PRELIMINARY GEOTECHNICAL INVESTIGATION
LOT 39 OF TRACT 7240
±10.01-ACRE SITE, (APN 290-190-083)
GLEN IVY SENIOR COMMUNITY AND
RETAIL/COMMERCIAL PROJECT
TEMESCAL VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

FOR

GLEN IVY PROPERTIES, LLC
34145 PACIFIC COAST HWY #621
DANA POINT, CALIFORNIA 92629

W.O. 7731-A-SC   MARCH 16, 2020

GeoSoils, Inc.

Geotechnical • Geologic • Coastal • Environmental

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March 16, 2020

W.O. 7731-A-SC

Glen Ivy Properties, LLC
34145 Pacific Coast Hwy #621
Dana Point, California 92629

Attention: Mr. Benjamin Day

Subject: Preliminary Geotechnical investigation, Lot 39 of Tract 7240, ±10.01-Acre Site (APN 290-190-083), Glen Ivy Senior Community and Retail/Commercial Project, Temescal Valley Area, Riverside County, California

Dear Mr. Day:

In accordance with your request and authorization, GeoSoils, Inc. (GSI) is presenting the results of our preliminary geotechnical investigation of the subject site in the Temescal Valley area of Riverside County, California. The purpose of this study was to evaluate the onsite soils and geologic conditions and their effects on the proposed senior community and retail/commercial development concept from a geotechnical viewpoint, supplement the previous fault finding investigation conducted onsite (GSI 1999a), and to provide code compliant conclusions and recommendations for future development of the property.

EXECUTIVE SUMMARY

Based on our current and previous field explorations, review of data obtained, and our geologic and geotechnical engineering analysis, the proposed development of the project appears suitable for its intended mixed-use development from a geotechnical viewpoint, provided the recommendations presented herein, and within the previous referenced report by GSI (1999a) are appropriately implemented during planning, design, and construction of the project. The primary developmental considerations are summarized below:

- Although preliminary in nature, GSI understands that proposed onsite improvements will consist of the construction of a one- to three-story senior living community and one- to two-story retail/commercial project onsite along with underground utility, associated infrastructure, driveway/parking areas, and offsite roadway improvements. Based upon our review of the new site and floor plans by

Douglas Pancake Architects (DPA, 2020) and constraints exhibit and topographic mapping by K&A Engineering, Inc. (K&A 2019), the site is relatively flat-lying and no significant slopes are currently anticipated onsite. No cut slopes are currently anticipated based on the remedial grading proposed, any proposed fill slopes constructed using onsite materials, should be grossly and surficially stable provided the recommendations contained herein are implemented during site development.

- As discussed above, a previous fault-finding study was performed onsite by GSI in 1999 (1999a), in conjunction with work on the commercial parcel to the north (GSI, 1999b and 1999c). The onsite fault finding investigation (GSI, 1999a) was conducted for the original ±14-acre property. It should be noted the southern ±4.15-acre portion (Parcel B) of the original property has been granted/sold and recently developed and fenced for the construction of a local flood control basin by the Riverside County Flood Control. As such, our current study covers the remaining northern ±10.01-acre portion of the property (Parcel A). See Plate 1 (Geotechnical Map).

- A number of previous geologic and geotechnical investigations have been performed on the adjacent properties. GSI performed studies in 1987, 1988, 1989, and 1999 (see GSI references in Appendix A [GSI; 1987, 1988, 1989, 1999b, and 1999c]) which encompassed the±776-acre Shea Homes for Active Adults project, and the commercial parcel (PA-18) to the north. A more recent geotechnical investigation (GSI, 2015) and geologic fault-finding study (GSI, 2007c and 2007b) for the adjacent property to the north (PA-18) were also performed. Responses to County of Riverside comments for the adjacent property to the north were also prepared in 2017 (GSI, 2017 - GEO 02541) and 2008 (GSI, 2008 - GEO 01709). To date it is unknown if these responses to County comments (GSI, 2017 and 2008) have been reviewed and/or approved by the County.

- In general, the site may be characterized as being underlain by Pleistocene-age alluvial fan deposits to the east and Holocene-age marsh deposits to the west which are locally mantled by undocumented artificial fill, younger alluvium, and topsoil/colluvium.

- Removal of all undocumented artificial fill, topsoil/colluvium, young alluvium and near surface weathered marsh deposits and Pleistocene-age alluvial fan deposits will be necessary prior to fill placement, in areas proposed for settlement-sensitive improvements. Approximate depths of removals are outlined in the conclusions and recommendations section of this report. For preliminary planning purposes, undocumented fill thicknesses (including previous fault trenches) are estimated to be on the order of ±5 feet to as much as ±20 feet in previous fault trenches; approximately ±5 to ±10 feet in areas delineated as younger alluvium (Qal), approximately ±10 feet in areas delineated as marsh deposits (Qm); and approximately ±5 feet in areas delineated as older alluvial fan deposits (Qf). Actual depths of removals will be evaluated in the field during grading by the geotechnical consultant. These are remedial removal recommendations and do not include geometric constraints based on the size of the building footprint.

- Our review indicates that strands of a known active fault associated with the Elsinore fault zone (EFZ) transect the site and portions of the site are included within an Alquist-Priolo Earthquake Fault Zone. Previous Alquist-Priolo (A-P) fault investigation studies onsite and the adjoining commercial property to the north by GSI (2007b, 2007c, and 1999a) provided detailed findings pertaining to this fault and the associated A-P zone. As previously indicated by GSI (2007b, 2007c, 1999a, and 1999c), the site lies within the EFZ, specifically the Glen Ivy North fault, which is considered active (i.e., movement within the Holocene Epoch), according to the State of California (California Geological Survey, 2018). Based on the above, setbacks from surface faulting encountered onsite are considered warranted, as habitable structures are proposed near areas transected by active faulting. The structural setback zones previously established are shown on the new constraints exhibit and topographic mapping by K&A (2019, see Plate 1 - Geotechnical Map).

- No California seismic hazard zone mapping is currently available for the Lake Mathews Quadrangle; however, the site is located within a "moderate" area of potential liquefaction (Riverside County Information Technology [RCIT], Graphic Information Services [GIS, 2018]). Although some paleoliquefaction related features (i.e., sand boils, soft sediment deformation, etc.), presumably associated with near-field seismic activity, were noted within the marsh deposits onsite and in other nearby trenches previously advanced on the adjoining properties (GSI, 1999a; Rockwell, et al., 1986), our evaluation indicates that these features can be reasonably mitigated by the use of appropriate remedial grading, building setbacks, and/or other foundation engineering design, since settlement-sensitive improvements are proposed within the areas delineated as "Quaternary marsh deposits" onsite (see Plate 1). It should be noted that <u>no</u> liquefaction related features have been identified within the Pleistocene-age alluvial fan deposits, as encountered during this, and previous studies (GSI; 2015, 2007b, 2007c, 1999a, 1999c, and 1987). Regardless, in accordance with County mapping and current standards of practice, our liquefaction analysis (pursuant to Special Publication 117A [SP 117A, California Geological Survey {CGS}, 2008]) indicates that the potential for liquefaction and associated adverse effects within the Pleistocene-age alluvial fan deposits is considered low, and perhaps moderate within the Holocene-age marsh deposits onsite.

- Our review of site conditions indicates regional seismic shaking, ranging from moderate to severe, may occur on the site associated with onsite and/or regional faults, and horizontal seismic accelerations at the site are anticipated to be approximately 0.83g, should the design earthquake occur. The potential for more onerous near-field seismic effects (based on the type and size of structures proposed, the seismic source, distance, and geological aspects, etc.), and appropriate mitigation, as warranted, is discussed herein. In view of the site seismic setting and the potential for seismic settlement, post-tensioned and/or mat foundations appear particularly appropriate for this project. Based on our site specific seismic hazard analysis, seismic design parameters per the 2019 CBC are provided herein.

- It should be noted, that the 2019 CBC indicates that removals of unsuitable soils be performed across all areas under the purview of a grading permit, not just within the influence of the building or structure. Relatively deep removals may also necessitate a special zone of consideration, on perimeter/confining areas. This zone would be approximately equal to the depth of removals, if removals cannot be performed onsite or offsite. Thus, any settlement-sensitive improvements (perimeter walls, curbs, flatwork, etc.), constructed within this zone may require deepened foundations, reinforcement, etc., or will retain some potential for settlement and associated distress, if not properly mitigated during grading.

- Expansion Index (E.I.) testing results indicate that the site soils tested are generally very low in expansion potential. However, the possible presence of soils with low expansion potentials exist onsite. Based on the relatively high site accelerations anticipated onsite, preliminary foundation recommendations for both post-tension and mat designs are provided herein. Post-tension or mat foundations should perform better under the design seismic event. Additional E.I. and Plasticity Index (P.I.) testing, if warranted, should be conducted during, or shortly after, site grading to further evaluate the preliminary test results obtained.

- Typical samples of the site materials have been analyzed for soluble sulfate/corrosion potential. For preliminary planning purposes, and based upon the soluble sulfate test results and the latest edition of the 2019 CBC, the soluble sulfate content is considered Class “S0" (per American Concrete Institute [ACI] 2014a), and sulfate-resistant concrete is currently not anticipated. Based on the results of the pH and saturated resistivity testing, the onsite soils (Marsh deposits) are generally considered extremely acid and are considered severely corrosive to ferrous metals in a saturated state. Additional sulfate/corrosion testing should be conducted during, or shortly after, site grading to further evaluate the preliminary test results obtained. Based on the conditions encountered, a consulting corrosion engineer should be considered to provide specific recommendations for foundations, piping, etc., as warranted.

