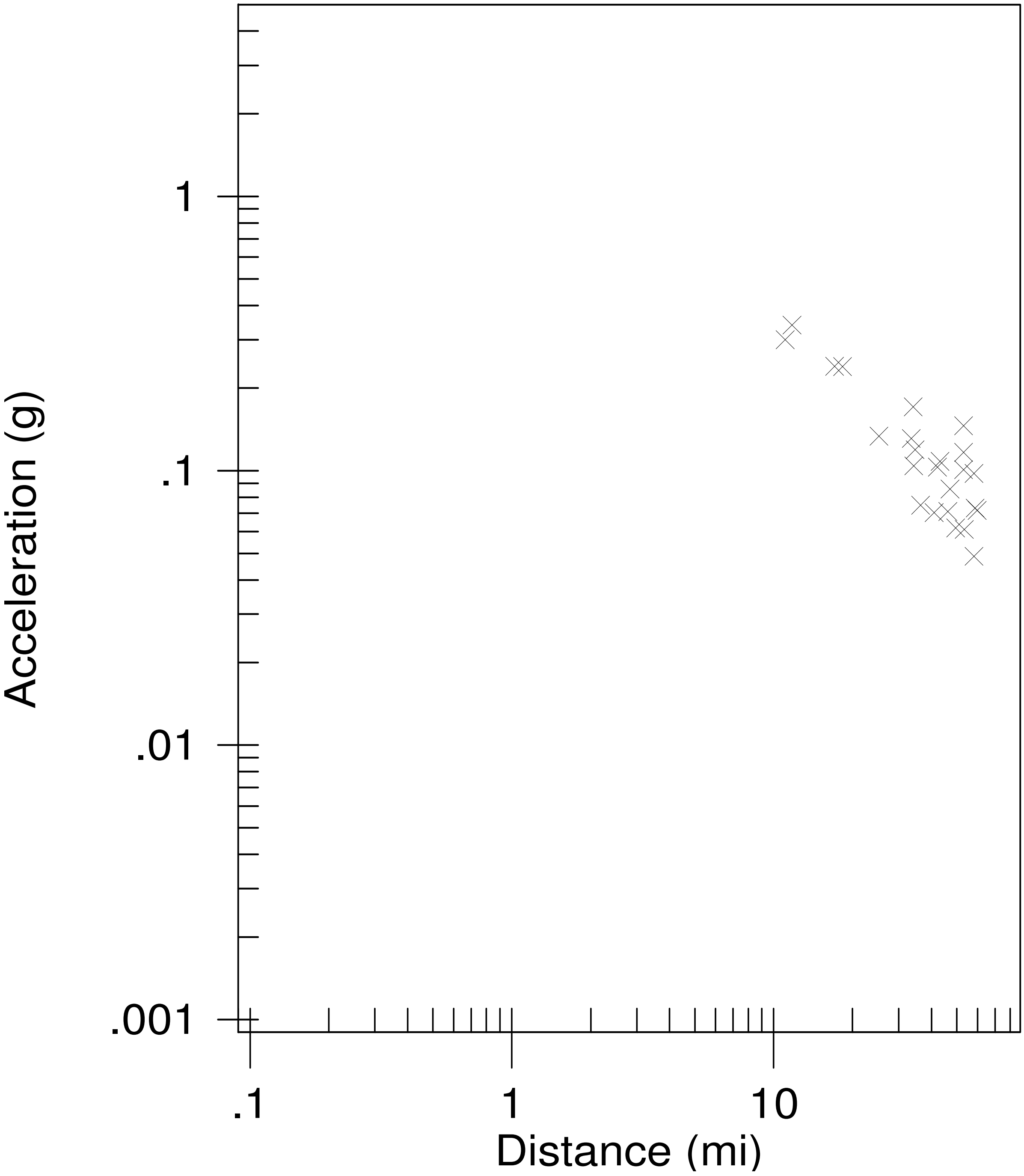
-END OF SEARCH-   25 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ELSINORE (TEMECULA)              FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 11.1 MILES (17.8 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.3400 g
```

MAXIMUM EARTHQUAKES

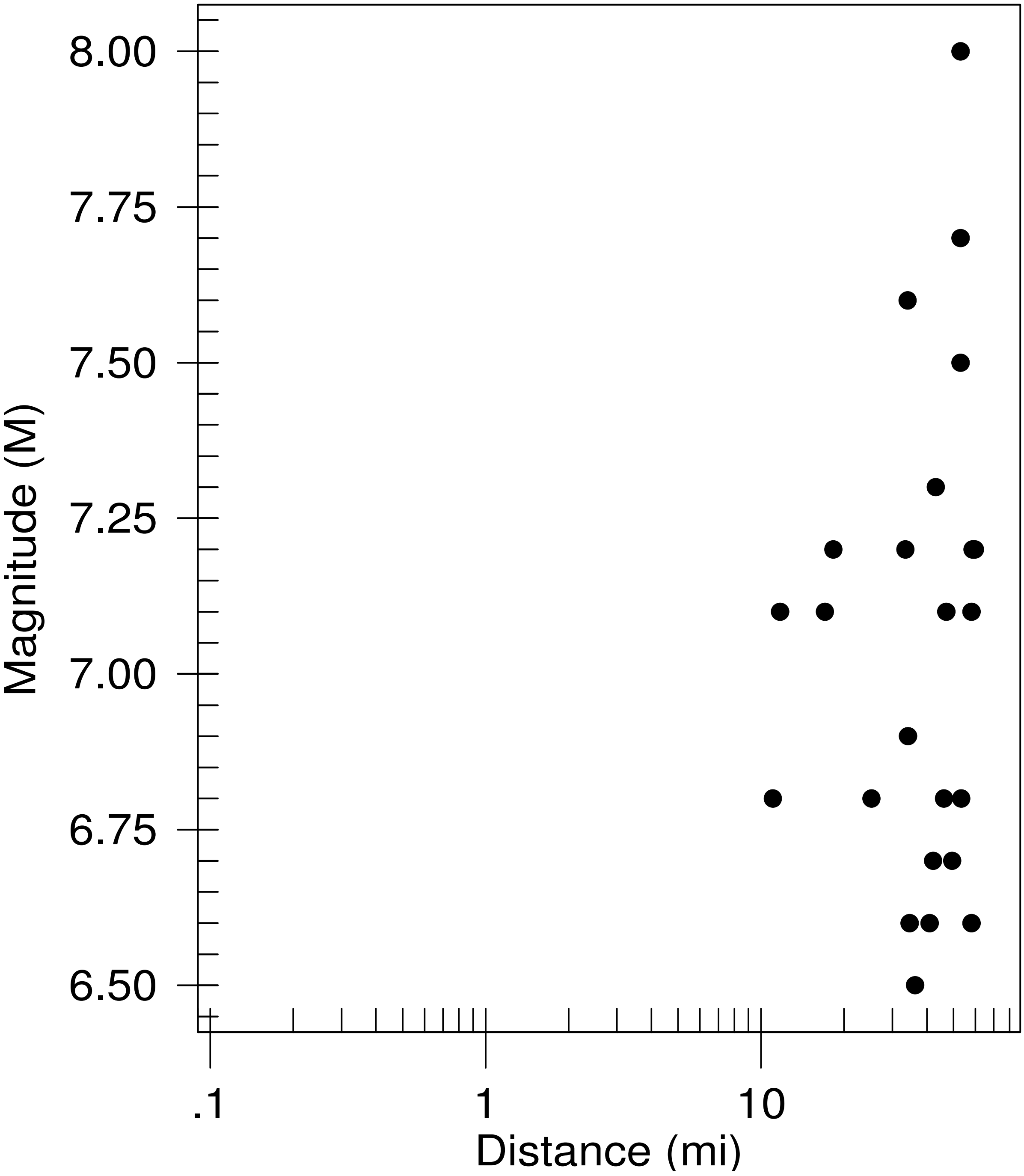
Vessels non-rock areas



W.O. 6688-A-SC
PLATE C-5

EARTHQUAKE MAGNITUDES & DISTANCES

Vessels non-rock areas



W.O. 6688-A-SC
PLATE C-6

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

                    DETERMINISTIC ESTIMATION OF
              PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 6688
                                                      DATE: 01-27-2015

JOB NAME: Vessels  rock

CALCULATION NAME: Test Run Analysis

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


SITE COORDINATES:
   SITE LATITUDE:  33.3027
   SITE LONGITUDE:  117.1933

SEARCH RADIUS:   62.4  mi

ATTENUATION RELATION:  13) Bozorgnia Campbell Niazi (1999) Hor.-Hard Rock-Cor.
   UNCERTAINTY (M=Median, S=Sigma): S       Number of Sigmas:  1.0
   DISTANCE MEASURE:  cdist
   SCOND:   1
   Basement Depth:  .01 km      Campbell SSR:  0     Campbell SHR:  1
   COMPUTE PEAK HORIZONTAL ACCELERATION

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


MINIMUM DEPTH VALUE (km):  3.0
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                       EQFAULT SUMMARY
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                DETERMINISTIC SITE PARAMETERS
                ----------------------------

Page  1
--------------------------------------------------------------------------------
                                |              |ESTIMATED MAX. EARTHQUAKE EVENT
                                | APPROXIMATE  |-------------------------------
          ABBREVIATED           |   DISTANCE   | MAXIMUM  |   PEAK   |EST. SITE
          FAULT  NAME           |   mi   (km)  |EARTHQUAKE|   SITE   |INTENSITY
                                |              | MAG.(Mw) | ACCEL. g |MOD.MERC.
================================|==============|==========|==========|=========
ELSINORE (TEMECULA)             |  11.1(  17.8)|   6.8    |   0.267  |   IX
ELSINORE (JULIAN)               |  11.7(  18.9)|   7.1    |   0.306  |   IX
NEWPORT-INGLEWOOD (Offshore)    |  17.1(  27.5)|   7.1    |   0.210  |  VIII
ROSE CANYON                     |  18.3(  29.5)|   7.2    |   0.210  |  VIII
ELSINORE (GLEN IVY)             |  25.2(  40.6)|   6.8    |   0.114  |  VII
SAN JACINTO-ANZA                |  33.5(  53.9)|   7.2    |   0.112  |  VII
CORONADO BANK                   |  34.1(  54.9)|   7.6    |   0.147  |  VIII
SAN JACINTO-SAN JACINTO VALLEY  |  34.2(  55.1)|   6.9    |   0.089  |  VII
SAN JOAQUIN HILLS               |  34.7(  55.8)|   6.6    |   0.102  |  VII
EARTHQUAKE VALLEY               |  36.4(  58.5)|   6.5    |   0.064  |  VI
SAN JACINTO-COYOTE CREEK        |  41.0(  66.0)|   6.6    |   0.060  |  VI
CHINO-CENTRAL AVE. (Elsinore)   |  42.2(  67.9)|   6.7    |   0.088  |  VII
PALOS VERDES                    |  43.2(  69.5)|   7.3    |   0.092  |  VII
WHITTIER                        |  46.2(  74.4)|   6.8    |   0.060  |  VI
NEWPORT-INGLEWOOD (L.A.Basin)   |  47.2(  75.9)|   7.1    |   0.073  |  VII
SAN JACINTO-SAN BERNARDINO      |  49.5(  79.6)|   6.7    |   0.053  |  VI
SAN ANDREAS - Whole M-1a        |  53.1(  85.4)|   8.0    |   0.125  |  VII
SAN ANDREAS - San Bernardino M-1|  53.1(  85.4)|   7.5    |   0.086  |  VII
SAN ANDREAS - SB-Coach. M-1b-2  |  53.1(  85.4)|   7.7    |   0.100  |  VII
SAN ANDREAS - SB-Coach. M-2b    |  53.1(  85.4)|   7.7    |   0.100  |  VII
ELSINORE (COYOTE MOUNTAIN)      |  53.4(  86.0)|   6.8    |   0.052  |  VI
PUENTE HILLS BLIND THRUST       |  58.2(  93.6)|   7.1    |   0.083  |  VII
SAN JACINTO - BORREGO           |  58.2(  93.6)|   6.6    |   0.041  |   V
PINTO MOUNTAIN                  |  58.7(  94.5)|   7.2    |   0.062  |  VI
SAN ANDREAS - Coachella M-1c-5  |  59.8(  96.3)|   7.2    |   0.061  |  VI
********************************************************************************




-END OF SEARCH-   25 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ELSINORE (TEMECULA)              FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 11.1 MILES (17.8 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.3057 g
```
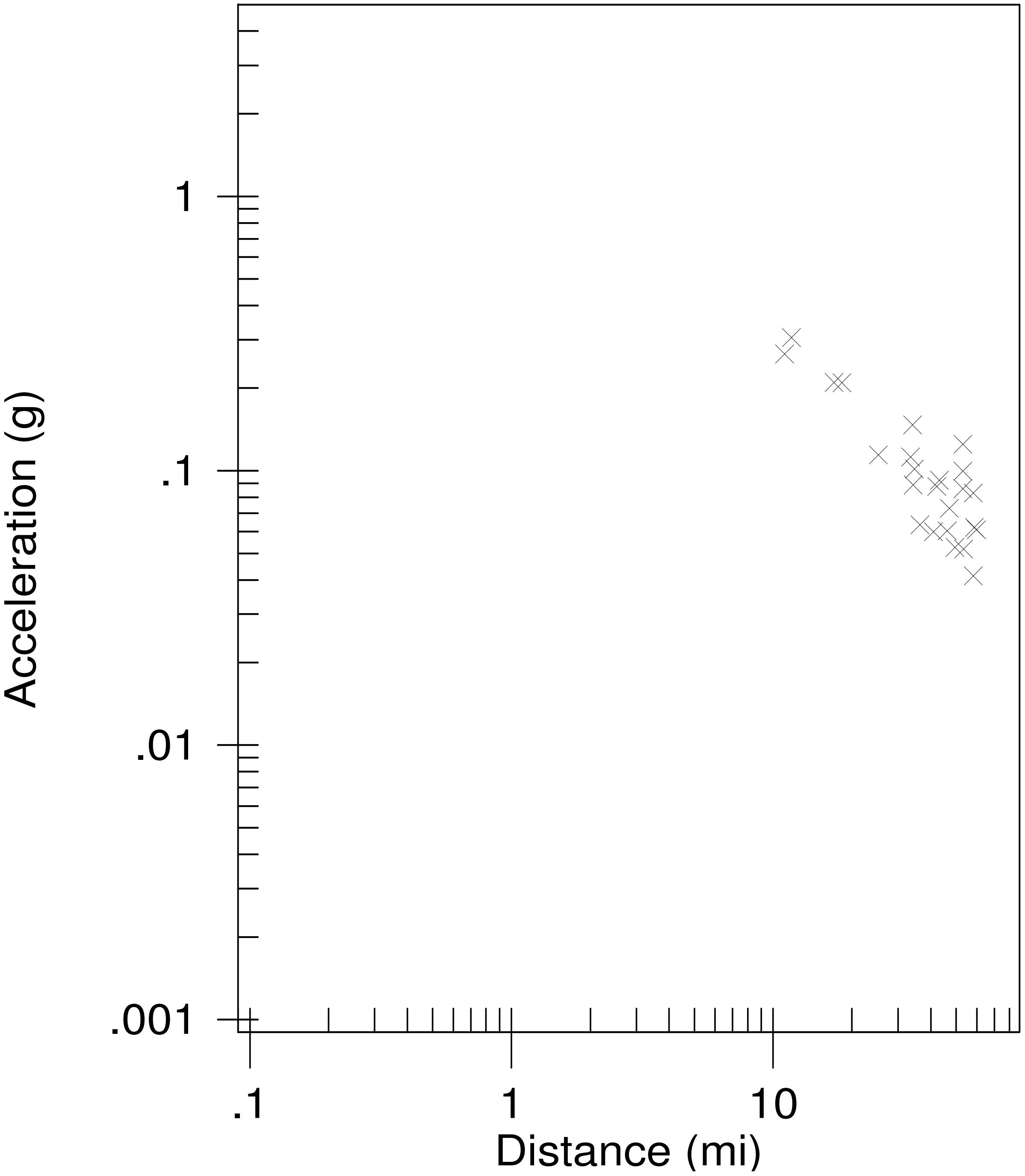

MAXIMUM EARTHQUAKES

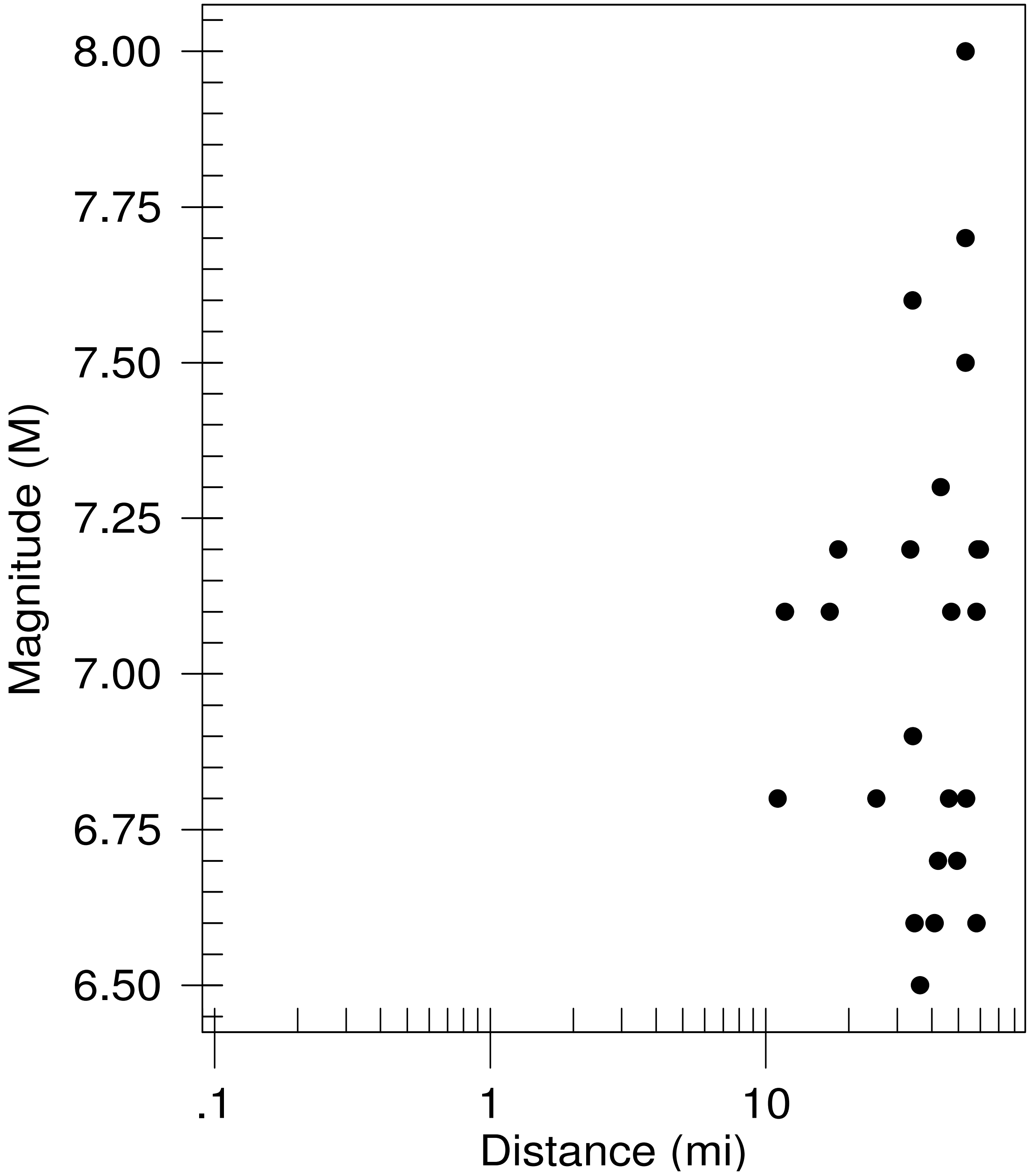
Vessels rock



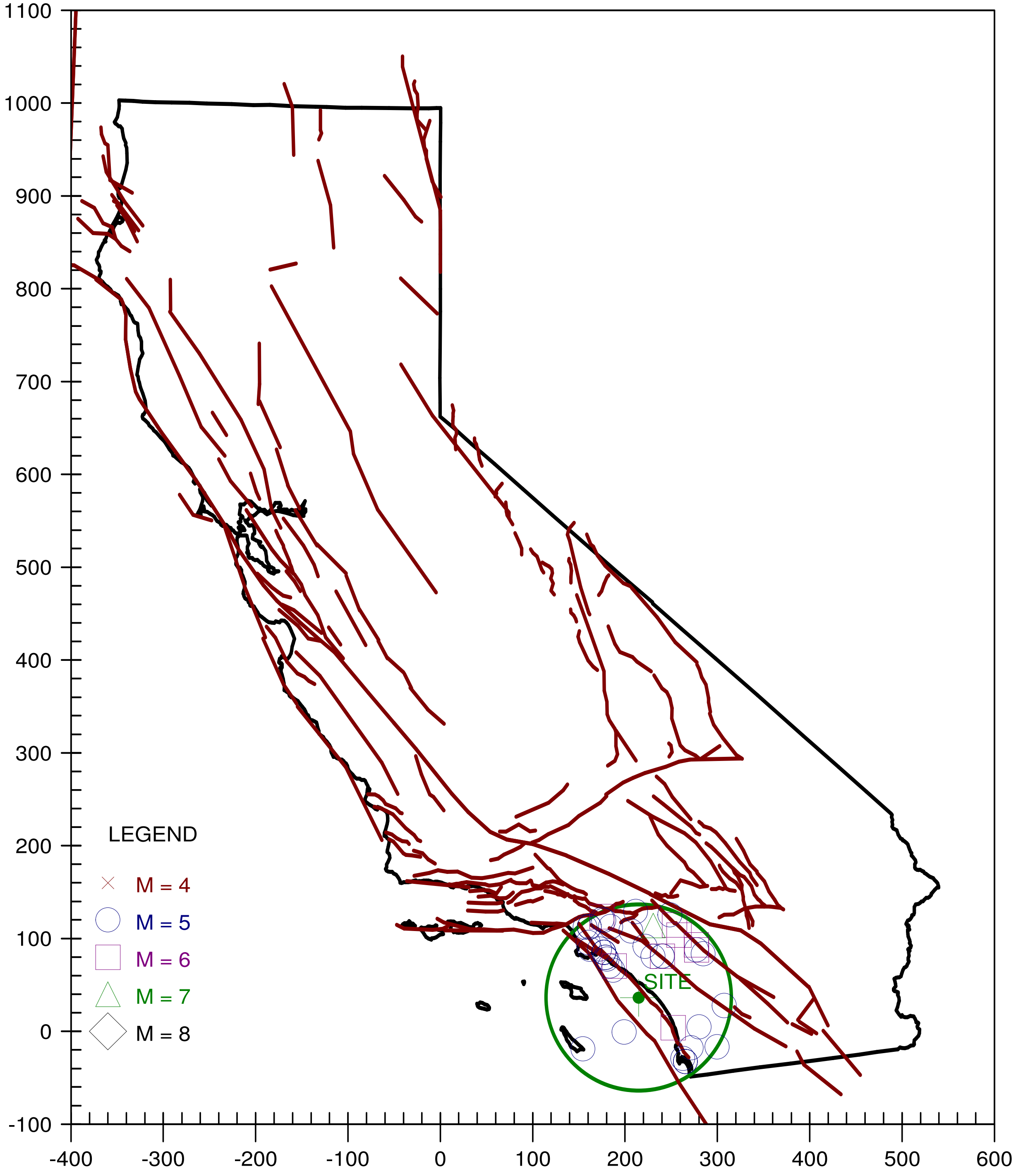
W.O. 6688-A-SC
PLATE C-9

EARTHQUAKE MAGNITUDES & DISTANCES

Vessels rock



W.O. 6688-A-SC
PLATE C-10



EARTHQUAKE EPICENTER MAP

vessels non rock

W.O. 6688-A-SC
PLATE C-11

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                           *                       *
                           *    E Q S E A R C H    *
                           *                       *
                           *     Version 3.00      *
                           *                       *
                           *************************

                                 ESTIMATION OF
                            PEAK ACCELERATION FROM
                        CALIFORNIA EARTHQUAKE CATALOGS


JOB NUMBER: 6688-A
                                                     DATE: 01-27-2015

JOB NAME: vessels non rock

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT


MAGNITUDE RANGE:
   MINIMUM MAGNITUDE:  5.00
   MAXIMUM MAGNITUDE:  9.00

SITE COORDINATES:
   SITE LATITUDE:  33.3027
   SITE LONGITUDE:  117.6917

SEARCH DATES:
           START DATE:   1800
           END DATE:   2014

SEARCH RADIUS:
           62.4 mi
           100.4 km

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

MINIMUM DEPTH VALUE (km):  3.0
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                          EARTHQUAKE SEARCH RESULTS
                          -------------------------

Page  1
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    |       |        |          |  TIME    |     |     | SITE   |SITE|   APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC)   |DEPTH|QUAKE|  ACC.  | MM |   DISTANCE
CODE| NORTH |  WEST  |          | H M Sec  | (km)| MAG.|   g    |INT.|   mi  [km]
----+-------+--------+----------+----------+-----+-----+--------+----+------------
PAS |32.9710|117.8700|07/13/1986|1347 8.2|  6.0| 5.30| 0.052 | VI | 25.1( 40.4)
DMG |33.5750|117.9830|03/11/1933| 518 4.0|  0.0| 5.20| 0.049 | VI | 25.2( 40.6)
DMG |33.6170|117.9670|03/11/1933| 154 7.8|  0.0| 6.30| 0.090 | VII| 26.9( 43.2)
DMG |33.6170|118.0170|03/14/1933|19 150.0|  0.0| 5.10| 0.040 |  V | 28.7( 46.1)
DMG |33.6990|117.5110|05/31/1938| 83455.4| 10.0| 5.50| 0.050 | VI | 29.3( 47.1)
DMG |33.0000|117.3000|11/22/1800|2130 0.0|  0.0| 6.50| 0.089 | VII| 30.8( 49.6)
DMG |33.7000|117.4000|05/15/1910|1547 0.0|  0.0| 6.00| 0.062 | VI | 32.2( 51.8)
DMG |33.7000|117.4000|04/11/1910| 757 0.0|  0.0| 5.00| 0.034 |  V | 32.2( 51.8)
DMG |33.7000|117.4000|05/13/1910| 620 0.0|  0.0| 5.00| 0.034 |  V | 32.2( 51.8)
DMG |33.6830|118.0500|03/11/1933| 658 3.0|  0.0| 5.50| 0.043 | VI | 33.4( 53.7)
MGI |33.8000|117.6000|04/22/1918|2115 0.0|  0.0| 5.00| 0.031 |  V | 34.7( 55.9)
DMG |33.7000|118.0670|03/11/1933| 85457.0|  0.0| 5.10| 0.033 |  V | 34.9( 56.2)
DMG |33.7000|118.0670|03/11/1933| 51022.0|  0.0| 5.10| 0.033 |  V | 34.9( 56.2)
DMG |33.7500|118.0830|03/11/1933| 323 0.0|  0.0| 5.00| 0.028 |  V | 38.2( 61.5)
DMG |33.7500|118.0830|03/11/1933| 2 9 0.0|  0.0| 5.00| 0.028 |  V | 38.2( 61.5)
DMG |33.7500|118.0830|03/11/1933| 230 0.0|  0.0| 5.10| 0.030 |  V | 38.2( 61.5)
DMG |33.7500|118.0830|03/13/1933|131828.0|  0.0| 5.30| 0.033 |  V | 38.2( 61.5)
DMG |33.7500|118.0830|03/11/1933| 910 0.0|  0.0| 5.10| 0.030 |  V | 38.2( 61.5)
DMG |33.7830|118.1330|10/02/1933| 91017.6|  0.0| 5.40| 0.032 |  V | 41.8( 67.2)
GSG |33.9530|117.7610|07/29/2008|184215.7| 14.0| 5.30| 0.028 |  V | 45.1( 72.5)
MGI |33.0000|117.0000|09/21/1856| 730 0.0|  0.0| 5.00| 0.024 | IV | 45.1( 72.6)
DMG |33.7830|118.2500|11/14/1941| 84136.3|  0.0| 5.40| 0.029 |  V | 46.2( 74.3)
MGI |32.8000|117.1000|05/25/1803| 0 0 0.0|  0.0| 5.00| 0.022 | IV | 48.8( 78.5)
MGI |34.0000|117.5000|12/16/1858|10 0 0.0|  0.0| 7.00| 0.076 | VII| 49.4( 79.5)
DMG |33.9000|117.2000|12/19/1880| 0 0 0.0|  0.0| 6.00| 0.039 |  V | 50.0( 80.5)
DMG |33.8500|118.2670|03/11/1933|1425 0.0|  0.0| 5.00| 0.021 | IV | 50.2( 80.8)
DMG |33.7500|117.0000|04/21/1918|223225.0|  0.0| 6.80| 0.065 | VI | 50.4( 81.1)
DMG |33.7500|117.0000|06/06/1918|2232 0.0|  0.0| 5.00| 0.021 | IV | 50.4( 81.1)
DMG |32.7000|117.2000|05/27/1862|20 0 0.0|  0.0| 5.90| 0.036 |  V | 50.4( 81.1)
DMG |32.8170|118.3500|12/26/1951| 04654.0|  0.0| 5.90| 0.036 |  V | 50.7( 81.7)
MGI |34.0000|118.0000|12/25/1903|1745 0.0|  0.0| 5.00| 0.021 | IV | 51.3( 82.6)
DMG |33.7100|116.9250|09/23/1963|144152.6| 16.5| 5.00| 0.020 | IV | 52.3( 84.2)
DMG |33.8000|117.0000|12/25/1899|1225 0.0|  0.0| 6.40| 0.048 | VI | 52.6( 84.6)
T-A |32.6700|117.1700|10/21/1862| 0 0 0.0|  0.0| 5.00| 0.020 | IV | 53.1( 85.5)
T-A |32.6700|117.1700|05/24/1865| 0 0 0.0|  0.0| 5.00| 0.020 | IV | 53.1( 85.5)
T-A |32.6700|117.1700|12/00/1856| 0 0 0.0|  0.0| 5.00| 0.020 | IV | 53.1( 85.5)
DMG |34.0000|117.2500|07/23/1923| 73026.0|  0.0| 6.25| 0.042 |  V | 54.4( 87.6)
PAS |34.0610|118.0790|10/01/1987|144220.0|  9.5| 5.90| 0.032 |  V | 56.9( 91.5)
DMG |33.2000|116.7000|01/01/1920| 235 0.0|  0.0| 5.00| 0.018 | IV | 57.7( 92.8)
GSP |34.1400|117.7000|02/28/1990|234336.6|  5.0| 5.20| 0.020 | IV | 57.8( 93.0)
T-A |34.0000|118.2500|01/10/1856| 0 0 0.0|  0.0| 5.00| 0.018 | IV | 57.9( 93.1)
T-A |34.0000|118.2500|09/23/1827| 0 0 0.0|  0.0| 5.00| 0.018 | IV | 57.9( 93.1)
T-A |34.0000|118.2500|03/26/1860| 0 0 0.0|  0.0| 5.00| 0.018 | IV | 57.9( 93.1)
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PAS |34.0730|118.0980|10/04/1987|105938.2|  8.2| 5.30| 0.022 | IV | 58.1( 93.5)
MGI |34.1000|117.3000|07/15/1905|2041 0.0|  0.0| 5.30| 0.021 | IV | 59.5( 95.7)
MGI |34.0000|118.3000|09/03/1905| 540 0.0|  0.0| 5.30| 0.021 | IV | 59.5( 95.7)
MGI |34.1000|118.1000|07/11/1855| 415 0.0|  0.0| 6.30| 0.039 |  V | 59.8( 96.3)
DMG |32.8000|116.8000|10/23/1894|23 3 0.0|  0.0| 5.70| 0.025 |  V | 62.2(100.1)

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-END OF SEARCH-   48 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH:   1800  TO  2014

LENGTH OF SEARCH TIME:   215  years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 25.1 MILES (40.4 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.090 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
  a-value=  1.062
  b-value=  0.392
  beta-value=  0.902


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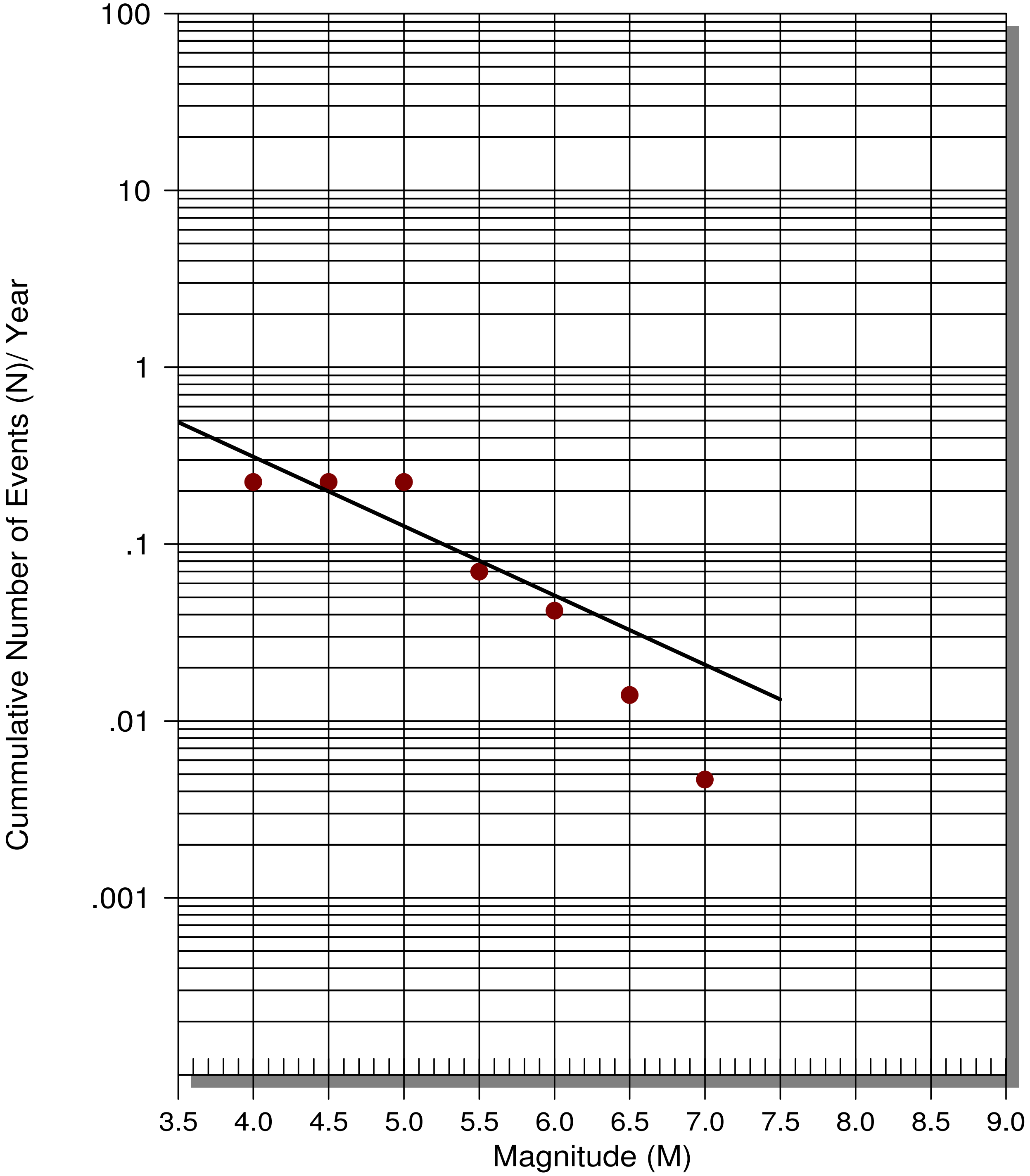
TABLE OF MAGNITUDES AND EXCEEDANCES:
------------------------------------

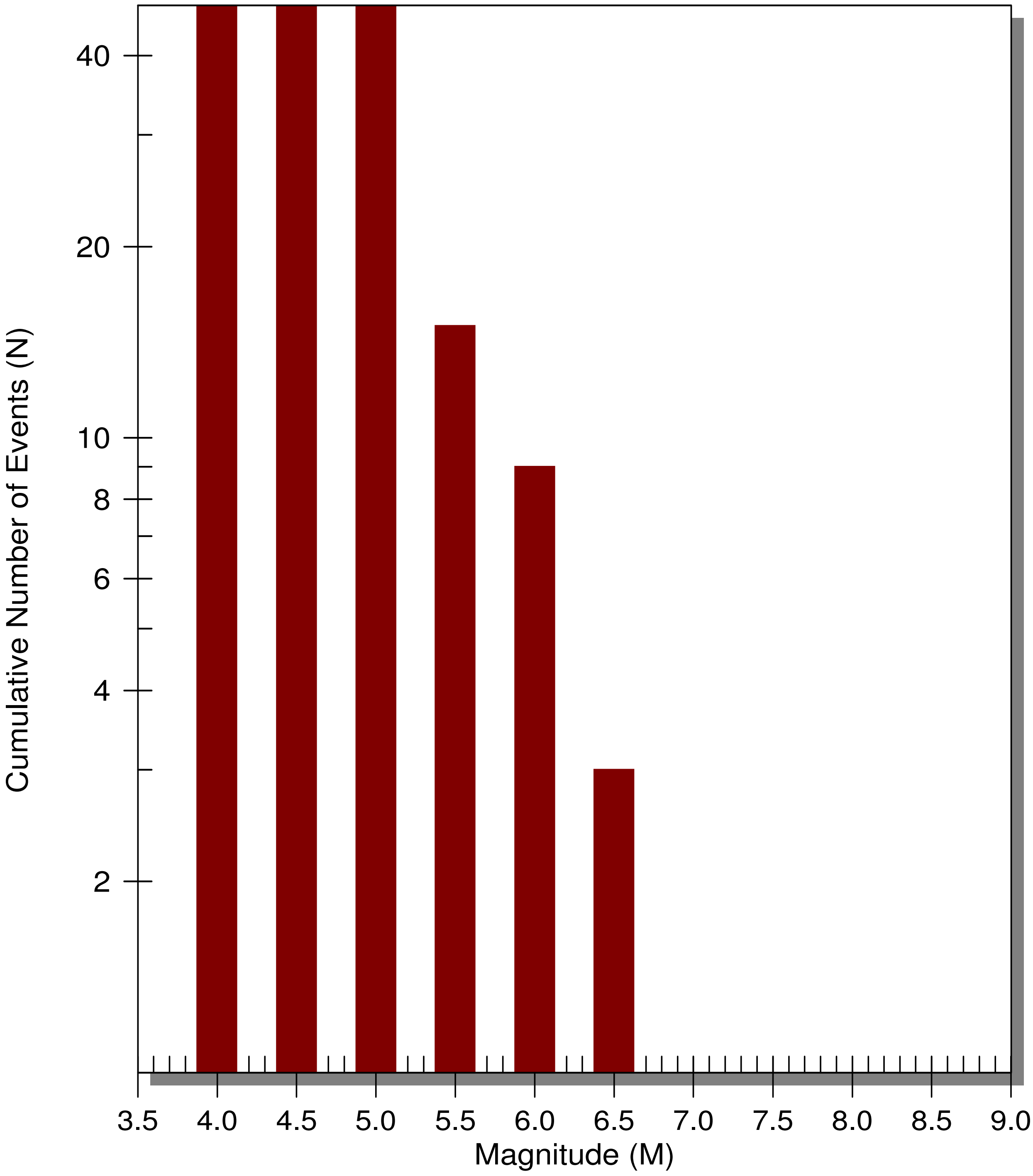
  Earthquake | Number of Times | Cumulative
   Magnitude |    Exceeded     | No. / Year
  -----------+-----------------+------------
     4.0     |       48        |   0.22326
     4.5     |       48        |   0.22326
     5.0     |       48        |   0.22326
     5.5     |       15        |   0.06977
     6.0     |        9        |   0.04186
     6.5     |        3        |   0.01395
     7.0     |        1        |   0.00465
```

EARTHQUAKE RECURRENCE CURVE

vessels non rock



W.O. 6688-A-SC
PLATE C-15



Number of Earthquakes (N) Above Magnitude (M)

vessels non rock

W.O. 6688-A-SC
PLATE C-16

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                           *************************
                           *                       *
                           *    E Q S E A R C H    *
                           *                       *
                           *     Version 3.00      *
                           *                       *
                           *************************

                                 ESTIMATION OF
                            PEAK ACCELERATION FROM
                        CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 6688-A
                                                     DATE: 01-27-2015

JOB NAME: vessels rock

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT


MAGNITUDE RANGE:
   MINIMUM MAGNITUDE:  5.00
   MAXIMUM MAGNITUDE:  9.00

SITE COORDINATES:
   SITE LATITUDE:  33.3027
   SITE LONGITUDE:  117.6917

SEARCH DATES:
           START DATE:   1800
           END DATE:   2014

SEARCH RADIUS:
           62.4 mi
           100.4 km

ATTENUATION RELATION:  13) Bozorgnia Campbell Niazi (1999) Hor.-Hard Rock-Cor.
   UNCERTAINTY (M=Median, S=Sigma): S       Number of Sigmas:  1.0
   ASSUMED SOURCE TYPE:  SS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
   SCOND:   1  Depth Source:  A
   Basement Depth:  .01 km      Campbell SSR:  0    Campbell SHR:  1
   COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km):  3.0
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                         EARTHQUAKE SEARCH RESULTS
                         -------------------------

Page  1
-------------------------------------------------------------------------------
    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   | (UTC)  |DEPTH|QUAKE| ACC.  | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+-----------
PAS |32.9710|117.8700|07/13/1986|1347 8.2|  6.0| 5.30| 0.044 | VI | 25.1( 40.4)
DMG |33.5750|117.9830|03/11/1933| 518 4.0|  0.0| 5.20| 0.041 |  V | 25.2( 40.6)
DMG |33.6170|117.9670|03/11/1933| 154 7.8|  0.0| 6.30| 0.077 | VII| 26.9( 43.2)
DMG |33.6170|118.0170|03/14/1933|19 150.0|  0.0| 5.10| 0.034 |  V | 28.7( 46.1)
DMG |33.6990|117.5110|05/31/1938| 83455.4| 10.0| 5.50| 0.042 | VI | 29.3( 47.1)
DMG |33.0000|117.3000|11/22/1800|2130 0.0|  0.0| 6.50| 0.076 | VII| 30.8( 49.6)
DMG |33.7000|117.4000|05/15/1910|1547 0.0|  0.0| 6.00| 0.052 | VI | 32.2( 51.8)
DMG |33.7000|117.4000|04/11/1910| 757 0.0|  0.0| 5.00| 0.029 |  V | 32.2( 51.8)
DMG |33.7000|117.4000|05/13/1910| 620 0.0|  0.0| 5.00| 0.029 |  V | 32.2( 51.8)
DMG |33.6830|118.0500|03/11/1933| 658 3.0|  0.0| 5.50| 0.037 |  V | 33.4( 53.7)
MGI |33.8000|117.6000|04/22/1918|2115 0.0|  0.0| 5.00| 0.026 |  V | 34.7( 55.9)
DMG |33.7000|118.0670|03/11/1933| 85457.0|  0.0| 5.10| 0.028 |  V | 34.9( 56.2)
DMG |33.7000|118.0670|03/11/1933| 51022.0|  0.0| 5.10| 0.028 |  V | 34.9( 56.2)
DMG |33.7500|118.0830|03/11/1933| 323 0.0|  0.0| 5.00| 0.024 | IV | 38.2( 61.5)
DMG |33.7500|118.0830|03/11/1933| 2 9 0.0|  0.0| 5.00| 0.024 | IV | 38.2( 61.5)
DMG |33.7500|118.0830|03/11/1933| 230 0.0|  0.0| 5.10| 0.025 |  V | 38.2( 61.5)
DMG |33.7500|118.0830|03/13/1933|131828.0|  0.0| 5.30| 0.028 |  V | 38.2( 61.5)
DMG |33.7500|118.0830|03/11/1933| 910 0.0|  0.0| 5.10| 0.025 |  V | 38.2( 61.5)
DMG |33.7830|118.1330|10/02/1933| 91017.6|  0.0| 5.40| 0.027 |  V | 41.8( 67.2)
GSG |33.9530|117.7610|07/29/2008|184215.7| 14.0| 5.30| 0.024 | IV | 45.1( 72.5)
MGI |33.0000|117.0000|09/21/1856| 730 0.0|  0.0| 5.00| 0.020 | IV | 45.1( 72.6)
DMG |33.7830|118.2500|11/14/1941| 84136.3|  0.0| 5.40| 0.025 |  V | 46.2( 74.3)
MGI |32.8000|117.1000|05/25/1803| 0 0 0.0|  0.0| 5.00| 0.019 | IV | 48.8( 78.5)
MGI |34.0000|117.5000|12/16/1858|10 0 0.0|  0.0| 7.00| 0.065 | VI | 49.4( 79.5)
DMG |33.9000|117.2000|12/19/1880| 0 0 0.0|  0.0| 6.00| 0.033 |  V | 50.0( 80.5)
DMG |33.8500|118.2670|03/11/1933|1425 0.0|  0.0| 5.00| 0.018 | IV | 50.2( 80.8)
DMG |33.7500|117.0000|04/21/1918|223225.0|  0.0| 6.80| 0.055 | VI | 50.4( 81.1)
DMG |33.7500|117.0000|06/06/1918|2232 0.0|  0.0| 5.00| 0.018 | IV | 50.4( 81.1)
DMG |32.7000|117.2000|05/27/1862|20 0 0.0|  0.0| 5.90| 0.031 |  V | 50.4( 81.1)
DMG |32.8170|118.3500|12/26/1951| 04654.0|  0.0| 5.90| 0.030 |  V | 50.7( 81.7)
MGI |34.0000|118.0000|12/25/1903|1745 0.0|  0.0| 5.00| 0.018 | IV | 51.3( 82.6)
DMG |33.7100|116.9250|09/23/1963|144152.6| 16.5| 5.00| 0.017 | IV | 52.3( 84.2)
DMG |33.8000|117.0000|12/25/1899|1225 0.0|  0.0| 6.40| 0.040 |  V | 52.6( 84.6)
T-A |32.6700|117.1700|10/21/1862| 0 0 0.0|  0.0| 5.00| 0.017 | IV | 53.1( 85.5)
T-A |32.6700|117.1700|05/24/1865| 0 0 0.0|  0.0| 5.00| 0.017 | IV | 53.1( 85.5)
T-A |32.6700|117.1700|12/00/1856| 0 0 0.0|  0.0| 5.00| 0.017 | IV | 53.1( 85.5)
DMG |34.0000|117.2500|07/23/1923| 73026.0|  0.0| 6.25| 0.035 |  V | 54.4( 87.6)
PAS |34.0610|118.0790|10/01/1987|144220.0|  9.5| 5.90| 0.027 |  V | 56.9( 91.5)
DMG |33.2000|116.7000|01/01/1920| 235 0.0|  0.0| 5.00| 0.016 | IV | 57.7( 92.8)
GSP |34.1400|117.7000|02/28/1990|234336.6|  5.0| 5.20| 0.017 | IV | 57.8( 93.0)
T-A |34.0000|118.2500|01/10/1856| 0 0 0.0|  0.0| 5.00| 0.015 | IV | 57.9( 93.1)
T-A |34.0000|118.2500|09/23/1827| 0 0 0.0|  0.0| 5.00| 0.015 | IV | 57.9( 93.1)
T-A |34.0000|118.2500|03/26/1860| 0 0 0.0|  0.0| 5.00| 0.015 | IV | 57.9( 93.1)
```
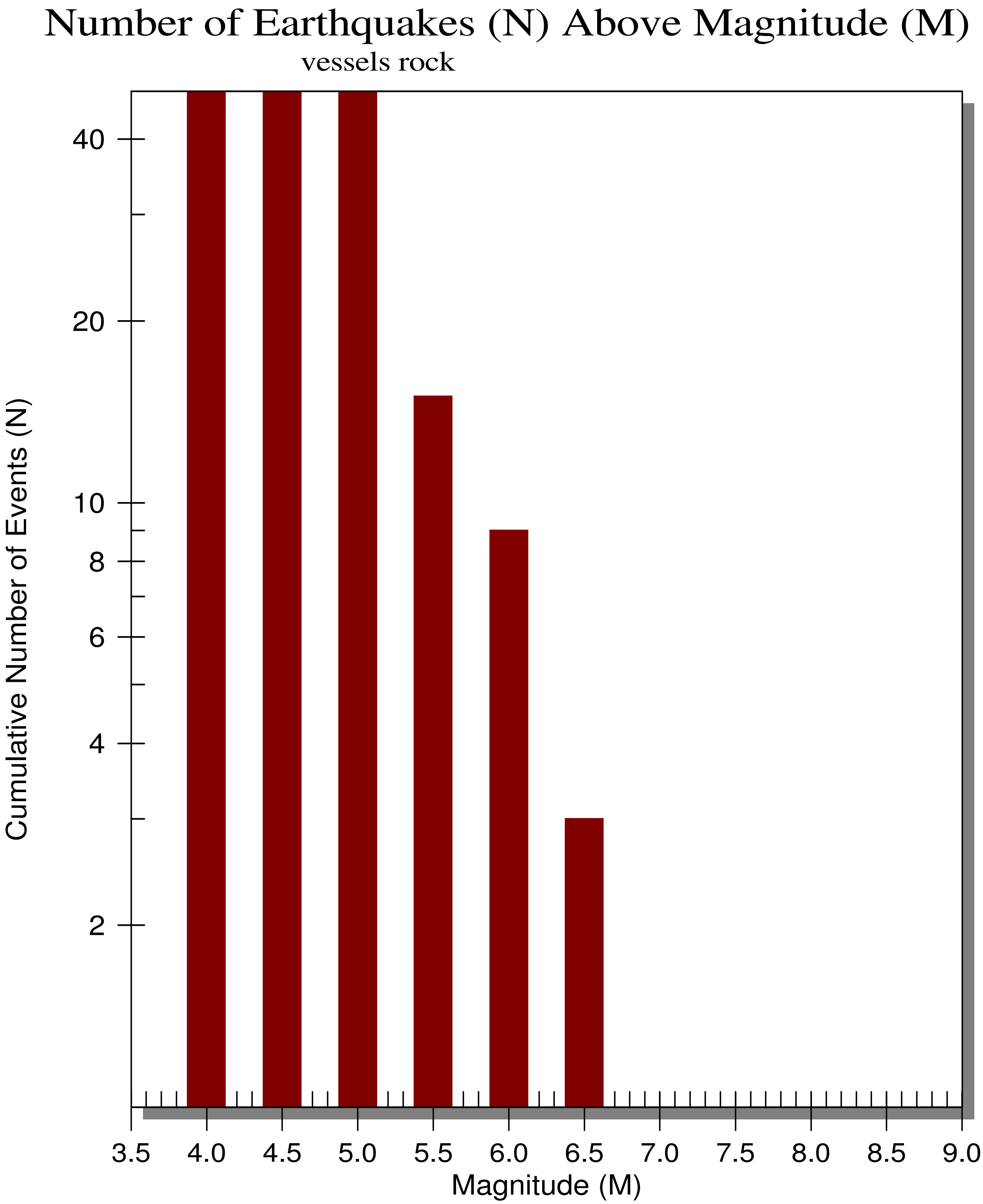
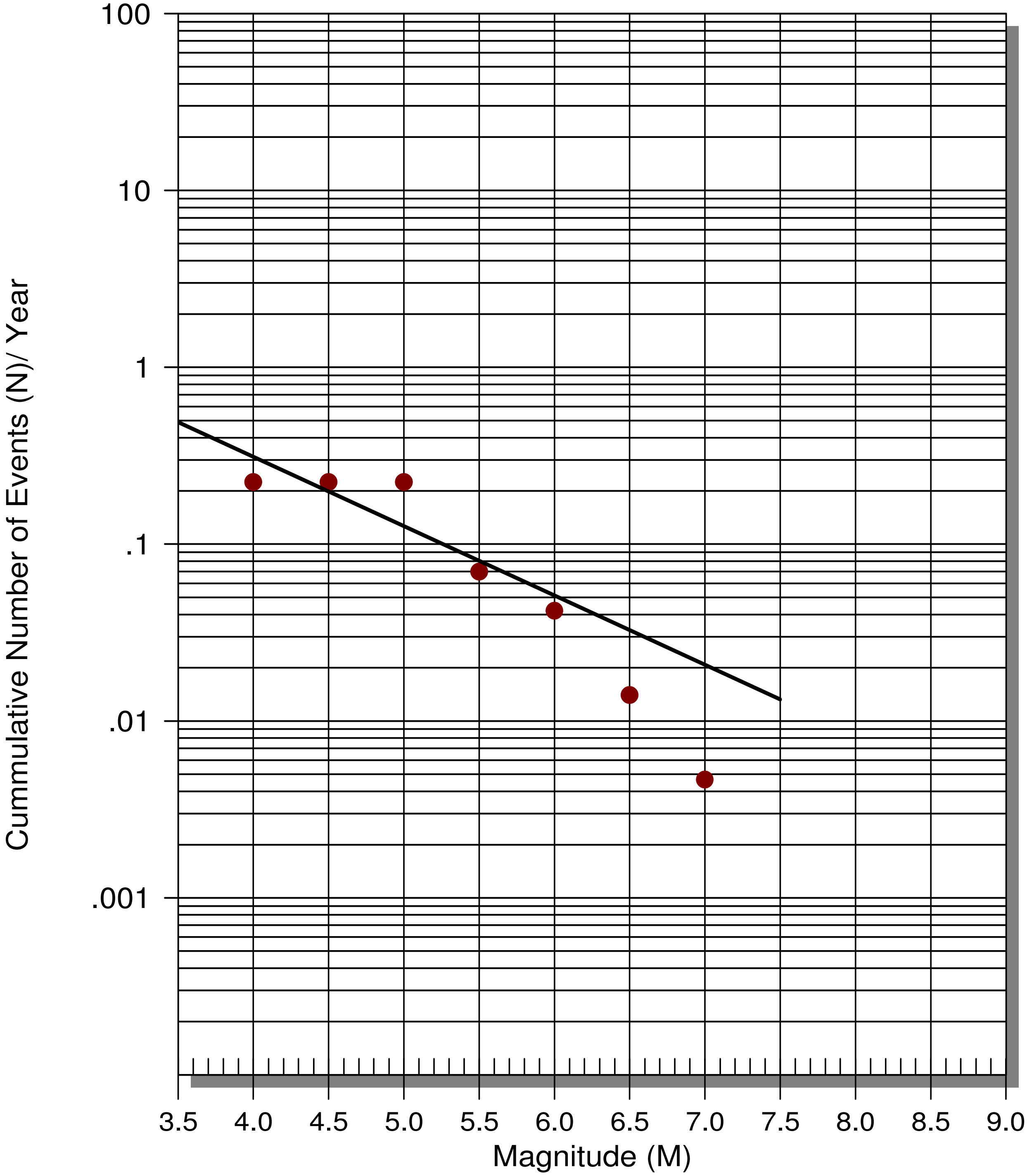

```
PAS |34.0730|118.0980|10/04/1987|105938.2|  8.2| 5.30| 0.018 | IV | 58.1( 93.5)
MGI |34.1000|117.3000|07/15/1905|2041 0.0|  0.0| 5.30| 0.018 | IV | 59.5( 95.7)
MGI |34.0000|118.3000|09/03/1905| 540 0.0|  0.0| 5.30| 0.018 | IV | 59.5( 95.7)
MGI |34.1000|118.1000|07/11/1855| 415 0.0|  0.0| 6.30| 0.033 |  V | 59.8( 96.3)
DMG |32.8000|116.8000|10/23/1894|23 3 0.0|  0.0| 5.70| 0.022 | IV | 62.2(100.1)

*******************************************************************************








-END OF SEARCH-   48 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH:   1800  TO  2014

LENGTH OF SEARCH TIME:   215  years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 25.1 MILES (40.4 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.077 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
  a-value=  1.062
  b-value=  0.392
  beta-value=  0.902


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

  Earthquake | Number of Times | Cumulative
   Magnitude |     Exceeded     | No. / Year
  -----------+-----------------+------------
     4.0     |       48        |   0.22326
     4.5     |       48        |   0.22326
     5.0     |       48        |   0.22326
     5.5     |       15        |   0.06977
     6.0     |        9        |   0.04186
     6.5     |        3        |   0.01395
     7.0     |        1        |   0.00465
```