- In general and based upon the available data to date, regional groundwater is not expected to be a major factor in the development of the site. During the previous fault evaluation studies onsite and the adjoining property to the north (GSI; 2007b, 2007c, 1999a, 1999b, and 1987) evidence of a relatively high long-term groundwater level was documented only within the marsh deposits onsite. This evidence was from the geologic past, in the form of peat deposits and soil mottling. During our field study perched groundwater was encountered in the marsh deposits at a depth of approximately ±32 feet below the ground surface (b.g.s.). These observations reflect site conditions at the time of our field investigation and do not preclude changes in local groundwater conditions. Shallower seepage and/or perched water may exist locally during and after development, owing to a combination of high rainfall, irrigation runoff, broken utilities, improper drainage, and/or relatively impermeable subsoils. Based on the above, subdrainage systems for the control of localized seepage and/or perched groundwater (and attendant excavation problems [i.e., soils too wet to compact]), may be encountered during grading and subsurface utility installation. This potential for seepage or perched water to occur after development should be disclosed to all interested/affected parties.

- Other than the presence of active faulting (discussed above), adverse geologic features that would preclude project feasibility (e.g., landslides, collapsible soils, etc.) were not encountered.

- The recommendations presented in this report should be incorporated into the planning, design, and construction considerations of the project. Unless specifically superceded herein, the conclusions and recommendations provided within the previous referenced report by GSI (1999a, see Appendix A) should be appropriately implemented during planning, design, and construction of the project

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

Respectfully submitted,

GeoSoils, Inc.

Todd A. Greer
Engineering Geologist, CEG 2377

David W. Skelly
Civil Engineer, RCE 47857

John P. Franklin
President, CEG 1340

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Distribution: (3) Addressee (2 wet signed)

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SCOPE OF SERVICES

The scope of our services has included the following:

1. Review of available soils and geologic data for the site, including previous geotechnical reports by GSI and others (Appendix A).

2. Geologic site reconnaissance and geologic mapping of surficial deposits and significant geologic structures.

3. Subsurface exploration consisting of the advancement of six (6) exploratory borings for geotechnical logging and soil sampling and six (6) Cone Penetration Tests (CPTs) for correlation with the borings, within the marsh and alluvial fan deposits onsite (Appendix B). In addition, the previous advancement of one (1) exploratory boring for geotechnical logging and soil sampling (GSI, 1987, Appendix B).

4. General areal seismicity evaluation (Appendix C).

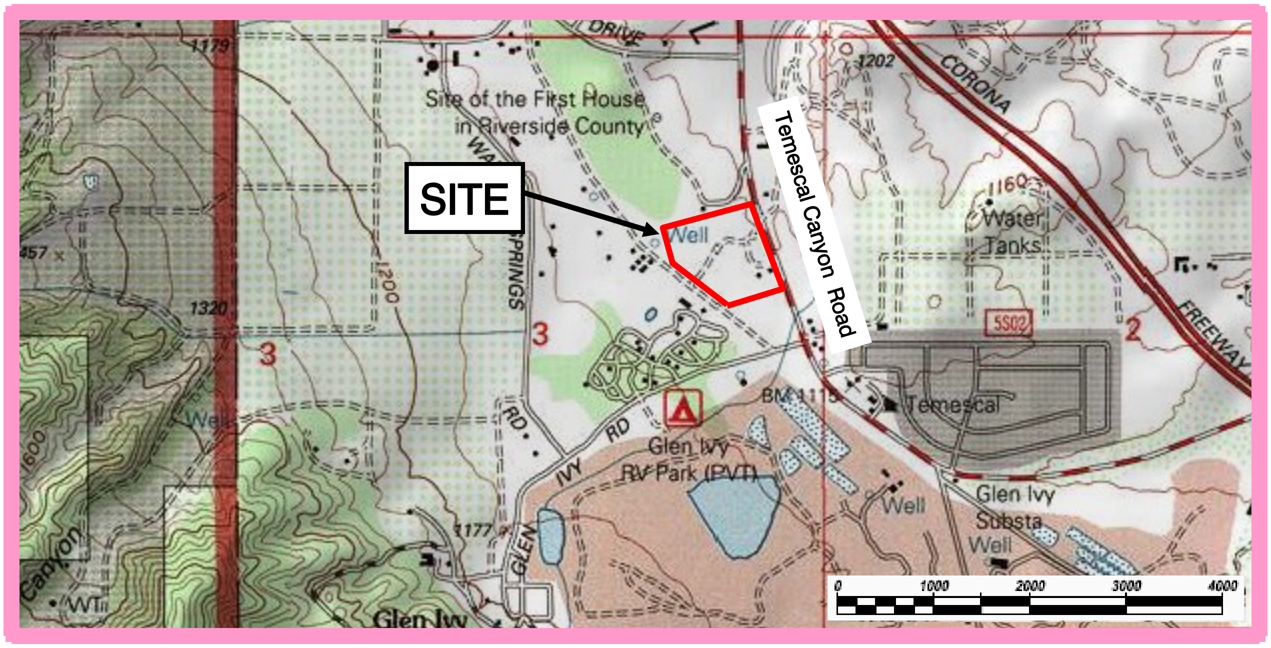
5. Pertinent laboratory testing of representative soil samples collected during this, and our previous subsurface exploration programs. Testing included in-situ moisture and density, maximum density testing, expansion index testing, sulfate/corrosion testing, sieve analysis, direct shear testing, and consolidation testing of the materials encountered during our field studies. Results of our laboratory testing are provided in Appendix D.

6. Preparation of a liquefaction analysis for this study based on the subsurface data obtained. Our liquefaction analysis is included in Appendix E.

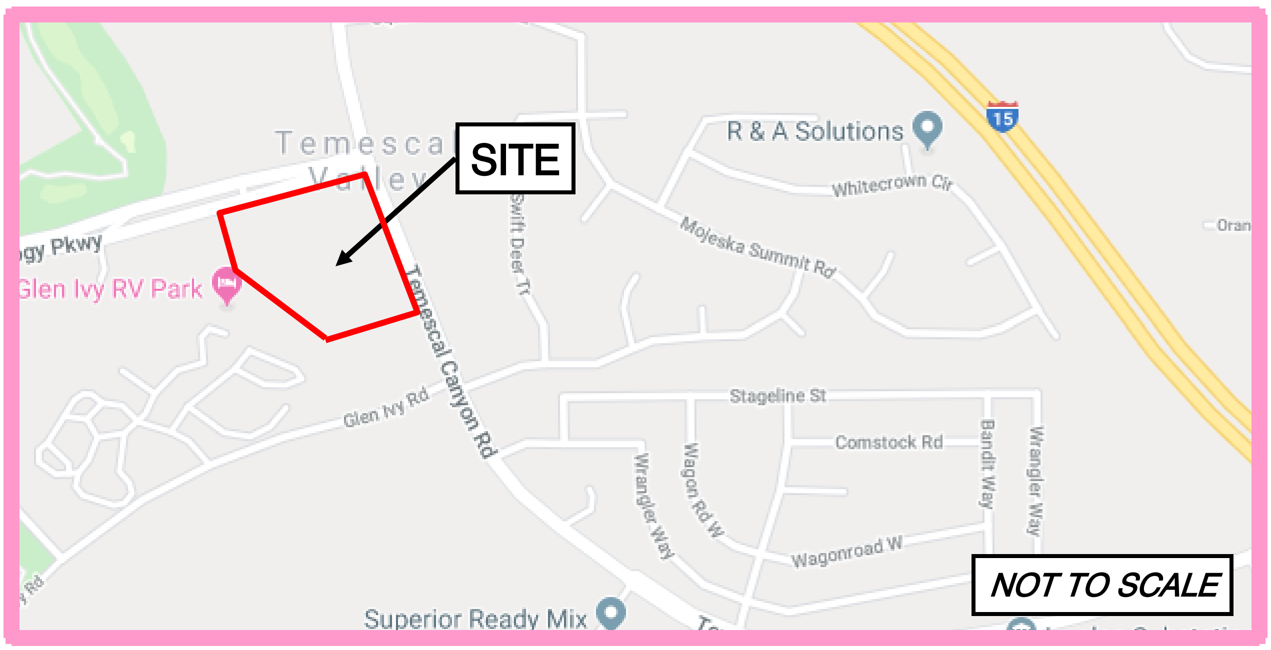
7. Appropriate engineering and geologic analyses of data collected and preparation of this report and accompaniments.

SITE DESCRIPTION

The subject property is located on the southwest corner of the intersection of Temescal Canyon Road and Trilogy Parkway in the unincorporated area of Temescal Valley, Riverside County, California. The property (APN 290-190-083) consists of an irregularly-shaped parcel, totaling approximately ±10.01 acres (see the Site Location Map, Figure 1).



Base Map: TOPO! Copyright 2003 National Geographic, USGS Lake Mathews Quadrangle, California -- Riverside Co., 7.5 Minute, dated 1967.



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

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SITE LOCATION MAP

Figure 1

With the exception of recent flood control improvements (perimeter grubbing and fence line) and localized landscape improvements on the southern margin and northern portion of the property, respectively, the site is generally undeveloped. Topographically, a majority of the project area is generally flat-lying. Based on the new constraints exhibit and topographic mapping by K&A (2019), site elevations range from a high of about ±1,100 feet Mean Sea Level (MSL), in the northwest portion of the site, to a low of about ±1,083 feet MSL within the closed depression in the central portion of the site, for an overall relief of about ±17 feet. Overall site drainage is generally to the east by sheetflow; however, drainage is variable and trapped in localized areas depending on the relief. Several oak trees, native weeds and grasses, and other vegetation were noted onsite, and the localized landscape area on the north, as previously discussed.

BACKGROUND

A number of previous geologic and geotechnical investigations have been performed on the adjacent properties. GSI performed studies in 1987, 1988, 1989, and 1999 (see GSI references in Appendix A [GSI; 1987, 1988, 1989, 1999b, and 1999c]) which encompassed the ±776-acre Shea Homes for Active Adults project, and the commercial parcel to the north. As discussed above, a fault-finding study was performed onsite by GSI in 1999 (1999a), in conjunction with work on commercial parcel to the north (GSI, 1999a). A more recent geotechnical investigation (GSI, 2015) and geologic fault-finding study (GSI, 2007b and 2007c) for the adjacent commercial property to the north of the subject site were also performed. A response to County of Riverside comments was also prepared in 2008 (GSI, 2008) and 2017 (GSI, 2017). To date it is unknown if these responses to County comments (GSI, 2008 and 2017) have been reviewed and/or approved by the County. For convenience, the previous GSI explorations are shown on Plate 1, and the logs of the previous GSI fault finding trenches advanced onsite are provided as Plates 2 through 5.

PROPOSED DEVELOPMENT

Based on conversations with the Client, GSI understands that the proposed development of the project would consist of the mixed-use development of the ±10.01-acre property for senior community living and retail/commercial use, along with the installation of underground utility, site infrastructure and street/parking improvements. GSI has assumed that site elevations may be raised on the order of ±3 to ±5 feet above existing grades. Building loads are assumed to be typical for this type of relatively light mixed-use development. Sewage disposal is to be accommodated by tying into the regional system.

FIELD STUDIES

As indicated above, field studies conducted during this preliminary geotechnical investigation of the property consisted of geologic reconnaissance mapping, the advancement of six (6) exploratory borings and six (6) Cone Penetration Tests (CPTs), for correlation with the borings, and for evaluation of near-surface soil and geologic conditions. Our field exploration was performed on January 23, 27, and February 7, 2020. The borings were logged and the CPTs were directed by engineering geologists from our firm who collected representative bulk and undisturbed soil samples for appropriate laboratory testing. The logs of the borings and the CPT printouts are presented in Appendix B. The approximate locations of the exploratory borings and CPTs advanced for this study, as well as previous GSI explorations, are presented on Plate 1 (Geotechnical Map).

REGIONAL AND SITE GEOLOGY

Regional Geologic Setting

The subject property is located on the western margin of the Perris Block of the Peninsular Ranges Geomorphic Province, which is characterized by northwest-trending, steep, elongated ranges and valleys. The Peninsular Ranges Geomorphic Province extends north to the base of the San Gabriel Mountains along the southern side of the Transverse Ranges Province, and south into Baja, California. The province is bounded by the 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 Perris Block is generally regarded as part of the Peninsular Ranges, and is considered to be a relatively stable structural block lying between the EFZ and San Jacinto fault zones (SJFZ). The Perris Block is bounded on the northeast by the SJFZ, on the north by the Cucamonga fault zone (CFZ) and the San Gabriel Mountains, and on the southwest by the EFZ and the Santa Ana Mountains. The Perris Block is bounded by two grabens, the Elsinore Trough on the west (part of the EFZ) and the Salton Trough on the east (Sharp, 1975). The uplift of the Perris Block has been less than that of the bounding mountain ranges, resulting in lower relief.

Local Geology

Most of the site is underlain by fluvial sediments emanating from Bixby and Anderson Canyons, and to a lessor extent Coldwater Canyon, where they coalesce as they flow out of the Santa Ana Mountains. Geomorphically, the alluvial fan deposits are only slightly dissected and bear weakly developed soil profiles (i.e., paleosols) indicating they are likely of latest Pleistocene- and Holocene-age (GSI, 2015, 2007b, 2007c,1999a, and 1999c). These sediments may reach several tens of feet in thickness before basement rock is reached. As encountered onsite, a relatively thin layer (i.e., ±2 to ±10 feet in thickness) of younger alluvial materials locally mantle portions of the Marsh deposits and the Pleistocene-age alluvial fan deposits onsite. Localized undocumented artificial fill (associated with stockpiling, the previous fault finding trenches and land use), and topsoil/colluvium mantle the Holocene-age marsh deposits and Pleistocene-age alluvial fan deposits onsite.

Site Geologic Units

Descriptions of site geologic units mentioned herein were observed during this study, our previous fault/seismic investigations (GSI, 2007b and 2007c) and/or were previously described in GSI (2015, 1999a, 1999b, and 1999c). The site geologic units consist of undocumented artificial fill, topsoil/colluvium, alluvium (younger), marsh deposits, and older alluvial fan deposits. The limits of mappable units are shown on Plate 1. The major geologic units are generally described as follows, from youngest to oldest:

Artificial Fill - Undocumented (Map Symbol - Afu)

Undocumented artificial fill was observed in localized areas generally associated with previous stockpiling, backfill of previous fault finding exploratory trenches, and dumped fills from previous land use. The undocumented fill materials are generally light yellowish brown to brown, silty to clayey sands with gravel, cobbles, and localized boulders derived offsite from the adjoining residential tract and onsite from the underlying alluvial fan and marsh sediments, and range in depth from approximately ±1 to as much as ±20 feet (within previous fault trenches). The undocumented fill materials are anticipated to have a very low to possibly low expansion potential based on visual classification. The undocumented fill materials are potentially compressible in their existing state and may settle appreciably under additional fill or foundation and improvement loadings, and therefore, should be removed and recompacted within the influence of settlement-sensitive improvements.

Topsoil/colluvium (Not Mapped)

Topsoil/colluvium was observed to discontinuously mantle portions of the site. Where encountered, the topsoil/colluvium ranges in thickness from about ±2 to ±3 feet. The topsoil/colluvium is generally silty to clayey, fine- to coarse-grained sands and silts. These materials are damp to wet, are generally loose/soft to medium dense/medium stiff, porous and bioturbated. Based on visual classification, the topsoil/colluvium typically has a low expansion potential. These materials are considered unsuitable for support of structures and/or improvements in their existing state.

Quaternary Alluvium - Younger (Qal)

Younger alluvium discontinuously mantles the older sediments onsite. The alluvium is generally silty sand, with minor to locally abundant pebbles, gravels and cobbles, to sands with pebbles, gravels and cobbles, to locally sandy gravels/gravelly sands with cobbles and minor boulders. It is generally light brown, brown, and grayish brown, dry to damp, and generally loose to medium dense with depth. These soils are visually classified as having a very low, expansion potential. Due to the potentially compressible nature of these younger alluvial materials, complete removals will be required in proposed structural/settlement-sensitive areas. The younger alluvial sediments are estimated to be Holocene-age.

Quaternary Marsh Deposits (Map Symbol - Qm)

Marsh deposits were previously encountered by GSI during our fault finding investigation (1999a), and within the exploratory borings and CPTs for this study associated with the Glen Ivy North marsh on the western portion of the site (Qm, see Plate 1). The marsh deposits are generally silty sands and clayey silts to clays, with some interbedded organic layers. Based on visual classification, these sediments have a very low to low expansion potential. The near surface marsh deposits are generally not well consolidated, however, are generally thin to medium-bedded and flat-lying, except where locally affected by faulting. The near surface marsh deposits are considered unsuitable for support of structures and/or improvements in their existing state. Based on radiocarbon age-dating of representative charcoal and organic/peat samples obtained during our previous fault/seismic and subsurface investigations on the adjoining commercial property to the north (GSI; 2007a, 2007b, and 2007c), this unit was determined to be late- to mid-Holocene in age (Beta Analytic, 2006).

Quaternary Alluvial Fan Deposits - Older (Map Symbol - Qf)

Older alluvial fan deposits underlie the eastern portions of the study area (see Plate 1). The alluvial fan deposits are generally silty to gravely sands, to sands with pebbles, gravels, cobbles, and minor boulders. They are generally pale brown to reddish yellow, dry to damp, and medium dense to dense. Additionally, they are thinly to medium bedded, and locally form grossly fining upward sequences. They are generally flat lying to gently inclined to the northeast. These sediments may reach several tens of feet in thickness. The older alluvial fan deposits are estimated to be latest Pleistocene-age (GSI; 2007b, 2007c, 1999a, and 1999c). Typically, these sediments have a very low to low expansion potential. The near surface older alluvial fan deposits are weathered and porous and are unsuitable for support of settlement-sensitive improvements in their existing state, and will require some removal and recompaction.

GROUNDWATER

Seeps or springs were not noted on the subject property during the time of our field investigation. During our field study, perched groundwater was encountered in the marsh deposits at a depth of approximately ±32 feet below the ground surface (b.g.s.). Based on water well data acquired from the California Department of Water Resources (CDWR, 2020), "Water Data Library," groundwater levels in other nearby wells were previously measured at depths ranging from ±22 feet (Well No. 337430N1174280W001 - November 13, 2018) to ±53 feet (Well No. 338227N1175072W001 - November 13, 2016) below the ground surface. However, it should be noted that these wells lie within alluvial valley areas, and based on previous studies on or near the site (Rockwell, et al., 1986), that perched water may exist locally during and after development, owing to a combination of high rainfall, irrigation runoff and seepage, broken utilities, improper drainage, and/or relatively impermeable subsoils. During these previous fault evaluation studies (GSI; 2007b, 2007c, 1999a, 1999b, and 1987) evidence of a relatively high long-term groundwater level was documented only within the marsh deposits onsite. This evidence was from the geologic past, in the form of peat deposits and soil mottling. In contrast, modern evidence, only in the form of localized seepage and perched groundwater within the zone of faulting or within the marsh deposits, was observed. No evidence for artesian/spring conditions were noted during our investigations and subsurface water was encountered during our study at a depth of approximately ±32 feet b.g.s. However, these observations reflect site conditions at the time of our investigation and do not preclude changes in local groundwater conditions in the future from heavy irrigation, precipitation, or other factors not obvious at the time of our field work. Perched groundwater may occur in the future due to increased precipitation or increased irrigation and runoff from urbanization, and/or along zones of contrasting permeabilities (i.e., marsh deposits, alluvium, and older alluvial fan deposit contacts, etc.).

FAULTING AND REGIONAL SEISMICITY

The seismicity and seismic history of the project site have been previously described in GSI (2015, 2007b, 2007c, 1999a, and 1999c). The seismicity of the region has not changed appreciably since the issuance of those reports. Those studies also provided fault finding investigations which identified active faulting onsite (GSI, 1999a) and on the adjoining property to the north associated with the Elsinore fault zone. As such, appropriate structural setbacks were provided in GSI (1999a). The recommended structural setbacks are depicted on Plate 1.

The possibility of ground shaking at the site may be considered similar to the southern California region as a whole. The site is situated in an area of active as well as potentially-active faults. The Elsinore fault zone, design fault for the site, is considered active and is included within an Alquist-Priolo Earthquake Fault Zone (CGS, 2018). A list of the major faults and fault zones in southern California that could have a significant effect on the site, should they experience activity, is provided in Appendix C. The relationship of the location of the project area to these major mapped faults is indicated on the California Fault Map (Appendix C).