W.O. 6688-A-SC
PLATE C-20

EARTHQUAKE RECURRENCE CURVE

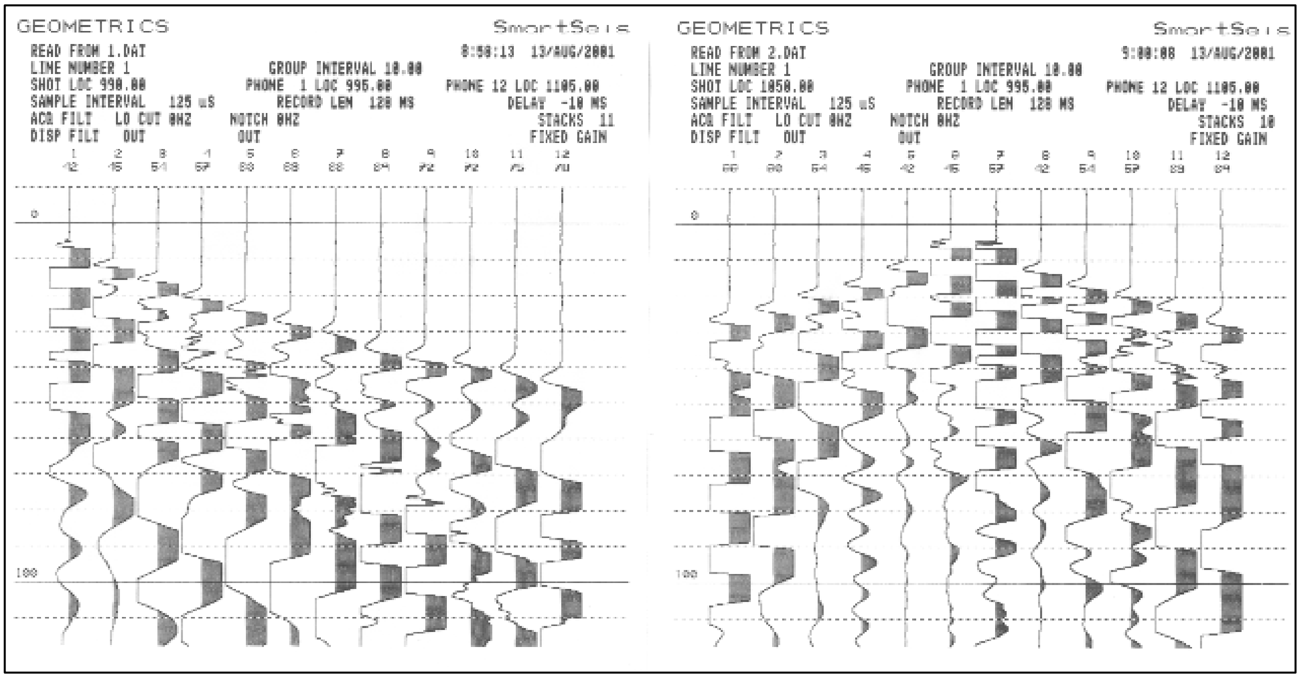
vessels rock



APPENDIX D

ROCK HARDNESS REFRACTION SURVEY

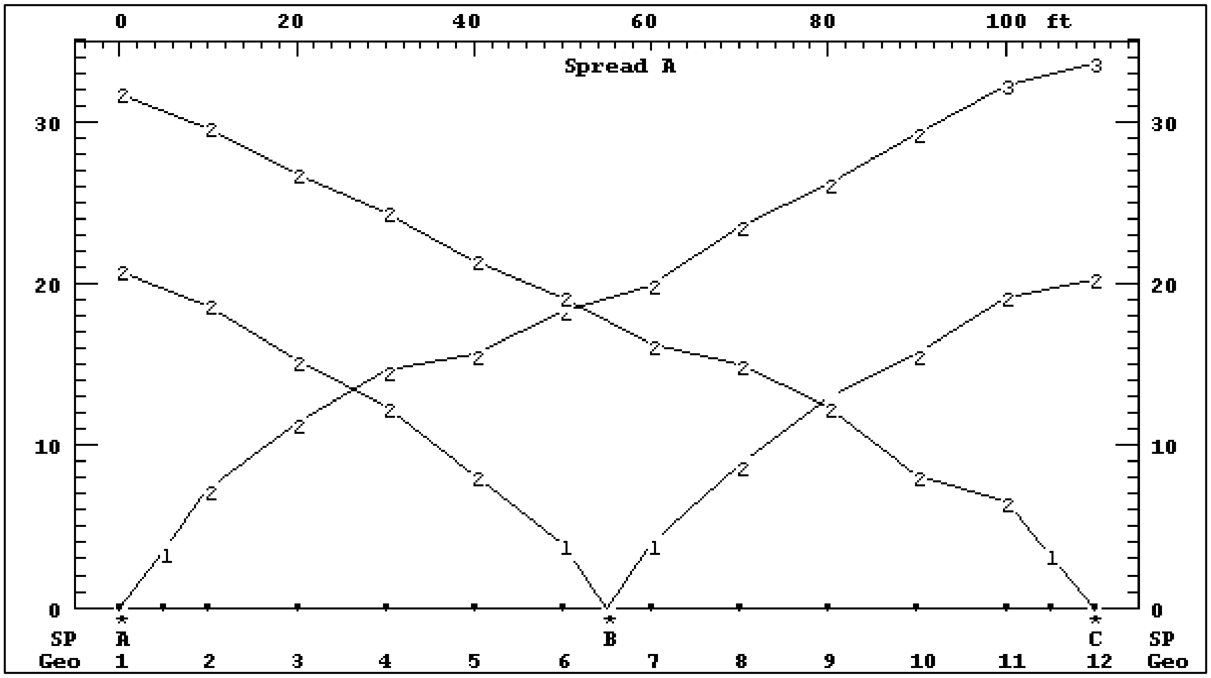
Example
Raw Seismic Data



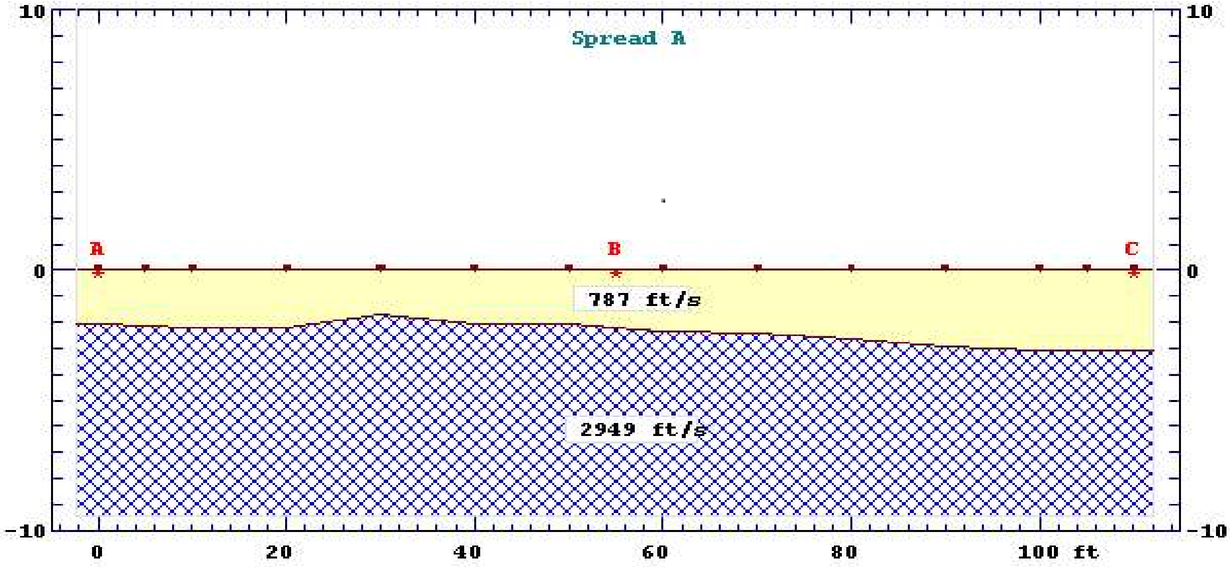
```
READ FROM 1.DAT                                    8:58:13  13/AUG/2001
LINE NUMBER 1                  GROUP INTERVAL 10.00
SHOT LOC 990.00             PHONE  1 LOC 995.00       PHONE 12 LOC 1105.00
SAMPLE INTERVAL   125 uS        RECORD LEN  120 MS            DELAY  -10 MS
ACQ FILT   LO CUT 0HZ     NOTCH 0HZ                              STACKS  11
DISP FILT   OUT              OUT                                FIXED GAIN
```

```
READ FROM 2.DAT                                    9:00:08  13/AUG/2001
LINE NUMBER 1                  GROUP INTERVAL 10.00
SHOT LOC 1050.00            PHONE  1 LOC 995.00       PHONE 12 LOC 1105.00
SAMPLE INTERVAL   125 uS        RECORD LEN  120 MS            DELAY  -10 MS
ACQ FILT   LO CUT 0HZ     NOTCH 0HZ                              STACKS  10
DISP FILT   OUT              OUT                                FIXED GAIN
```

Example
Seismic Line



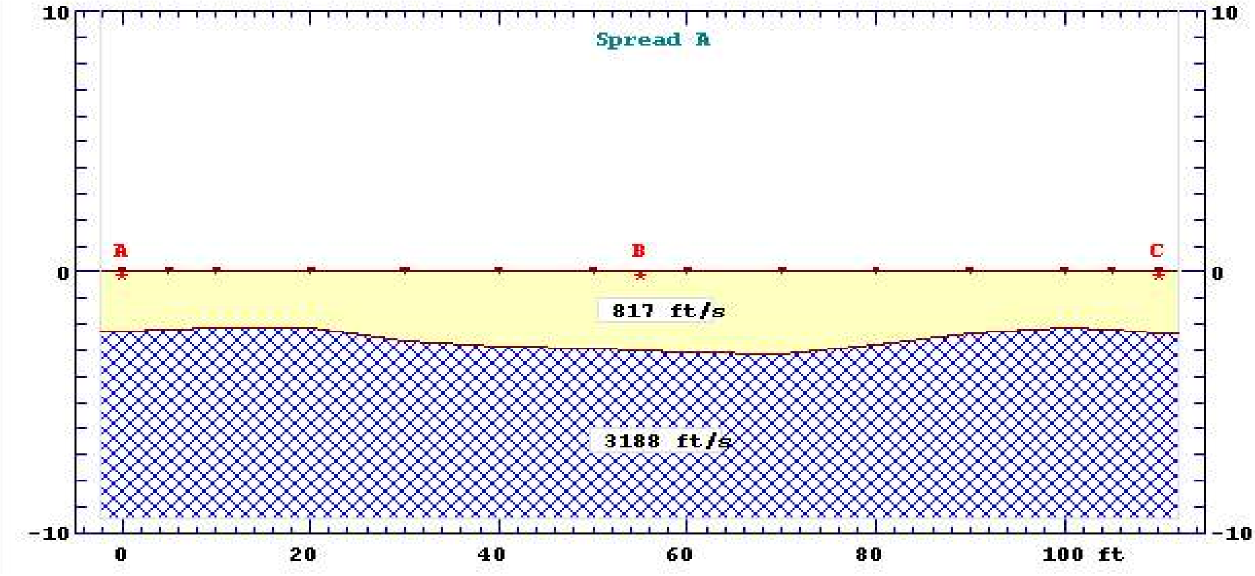
Seismic Line
SL-1



North

South

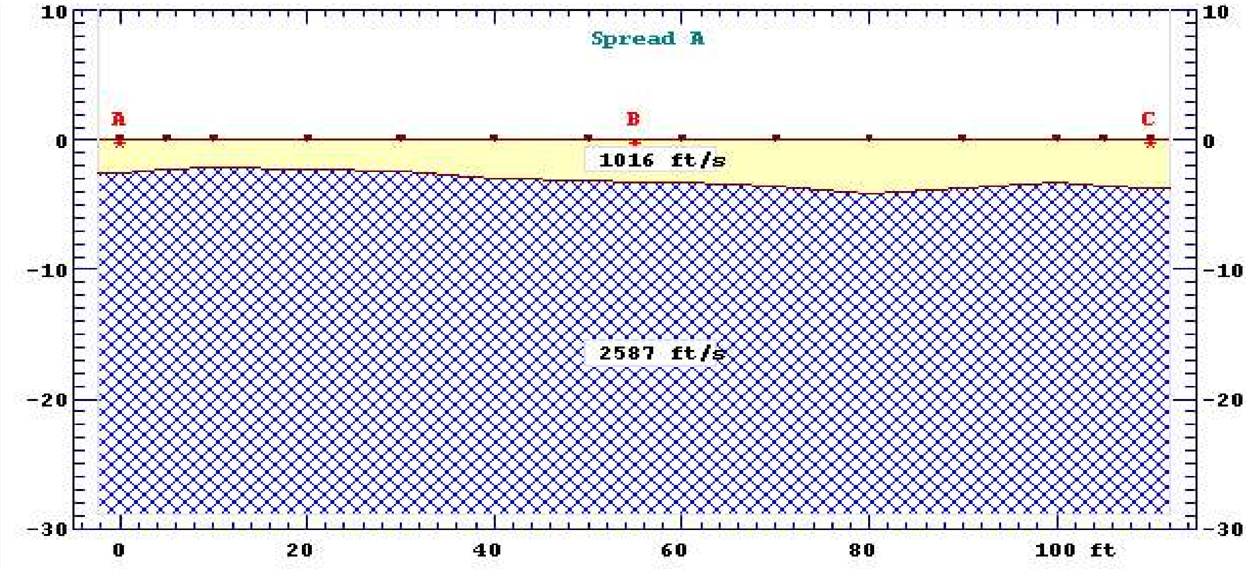
Seismic Line
SL-2



Northwest

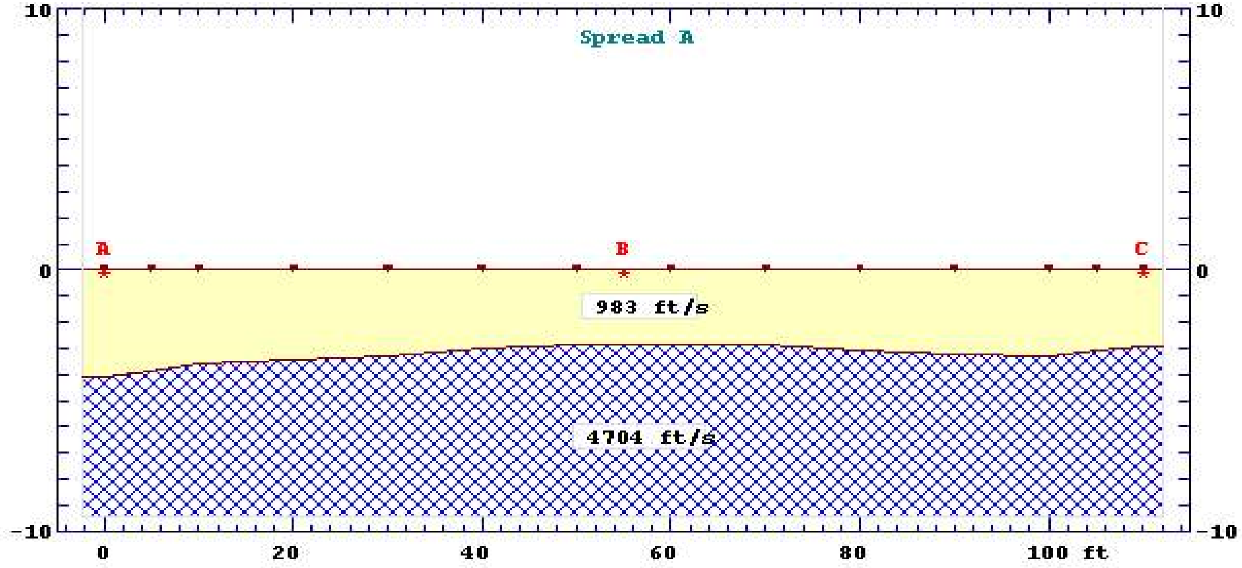
Southeast

Seismic Line
SL-3



North South

Seismic Line
SL-4

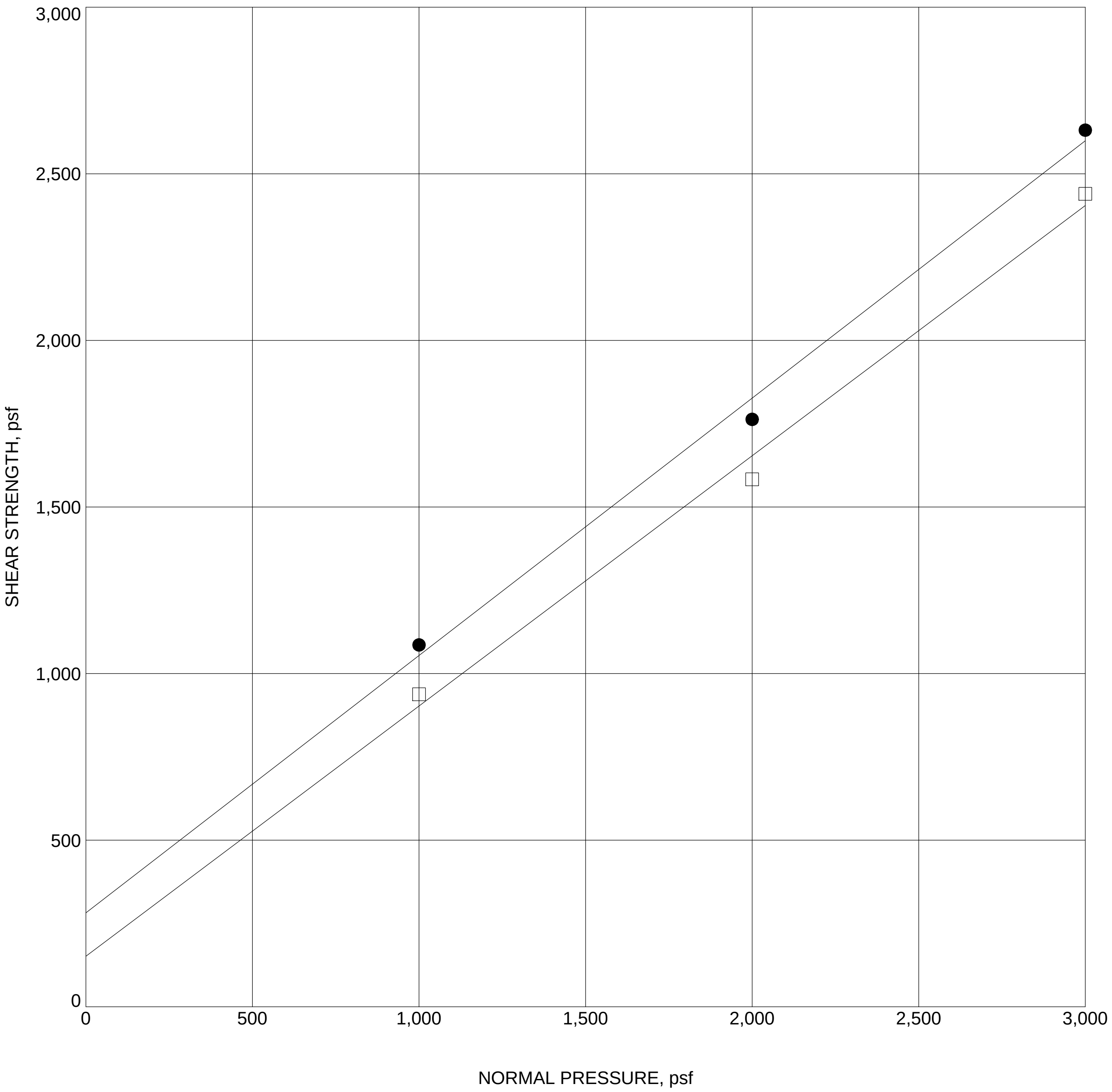


Northwest

Southeast

APPENDIX E

LABORATORY DATA



	Sample	Depth/El.	Range	Classification	Primary/Residual	Sample Type	γ_d	MC%	*c*	ϕ
●	TP-7	4.0		Silty Sand	Primary Shear	Remolded	115.2	11.0	282	38
☐	TP-7	4.0			Residual Shear	Remolded	115.2	11.0	152	37
					Reshear Shear	Remolded				
					Reshear Shear	Remolded				

Note: Sample Innundated Prior To Test

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

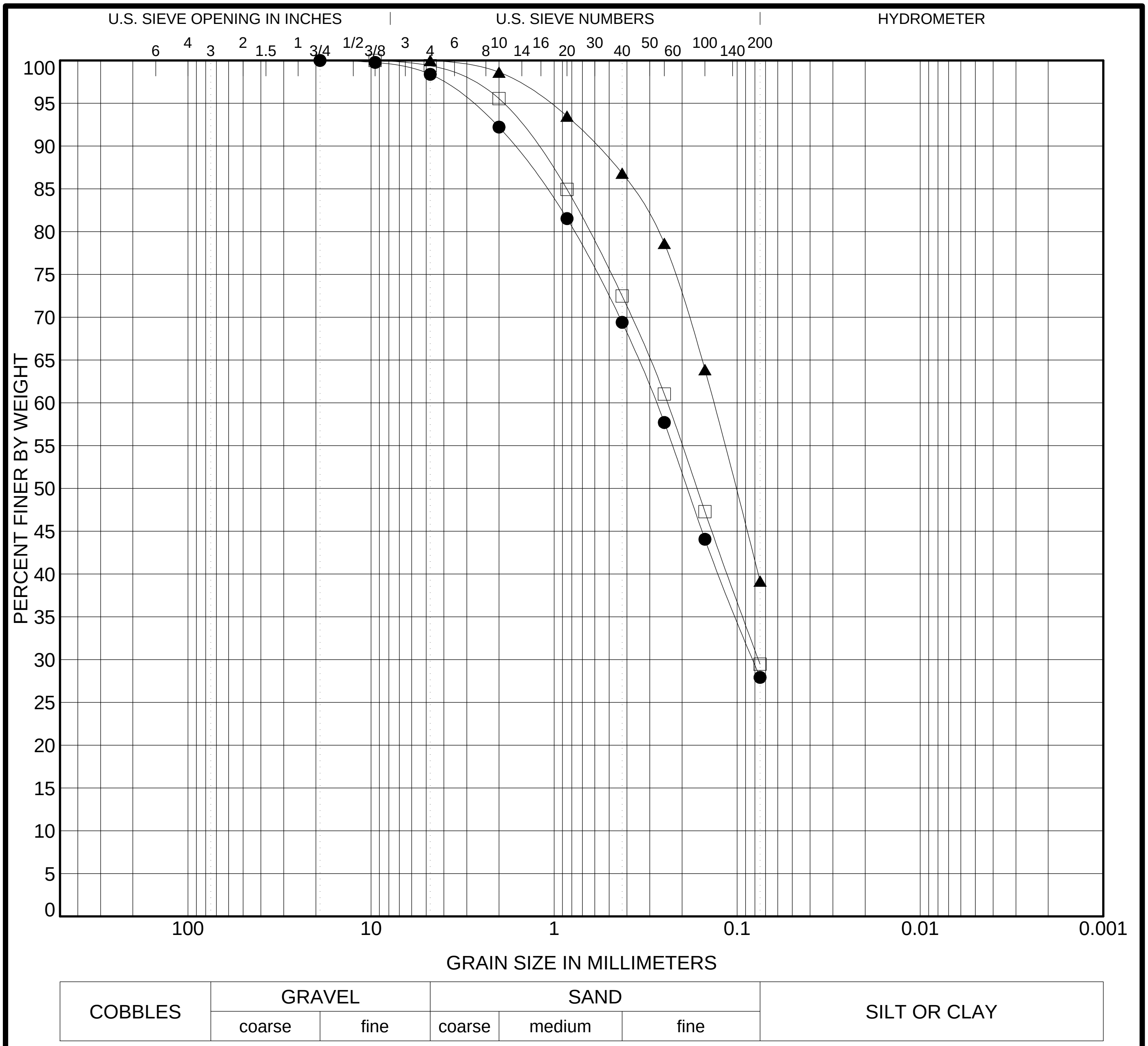
DIRECT SHEAR TEST

Project: VESSEL'S STALLION RANCH

Number: 6688-A-SC

Date: January 2015

Plate: E - 1



	Sample	Depth	Range	Visual Classification/USCS CLASSIFICATION	LL	PL	PI	Cc	Cu
●	TP-1	0.0		Silty Sand					
☐	TP-2	8.0		Silty Sand					
▲	TP-6	6.0		Silty Sand					

	Sample	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	TP-1	0.0	19	0.277	0.082		1.6	70.5	27.9	
☐	TP-2	8.0	9.5	0.241	0.077		0.6	69.9	29.5	
▲	TP-6	6.0	4.75	0.135			0.0	60.8	39.2	

GeoSoils, Inc.

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

GRAIN SIZE DISTRIBUTION

Project: VESSEL'S STALLION RANCH

Number: 6688-A-SC

Date: January 2015

Plate: E - 2

Cal Land Engineering, Inc.
dba Quartech Consultant
Geotechnical, Environmental, and Civil Engineering

SUMMARY OF LABORATORY TEST DATA

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

QCI Project No.: 14-029-03g
Date: March 20, 2014
Summarized by: ABK

Client: Vessel's Stallion Ranch
W.O. 6688-A- SC

Corrosivity Test Results

Sample ID	Sample Depth	pH CT-532 (643)	Chloride CT-422 (ppm)	Sulfate CT-417 % By Weight	Resistivity CT-532 (643) (ohm-cm)
TP-1	1' -.4'	6.99	122	0.0110	1,800