Seismicity

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

The program estimates the closest distance between each fault and a given site. If a fault is found to be within a user-selected radius, the program estimates peak horizontal ground acceleration that may occur at the site from an upper bound (formerly "maximum credible earthquake"), on that fault. Upper bound refers to the maximum expected ground acceleration produced from a given fault. Site acceleration (g) was computed by one user-selected acceleration-attenuation relation that is contained in EQFAULT. Based on the EQFAULT program, a peak horizontal ground acceleration from an upper bound event on the Glen Ivy segment of the Elsinore fault may be on the order of 0.83g. 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 August 15, 2018). This program performs a search of the historical earthquake records for magnitude 5.0 to 9.0 seismic events within a 100-kilometer radius, between the years 1800 through August 15, 2018. Based on the selected acceleration-attenuation relationship, a peak horizontal ground acceleration is estimated, which may have affected the site during the specific event listed. Based on the available data and the attenuation relationship used, the estimated maximum (peak) site acceleration during the period 1800 through August 15, 2018 was about 0.32g. A historic earthquake epicenter map and a seismic recurrence curve are also estimated/generated from the historical data. Computer printouts of the EQSEARCH program are presented in Appendix C.

Seismic Shaking Parameters

The following tables summarize the reevaluated site-specific design criteria obtained from the 2019 CBC, Chapter 16 Structural Design, Section 1613, Earthquake Loads for a Site Class of D, as determined by actual testing (see Appendix B). The computer program Seismic Design Maps, provided by the California Office of Statewide Health Planning and Development (OSHPD, 2019) has been utilized to aid in design (https://seismicmaps.org). The short spectral response utilizes a period of 0.2 seconds.

2019 CBC SEISMIC DESIGN PARAMETERS			
PARAMETER	**VALUE**	**VALUE per ASCE 7-16**	**2019 CBC or REFERENCE**
Risk Category	II	-	Table 1604.5
Site Class	D	-	Section 1613.2.2/Chap. 20 ASCE 7-16 (p. 203-204)
Spectral Response - (0.2 sec), S_s	2.458 g	-	Section 1613.2.1 Figure 1613.2.1(1)
Spectral Response - (1 sec), S_1	0.981 g	-	Section 1613.2.1 Figure 1613.2.1(2)
Site Coefficient, F_a	1.0	-	Table 1613.2.3(1)
Site Coefficient, F_v	null - see Section 11.4.8 ASCE 7-16	2.5 (1) (Section 21.3)	Table 1613.2.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec), S_{MS}	2.458 g	-	Section 1613.2.3 (Eqn 16-36)

2019 CBC SEISMIC DESIGN PARAMETERS			
PARAMETER	**VALUE**	**VALUE per ASCE 7-16**	**2019 CBC or REFERENCE**
Maximum Considered Earthquake Spectral Response Acceleration (1 sec), S_{M1}	null - see Section 11.4.8 ASCE 7-16	1.996 (2) (Section 21.4)	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	1.638 g	-	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	null - see Section 11.4.8 ASCE 7-16	1.331 (3) (Section 21.4)	Section 1613.2.4 (Eqn 16-39)
PGA_M - Probabilistic Vertical Ground Acceleration may be assumed as about 50% of these values.	1.139 g	-	ASCE 7-16 (Eqn 11.8.1)
Seismic Design Category	null - see Section 11.4.8 ASCE 7-16	E (3) (Section 11.6)	Section 1613.2.5/ASCE 7-16 (p. 85: Table 11.6-1 or 11.6-2)
1. $F_V = 2.5$ $S_1 > 0.2$ per Section 21.3 2. $S_{M1} = (1.5)S_{D1} = (1.5)(1.331) = 1.996$ per Section 21.4 3. $S_1 \geq 0.75 => 0.981 \geq 0.75$, per Section 11.6 site is in Risk Category E			

GENERAL SEISMIC PARAMETERS	
PARAMETER	**VALUE**
Distance to Seismic Source (B fault) (1)	±0.0 mi (0.0km) (2)
Upper Bound Earthquake (Elsinore - Glen Ivy fault)	$M_W = 6.8$ (1)
(1) - Cao, et al. (2003) (2) - Blake (2000)	

Conformance to the criteria above for seismic design does not constitute any kind of guarantee or assurance that significant structural damage, ground failure, or surface manifestations will not occur in the event of a large earthquake in this region. The primary goal of seismic design is to protect life, not to eliminate all damage, since such design may be economically prohibitive. Cumulative effects of seismic events are not addressed in the 2019 CBC and regular maintenance and repair following locally significant seismic events (i.e., M_w 5.5) will likely be necessary.

It is important to keep in perspective that in the event of a maximum probable or credible 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. This potential would be no greater than that for other existing structures and improvements in the immediate vicinity.

LIQUEFACTION POTENTIAL

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

One of the primary factors controlling the potential for liquefaction is 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 for depths below 60 feet when relative densities (D_r) are 40 to 60 percent and 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 should be concurrently present for liquefaction to occur: 1) sediments must be relatively young in age and not have developed a large amount of cementation; 2) sediments generally consist 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 experience a seismic event of a sufficient duration and magnitude, to induce straining of soil particles. Only two or three of the necessary five concurrent conditions have the potential to affect the Pleistocene-age fan deposits, after remedial grading. The marsh deposits appear to have the potential for all five necessary concurrent conditions for liquefaction.

The condition of liquefaction has two principal effects. One is the consolidation of loose sediments with resultant settlement of the ground surface. The other effect is lateral sliding. Significant permanent lateral movement generally occurs only when there is significant differential loading, such as fill or natural ground slopes within susceptible materials. Few such loading conditions exist on the site. In the site area, we found there is a potential for seismic activity. However, a high groundwater table (30 to 60 feet below the ground surface) does not exist within the older alluvial fan deposits that were silty, fine to coarse grained, massively bedded and become dense with depth. Soft sediment deformation features were noted only within the marsh deposits onsite. These features would only be expected if the site area had been subject to liquefaction in the past (Obermeier, 1996). Inasmuch as the future performance of the site with respect to liquefaction should be similar to the past, excluding the effects of urbanization (irrigation), GSI concludes that alluvial fan deposits generally have not been subject to liquefaction in the geologic past, regardless of the depth of the regional water table.

As previously discussed, no California seismic hazard zone mapping is available for the Lake Mathews Quadrangle. However, based on our review the site is located within an area designated as having a "moderate" potential for liquefaction (RCIT-GIS, 2018). Although some paleoliquefaction related features (i.e., sand boils, soft sediment deformation, etc.), presumably associated with near-field seismic activity, were noted within the marsh deposits in other nearby trenches (Rockwell, et al., 1986) and onsite (GSI, 1999a), our evaluation indicates that these features can be reasonably mitigated by use of appropriate remedial grading, building setbacks, and/or other foundation engineering design, as settlement-sensitive improvements are proposed within the areas delineated as "Quaternary marsh deposits" onsite (see Plate 1). It should be noted that no liquefaction related features have been identified within the Pleistocene-age alluvial fan deposits, as encountered during this, our fault finding investigation (GSI, 1999a), or our previous studies on the adjoining commercial property to the north (GSI; 2007b, 2007c, 1999c, and 1987). Based on the Quaternary marsh deposits encountered onsite, and in accordance with County mapping and current standards of practice, our evaluation and liquefaction analysis (pursuant to Special Publication 117A [SP 117A, 2008] see Appendix E) indicates that the potential for liquefaction and associated adverse effects within the Pleistocene-age alluvial fan deposits is considered low, and perhaps moderate within the Holocene-age marsh deposits onsite. The site conditions will also be improved by removal and recompaction of low density near-surface soils. Due to the relatively high peak horizontal ground acceleration anticipated onsite during the design seismic event, the vertical deformation due to densification of the fill and underlying fan deposits is possible, which may contribute to the differential seismic deformation across the buildings.

Seismic Densification

Seismic densification is a phenomenon that typically occurs in low relative density granular soils (i.e., classified as SP or SM) that are above the groundwater table and are significantly dry of optimum moisture content. During seismic-induced ground shaking, these natural sediments may deform under loading and volumetrically strain, resulting in ground surface settlements and densification. However, it was determined from current and previous geotechnical borings that most of these potentially susceptible materials are likely at or above optimum moisture content and/or are medium dense to dense. Therefore, the potential for seismically induced densification is considered low. Some minor densification of the fill and older alluvial fan deposits could occur.

SUBSIDENCE

Our experience in the site vicinity and review of readily available data indicates that there is no evidence that the overall project area is subsiding due to groundwater withdrawal. However, subsidence also occurs at the transition/slope condition between materials of substantially different engineering properties (i.e., bedrock vs. alluvium), or along active fault zones. Based on our review and field observations, localized active faulting has apparently created the Glen Ivy Marsh, which is located just northwest of the property. Based on our review, the Glen Ivy Marsh has resulted from the lateral displacement of topographically higher fan surfaces on the northeast side of the fault, resulting in a series of blocked drainages and ponding along the fault (Millman and Rockwell, 1986). In addition, the northeast side of the Glen Ivy North fault is maintaining nearly the same structural elevation with respect to the Santa Ana Mountains (Millman and Rockwell, 1986).

Also, our review of available stereoscopic aerial photographs (USDA, 1980, Appendix A) showed no features generally associated with areal subsidence (i.e., radially-directed drainages flowing into a depression(s), linearity of connecting depressions associated with mountain fronts, etc.). Furthermore, ground fissures are generally associated with excessive groundwater withdrawal and associated subsidence, or active faults. Our review did not indicate that excessive groundwater withdrawal and/or ground fissures in the specific site vicinity are occurring at this time. Nonetheless, with respect to the active fault zone and potential for subsidence of the marsh deposits, should tectonic induced subsidence (or uplift) occur, it should be inherently mitigated by removal and recompaction of near-surface low density soils, appropriate foundation design, and the recommended fault setback zone associated with the Glen Ivy North fault onsite (see Plate 1).

OTHER GEOLOGIC HAZARDS

Mass Wasting

Mass wasting refers to the various processes by which earth materials are moved down slope in response to the force of gravity. Indications of deep-seated landsliding, slope creep, or surficial failures on the site were not observed during our site investigations, and should not affect the site, provided our recommendations for development are properly implemented. Therefore, due to the generally flat lying nature of the site, the potential for mass wasting phenomena to effect the site is considered low. Likewise, the potential for seismically induced landsliding is considered low.

Due to the non-cohesive materials that may exist on portions of the site, caving and sloughing should be anticipated in all subsurface excavations and trenching. Appropriate safety considerations for potential caving and sloughing, such as shoring or layback cuts, should be incorporated into the construction design details.

The potential for surface flooding, although considered low, cannot be entirely precluded. Hence, this should be further evaluated by the design civil engineer due to the proximity of the site to the Glen Ivy Marsh and the new Coldwater Wash RCFC basin that adjoins the property on the South.

LABORATORY TESTING

Classification

Soils were classified visually according to the Unified Soils Classification System. The soil classifications are shown on the Boring Logs from this study, as wells as previous exploratory borings advanced onsite (Appendix B).

Moisture Density

The field moisture contents and dry unit weights were determined for undisturbed ring samples for the soils encountered in the exploratory borings. The dry unit weight was determined in pounds per cubic foot and the field moisture content was determined as a percentage of the dry unit weight. The results of these tests are shown on the Boring Logs (Appendix B).

Laboratory Standard

The maximum density and optimum moisture content was determined for the major soil type encountered in the exploratory borings. The laboratory standard used was ASTM D 1557. The moisture-density relationship obtained for the site soil is shown below:

SOIL TYPE	LOCATION & DEPTH (ft.)	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
Sandy SILT, Black w/ few Gravels (Marsh Deposits)	B-1 @ 0-5	106.2	15.8

Expansion Potential

Expansion Index (E.I.) tests were performed on a representative sample of site earth materials. The E.I. testing was performed in general accordance with ASTM Standard D 4829. Expansion index test results of <5 indicate that site soils tested are generally very low in expansion potential (E.I. 0-20). However, the presence of soils with a low expansion potential (E.I. 21-50) cannot be precluded. Additional E.I. testing should be conducted at the conclusion of site grading to further evaluate the preliminary test results obtained. The following table presents the results.

SAMPLE LOCATION AND DEPTH (FT)	EXPANSION INDEX	EXPANSION POTENTIAL*
B-1 @ 0-5	<5	Very Low

Grain Size Distribution

An evaluation was previously performed (GSI, 2007a) on selected representative soil samples in general accordance with ASTM D 422. The grain-size distribution curves are presented in Appendix D.

Direct Shear Test

Shear tests were performed on representative relatively undisturbed samples of site soils in general accordance with ASTM Test Method D 3080 in a Direct Shear Machine of the strain control type. The shear test results are presented in Appendix D.

Consolidation Testing

Consolidation testing was performed on relatively undisturbed soil samples in general accordance with ASTM Test Method D 2435. The consolidation test results are presented in Appendix D.

Soluble Sulfates/Corrosion

A typical sample of the site materials was analyzed for soluble sulfates, chloride, pH, and resistivity. The soluble sulfate and corrosion potential results are shown as follows:

LOCATION AND DEPTH (FT.)	SOLUBLE SULFATES PERCENTAGE BY WEIGHT	CHLORIDE (ppm)	PH	SATURATED RESISTIVITY (Ohms-cm)
B-1 @ 0-5	0.0620	30	4.1	930

A typical sample of the site materials was analyzed for soluble sulfate and corrosion potential. Based upon the soluble sulfate test results and the American Concrete Institute (ACI, 2014a), the soluble sulfate content for the subject site is categorized as Class "S0" (0.00 to 0.10 Water-Soluble Sulfate in Soil, Percentage by Mass for class S0), therefore on a preliminary sulfate-resistant concrete is not anticipated. Based on the results of the resistivity and pH testing, the onsite soils are generally considered extremely acid (a pH of 3.5 to 4.4 is considered extremely acid) and are generally severely corrosive to ferrous metals in a saturated state (a soil resistivity of below <1,000 is considered severely corrosive). Chloride levels are generally low. The soluble sulfate and corrosion test results are provided within Appendix D.

Although the site soils are categorized as severely corrosive to ferrous metals, other than Exposure Categories S0 and C1, no exposure conditions indicated in Table 19.3.1.1 of the ACI (2014a), were warranted, based on our preliminary laboratory testing. It is our understanding that ferrous metals embedded in properly placed and formed concrete should be adequately protected from these conditions. Typical development of this type does not generally use significant amounts of exposed metal piping and/or other buried metal improvements. Additional corrosion testing should be conducted at the conclusion of site grading to further evaluate the preliminary test results obtained. Based on the conditions encountered, a consulting corrosion engineer should be considered to provide recommendations for foundations, piping, etc., as warranted.

PRELIMINARY EARTHWORK FACTORS

Preliminary earthwork factors (shrinkage and bulking) for the subject property have been estimated based upon our field and laboratory testing, visual site observations, and experience in the site area. 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 exploring. However, all these factors are difficult to define in a three-dimensional fashion.

Therefore, the information presented below represents average shrinkage/bulking values:

Undocumented Artificial Fill	15% to 20% shrinkage
Topsoil/Colluvium	10% to 15% shrinkage
Alluvium - younger	12% to 15% shrinkage
Marsh Deposits	10% to 20% shrinkage
Older Alluvial Fan Deposits	5% to 10% shrinkage

An additional shrinkage factor item would include the removal of root systems of individual large plants or trees. These plants and trees vary in size, but when pulled, they may generally result in a loss of $\pm$½ to $\pm$1 cubic yards, to locally greater than $\pm$1½ cubic yards of volume, respectively. The above facts indicate that earthwork balance for the site would be difficult to define and flexibility in design is essential to achieve a balanced end product. Subsidence due to equipment loadings (dynamic compaction) may be on the order of up to 0.10 feet within the marsh deposits and 0.05 feet within the older alluvial fan deposits, but will depend on haul routes, etc.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on our previous (GSI, 1999a) and current field exploration, laboratory testing, and our engineering and geologic analyses, it is our opinion that the project site appears suited for the proposed mixed use development from a soils engineering and geologic viewpoint. The recommendations presented below should be incorporated in the design, grading, and construction considerations.

General

1. Soils engineering and compaction testing services should be provided during grading operations to assist the contractor in removing unsuitable soils and in his effort to compact the fill.

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

3. In general and based upon the available data to date, groundwater is not expected to be a significant factor in development of the site. However, due to the nature of the site materials, seepage may be encountered throughout the site along with <u>seasonal</u> perched water within drainage areas. In addition, seepage may be encountered in "daylighted" joint or discontinuity systems or sandy lenses within the native onsite earth materials. Therefore, subdrainage systems for the control of localized groundwater seepage should be anticipated.

4. Based upon our field explorations earth materials throughout the site should be rippable to the depths proposed.

5. Preliminary E.I. testing indicates that the near-surface onsite soils are not detrimentally expansive. However, additional E.I. testing will need to be performed during, or shortly after the conclusion of grading to further evaluate if the proposed structures will need to incorporate specific structural design for the mitigation of expansive soils, if encountered.

6. Preliminary soluble sulfate and corrosion testing on the marsh deposits indicates that the near-surface soils are generally considered extremely acid with respect to soil acidity/alkalinity; are severely corrosive to exposed buried metals when saturated. The soluble sulfate exposure is considered Class “S0", and the chloride content is generally low. Reinforced concrete mix design should minimally conform to Exposure Class “S0" and “C1" in Table 19.3.2.1 of ACI (2014a) since concrete would be exposed to moisture, and as per the corrosion engineer. Additional corrosion testing will need to be performed during, or shortly after the conclusion of grading.

7. The onsite soils are considered erosive. Thus, surface drainage should be designed to direct surface runoff water from the tops of any slopes and foundations. Vegetative covering should be established soon after site earthwork with plants that is capable of surviving the prevailing (semi-arid) climate. In the interim, temporary erosion control measures should be employed.

8. Due to the nature of some of the onsite materials, some caving and sloughing may be anticipated to be a factor in temporary subsurface excavations and trenching. Therefore, current local, state (CAL-OHSA), and federal safety ordinances for subsurface trenching should be enforced. On a preliminary basis, temporary slopes should be constructed in general accordance with CAL-OSHA guidelines for Type “B” soil conditions (i.e., 1:1 [h:v] slope) provided groundwater or running sands are not present. GSI does not consult in the area of safety engineering.

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

Demolition/Grubbing

1. Any existing surficial/subsurface structures (i.e., wells, septic systems, etc.), major vegetation, trees, and any miscellaneous debris should be removed from the areas of proposed grading.

2. The project geotechnical consultant should be notified of any previous foundation, irrigation lines, cesspools, septic tanks, leach fields, or other subsurface structures that are uncovered during the recommended removals, so that appropriate remedial recommendations can be provided.

3. Cavities or loose soils (including all previous fault finding trenches) remaining after demolition and site clearance should be cleaned out, observed by the soils engineer, processed, and replaced with fill that has been moisture conditioned to at least optimum moisture content and compacted to at least 90 percent of the laboratory standard (ASTM D 1557).

Treatment of Existing Ground

1. All undocumented artificial fill (including all previous fault finding trenches), topsoil/colluvium, and alluvium should be completely removed. Near surface weathered marsh deposits and older alluvial fan deposits should be removed to competent marsh deposits and older alluvial fan deposits (i.e., greater than or equal to 85 percent saturation, and/or greater than or equal to 105 pcf for in-place native materials), if not removed by proposed excavation within areas proposed for settlement-sensitive improvements. For preliminary planning purposes, undocumented fill thicknesses (including previous fault trenches) are estimated to be on the order of ±5 feet to as much as ±20 feet in previous fault trenches; approximately ±5 to ±10 feet in areas delineated as younger alluvium (Qal), approximately ±10 feet in areas delineated as marsh deposits (Qm); and approximately ±5 feet in areas delineated as older alluvial fan deposits (Qf). Variations of remedial removal thicknesses should be anticipated. Actual depths of removals would be evaluated in the field during grading by the geotechnical consultant.