APPENDIX F

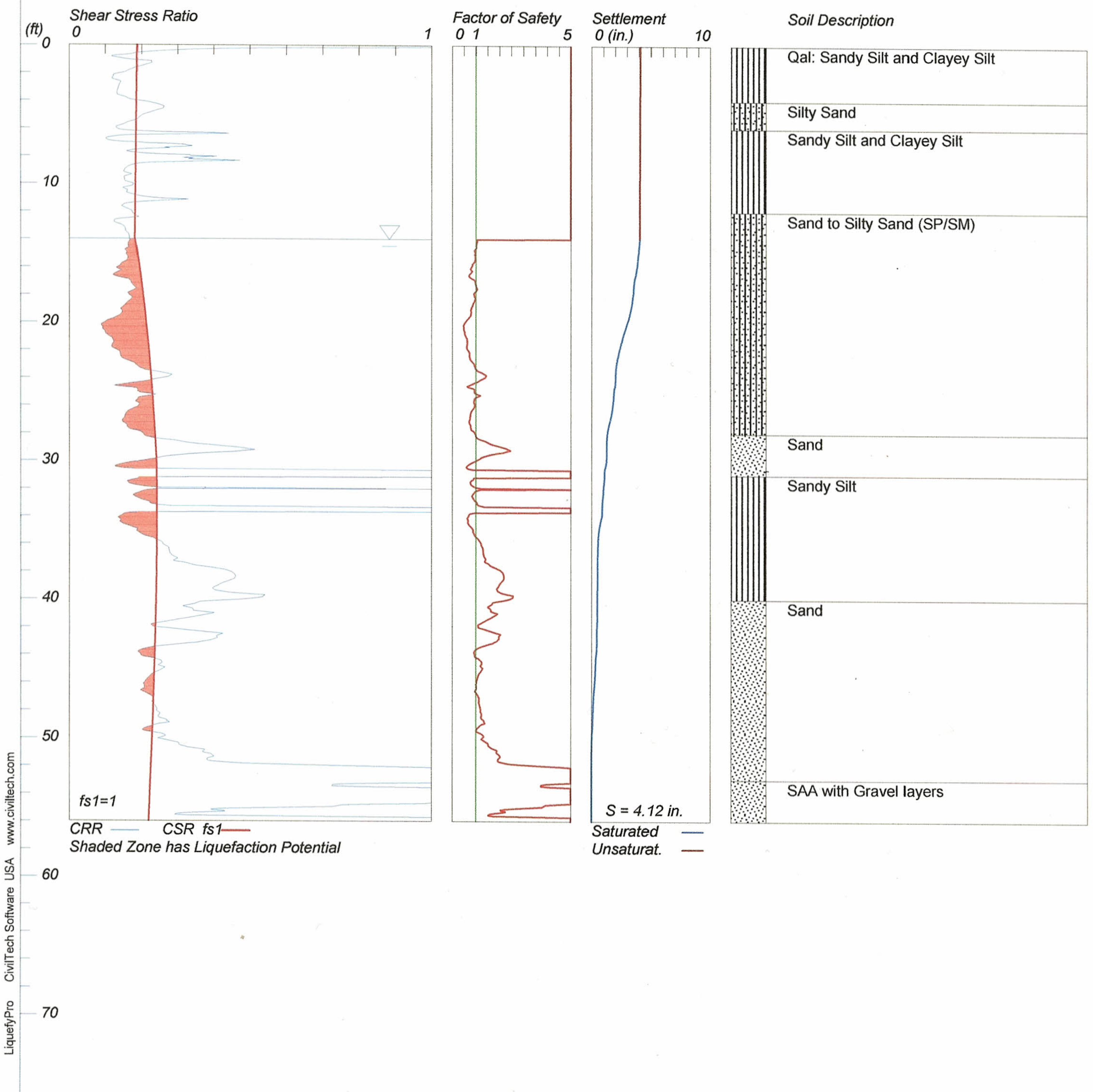
LIQUEFACTION ANALYSIS

Seismic Vertical Deformation Analysis

Vessels Ranch

Hole No.=CPT 1   Water Depth=14 ft   Surface Elev.=194

Magnitude=7.1
Acceleration=0.29g



GeoSoils, Inc.   **W.O. 6688-A-SC**   **PLATE F-1**

NO COVER 0.29

Seismic Vertical Deformation Analysis

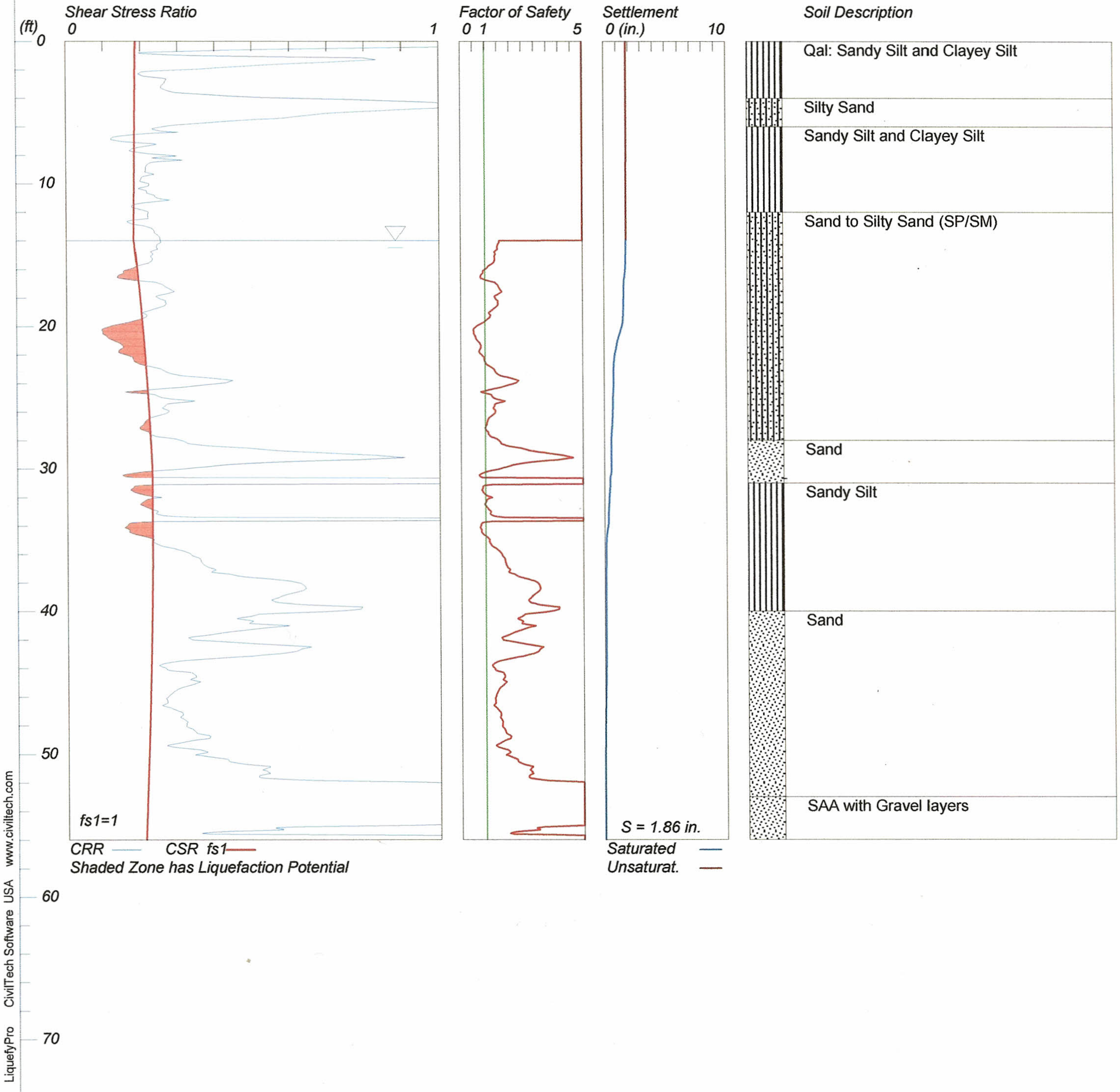
Vessels Ranch

Hole No.=CPT 1 Water Depth=14 ft Surface Elev.=194

Ground Improvement of Fill=5 ft

Magnitude=7.1

Acceleration=0.29g



5' Case 0.29

Seismic Vertical Deformation Analysis

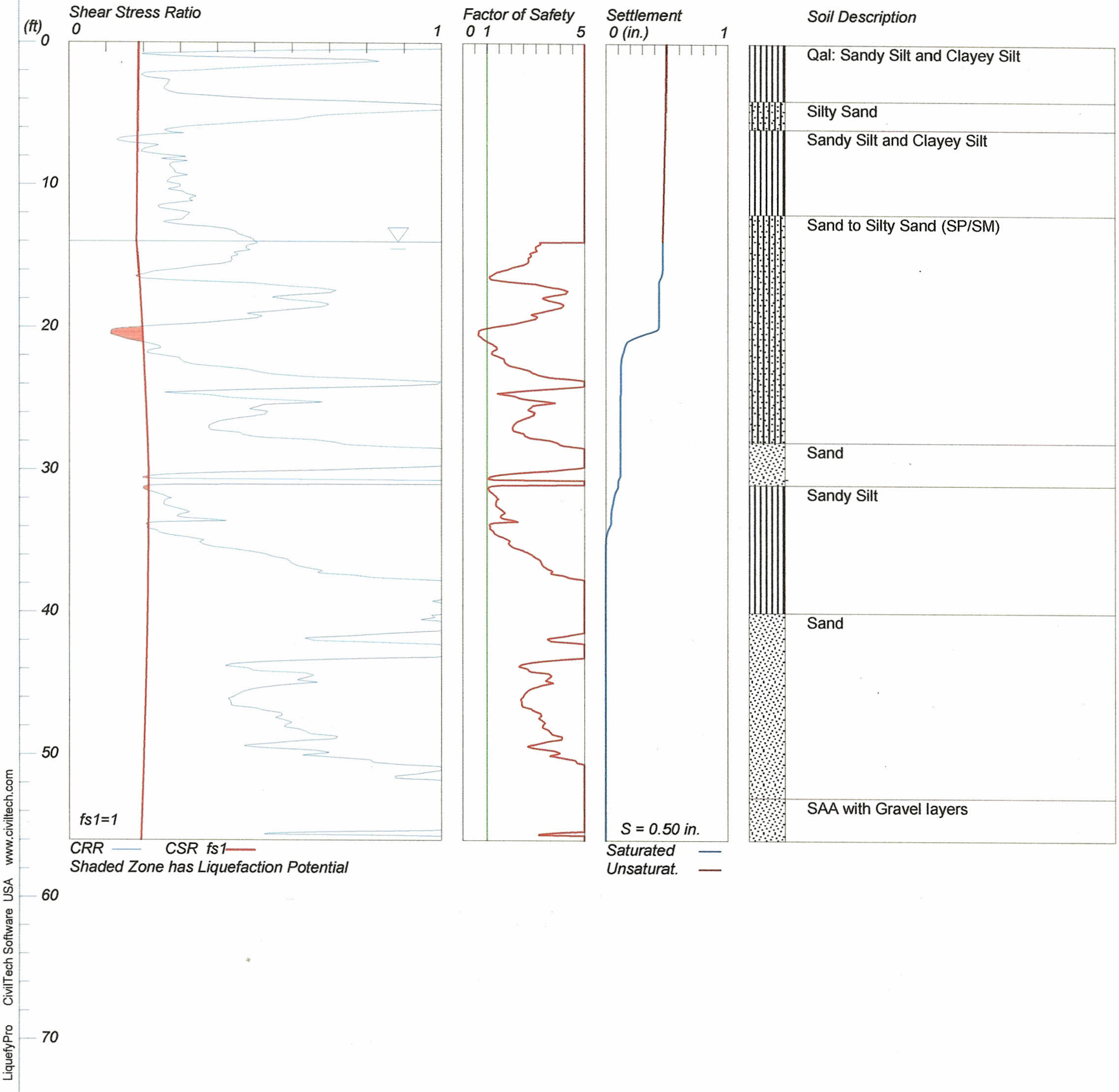
Vessels Ranch

Hole No.=CPT 1 Water Depth=14 ft Surface Elev.=194

Ground Improvement of Fill=15 ft

Magnitude=7.1

Acceleration=0.29g



GeoSoils, Inc. **W.O. 6688-A-SC** **PLATE F-3**

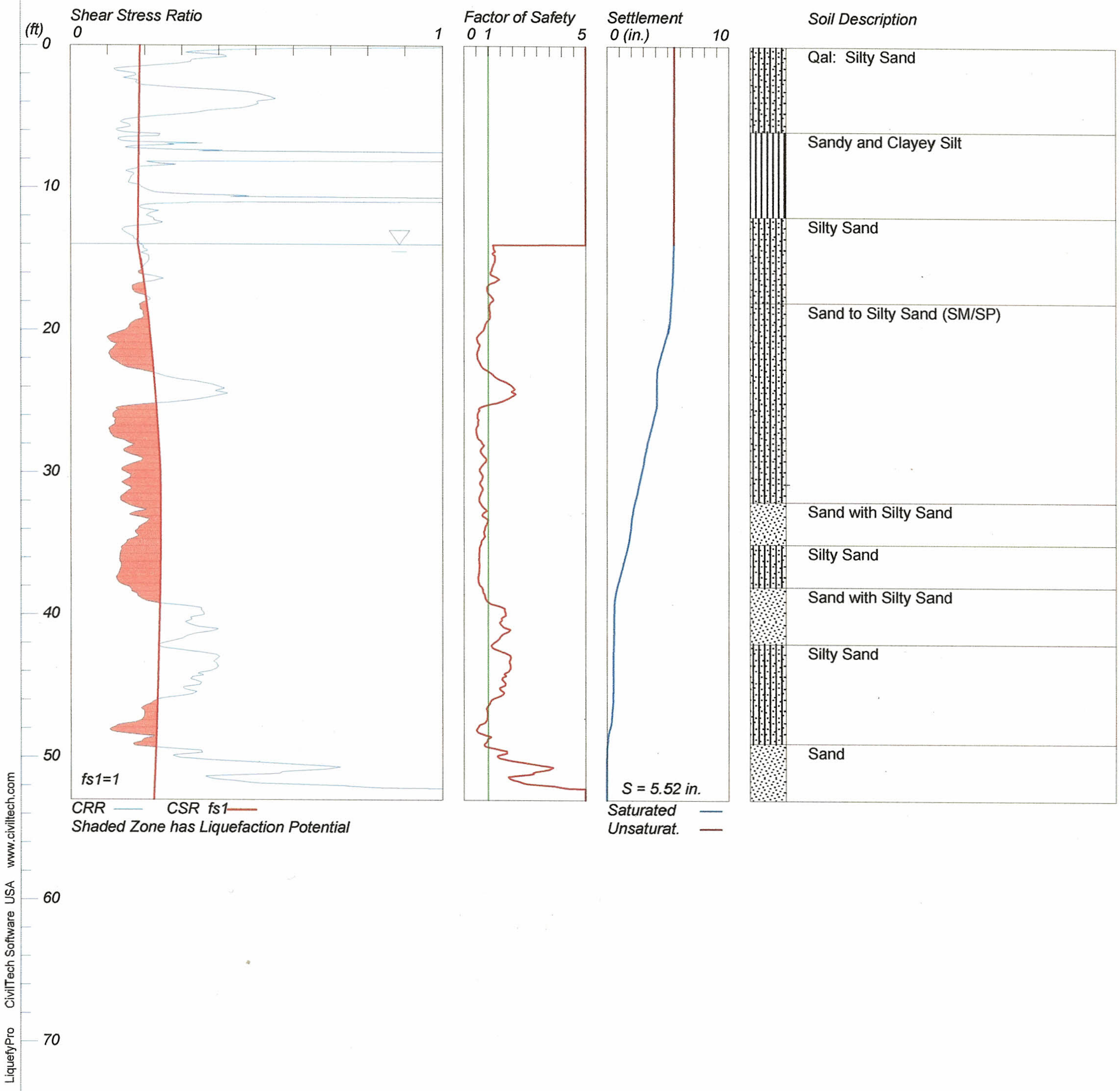
15' cover 0.29

Seismic Vertical Deformation Analysis

Vessels Ranch CPT 1A

Hole No.=CPT 1A   Water Depth=14 ft   Surface Elev.=194

Magnitude=7.1
Acceleration=0.29g



Seismic Vertical Deformation Analysis

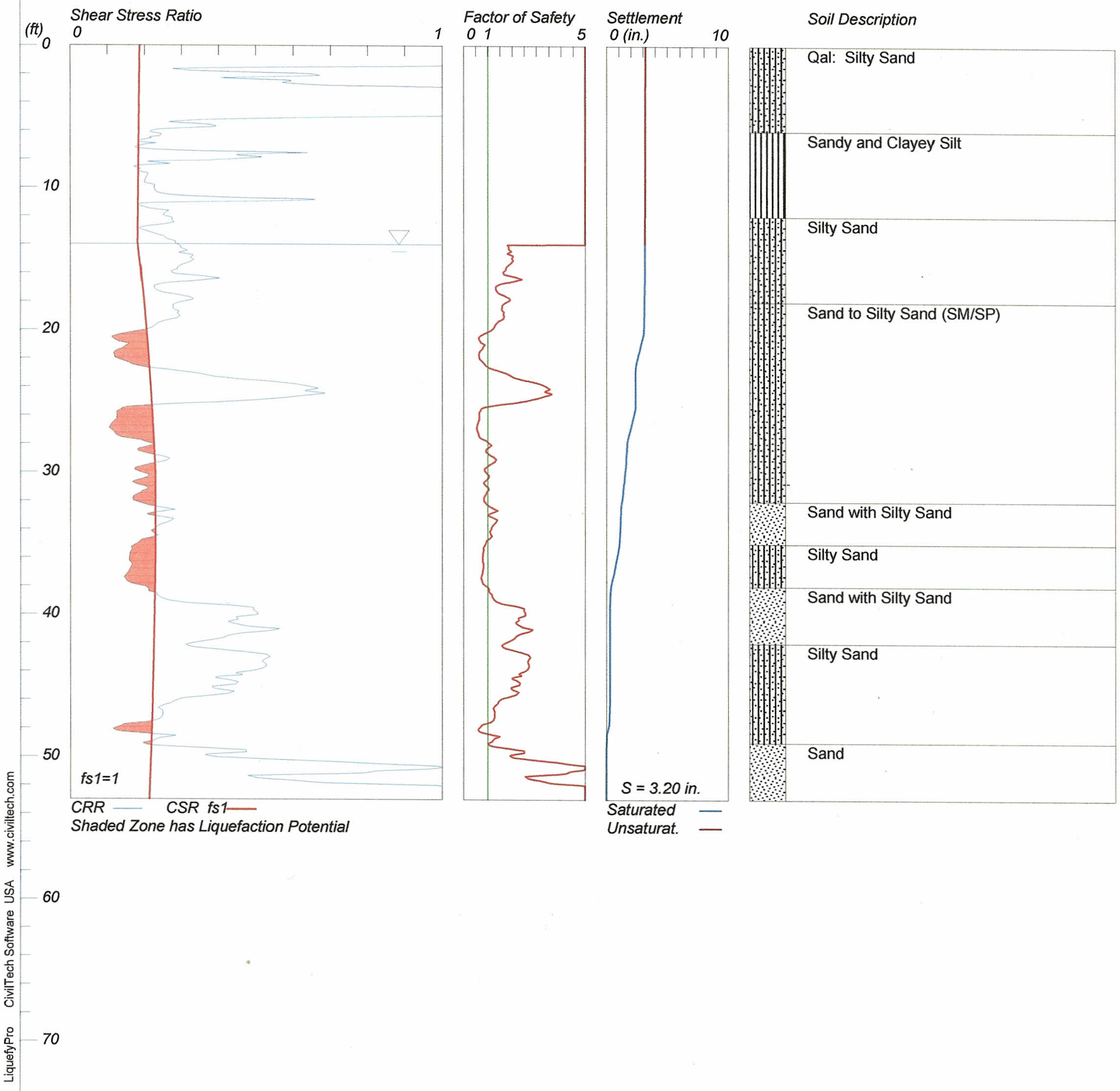
Vessels Ranch CPT 1A

Hole No.=CPT 1A Water Depth=14 ft Surface Elev.=194

Ground Improvement of Fill=5 ft

Magnitude=7.1

Acceleration=0.29g

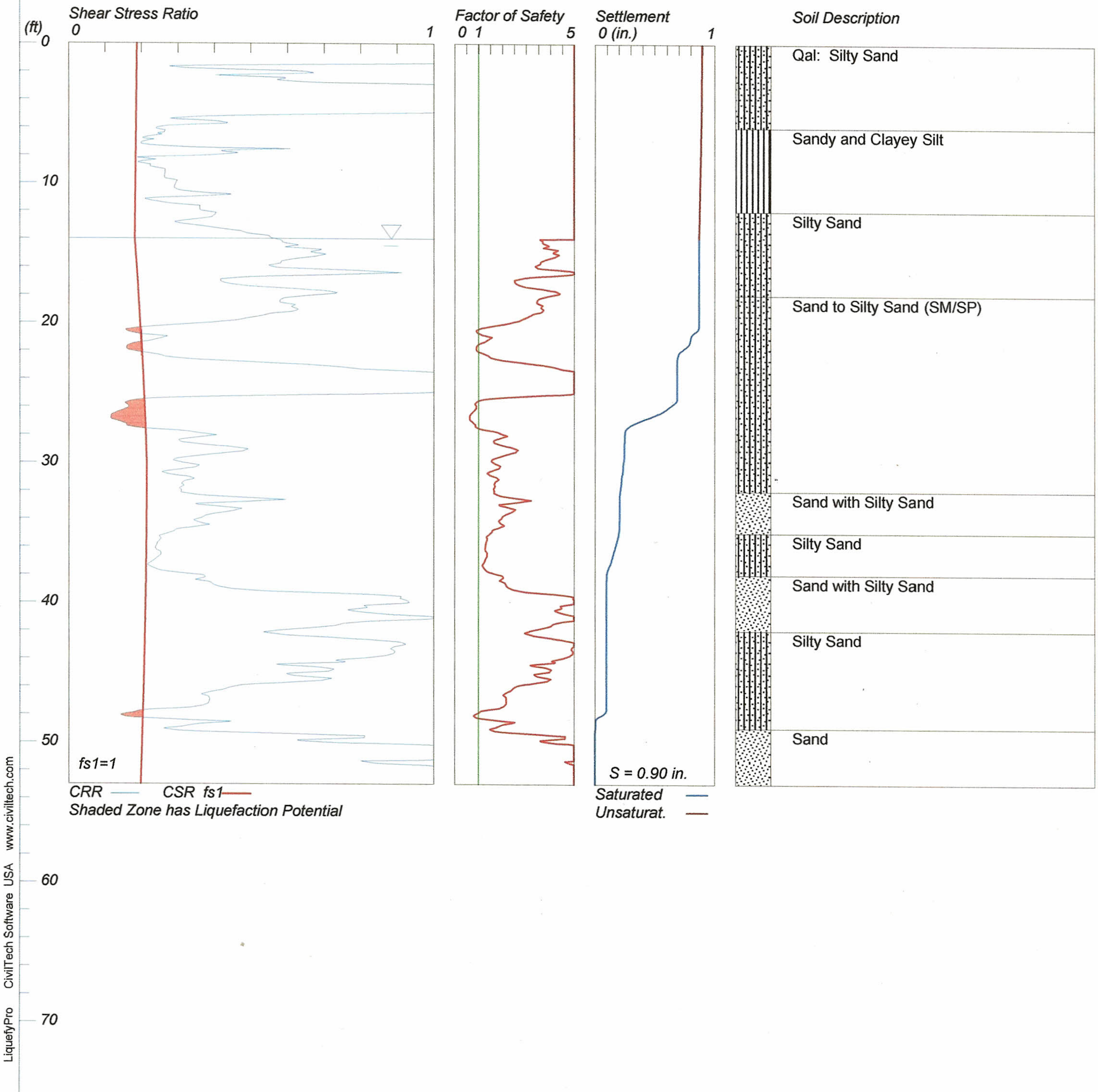


Seismic Vertical Deformation Analysis

Vessels Ranch CPT 1A

Hole No.=CPT 1A Water Depth=14 ft Surface Elev.=194
Ground Improvement of Fill=15 ft

Magnitude=7.1
Acceleration=0.29g

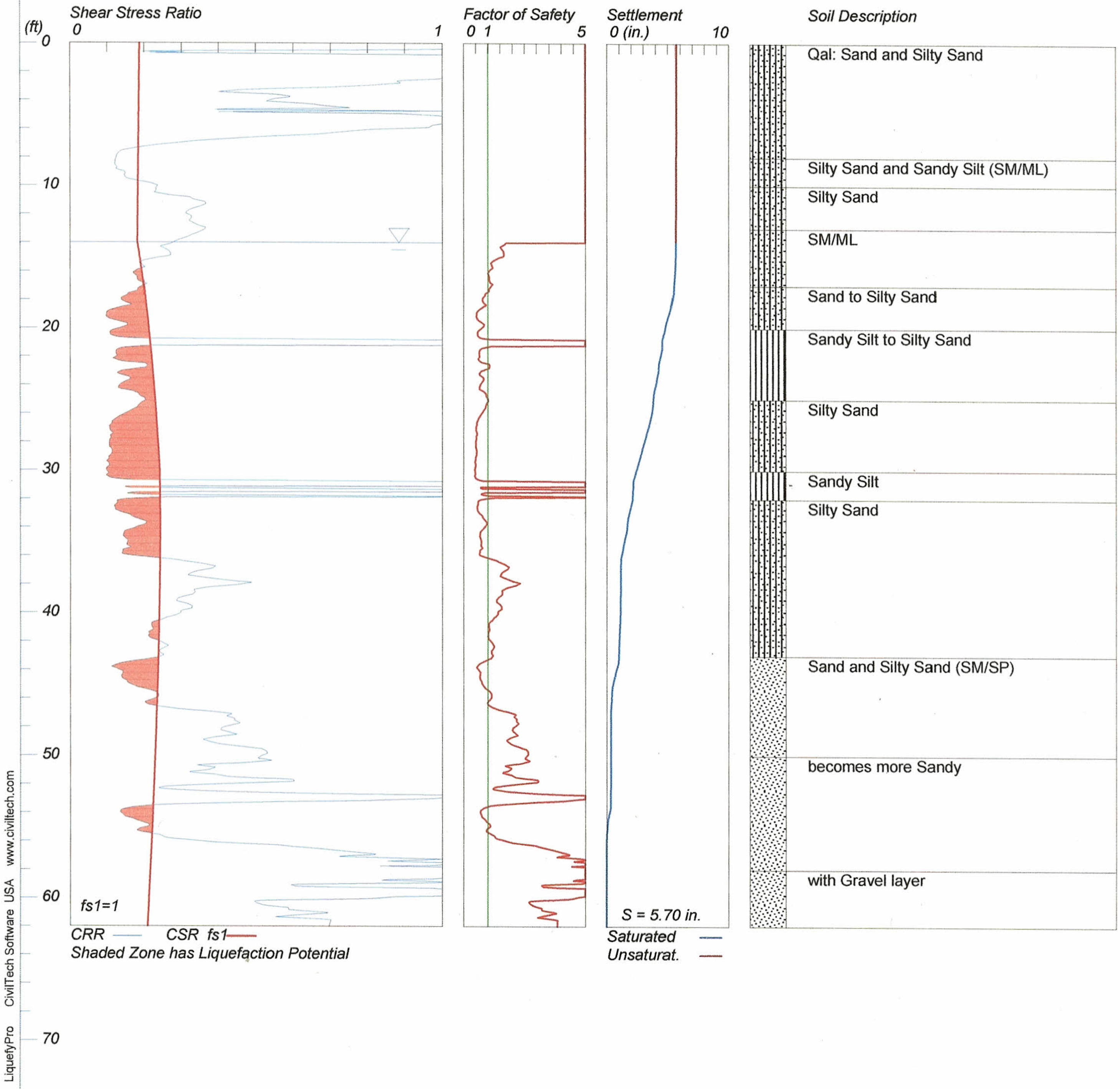


Seismic Vertical Deformation Analysis

Vessels Ranch CPT 2

Hole No.=CPT 2 Water Depth=14 ft Surface Elev.=201

Magnitude=7.1
Acceleration=0.29g



No Calc 0.29

Seismic Vertical Deformation Analysis

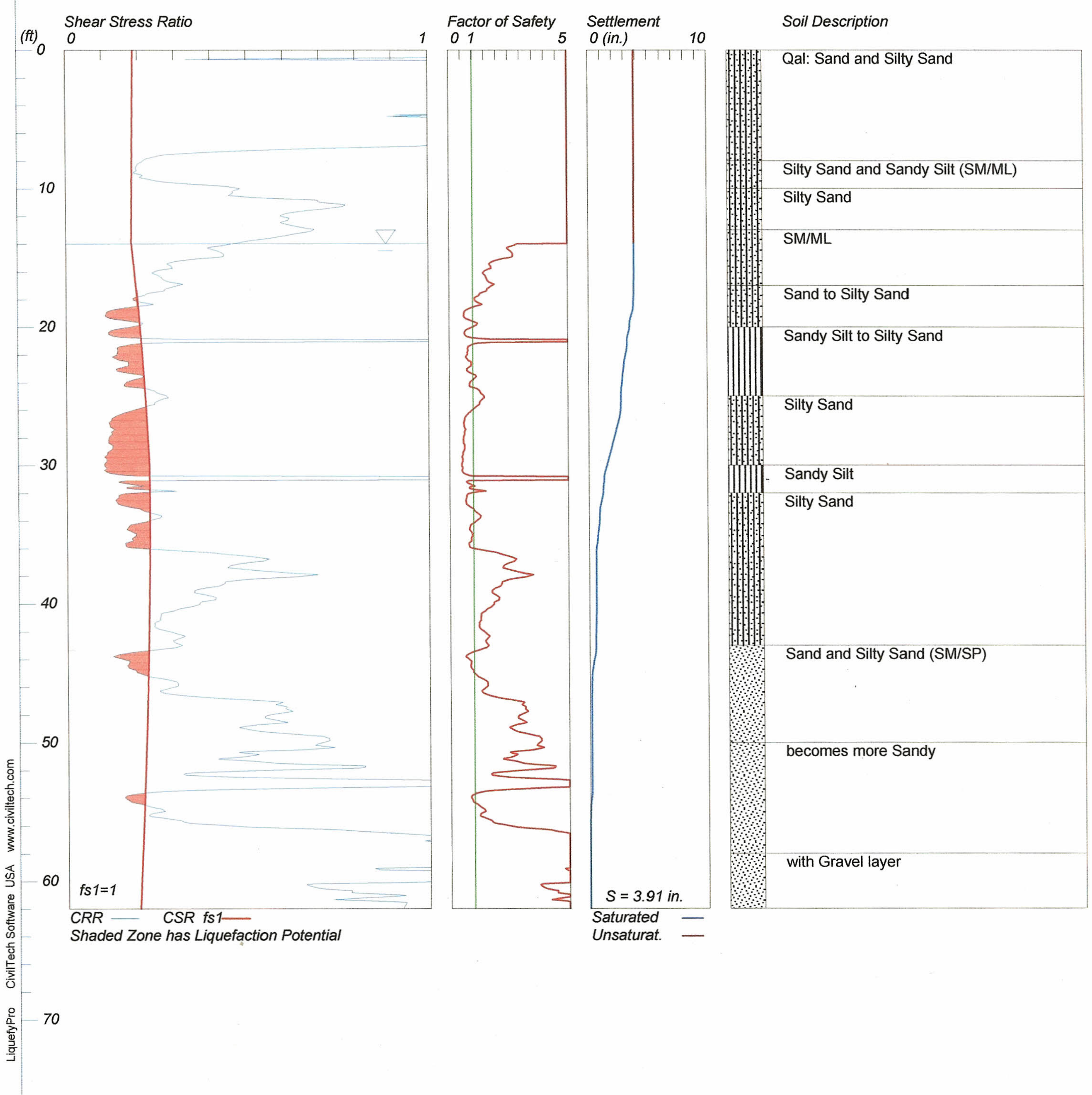
Vessels Ranch CPT 2

Hole No.=CPT 2 Water Depth=14 ft Surface Elev.=201

Magnitude=7.1

Ground Improvement of Fill=5 ft

Acceleration=0.29g



5' Cavega 0-29

Seismic Vertical Deformation Analysis

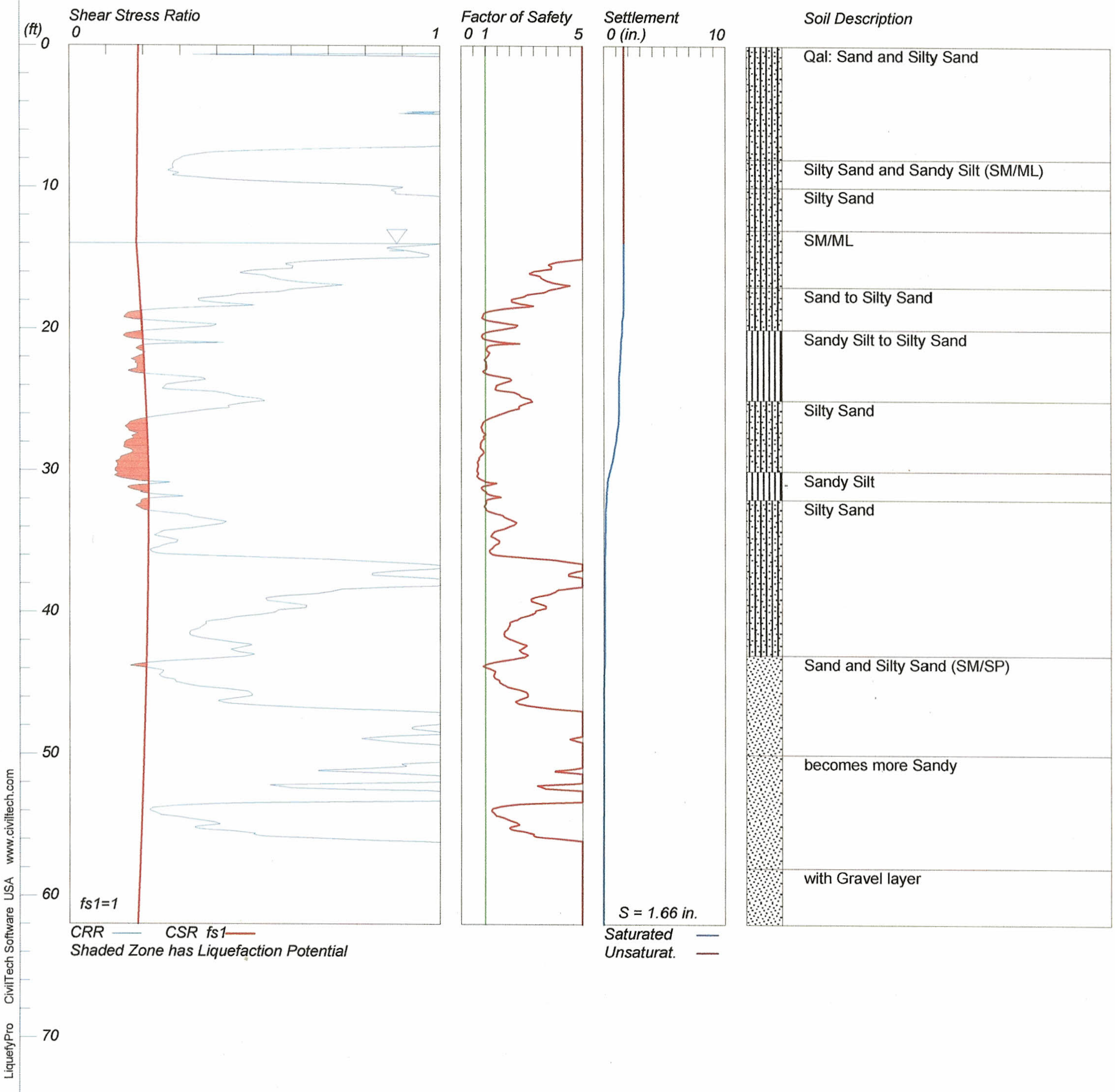
Vessels Ranch CPT 2

Hole No.=CPT 2 Water Depth=14 ft Surface Elev.=201

Ground Improvement of Fill=15 ft

Magnitude=7.1

Acceleration=0.29g



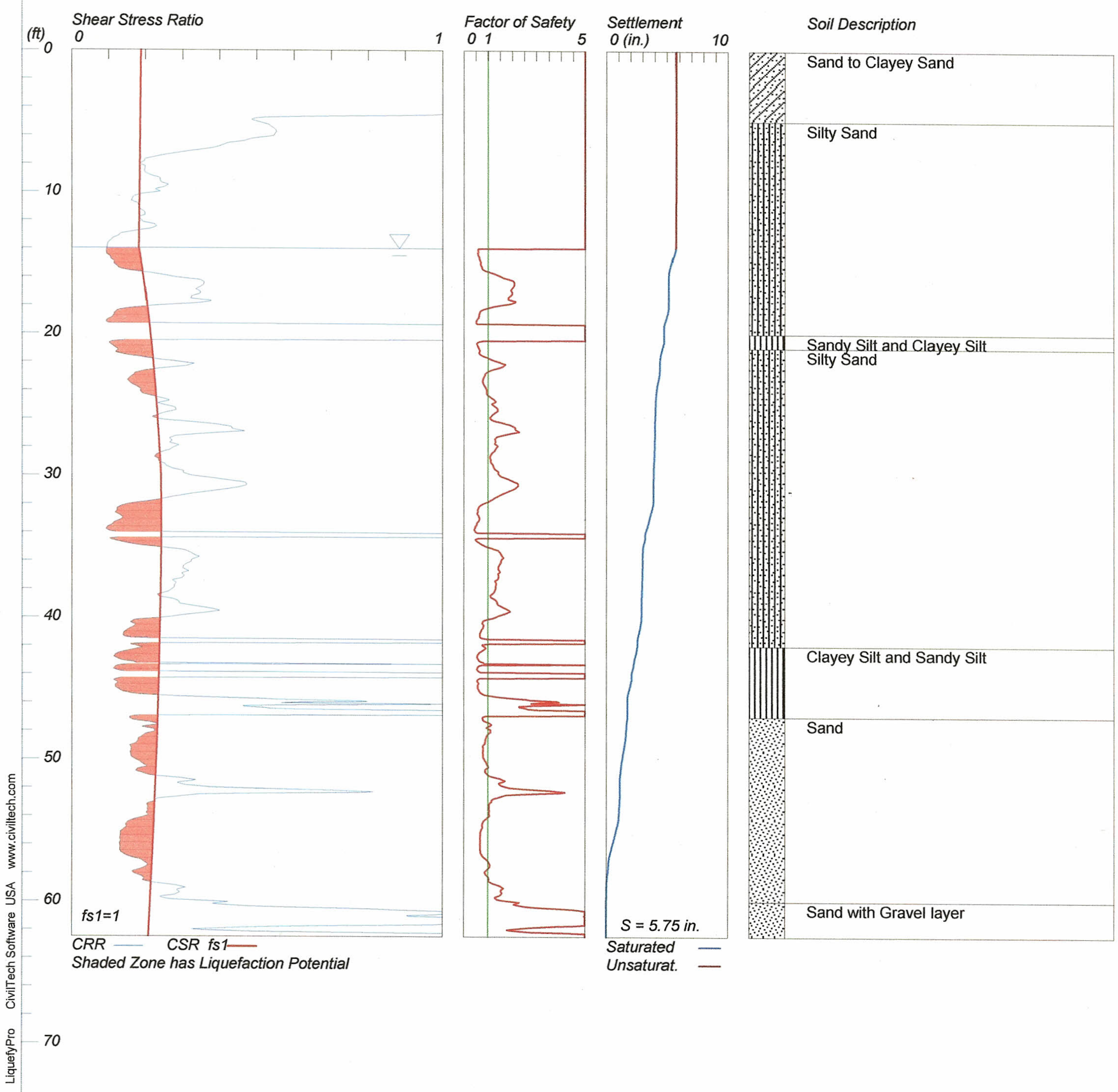
15' cover 0.29

Seismic Vertical Deformation Analysis

Vessels Ranch CPT 3

Hole No.=CPT 3 Water Depth=14 ft Surface Elev.=206

Magnitude=7.1
Acceleration=0.29g



Seismic Vertical Deformation Analysis

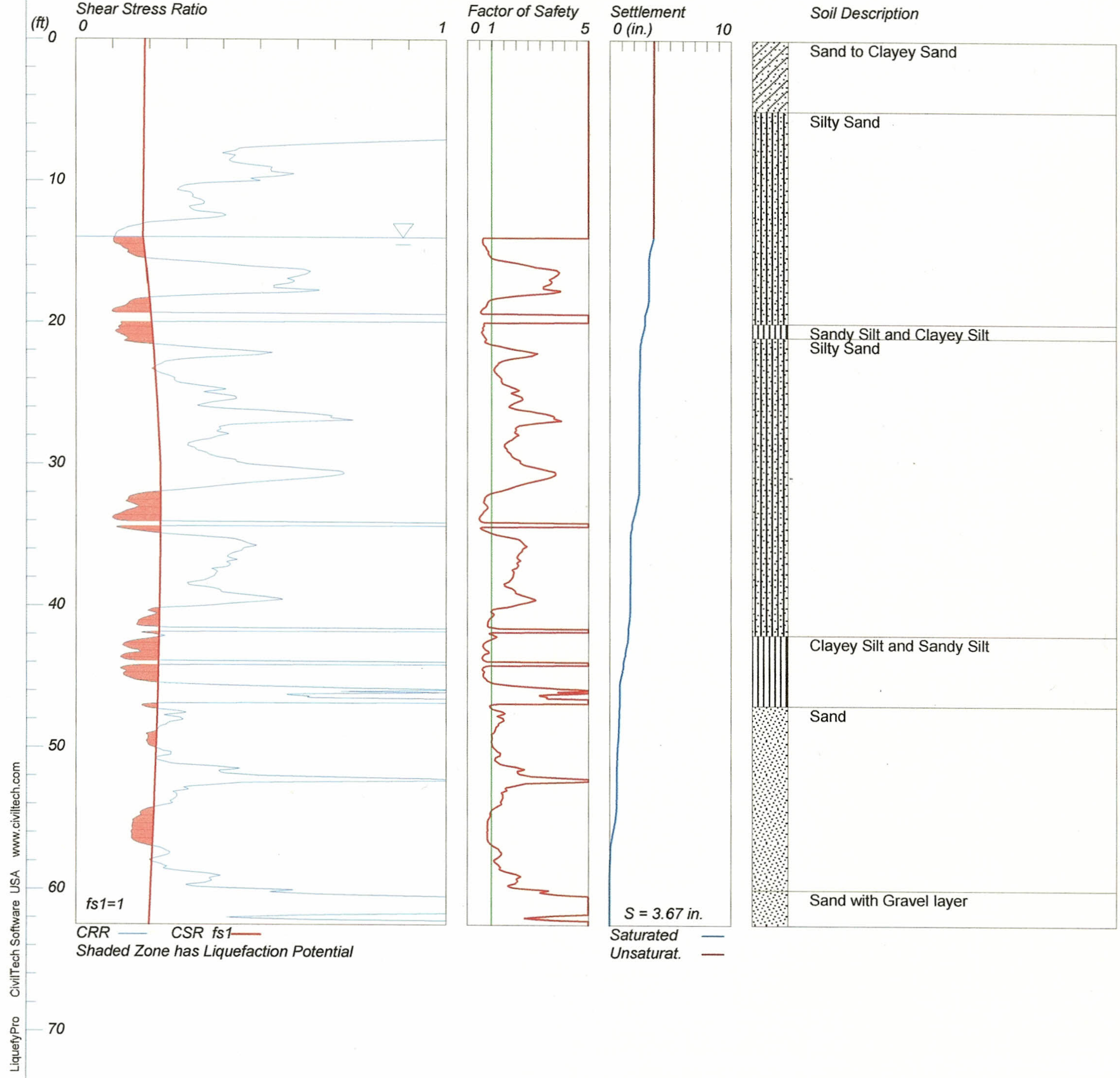
Vessels Ranch CPT 3

Hole No.=CPT 3 Water Depth=14 ft Surface Elev.=206

Ground Improvement of Fill=5 ft

Magnitude=7.1

Acceleration=0.29g



Seismic Vertical Deformation Analysis

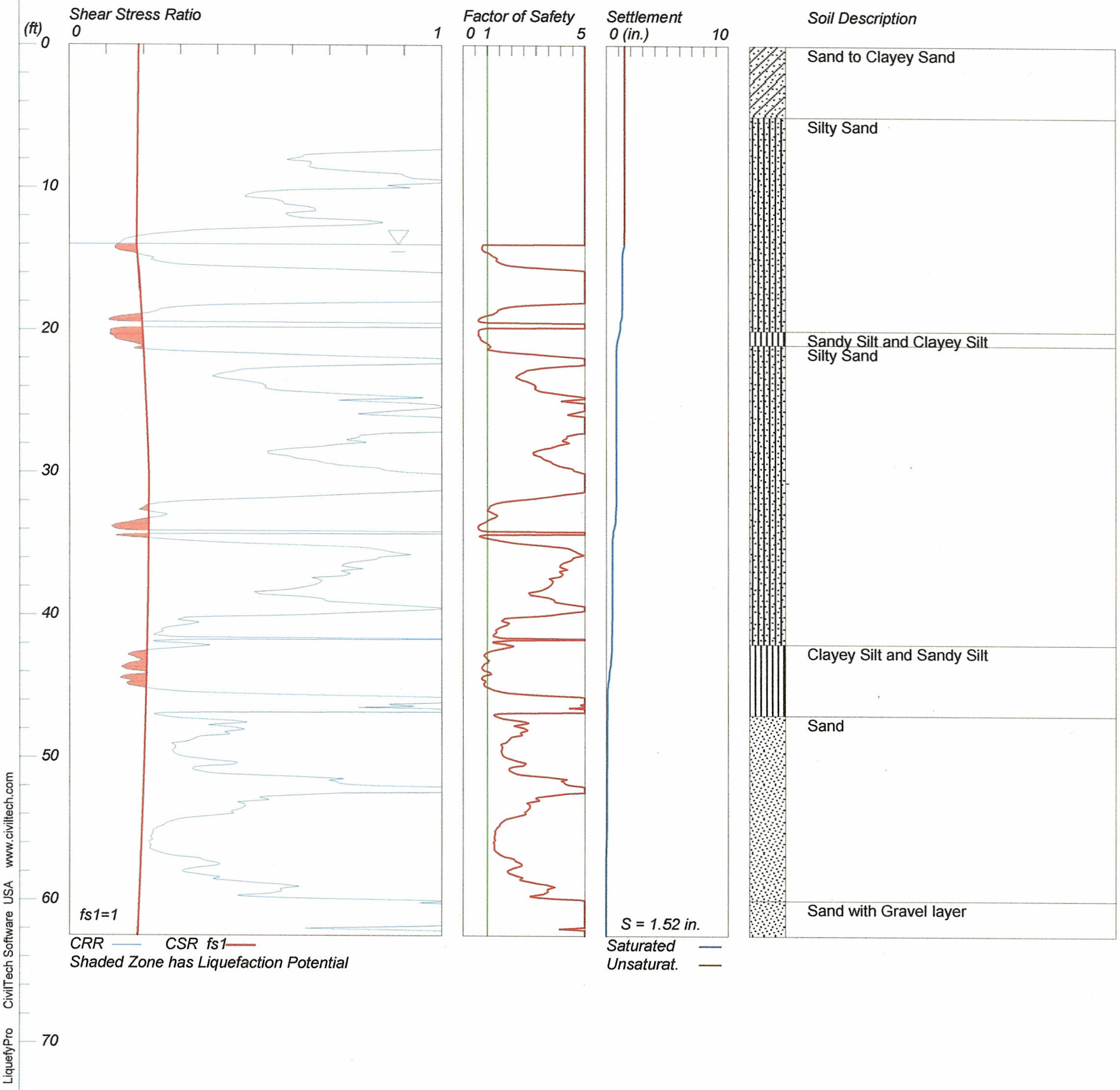
Vessels Ranch CPT 3

Hole No.=CPT 3 Water Depth=14 ft Surface Elev.=206

Ground Improvement of Fill=15 ft

Magnitude=7.1

Acceleration=0.29g

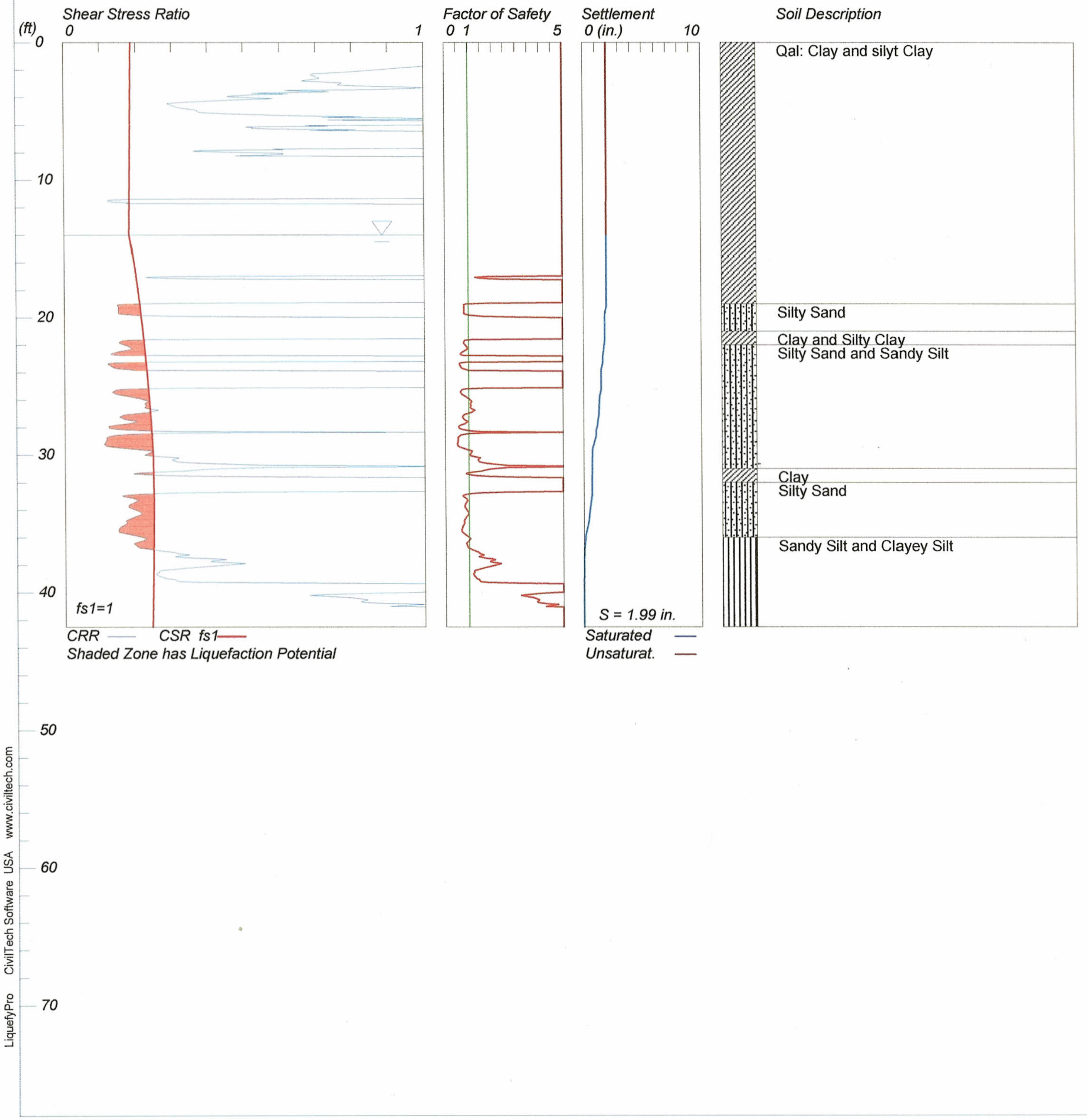


Seismic Vertical Deformation Analysis

Vessels Ranch CPT 4

Hole No.=CPT 4 Water Depth=14 ft Surface Elev.=212

Magnitude=7.1
Acceleration=0.29g



No Cover 0.29

Seismic Vertical Deformation Analysis

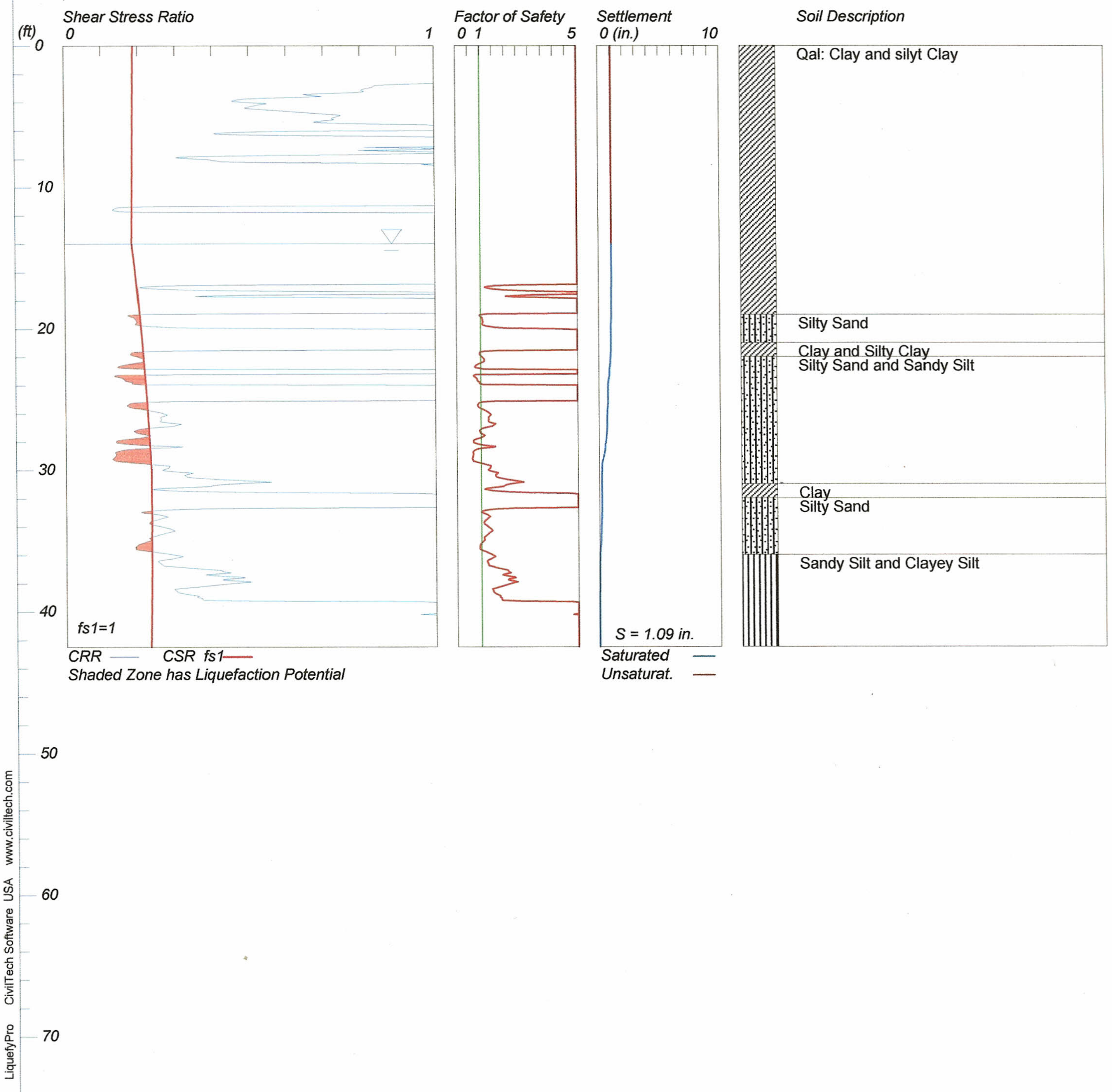
Vessels Ranch CPT 4

Hole No.=CPT 4 Water Depth=14 ft Surface Elev.=212

Ground Improvement of Fill=5 ft

Magnitude=7.1

Acceleration=0.29g



LiquefyPro CivilTech Software USA www.civiltech.com

GeoSoils, Inc. **W.O. 6688-A-SC** **PLATE F-14**

Seismic Vertical Deformation Analysis

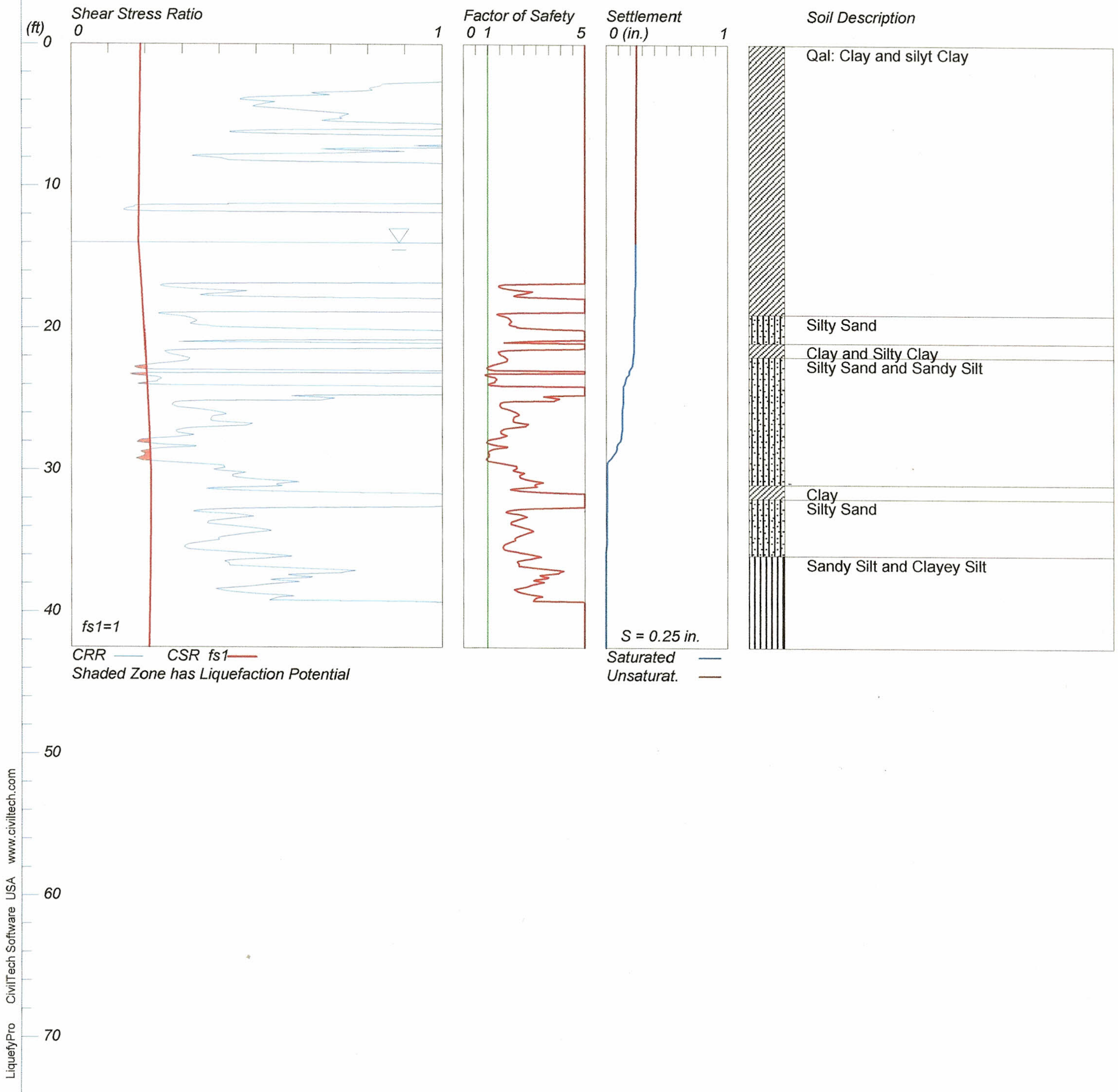
Vessels Ranch CPT 4

Hole No.=CPT 4 Water Depth=14 ft Surface Elev.=212

Ground Improvement of Fill=15 ft

Magnitude=7.1

Acceleration=0.29g



APPENDIX G

GENERAL EARTHWORK, GRADING GUIDELINES AND PRELIMINARY CRITERIA

GENERAL EARTHWORK, GRADING GUIDELINES, AND PRELIMINARY CRITERIA

General

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

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

EARTHWORK OBSERVATIONS AND TESTING

Geotechnical Consultant

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

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

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

Laboratory and Field Tests

Maximum dry density tests to determine the degree of compaction should be performed in accordance with American Standard Testing Materials test method ASTM designation

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

Contractor's Responsibility

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

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

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

SITE PREPARATION

All major vegetation, including brush, trees, thick grasses, organic debris, and other deleterious material, should be removed and disposed of off-site. These removals must be 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

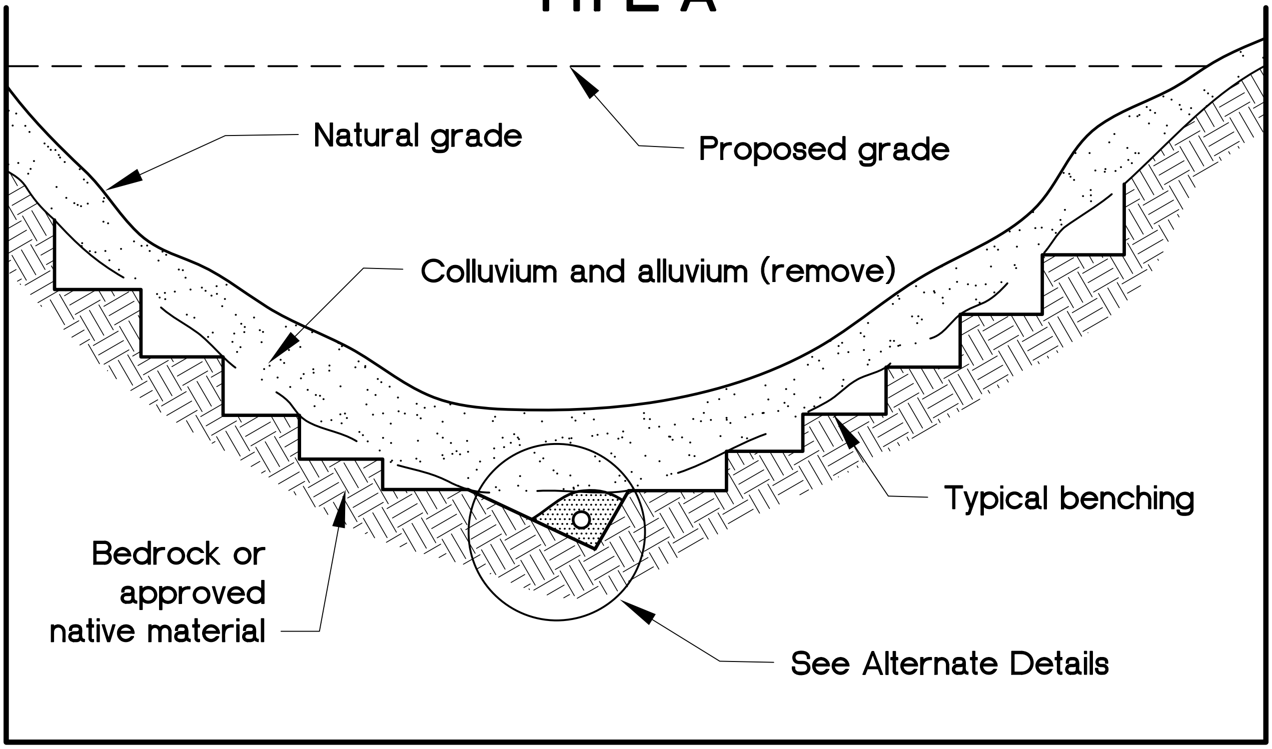
It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Our personnel are directed not to enter any excavation or vertical cut which: 1) is 5 feet or deeper unless shored or laid back; 2) displays any evidence of instability, has any loose rock or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

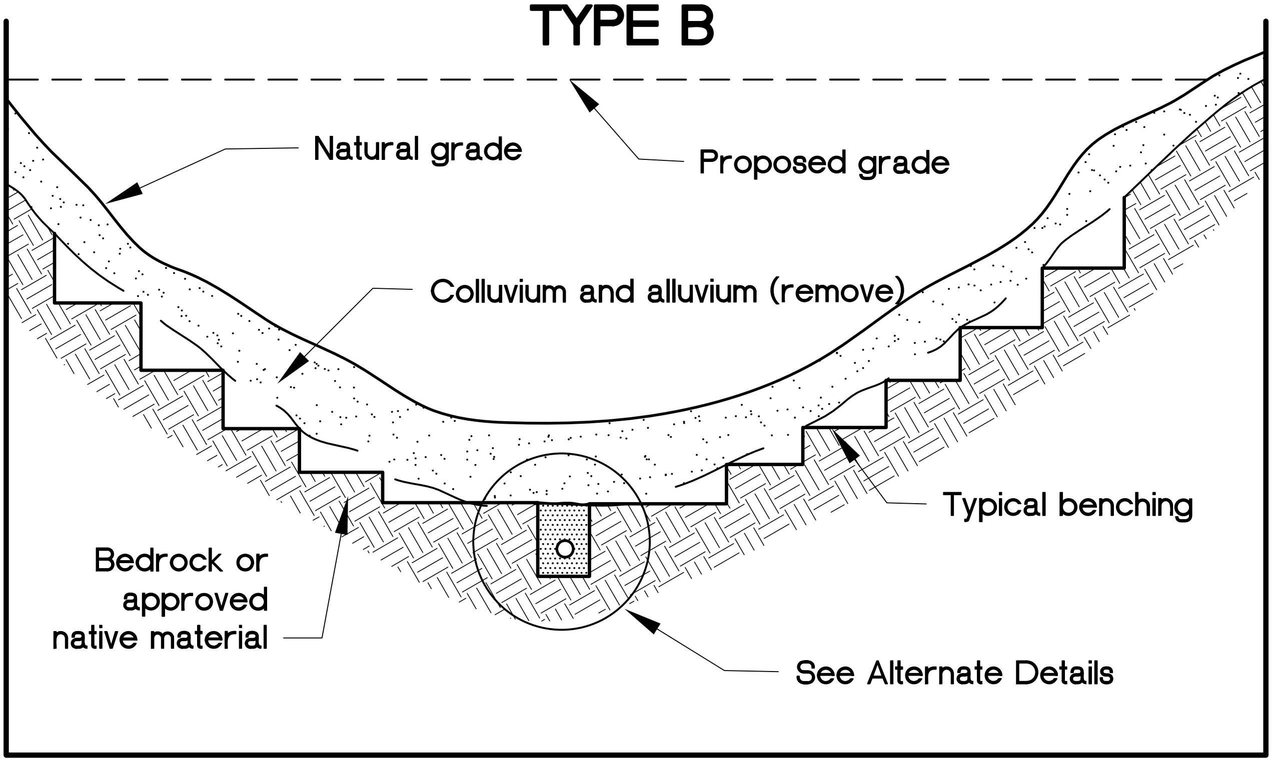
If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraw and notify his/her supervisor. The contractor's representative will be contacted in an effort to affect a solution. All backfill not tested due to safety concerns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

TYPE A

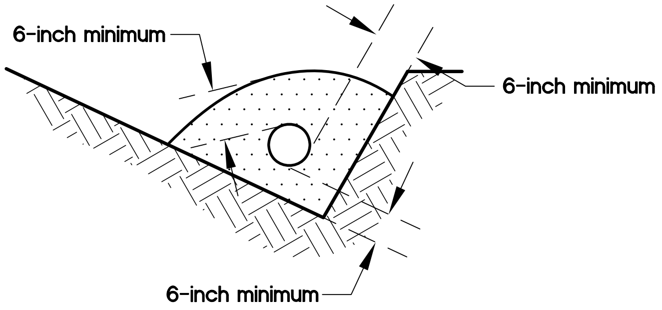


TYPE B

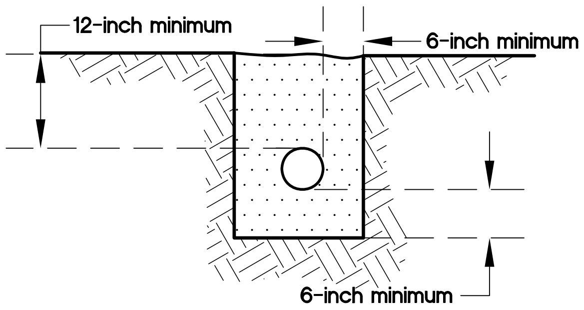


Selection of alternate subdrain details, location, and extent of subdrains should be evaluated by the geotechnical consultant during grading.

GeoSoils, Inc.	CANYON SUBDRAIN DETAIL	Plate G-1



A-1



B-1

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

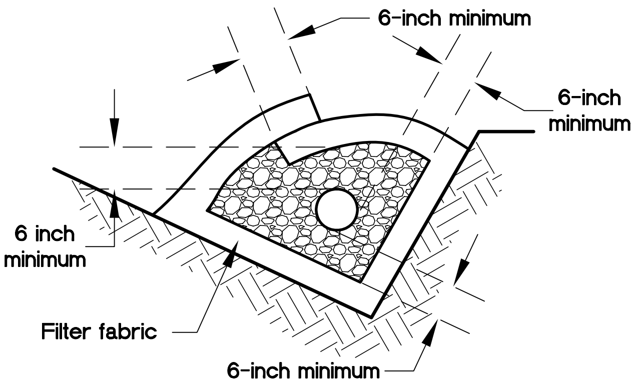
Perforated pipe: 6-inch-diameter ABS or PVC pipe or approved substitute with minimum 8 perforations ($^{1}/_{4}$-inch diameter) per lineal foot in bottom half of pipe (ASTM D-2751, SDR-35, or ASTM D-1527, Schd. 40).