2. Subsequent to the above removals, the upper 6 inches of the exposed subsoils 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).

3. The existing artificial fill, topsoil/colluvium, alluvium, marsh and older fan deposits, etc., may be reused as compacted fill provided that major concentrations of vegetation and miscellaneous trash and debris are removed prior to or during fill placement.

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

Fill Placement

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

2. In general, fill materials should be brought to at least optimum moisture, placed in thin 6- to 8-inch lifts and mechanically compacted to obtain a minimum relative compaction of 90 percent of the laboratory standard (ASTM D 1557).

3. If encountered, any oversized rock materials greater than 12 inches in diameter should be placed under the observation of the soils engineer and limited to depths in excess of 10 feet beneath any finish grade. Should appreciable amounts of rock be encountered, recommendations for placement could be provided at that time.

4. Any import materials should be observed and determined suitable by the soils engineer prior to placement on the site. Foundation designs may be altered if import materials have a greater sulfate and/or expansion values than the onsite materials encountered during our investigations.

Transition Areas/Overexcavation

In order to reduce the potential for differential settlement within transition areas, mitigate non-uniform subgrade soils, the entire cut area should generally be overexcavated to the depth of recommended remedial grading, a minimum depth of 3 feet below finish grade, or 2 feet below the foundation, whichever is greater, and/or to a maximum ratio of fill thickness on the building pad of 3:1 (maximum:minimum), and replaced with compacted fill. The County of Riverside requires that the minimum fill thickness beneath a building pad be at least half of the maximum fill thickness on the building pad.

Slope Considerations and Slope Design

As discussed previously, based on the relatively flat-lying nature of the project site, no significant slopes and/or cut slopes are currently anticipated. Any proposed fill slopes constructed using onsite materials, should be grossly and surficially stable provided the recommendations contained herein are implemented during site development.

All slopes should be designed and constructed in accordance with the minimum requirements of the 2019 CBC, County guidelines, and the recommendations contained in the General Earthwork, Grading Guidelines, and Preliminary Criteria section of this report (Appendix F), and the following:

1. Fill slopes should be designed at a 2:1 (h:v) gradients or flatter and should not exceed 15 feet in vertical height without further evaluation. Fill slopes should be properly built and compacted to a minimum relative compaction of 90 percent throughout (per ASTM D 1557), including the slope surfaces.

2. An evaluation of any proposed cut slopes prior to grading would be necessary in order to identify cut slopes that lie within areas of remedial grading and/or areas of non-cohesive materials. Should these conditions or materials be exposed during construction, the geotechnical engineer/geologist, would access the magnitude and extent of the materials and their potential affect on long-term maintenance or possible slope failures. Recommendations would then be made at the time of the field observation.

3. Surficial site soils are primarily granular (very low to potentially low expansive) with sandy silts and silty sands and considered erosive if subjected to rain or irrigation of sufficient intensity and/or duration. Recommendations for mitigation are provided herein.

Faulting/Seismicity Conclusions and Recommendations

1. Our previous onsite study (GSI; 1999a) and studies on the adjoining commercial property to the north (GSI; 2007b, 2007c, 1999c, and 1987) indicate that the onsite zone of faulting within the Elsinore fault is active (i.e., movement within the Holocene epoch, or last 11,000± years), according to the State of California (CGS, 2018). As such, appropriate structural setbacks were provided in GSI (1999a). The potential for surface fault rupture onsite will be mitigated by the recommended structural setback zones. The recommended structural setbacks are depicted on Plate 1 (Geotechnical Map). Based on our review of available data and literature, subsurface investigation, and soil stratigraphy, GSI concludes that active faults likely do not exist within the remainder of the property.

2. Severe seismic shaking may occur throughout the site, should an earthquake occur on one of the nearby active faults. Sympathetic movement along active and/or inactive fault planes is possible if an earthquake occurs on a nearby fault segment.

3. Should any utilities cross the fault zone they should be constructed at high angles to the fault trace in order to minimize the amount of damage should movement or subsidence occur. As such, appropriately located up-stream and down-stream cut-off valves for pressurized utilities (i.e., gas, water) to facilitate repair, should be considered for main-lines crossing the fault zone.

RECOMMENDATIONS - FOUNDATIONS

The proposed foundation systems should be designed and constructed in accordance with current standards of practice, the guidelines contained within the 2019 CBC, the

ACI (2014a), and the differential settlement and expansion potential values anticipated. The onsite soils expansion potentials for the project have been evaluated to be very low (E.I. 0 to 20). However, the presence of soils with a low expansion potential cannot be ruled out.

For the purpose of our geotechnical review and analyses, we understand that the buildings are proposed as one- to three-story structures, with slab-on-grade/continuous footings, utilizing typical wood-frame type of construction. Therefore, residential wall loads for one- to three-story structures are anticipated to be 1 to 3 kips per lineal foot of wall and 20 to 50 psf of concrete floor load. Isolated column loads are anticipated to be in the range of 10 to 50 kips. All footings are recommended to embed into compacted fill, as indicated in this report. When the final foundation loads are completed, GSI should be provided with this information for comment and/or review, as necessary.

Due to non-uniform bearing conditions and the potential for seismic settlement onsite, general recommendations for post-tension and mat foundation systems are provided in the following sections, and are not intended to preclude the transmission of water or water vapor through the foundations or slabs. Further discussion and recommendations are provided within the soil moisture transmission considerations section of this report. Unless specifically superceded in this report, all findings, conclusions and recommendations in previous referenced report (GSI, 1999a) remain pertinent and applicable, and should be incorporated into project plans and construction details.

General Foundation Design

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

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

3. Passive earth pressure in properly compacted sandy fill may be computed as an equivalent fluid having a density of 250 pcf, with a maximum earth pressure of 2,500 psf for footings founded into properly engineered fill. Lateral passive pressures for shallow foundations within 2019 CBC setback zones or within the influence of retaining walls should be reduced following a review by the geotechnical engineer unless proper setbacks can be established.

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

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

6. All footing setbacks from slopes should comply with Figure 1808.7.1 of the 2019 CBC. GSI recommends a minimum horizontal setback distance of 7 feet as measured from the bottom (i.e., bearing elevation), outboard edge of the footing to the slope face.

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

8. All interior and exterior column footings should be minimally tied to the perimeter wall footings in at least one direction for very low expansive soils, and two directions for low expansive soils, if encountered. The base of the reinforced grade beam should be at the same elevation as the adjoining footings.

9. Owing to regional seismicity, a minimum concrete slab-on-grade thickness of 5.0 inches is recommended.

10. The project structural engineer should consider the use of transverse and longitudinal control joints to help control slab cracking due to concrete shrinkage or expansion. Two of the best ways to control this movement are: 1) add a sufficient amount of reinforcing steel to increase the tensile strength of the slab; and 2) provide an adequate amount of control and/or expansion joints to accommodate anticipated concrete shrinkage and expansion. Transverse and longitudinal crack control joints should be spaced no more than 13 feet on center and constructed to a minimum depth of T/4, where "T" equals the slab thickness in inches. Per Portland Cement Association (PCA) and ACI guidelines, joints are commonly spaced at distances equal to 24 to 30 times the slab thickness. Joint spacing that is greater than 15 feet require the use of load transfer devices (dowels or diamond plates).

11. Provided the recommendations in this report are properly implemented, foundation systems should be minimally designed to accommodate the total and differential settlements provided herein. These settlement values do not apply to improvements constructed within 2019 CBC setbacks or within the influence of unmitigated soils.

Post-Tensioned Foundations

Post-tension foundations may be used to mitigate the damaging effects of differential settlement on the planned residential foundations and slab-on-grade floors. The post-tension foundation designer may elect to exceed these minimal recommendations to increase slab stiffness performance. Post-tension (PT) design may be either ribbed or mat-type. The latter is also referred to as uniform thickness foundation (UTF). The use of a UTF is an alternative to the traditional ribbed-type. The UTF offers a reduction in grade beams. That is to say a UTF typically uses a single perimeter grade beam and possible "shovel" footings, but has a thicker slab than the ribbed-type.

he information and recommendations presented in this section are not meant to supercede design by a registered structural engineer or civil engineer qualified to perform post-tensioned design. Post-tensioned foundations should be designed using sound engineering practice and be in accordance with local and 2019 CBC code requirements. Upon request, GSI can provide additional data/consultation regarding soil parameters as related to post-tensioned foundation design.

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

Slab Subgrade Pre-Soaking

Pre-moistening of the slab subgrade soil is recommended. The moisture content of the subgrade soils should be equal to or greater than optimum moisture to a depth equivalent to the perimeter grade beam or cut-off wall depth in the slab areas (typically 12 inches for very low expansive soil conditions).

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 to Low (0-50)	Upper 12 inches of pad soil moisture 2 percent over optimum (or 1.2 times)	Wetting and/or reprocessing	Periodically wet or cover with plastic after trenching. Evaluation 72 hours prior to placement of concrete.

Perimeter Cut-Off Walls

Perimeter cut-off walls should be at least 12 inches deep for very low to low expansive soil conditions. The cut-off walls may be integrated into the slab design or independent of the slab. The cut-off walls should be a minimum of 6 inches thick (wide). The bottom of the perimeter cut-off wall should be designed to resist tension, using cable or reinforcement per the structural engineer.

Post-Tensioned Foundation Design

The following recommendations for design of post-tensioned slabs have been prepared in general compliance with the requirements of the recent Post Tensioning Institute's (PTI's) publication titled "Standard Requirements for Design and Analysis of Shallow Post-tensioned Concrete Foundations on Expansive Soils" (PTI, 2012), together with it's subsequent erratas (PTI, 2013 and 2014).

Soil Support Parameters

The recommendations for soil support parameters have been provided based on the typical soil index properties for soils that are very low to low in expansion potential. The soil index properties are typically the upper bound values based on our experience and practice in the southern California area. The following table presents suggested minimum coefficients to be used in the Post-Tensioning Institute design method.