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

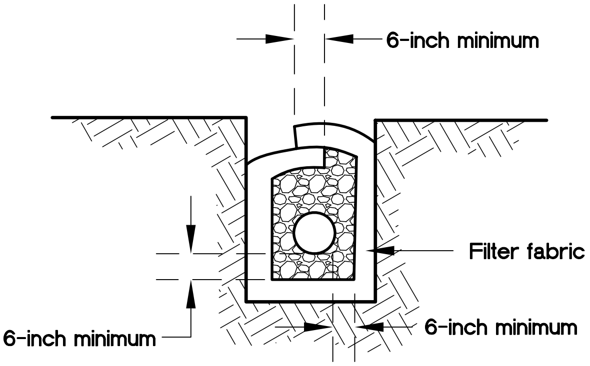
FILTER MATERIAL

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

ALTERNATE 1: PERFORATED PIPE AND FILTER MATERIAL



A-2



B-2

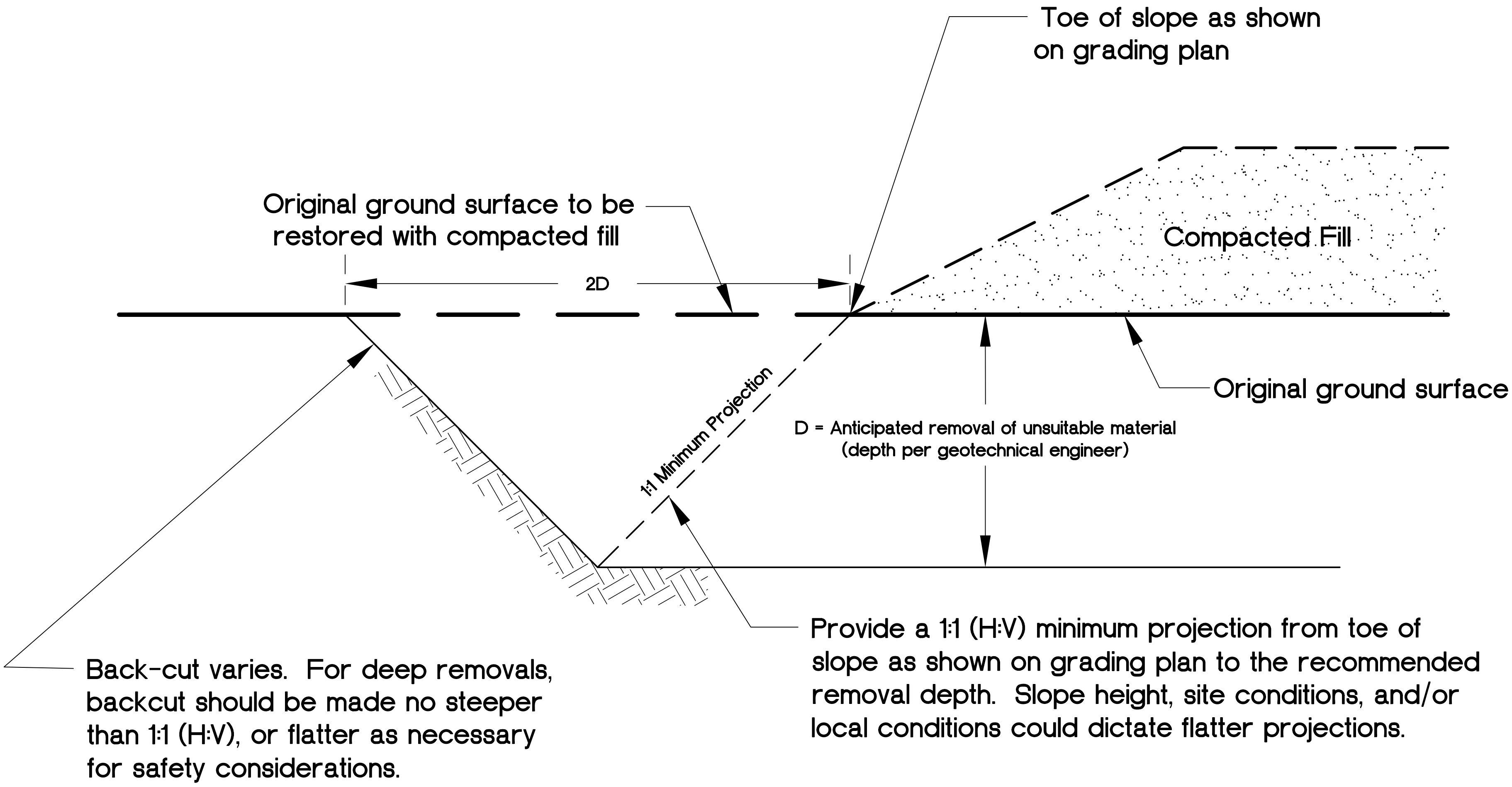
Gravel Material: 9 cubic feet per lineal foot.
Perforated Pipe: See Alternate 1
Gravel: Clean $^{3}/_{4}$-inch rock or approved substitute.
Filter Fabric: Mirafi 140 or approved substitute.

ALTERNATE 2: PERFORATED PIPE, GRAVEL, AND FILTER FABRIC

GeoSoils, Inc.

CANYON SUBDRAIN ALTERNATE DETAILS

Plate G-2



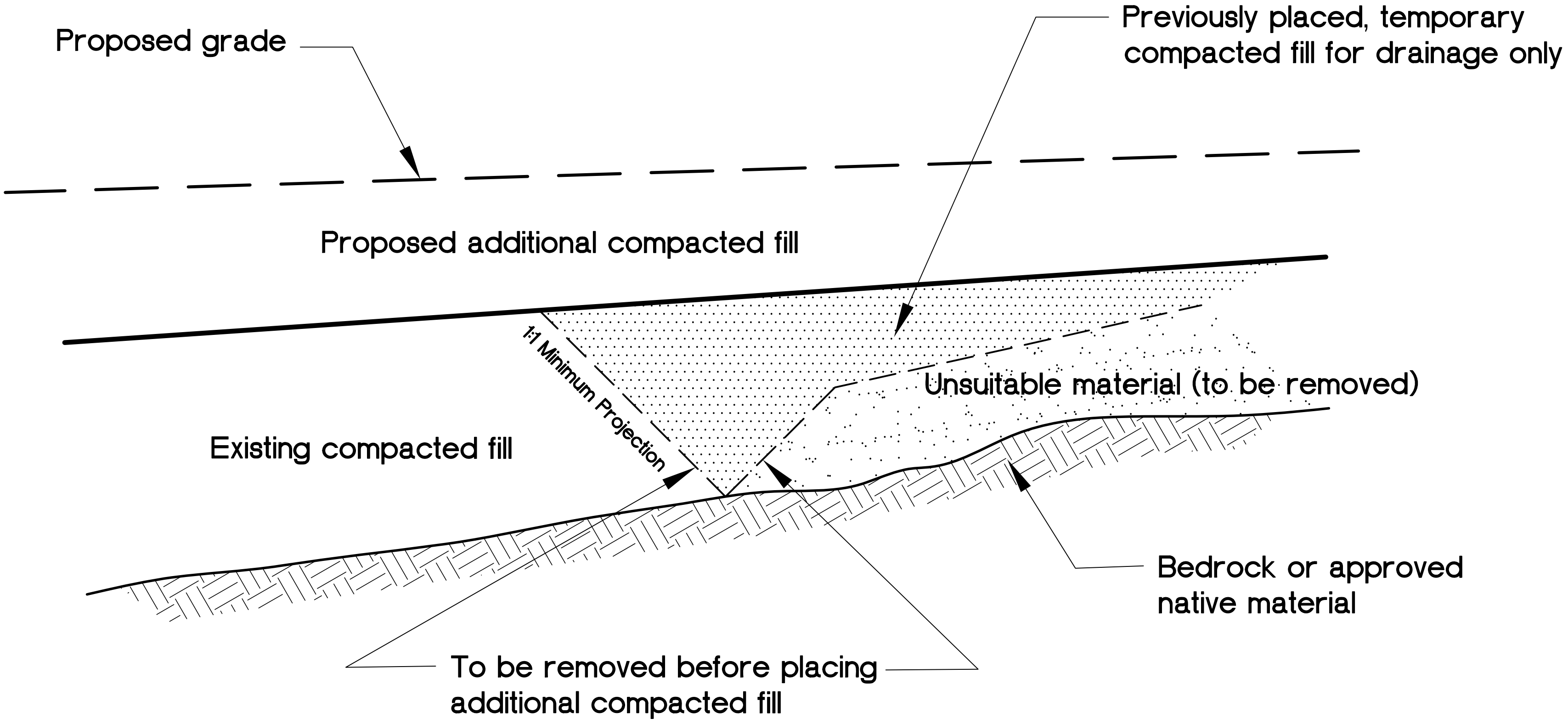
Back-cut varies. For deep removals, backcut should be made no steeper than 1:1 (H:V), or flatter as necessary for safety considerations.

Provide a 1:1 (H:V) minimum projection from toe of slope as shown on grading plan to the recommended removal depth. Slope height, site conditions, and/or local conditions could dictate flatter projections.

GeoSoils, Inc.

FILL SLOPE TOEING OUT ON FLAT ALLUVIATED CANYON DETAIL

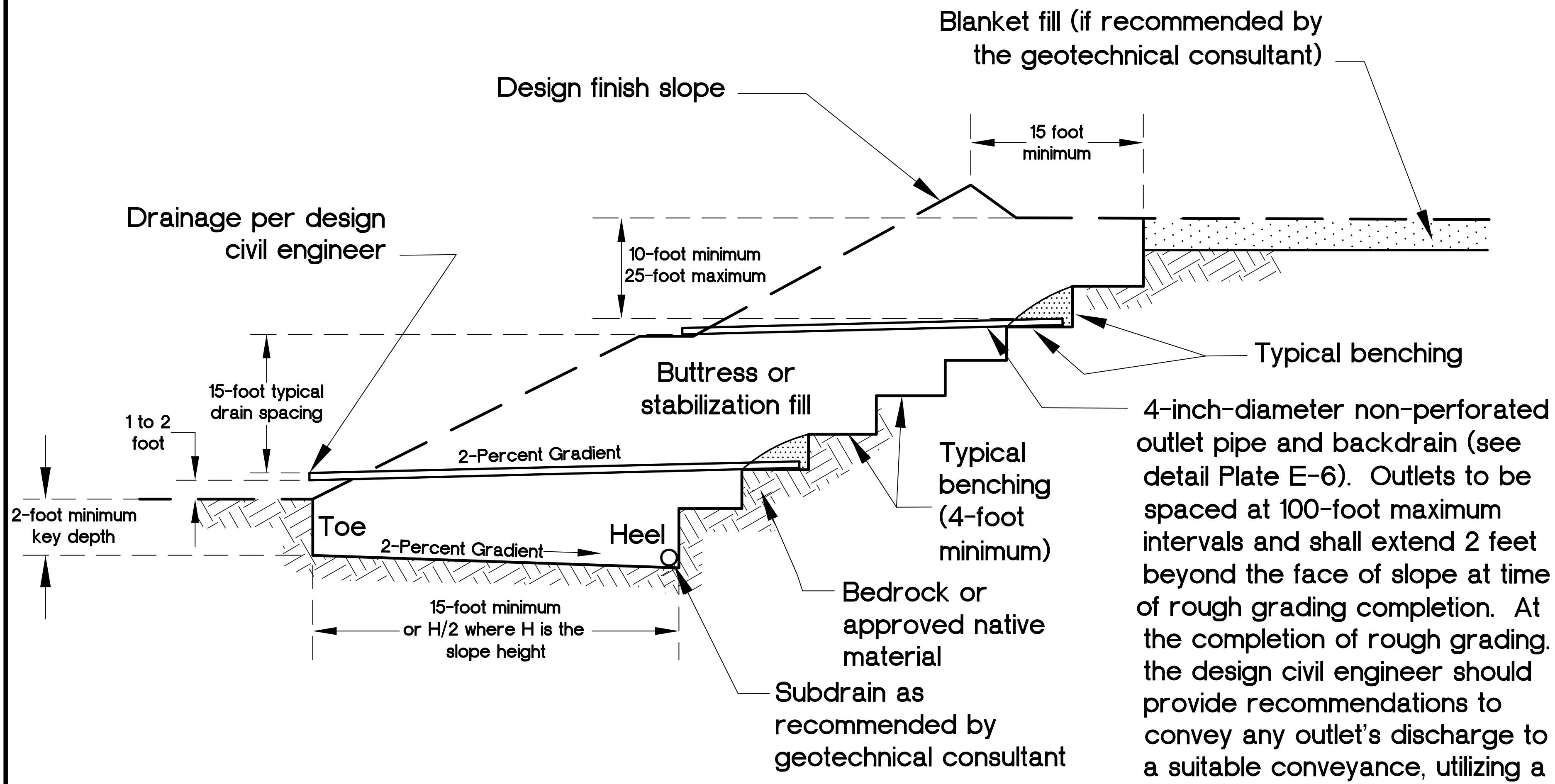
Plate G-3



GeoSoils, Inc.

REMOVAL ADJACENT TO EXISTING FILL ADJOINING CANYON FILL DETAIL

Plate G-4

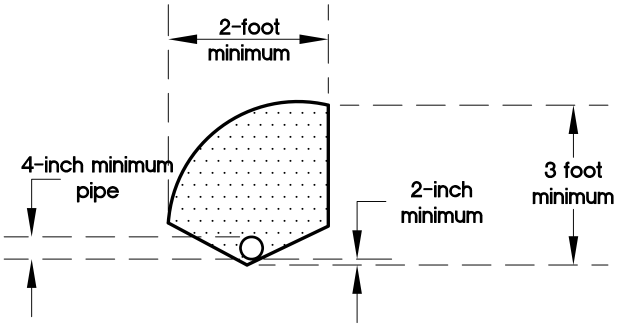


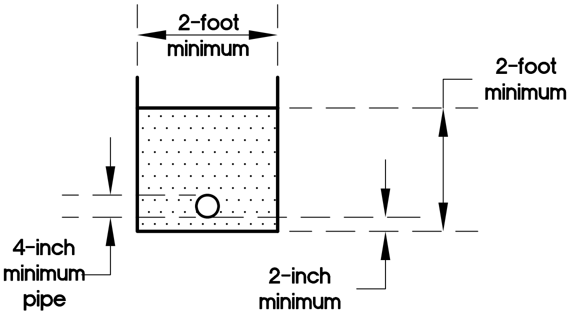
4-inch-diameter non-perforated outlet pipe and backdrain (see detail Plate E-6). Outlets to be spaced at 100-foot maximum intervals and shall extend 2 feet beyond the face of slope at time of rough grading completion. At the completion of rough grading. the design civil engineer should provide recommendations to convey any outlet's discharge to a suitable conveyance, utilizing a non-erosive device.

GeoSoils, Inc.

TYPICAL STABILIZATION / BUTTRESS FILL DETAIL

Plate G-5





<u>Filter Material</u>: 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.

<u>Alternative in Lieu of Filter Material</u>: 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.

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

Notes: 1. Trench for outlet pipes to be backfilled and compacted with onsite soil.

2. Backdrains and lateral drains shall be located at elevation of every bench drain. First drain located at elevation just above lower lot grade. Additional drains may be required at the discretion of the geotechnical consultant.

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

Sieve Size	Percent Passing
1 inch	100
¾ inch	90-100
⅜ inch	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

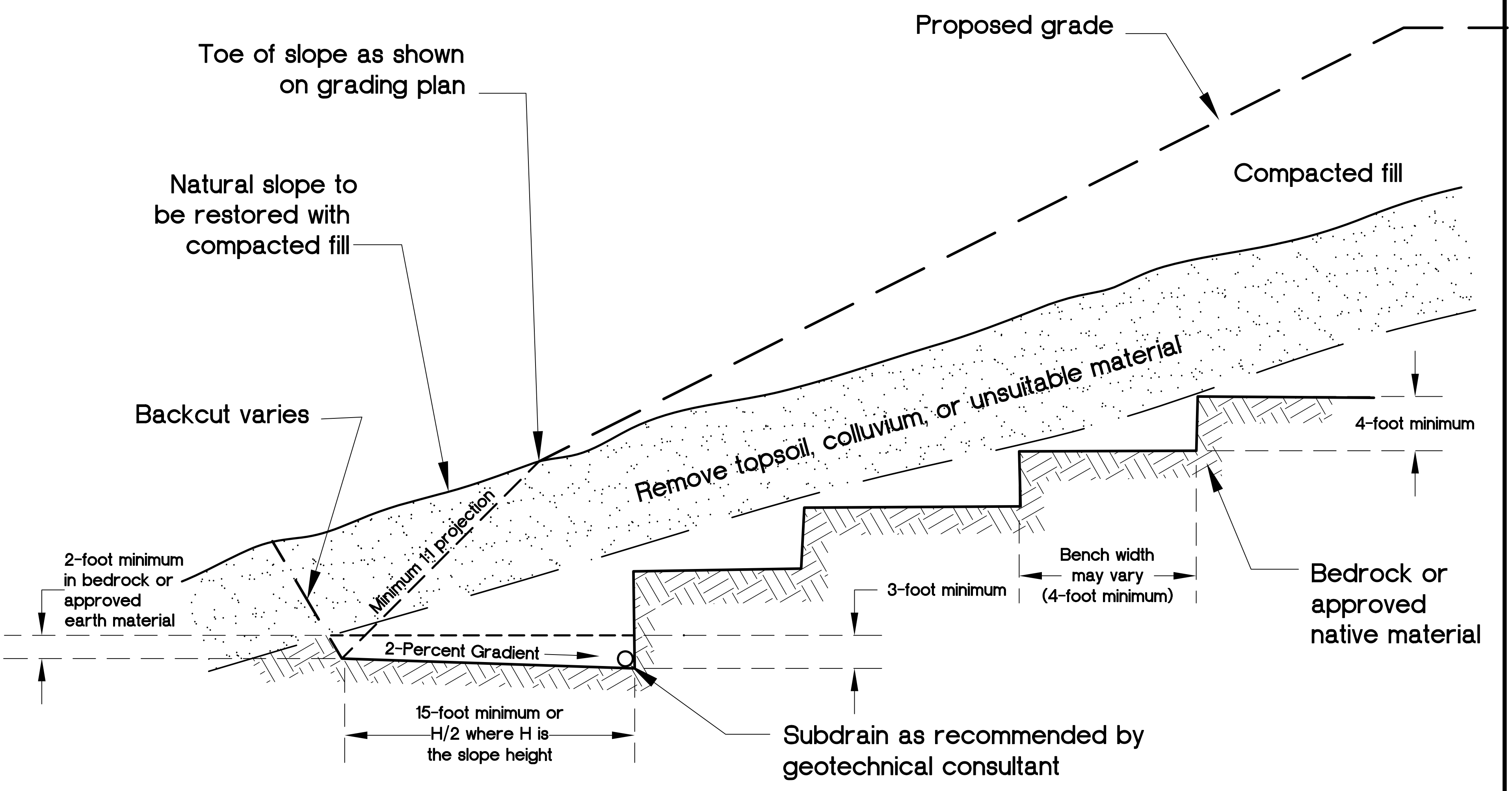
Gravel shall be of the following specification or an approved equivalent.

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

GeoSoils, Inc.

TYPICAL BUTTRESS SUBDRAIN DETAIL

Plate G-6



NOTES:

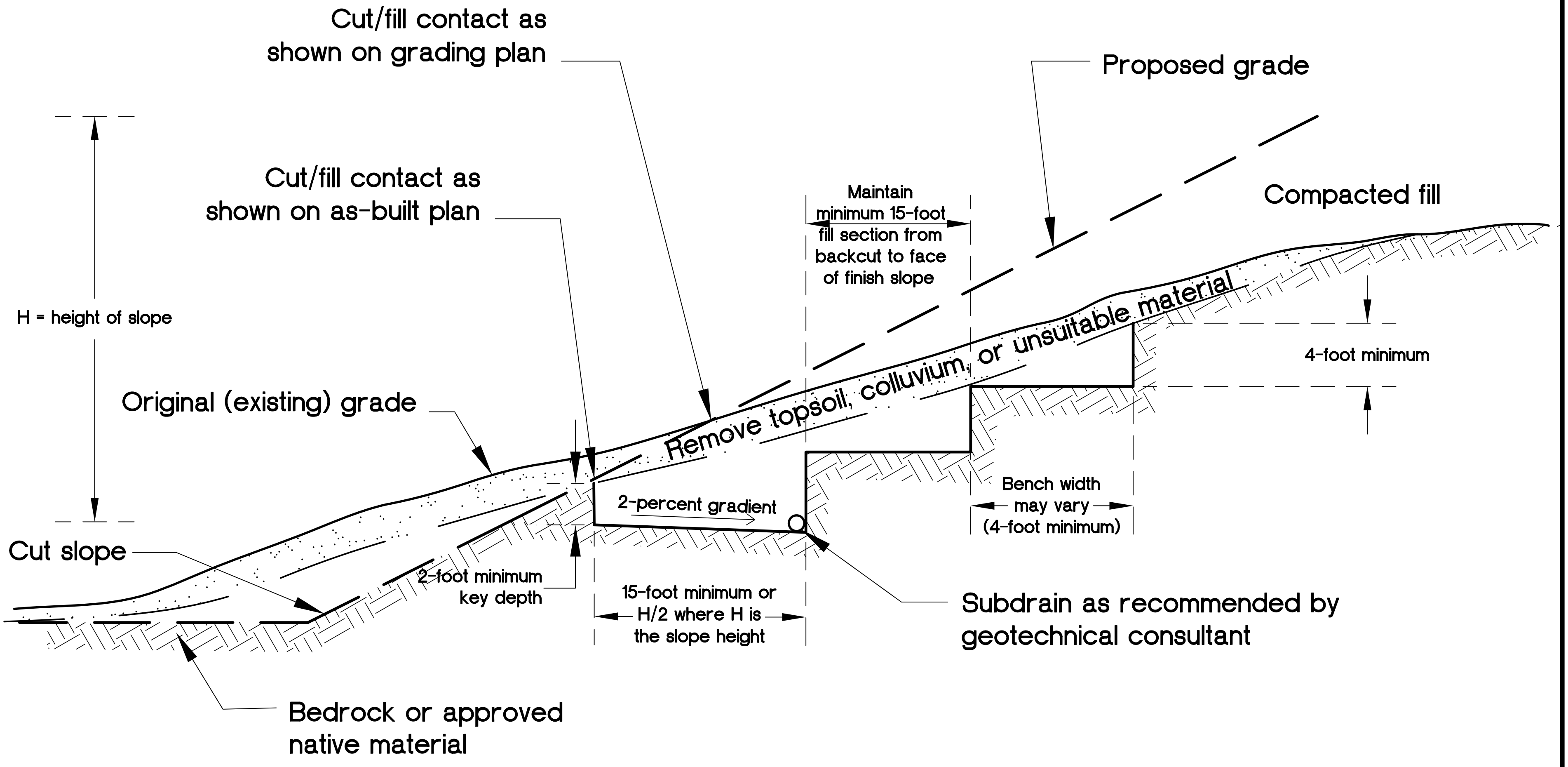
1. Where the natural slope approaches or exceeds the design slope ratio, special recommendations would be provided by the geotechnical consultant.

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

GeoSoils, Inc.

FILL OVER NATURAL (SIDEHILL FILL) DETAIL

Plate G-7

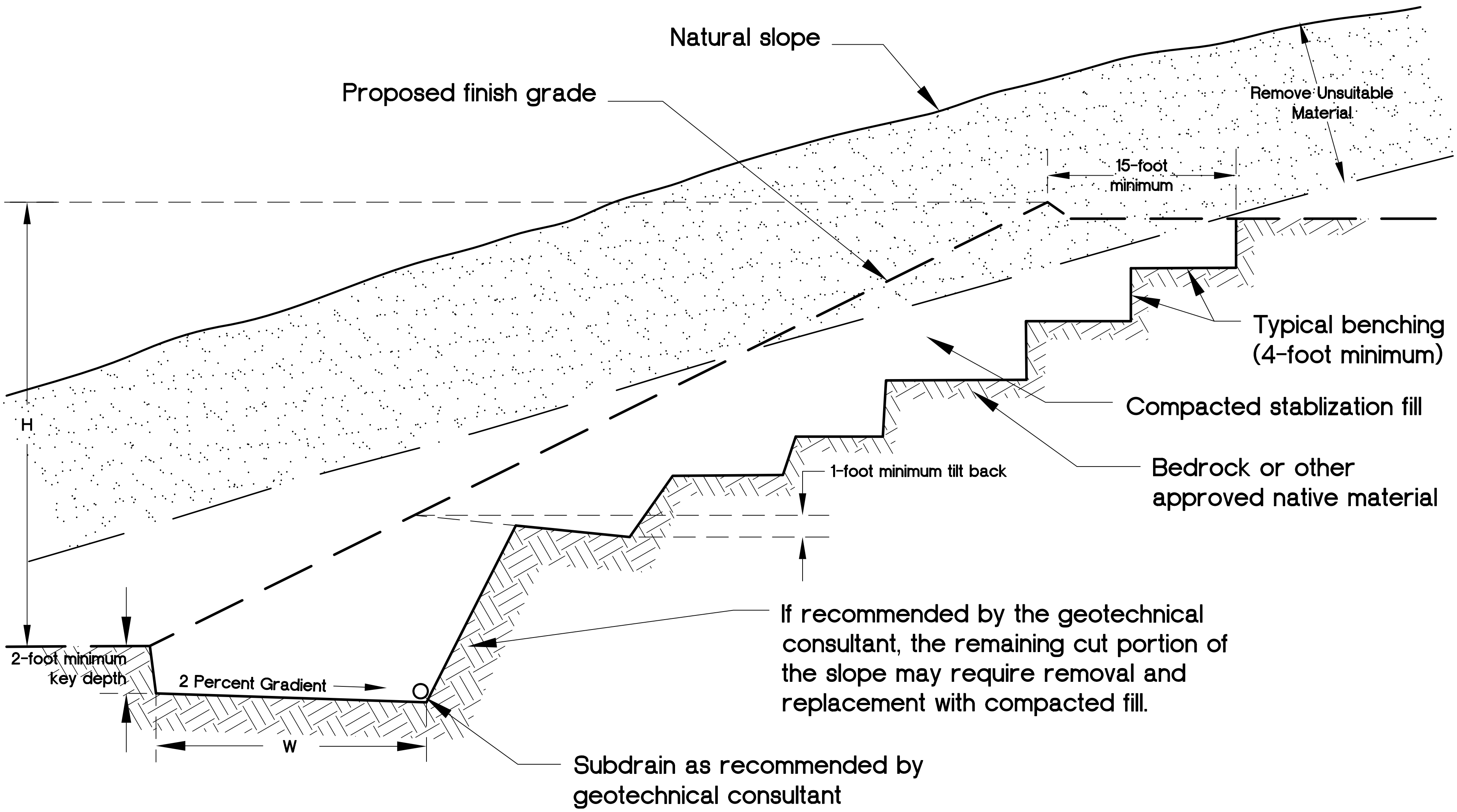


NOTE: The cut portion of the slope should be excavated and evaluated by the geotechnical consultant prior to construction of the fill portion.

GeoSoils, Inc.

FILL OVER CUT DETAIL

Plate G-8



If recommended by the geotechnical consultant, the remaining cut portion of the slope may require removal and replacement with compacted fill.

NOTES:

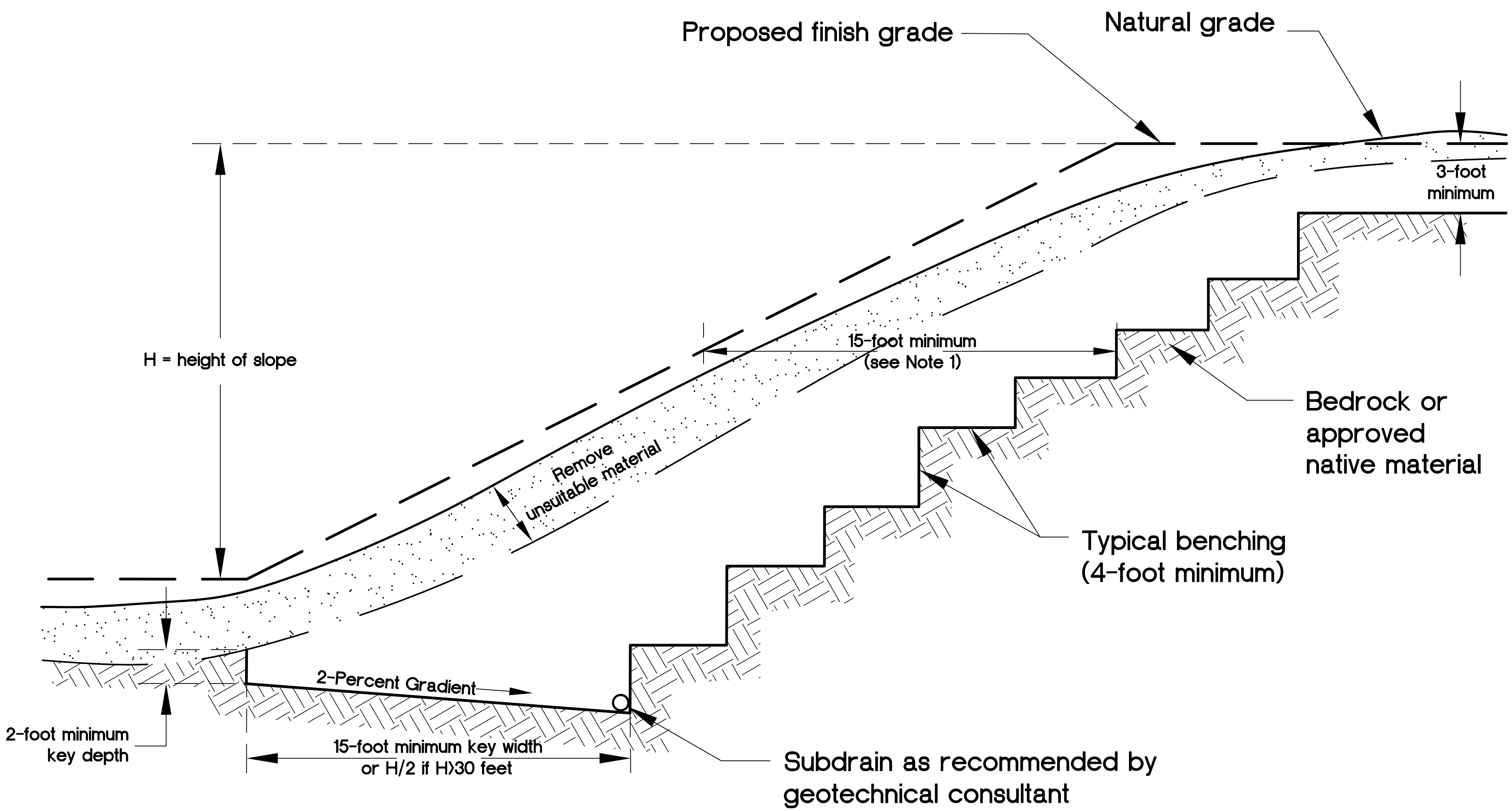
1. Subdrains may be required as specified by the geotechnical consultant.

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

GeoSoils, Inc.

STABLIZATION FILL FOR UNSTABLE MATERIAL EXPOSED IN CUT SLOPE DETAIL

Plate G-9



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

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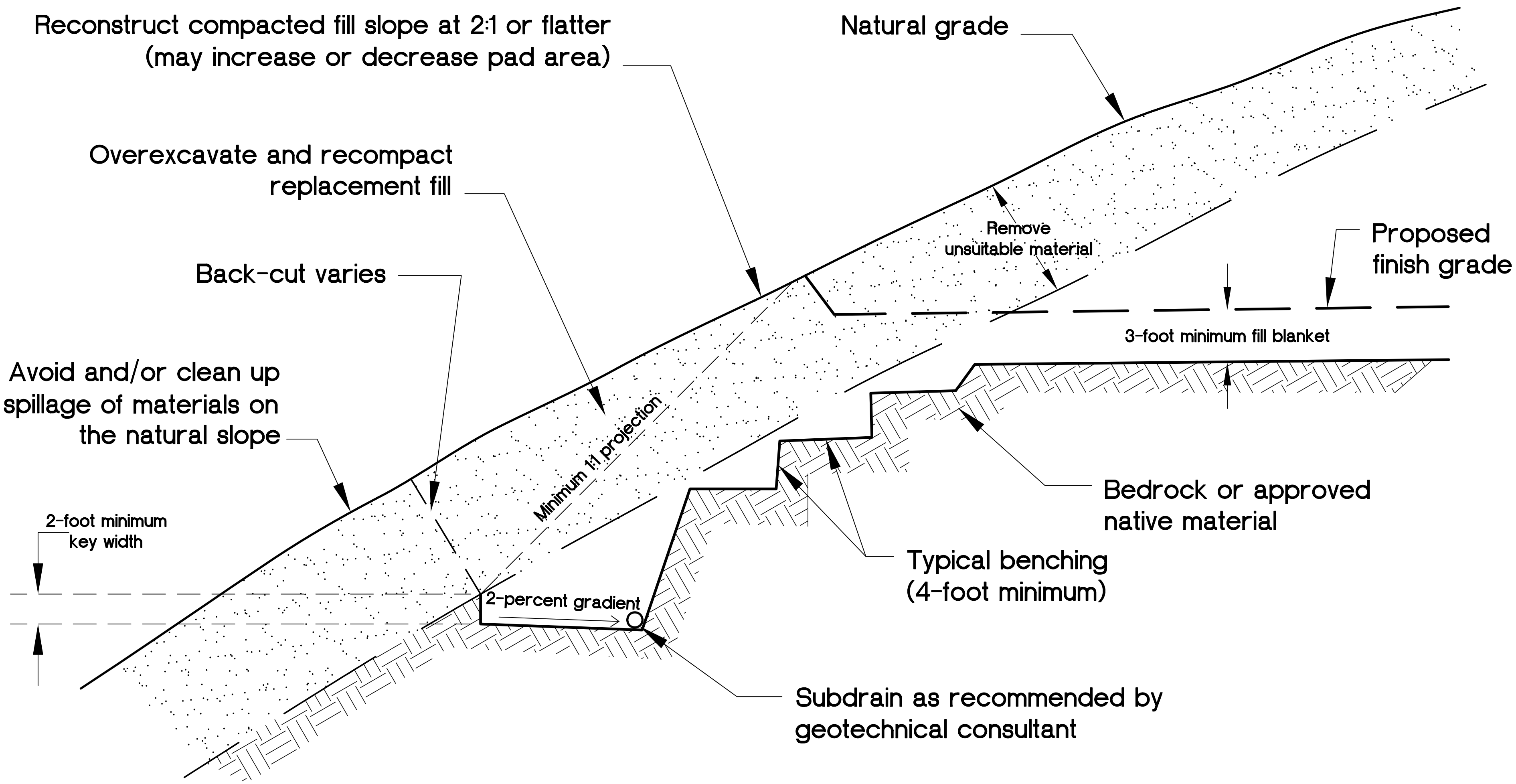
SKIN FILL OF NATURAL GROUND DETAIL

Plate G-10



NOTES:

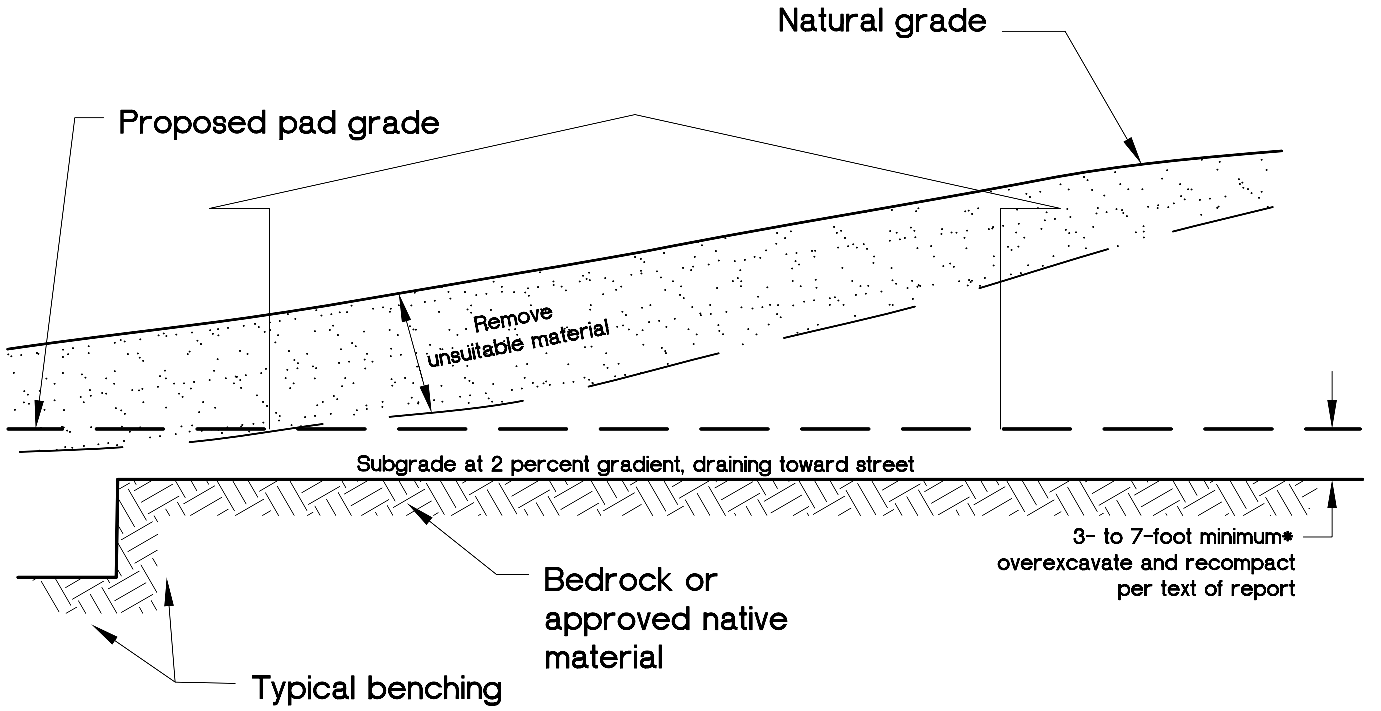
1. Subdrain and key width requirements will be evaluated based on exposed subsurface conditions and thickness of overburden.

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

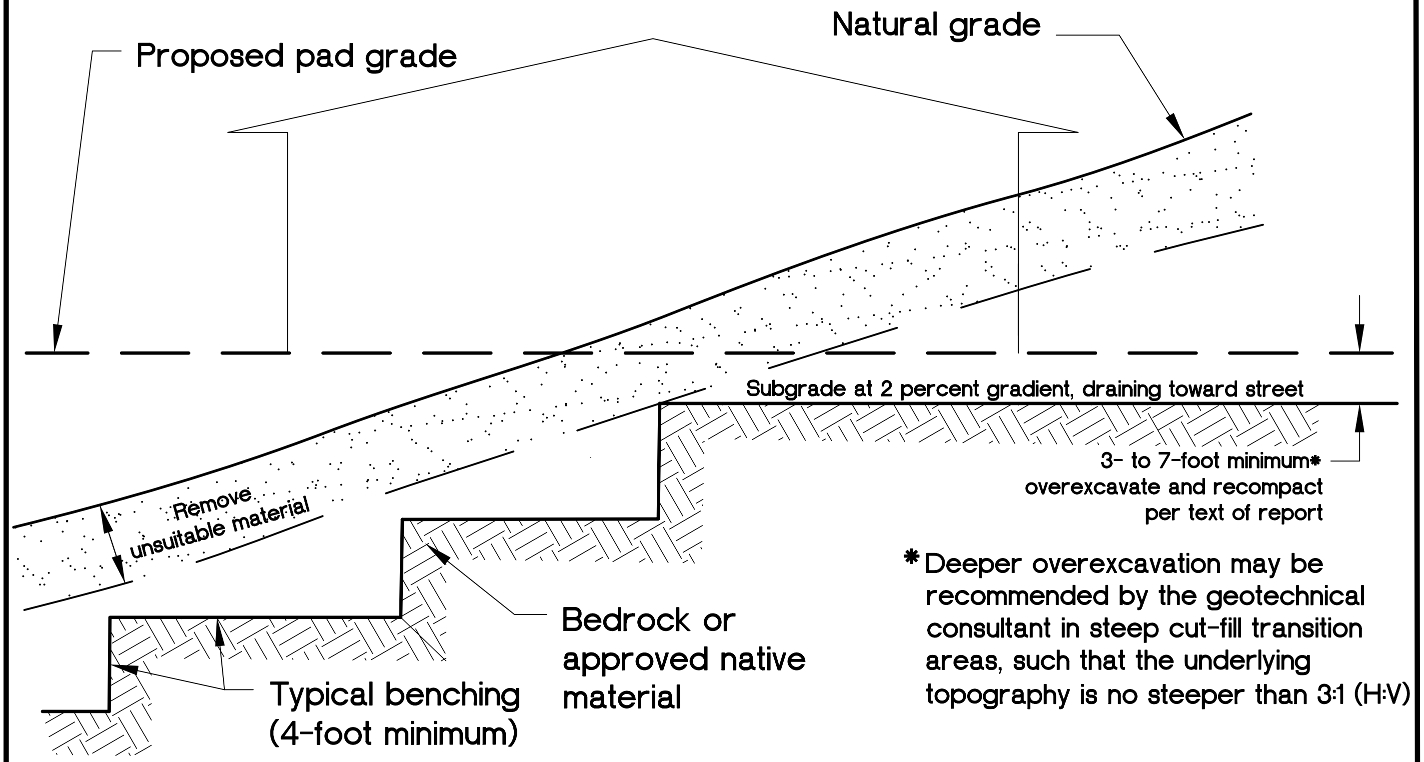
GeoSoils, Inc.

DAYLIGHT CUT LOT DETAIL

Plate G-11



CUT LOT OR MATERIAL-TYPE TRANSITION



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

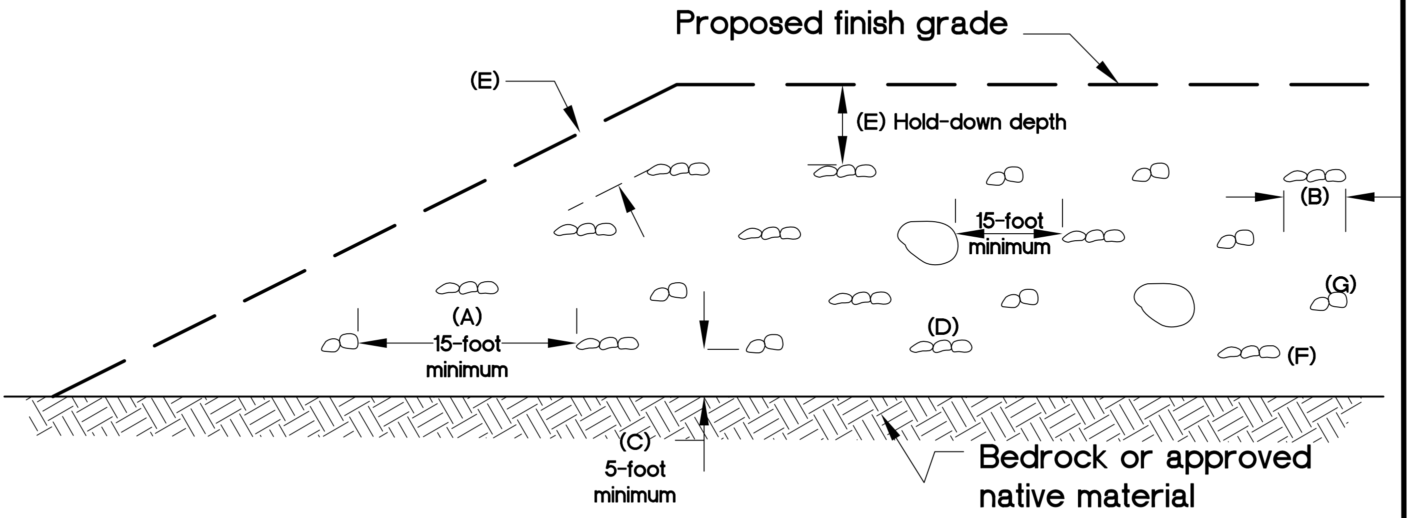
CUT-FILL LOT (DAYLIGHT TRANSITION)

GeoSoils, Inc.

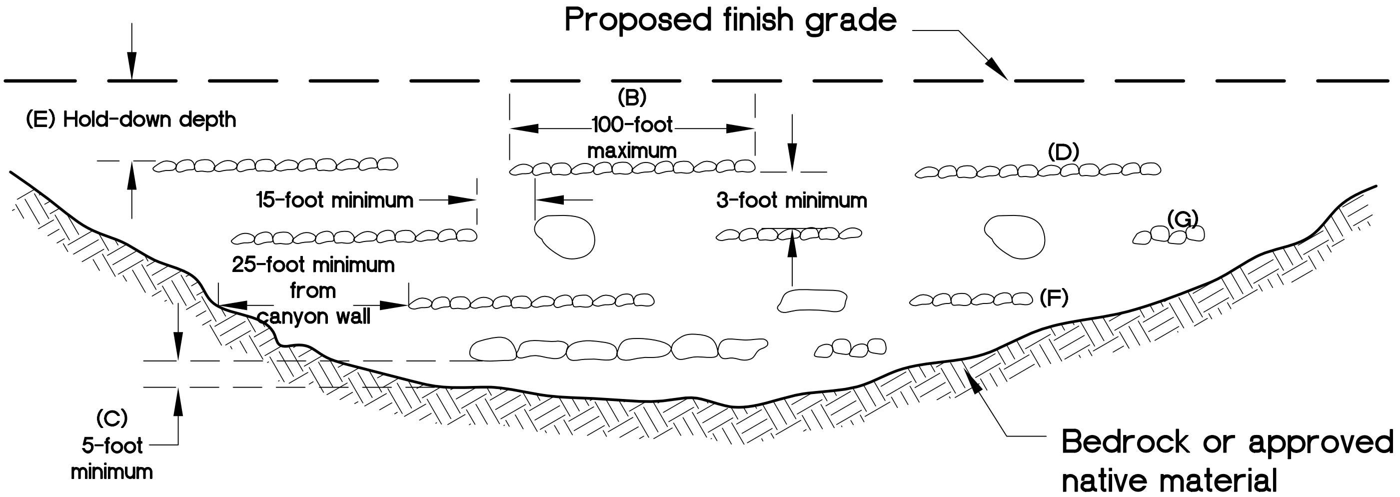
TRANSITION LOT DETAILS

Plate G-12

VIEW NORMAL TO SLOPE FACE



VIEW PARALLEL TO SLOPE FACE



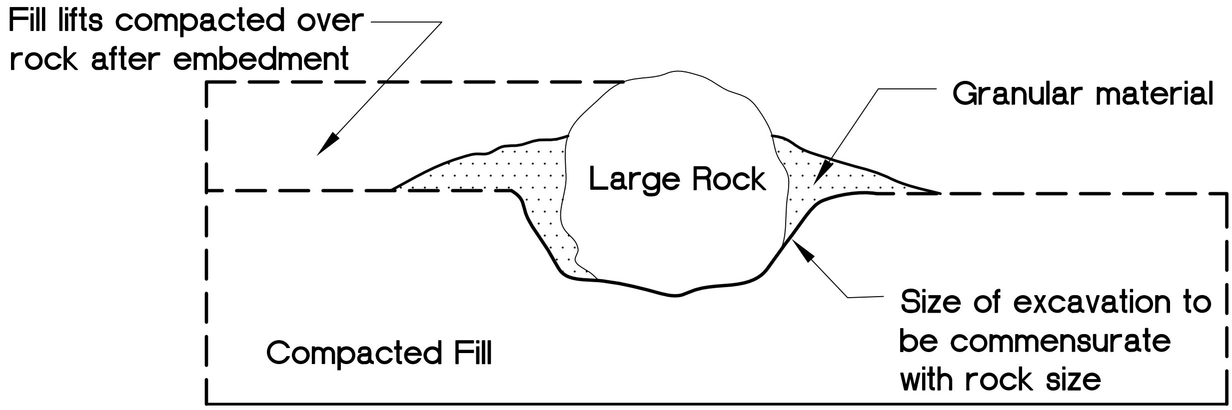
NOTES:

A. One equipment width or a minimum of 15 feet between rows (or windrows).
B. Height and width may vary depending on rock size and type of equipment. Length of windrow shall be no greater than 100 feet.
C. If approved by the geotechnical consultant, windrows may be placed direclty on competent material or bedrock, provided adequate space is available for compaction.
D. Orientation of windrows may vary but should be as recommended by the geotechnical engineer and/or engineering geologist. Staggering of windrows is not necessary unless recommended.
E. Clear area for utility trenches, foundations, and swimming pools; Hold-down depth as specified in text of report, subject to governing agency approval.
F. All fill over and around rock windrow shall be compacted to at leas
ompaction or as recommended.
G. After fill between windrows is placed and compacted, with the lift of fill covering windrow, windrow should be proof rolled with a D-9 dozer or equivalent.

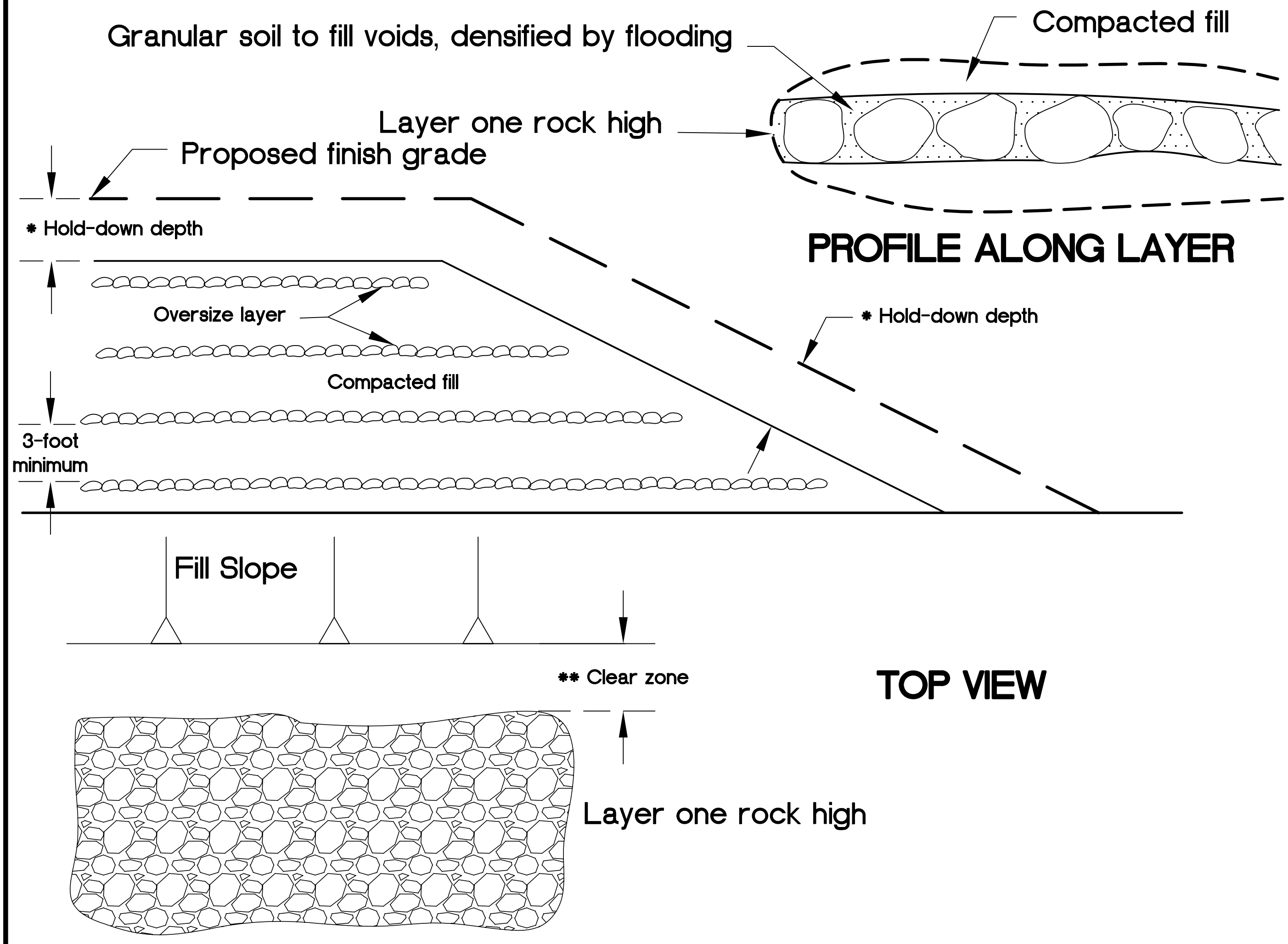
VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE
ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED

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

ROCK DISPOSAL PITS



ROCK DISPOSAL LAYERS

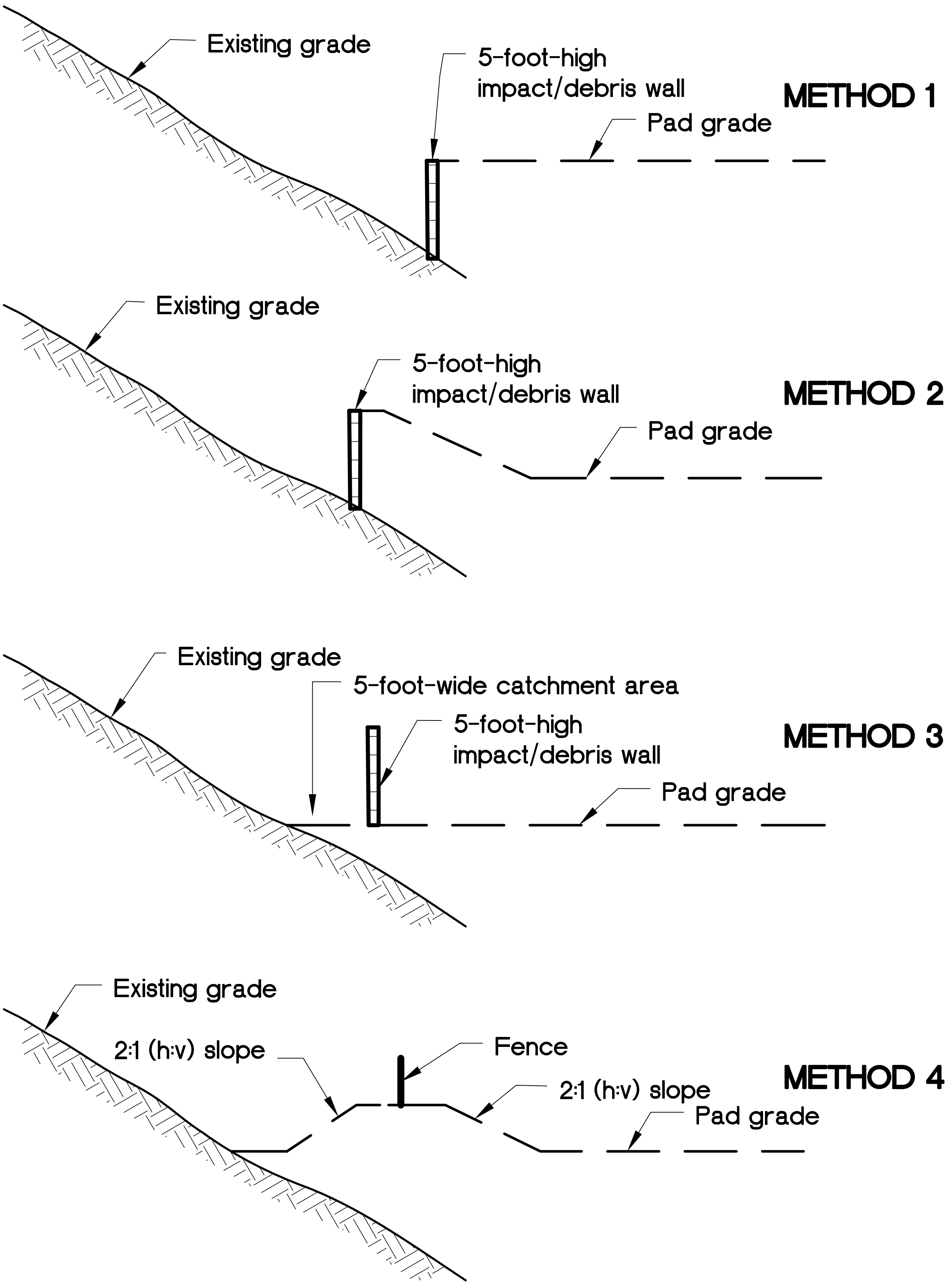


✱ Hold-down depth or below lowest utility as specified in text of report, subject to governing agency approval.