Thornthwaite Moisture Index	-20 inches/year
Correction Factor for Irrigation	20 inches/year
Depth to Constant Soil Suction	7 feet or overexcavation depth, whichever is greater

Constant soil Suction (pf)	3.6
Moisture Velocity	0.7 inches/month
Plasticity Index (P.I.)*	15-45
* - The effective plasticity index should be evaluated for the upper 7 to 15 feet of earth materials.	

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

DESIGN PARAMETERS	VERY LOW TO LOW EXPANSION (E.I. = 0-50)
e_m center lift	9.0 feet
e_m edge lift	5.2 feet
y_m center lift	0.4 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 12 inches, then by 20 percent for each additional foot of embedment to a maximum of 2,500 psf. [(2)] As measured below the lowest adjacent compacted subgrade surface without landscape layer or sand underlayment. Note: The use of open bottomed raised planters adjacent to foundations will require more onerous design parameters.	

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

Mat Foundations

In lieu of using a post-tensioned foundation to resist differential settlement and/or expansive soil effects, the Client may consider a mat foundation which uses steel bar reinforcement instead of post-tensioned cables. The structural engineer may supercede the following recommendations based on the planned building loads and use. Wire Reinforcement Institute (WRI, 2016) methodologies for design may be used.

Mat Foundation Design

The design of mat foundations should incorporate the vertical modulus of subgrade reaction. This value is a unit value for a 1-foot square footing and should be reduced in accordance with the following equation when used with the design of larger foundations. This assumes that the bearing soils will consist of engineered fills with an average relative compaction of 90 percent of the laboratory (ASTM D 1557).

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

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

The modulus of subgrade reaction (K_S) and effective plasticity index (PI) to be used in mat foundation design for various expansive soil conditions are presented in the following table.

VERY LOW TO LOW EXPANSION (E.I. = 0-50)
K_S =100 pci/inch, PI <15

Reinforcement bar sizing and spacing for mat slab foundations should be provided by the structural engineer. Mat slabs may be uniform thickness foundations (UTF) or may incorporate the use of edge footings for moisture cut-off barriers as recommended herein for post-tension foundations. Edge footings should be a minimum of 6 inches thick. The bottom of the edge footing should be designed to resist tension, using reinforcement per the structural engineer. The need and arrangement of interior grade beams (stiffening beams) will be in accordance with the structural consultant's recommendations. The recommendations for a mat type of foundation assume that the soils below the slab are compacted fill. The parameters herein are to mitigate the effects of expansive soils and should be modified to mitigate the effects of the total and differential settlements reported in the "Foundation and Improvement Settlements" section of this report. Specific pre-moistening/pre-soaking and moisture testing of the slab subgrade are recommended, as previously provided in this report. Slab subgrade moisture conditioning/pre-soaking should conform to the recommendations previously provided for post-tension foundation systems.

Confirmation Testing for Final Foundation Design

Following the completion of site grading, the expansion index, subgrade modulus, and corrosion potential of soils exposed near finish pad grades should be re-evaluated. Although not anticipated, the results of the recommended testing may require amendments to these preliminary recommendations.

FOUNDATION SETTLEMENTS

In designing foundations for the existing soil conditions, the estimated settlement and angular distortion values that an individual structure could be subjected to should be evaluated by a qualified structural engineer. In addition, significant site improvements such as retaining walls, sound walls, or other settlement-sensitive improvements should be evaluated by a structural engineer given the site conditions and geotechnical parameters expressed in this report. The levels of angular distortion were evaluated on a 40-foot length. This also applies to the other site improvements previously discussed.

The newly proposed building pad areas will be underlain by compacted fills, and at depth by Holocene-age marsh deposits and Pleistocene-age older alluvial fan deposits. GSI has assumed that site elevations may be raised on the order of ±3 to ±5 feet above existing grades. Post-construction settlement of improvements within the area under the purview of this report has been estimated to be approximately ±2 inches. The static differential settlement has been evaluated to be about 1.0 inch in 40 feet (i.e., 1 inch in a 40-foot span [1/480]).

Therefore, in addition to designing slab systems for the above soil conditions, the following settlement values should be utilized by the project structural engineer.

ESTIMATED TOTAL SETTLEMENT (INCHES)	ESTIMATED DIFFERENTIAL SETTLEMENT (INCHES)	ESTIMATED ANGULAR DISTORTION
<2.0	1.0	1/480

The estimates provided herein also include any contributions from post-construction settlement of the fill. These values do not take seismic effects from strong ground motion into account, nor do they apply to improvements constructed within the 2019 CBC setback zone. The designer should consider all the above-mentioned conditions in their design.

Post-construction settlement of the fill should be mitigated by remedial removals of near-surface low density soils, the structural setback zones, and proper foundation design, provided the design parameters provided herein are properly utilized for design of the foundation systems. In addition to the above, the structural engineer should also consider estimated settlements due to short duration seismic loading and applicable load combinations, as required by the City/County and/or the 2019 CBC. GSI should review the building loads, final building configurations, and revise the settlement estimates, if necessary.

SOIL MOISTURE TRANSMISSION CONSIDERATIONS

GSI has evaluated the potential for vapor or water transmission through the slabs, in light of typical floor coverings and improvements. Please note that typical slab moisture emission rates range from about 2 to 27 lbs/24 hours/1,000 square feet from a normal slab (Kanare, 2005), while floor covering manufacturers generally recommend about 3 lbs/24 hours as an upper limit. The recommendations in this section are not intended to preclude the transmission of water or vapor through the foundation or slabs. Foundation systems and slabs shall not allow water or water vapor to enter into the structure so as to cause damage to another building component or to limit the installation of the type of flooring materials typically used for the particular application (State of California, 2020). These recommendations may be exceeded or supplemented by a water "proofing" specialist, project architect, or structural consultant. Thus, the client will need to evaluate the following in light of a cost vs. benefit analysis (owner expectations and repairs/replacement), along with disclosure to all interested/affected parties.

Considering the E.I. test results, the anticipated typical water vapor transmission rates, floor coverings, and improvements (to be chosen by the owner) that can tolerate vapor transmission rates without significant 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 2019 CBC and the manufacturer's recommendation. The vapor retarder should comply with the ASTM E 1745 - Class A criteria, and be installed in accordance with ACI (2014a).

- 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.).

- Concrete slabs, including garages, shall be underlain by 2 inches of clean, washed sand (SE$\geq$30) above a 15-mil vapor retarder (ASTM E 1745 - Class A, per Engineering Bulletin 119 [Kanare, 2005]). The vapor retarder shall in-turn, be underlain by 2 inches of sand (SE$\geq$30) placed directly on the prepared, moisture conditioned, subgrade. The vapor retarder should be sealed to provide a continuous retarder under the entire slab and should be installed per the recommendations of the manufacturer, including all penetrations (i.e., pipe, ducting, rebar, etc.). The manufacturer shall provide instructions for lap sealing, including minimum width of lap, method of sealing, and either supply or specify suitable products for lap sealing (ASTM E 1745), and per code.

ACI 302.1R-04 (2004) states "If a cushion or sand layer is desired between the vapor retarder and the slab, care must be taken to protect the sand layer from taking on additional water from a source such as rain, curing, cutting, or cleaning. Wet cushion or sand layer has been directly linked in the past to significant lengthening of time required for a slab to reach an acceptable level of dryness for floor covering applications." Therefore, additional observation and/or testing will be necessary for the cushion or sand layer for moisture content, and relatively uniform thicknesses, prior to the placement of concrete.

- Concrete should have a maximum water/cement ratio of 0.50. This does not supercede Table 4.3.1 of Chapter 4 the ACI (2008) for corrosion or other corrosive requirements. Additional concrete mix design recommendations should be provided by the structural consultant and/or waterproofing specialist. Concrete finishing and workablity should be addressed by the structural consultant and a waterproofing specialist.

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

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

- Additional recommendations regarding water or vapor transmission should be provided by the architect/structural engineer/slab or foundation designer and should be consistent with the specified floor coverings indicated by the architect. Regardless of the mitigation, some limited moisture/moisture vapor transmission through the slab should be anticipated. Construction crews may require special training for installation of certain product(s), as well as concrete finishing techniques. The use of specialized product(s) should be approved by the slab designer and water-proofing consultant. A technical representative of the flooring contractor should review the slab and moisture retarder plans and provide comment prior to the construction of the foundations or improvements. The vapor retarder contractor should have representatives onsite during the initial installation.

PRELIMINARY PAVEMENT DESIGN RECOMMENDATIONS

General

The governing agency may retain the authority to approve the final structural design sections after subgrade elevations and actual resistance values (R-values) have been obtained at the conclusion of earthwork. Based on an assumed R-value of 16 (marsh deposits) and 30 (fan deposits), general review of pavement designs for other nearby projects, and for estimation and bidding purposes, the pavement sections provided herein should be considered for preliminary design. Typically actual pavement sections will likely vary, therefore final pavement sections should be based on actual R-value testing performed during, or shortly after, roadway grading for any proposed street and driveway/parking area improvements.

The preliminary pavement sections presented in the following table are based on general Traffic Indices (T.I.'s), utilized by the controlling authorities for an access road/local street, an enhanced local street, a collector street, an industrial collector, a secondary highway, and the guidelines presented in the latest revision to the California Department of Transportation "Highway Design Manual" sixth edition. It is our understanding that the minimum pavement section required by the controlling authorities for an access road/local street is 3 inches of AC (asphaltic concrete) on 6 inches of Class 2 aggregate base for a traffic index of 5.5. Based on assumed R-values (i.e., R=16 and 30), the following preliminary pavement designs are presented. Applicable sections of City/County ordinances should be followed during design of public roads, fire access lanes, etc.