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

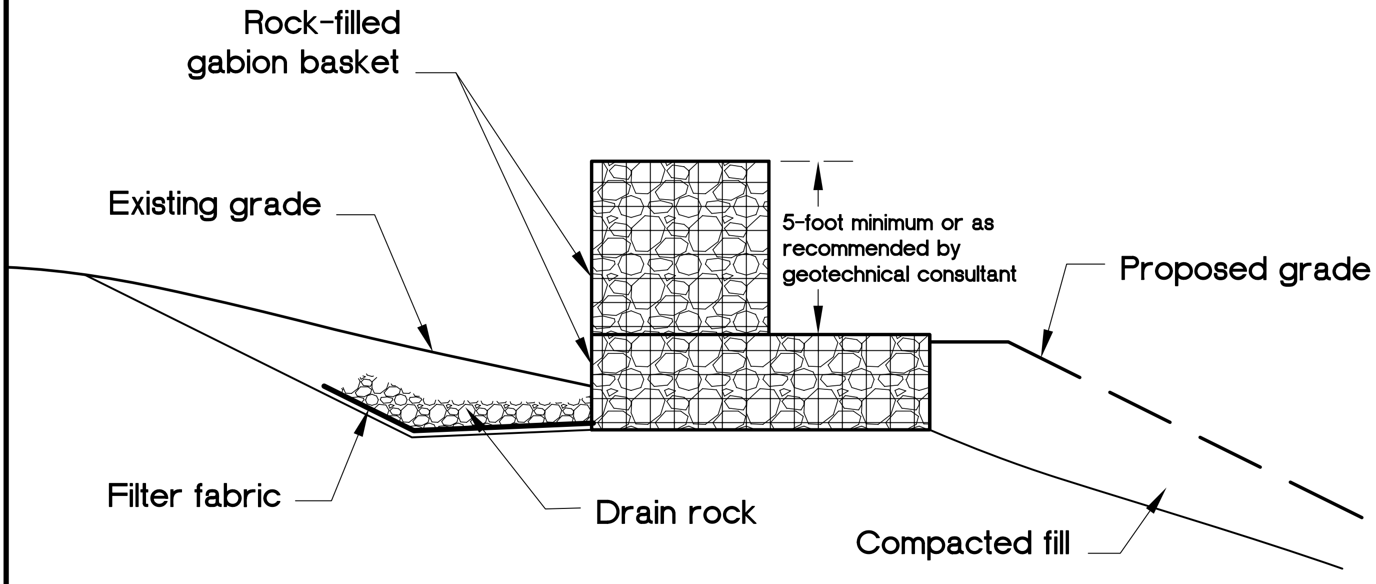
VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE
ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED IN

GeoSoils, Inc.	ROCK DISPOSAL DETAIL	Plate G-14



NOT TO SCALE

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



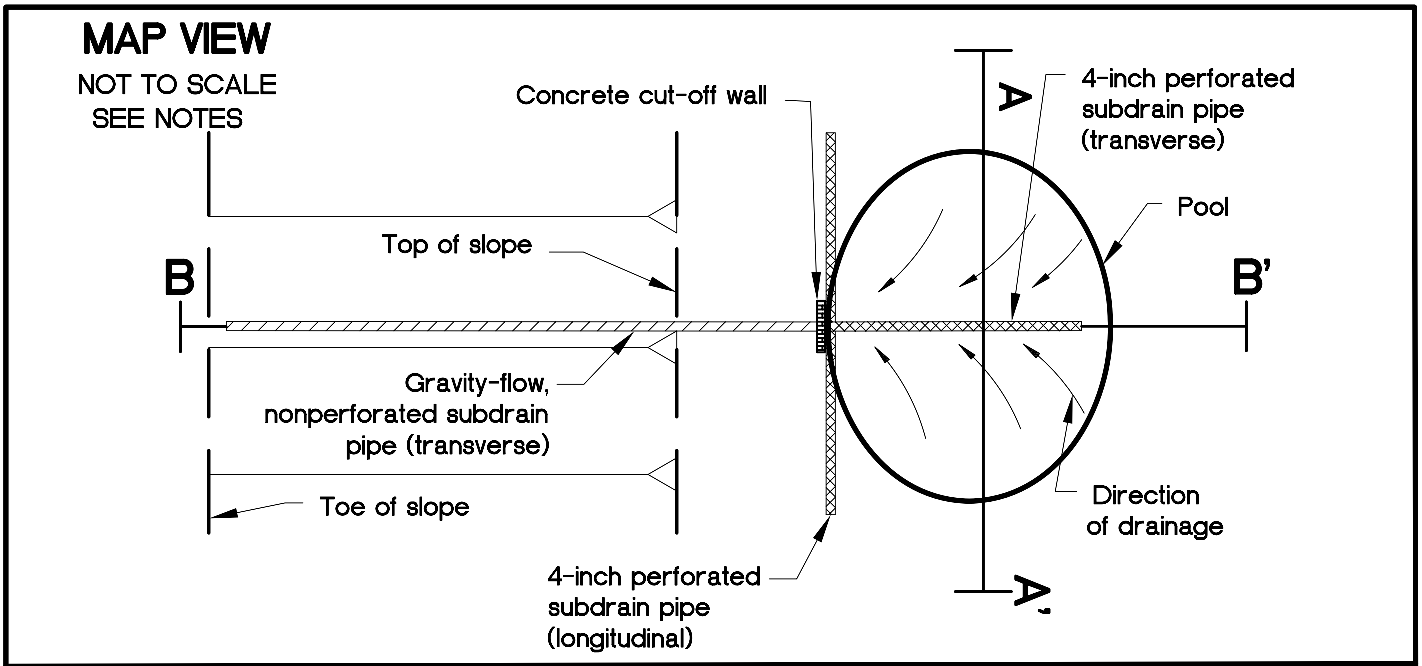
Gabion impact or diversion wall should be constructed at the base of the ascending slope subject to rock fall. Walls need to be constructed with high segments that sustain impact and mitigate potential for overtopping, and low segment that provides channelization of sediments and debris to desired depositional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

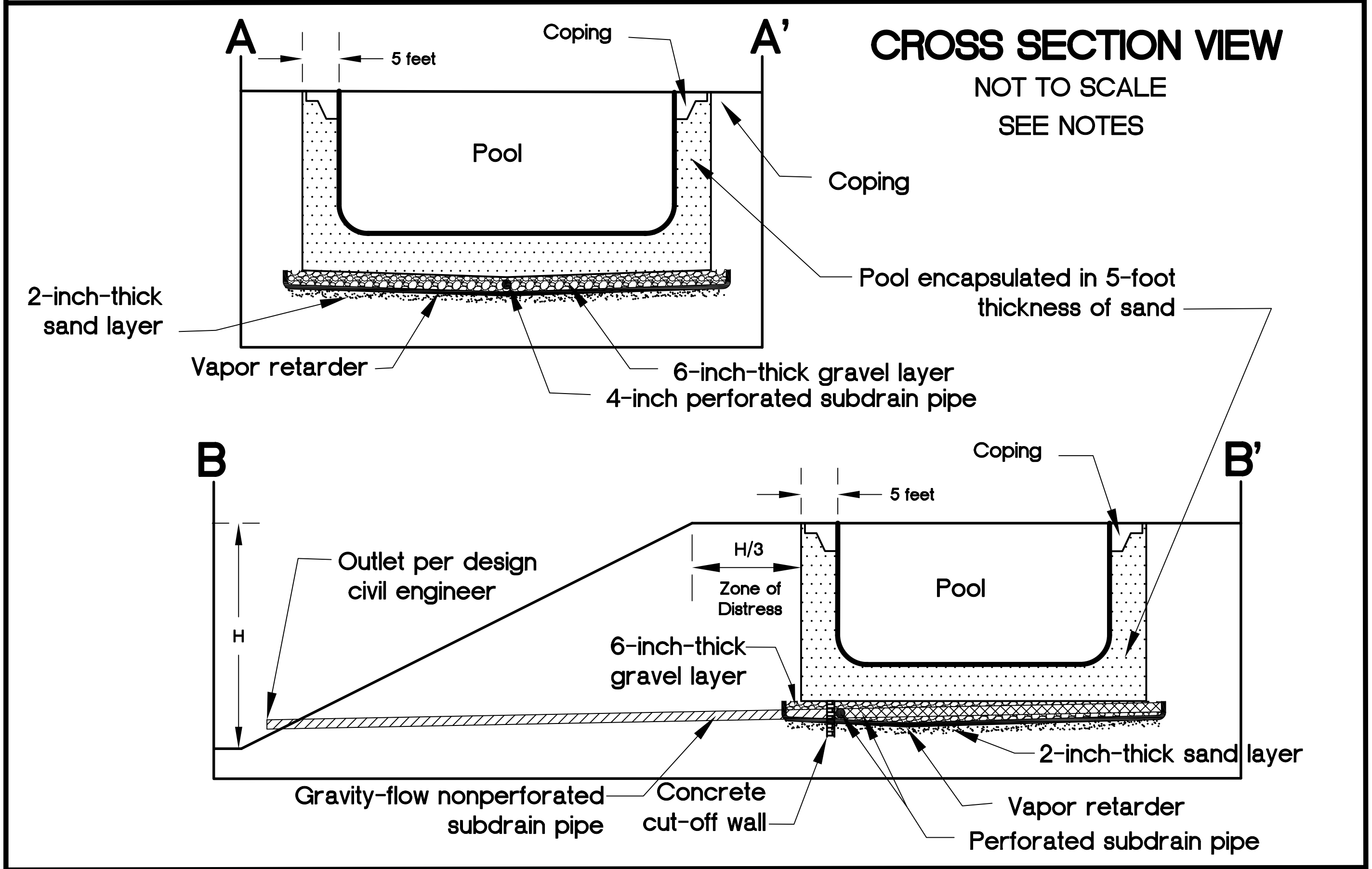
From GSA, 1987

GeoSoils, Inc.

ROCK FALL MITIGATION DETAIL

Plate G-16





NOTES:

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

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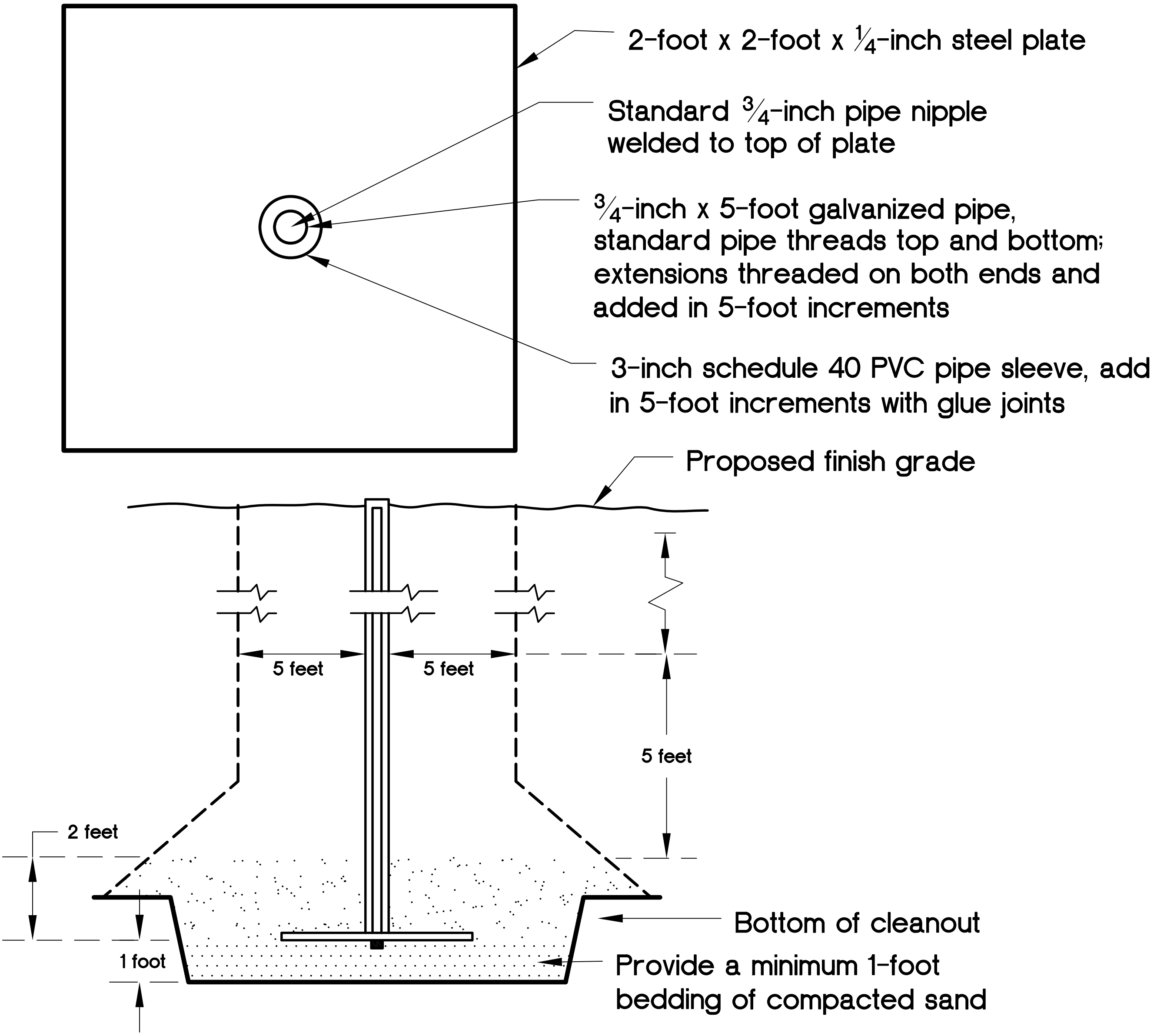
TYPICAL POOL/SPA DETAIL

Plate G-17



NOTES:

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

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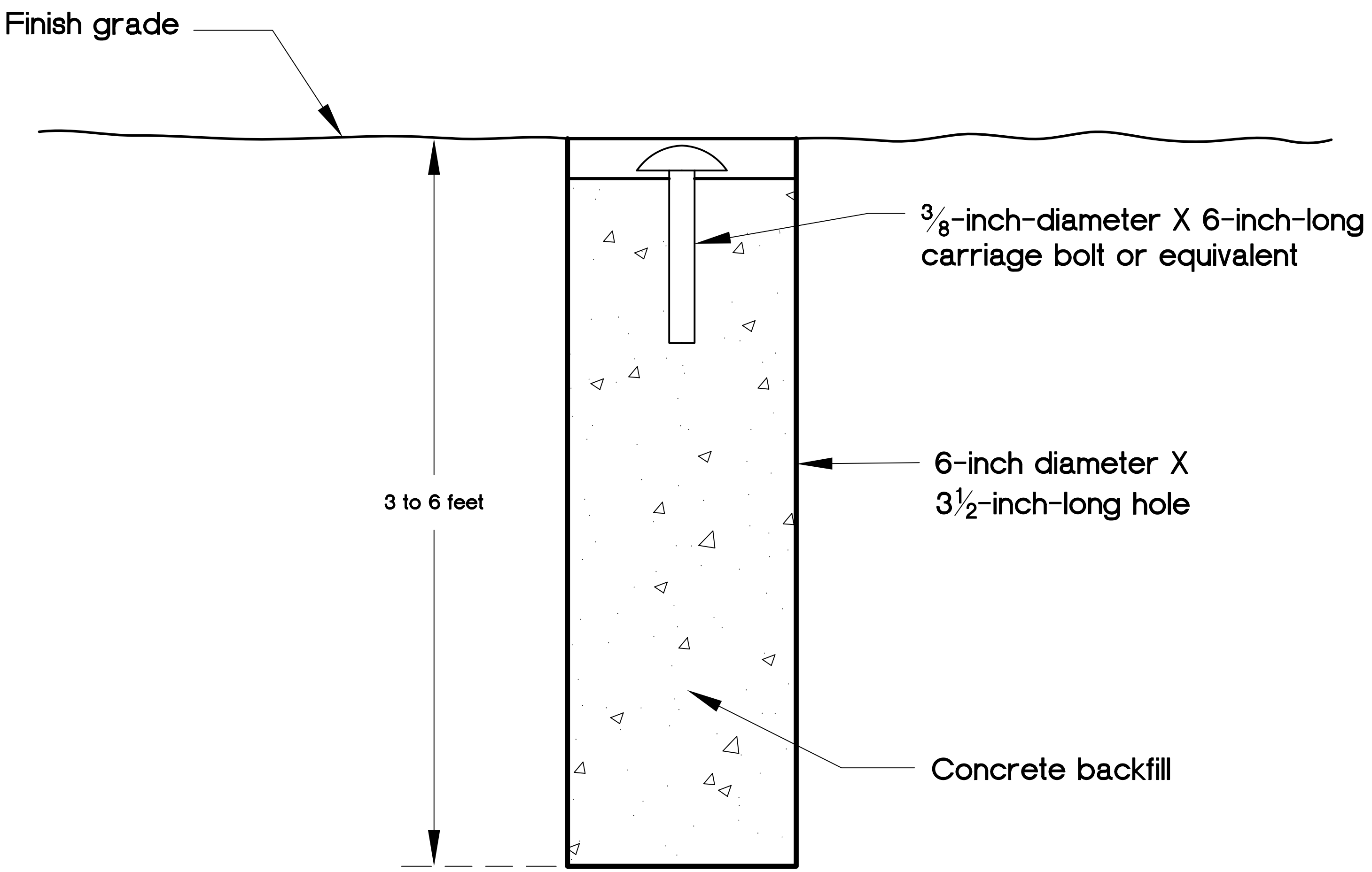
SETTLEMENT PLATE AND RISER DETAIL

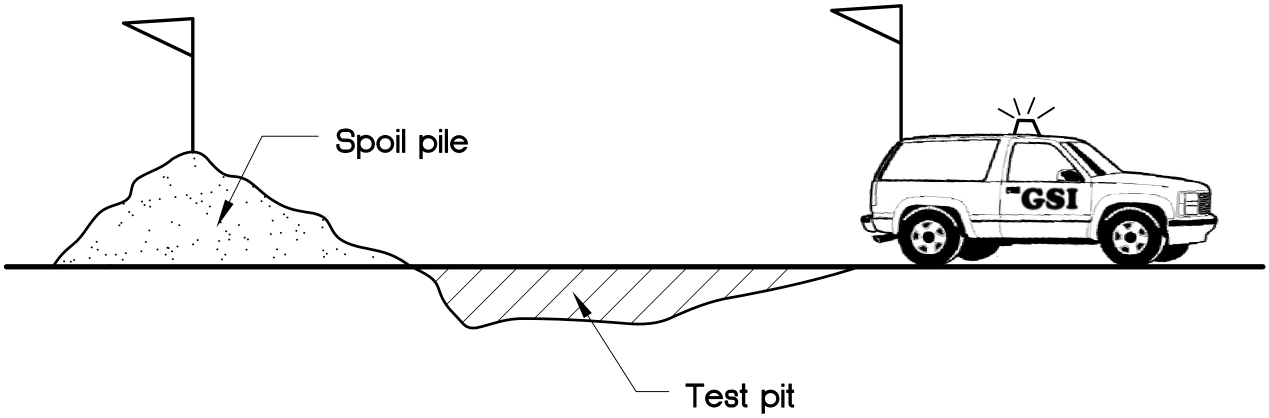
Plate G-18



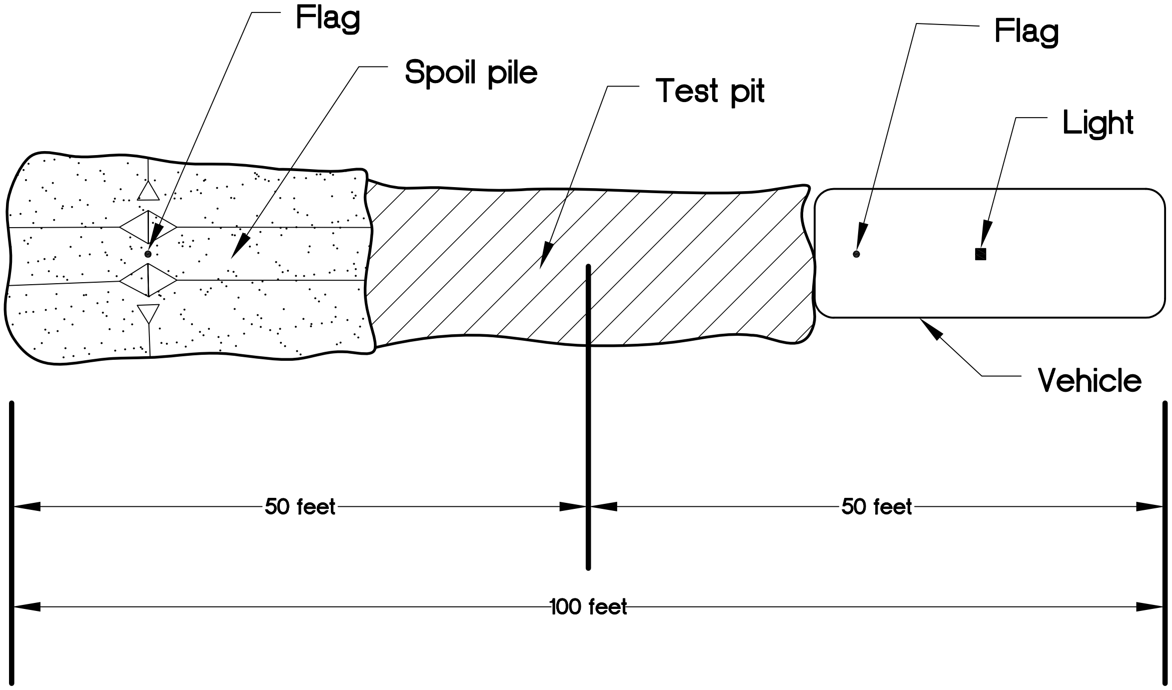
TYPICAL SURFACE SETTLEMENT MONUMENT

Plate G-19

SIDE VIEW



TOP VIEW



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TEST PIT SAFETY DIAGRAM

Plate G-20

GSI LEGEND

afu	— EXISTING ARTIFICIAL FILL – UNDOCUMENTED
Qal	— QUATERNARY ALLUVIUM
Qoa	— QUATERNARY OLDER ALLUVIAL FLOOD PLAIN DEPOSITS
Kcc	— CRETACEOUS–AGE GRANITIC BEDROCK (TONALITE OF COUSER CANYON), CIRCLED WHERE BURIED
PA-5	— APPROXIMATE LIMITS OF PLANNING AREAS
?	— APPROXIMATE LOCATION OF GEOLOGIC CONTACT (QUERIED WHERE UNCERTAIN)
89°	— JOINT ATTITUDE WITH DIP IN DEGREES
CPT-4	— APPROXIMATE LOCATION OF CONE PENETRATION TEST
TP-11	— APPROXIMATE LOCATION OF EXPLORATORY TEST PIT
SL-4	— APPROXIMATE LOCATION OF SEISMIC REFRACTION SURVEY
B B'	— APPROXIMATE LOCATION OF GEOLOGIC CROSS SECTION

ALL LOCATIONS ARE APPROXIMATE

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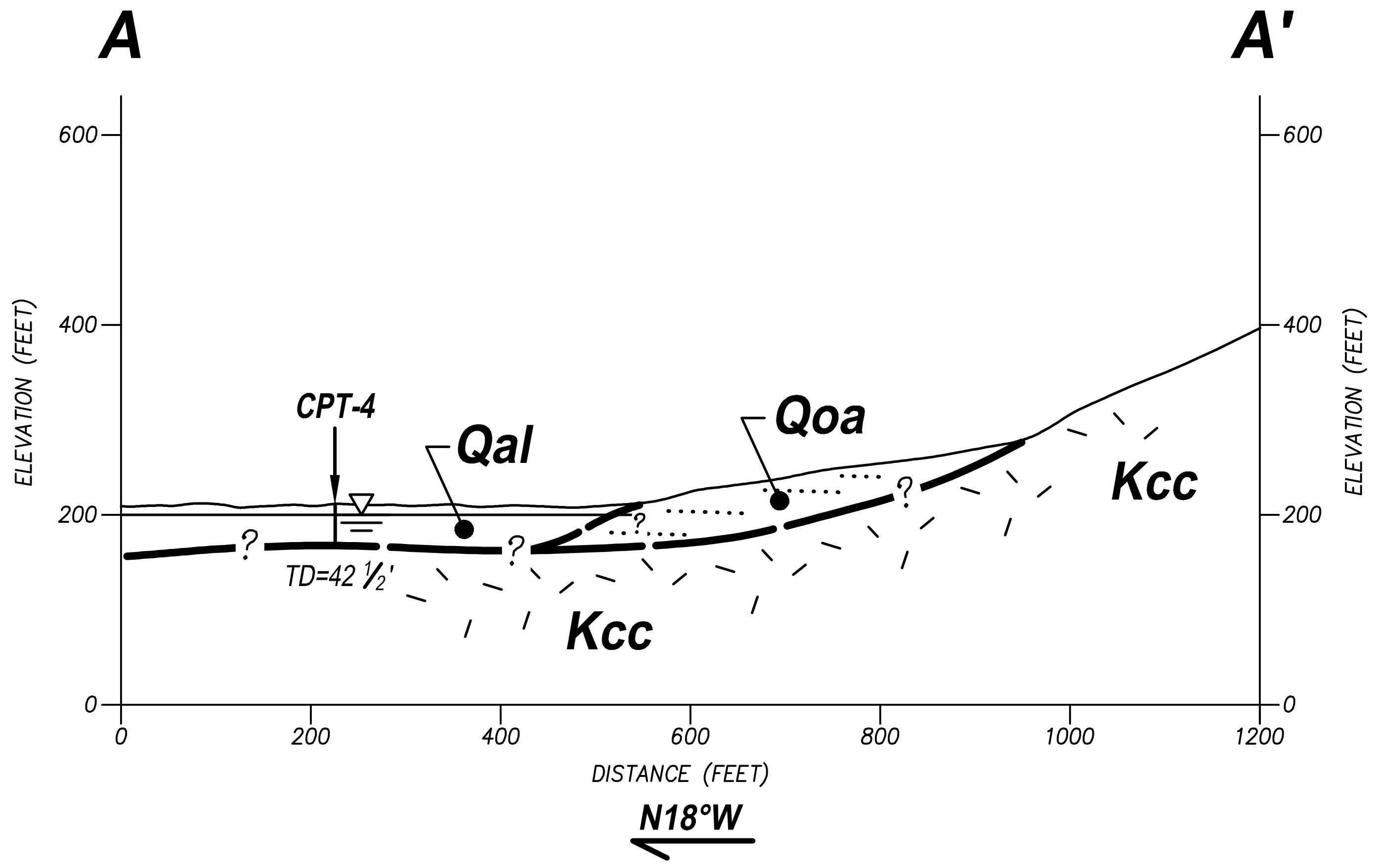
GeoSoils, Inc.

GEOTECHNICAL MAP

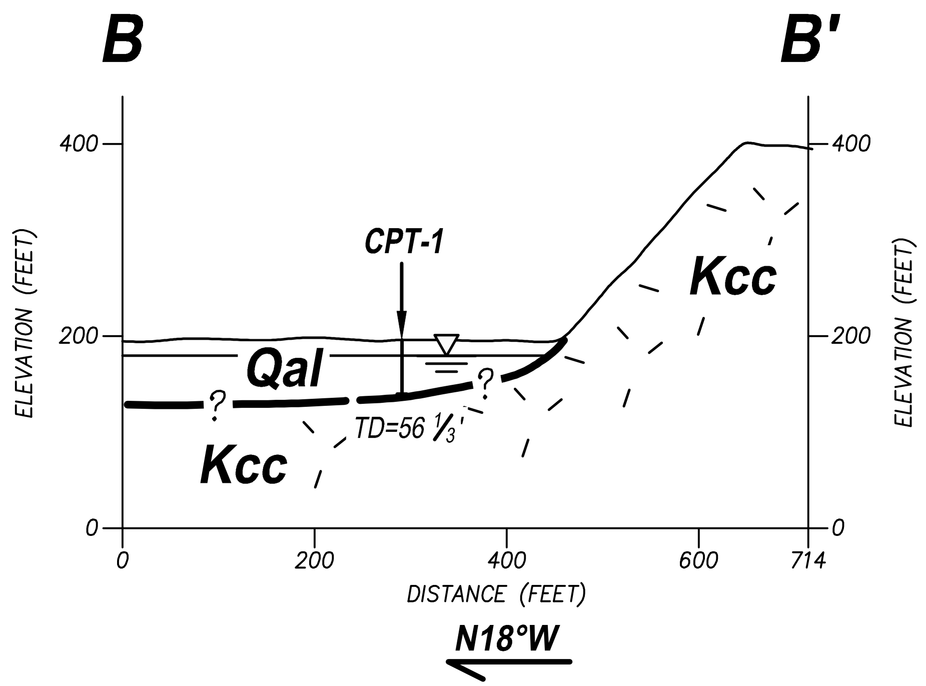
Plate 1

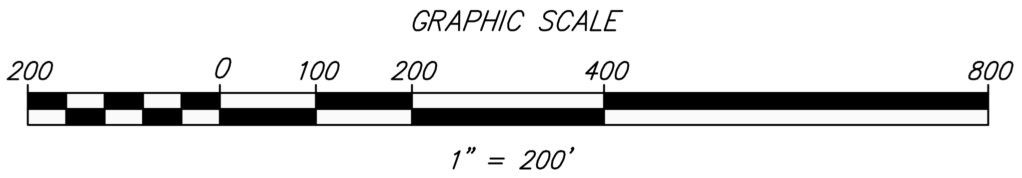
W.O. 6688-A-SC | DATE: 01/15 | SCALE: 1" = 400'



GSI LEGEND

Qal	— QUATERNARY ALLUVIUM
Qoa	— QUATERNARY OLDER ALLUVIUM
Kcc	— GRANITIC BEDROCK
▬?▬	— APPROXIMATE LOCATION OF GEOLOGIC CONTACT (QUERIED WHERE UNCERTAIN)
▽	— APPROXIMATE ELEVATION OF GROUNDWATER SURFACE
CPT-4 TD=56 ⅓'	— APPROXIMATE LOCATION (PROJECTED) OF CONE PENETRATION TEST SOUNDING WITH TOTAL DEPTH IN FEET





ALL LOCATIONS ARE APPROXIMATE

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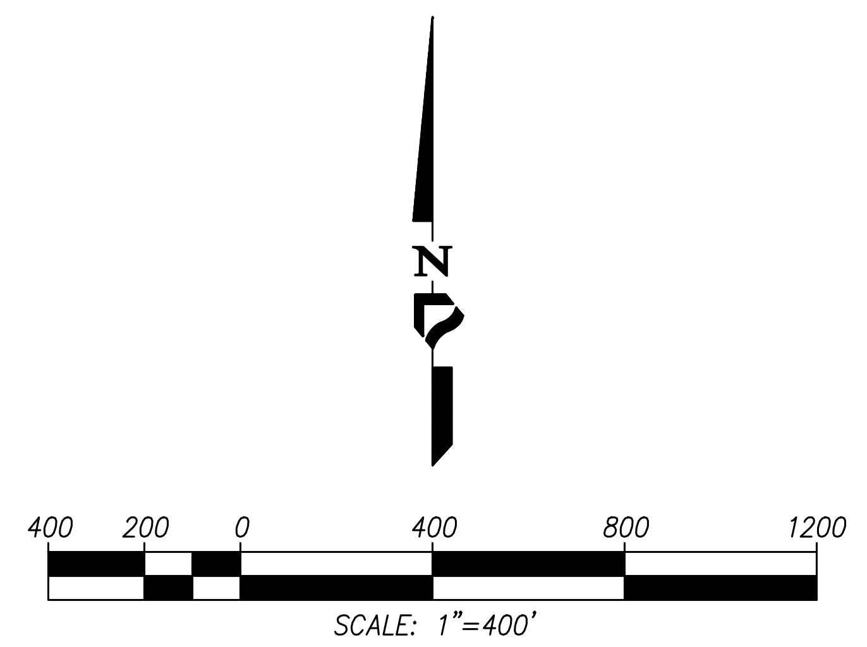
GEOLOGIC CROSS SECTIONS
A-A', B-B'

Plate 2

W.O. 6688-A-SC	DATE: 01/15	SCALE: 1" = 200'



GSI LEGEND

afu	—	ARTIFICIAL FILL - UNDOCUMENTED
Qal	—	QUATERNARY ALLUVIUM, CIRCLED WHERE BURIED
Qoa	—	QUATERNARY OLDER ALLUVIUM, CIRCLED WHERE BURIED
Kcc	—	CRETACEOUS-AGE GRANITIC BEDROCK (TONOLITE OF COUGAR CANYON)
Ki	—	CRETACEOUS-AGE GRANITIC BEDROCK (GRANODIORITE OF INDIAN MOUNTAIN)
KJm	—	CRETACEOUS/JURASSIC-AGE METAVOLCANIC AND METASEDIMENTARY ROCKS (UNDIVIDED)
89°	—	JOINT / FRACTURE ATTITUDE WITH DIP IN DEGREES
?	—	APPROXIMATE LOCATION OF GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN
TP-108 ◩	—	APPROXIMATE LOCATION OF EXPLORATORY TEST PIT (GSI, 2016)
TP-11 ⧓	—	APPROXIMATE LOCATION OF EXPLORATORY TEST PIT (GSI, 2015)
CPT-104 ●	—	APPROXIMATE LOCATION OF CONE PENETRATION TEST WITH TOTAL DEPTH IN FEET (GSI, 2016)
CPT-4 ⊕	—	APPROXIMATE LOCATION OF CONE PENETRATION TEST WITH TOTAL DEPTH IN FEET (GSI, 2015)
HSA-11 ◑ TD=51 ½'	—	APPROXIMATE LOCATION OF HOLLOW-STEM AUGER BORING WITH TOTAL DEPTH IN FEET
SL-4	—	APPROXIMATE LOCATION OF SEISMIC SURVEY LINE (GSI, 2015)
SL-104	—	APPROXIMATE LOCATION OF SEISMIC SURVEY LINE (GSI, 2016)

ALL LOCATIONS ARE APPROXIMATE

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GeoSoils, Inc.

GEOTECHNICAL MAP

Plate 1

W.O. 6960-A8-SC	DATE: 08/19	SCALE: 1" = 400'