STREET CLASSIFICATION	TRAFFIC INDEX (T.I.)[1]	STANDARD PAVEMENT DESIGNS		
		R-VALUE	AC* INCHES	CLASS 2 BASE ROCK[2] INCHES*
Access Road/Local Streets	5.5	16/30	3.0/3.0	10.0/7.0
Enhanced Local Street	6.5	16/30	3.6**/3.6**	12.0/9.0
Collector Street	7.0	16/30	4.0**/4.0**	12.5/9.5
Industrial Collector Street	8.0	16/30	4.7**/4.7**	15.0/11.0
Secondary Highway	8.5	16/30	5.2**/5.2**	16.0/12.0

[1] T.I.s are assumed based on County street/highway criteria.
[2] Assumed R-values for base rock R=78 - Cal-Trans standard Class 2 base rock.
* County minimum asphaltic concrete and/or crushed aggregate base requirements may vary.
** County minimum asphaltic concrete

The preliminary 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 T.I. used for design, increased maintenance and repair could be required for the pavement section. Consideration should be given to the increased potential for distress from overuse of paved street areas by heavy equipment and/or construction related heavy traffic (e.g., concrete trucks, loaded supply trucks, etc.), particularly when the final section is not in place (i.e., topcoat). Best management construction practices should be followed at all times, especially during inclement weather.

PAVEMENT GRADING RECOMMENDATIONS

General

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

Subgrade

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

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

Crushed 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 determined by ASTM test method D 1557 and/or Caltrans Test Method Number California 216.

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 geotechnical consultant.

Drainage

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

Additional Considerations

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

PRELIMINARY 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 a maximum E.I. of 50 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 should also be provided for site retaining walls in order to reduce the potential for efflorescence staining.

Retaining Wall Foundation Design

Foundation design for retaining walls should incorporate the following recommendations:

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

Minimum Footing Width - 24 inches

Allowable Vertical Bearing Pressure - An allowable vertical bearing pressure of 2,500 pcf may be used in the preliminary design of retaining wall foundations provided that the footing maintains a minimum width of 24 inches and extends at least 18 inches into approved engineered fill overlying dense formational materials. This pressure may be increased by one-third for short-term wind and/or seismic loads.

Passive Earth Pressure - A passive earth pressure of 250 pcf with a maximum earth pressure of 2,500 psf may be used in the preliminary design of retaining wall foundations provided the foundation is embedded into properly compacted silty to clayey sand fill.

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

Backfill Soil Density - A soil density of 120 to 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).

Settlement - Provided that the earthwork and foundation recommendations in this report are adhered, foundations bearing on approved non-detrimentally expansive, engineered fill should be minimally designed to accommodate a total static settlement of 2 inches and a differential static settlement of 1 inch over a 40-foot horizontal span (angular distortion = 1/480).

Any retaining wall footings near the perimeter of the site, or not within areas of placed compacted fills will likely need to be deepened into unweathered dense formational materials for adequate vertical and lateral bearing support. All retaining wall footing setbacks from slopes should comply with Figure 1808.7.1 of the 2019 CBC. GSI recommends a minimum horizontal setback distance of 7 feet as measured from the bottom, outboard edge of the footing to the 2:1 (h:v) 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 to low expansive (E.I. ≤ 50, P.I. <15) native (onsite) backfill, respectively. The design should include any applicable surcharge loading. For areas of male or re-entrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall (2H) laterally from the corner.

Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls up to 10 feet high. Design parameters for walls less than 3 feet in height may be superceded by Los Angeles County 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 loads on the back of retaining walls where vehicular traffic could occur within horizontal distance "H" from the back of the retaining wall (where "H" equals the wall height). The traffic surcharge may be taken as 100 psf/ft in the upper 5 feet of backfill for light truck and cars traffic. This does not include the surcharge of parked vehicles which should be evaluated at a higher surcharge to account for the effects of seismic loading. Equivalent fluid pressures for the design of cantilevered retaining walls are provided in the following table:

SURFACE SLOPE OF RETAINED MATERIAL (HORIZONTAL:VERTICAL)	EQUIVALENT FLUID WEIGHT P.C.F. (SELECT BACKFILL)[2]	EQUIVALENT FLUID WEIGHT P.C.F. (NATIVE BACKFILL)[3]
Level[1]	38	50
2 to 1	55	65

[1] Level backfill behind a retaining wall is defined as compacted earth materials, properly drained, without a slope for a distance of 2H behind the wall, where H is the height of the wall.
[2] SE ≥ 30, P.I. < 15, E.I. < 21, and ≤ 10% passing No. 200 sieve.
[3] E.I. = 0 to 50, SE ≥ 30, P.I. < 15, E.I. < 21, and ≤ 15% passing No. 200 sieve. Assumes 1 to 2 feet of gravel drain backfill be incorporated (see Details herein).

Seismic Surcharge

For engineered retaining walls with more than 6 feet of retained materials, as measured vertically from the bottom of the wall footing at the heel to daylight , GSI recommends that the walls be evaluated for a seismic surcharge (in general accordance with 2019 CBC requirements). The site walls in this category should maintain an overturning Factor-of-Safety (FOS) of approximately 1.25 when the seismic surcharge (increment), is applied. For restrained walls, the seismic surcharge should be applied as a uniform surcharge load from the bottom of the footing (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 applied as an inverted triangular distribution using 15H. For restrained walls, the pressure should be applied as a rectangular distribution. Please note this is for local wall stability only.

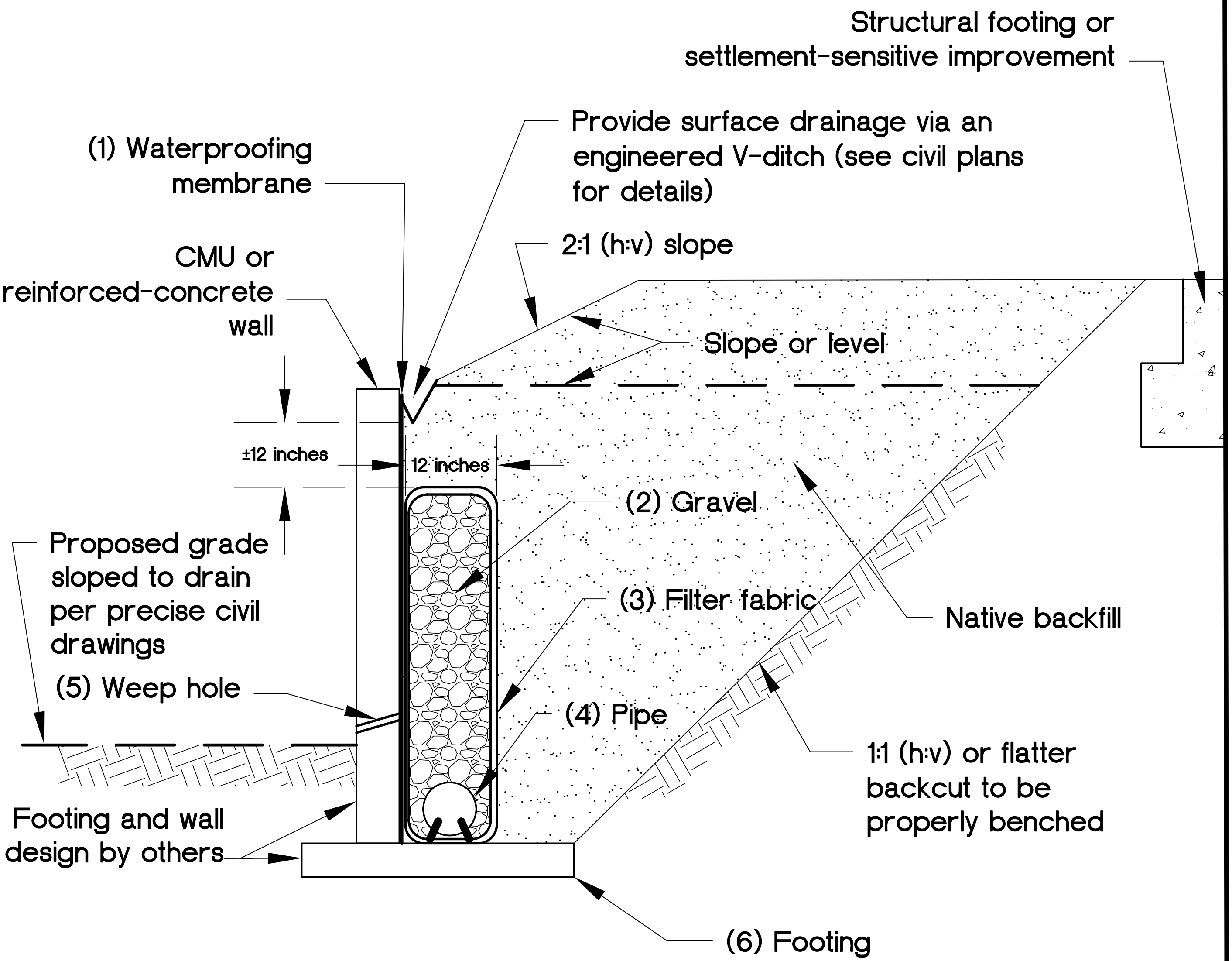
The 15H is derived from a Mononobe-Okabe solution for both restrained cantilever walls. This accounts for the increased lateral pressure due to shakedown or movement of the sand fill soil in the zone of influence from the wall or roughly a 45˚ - ϕ/2 plane away from the back of the wall. The 15H seismic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

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

Where:	P_h	=	Seismic increment
	a_h	=	Probabilistic horizontal site acceleration with a percentage of "g."
	γ_t	=	total unit weight (120 to 125 pcf for site soils @ 90% relative compaction).
	H	=	Height of the wall from the bottom of the footing or point of pile fixity.

Retaining Wall Backfill and Drainage

Positive drainage must be provided behind all retaining walls in the form of gravel wrapped in geofabric and outlets. A backdrain system is considered necessary for retaining walls that are 2 feet or greater in height. Details 1, 2, and 3, present the 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 select backfill, the filter material should extend a minimum of 1 horizontal foot behind the base of the walls and upward at least 1 foot. For native backfill that has up to E.I. = 50 (P.I. < 15), continuous Class 2 permeable drain materials should be used behind the wall. This material should be continuous (i.e., full height) behind the wall, and it should be constructed in accordance



(1) Waterproofing membrane.

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

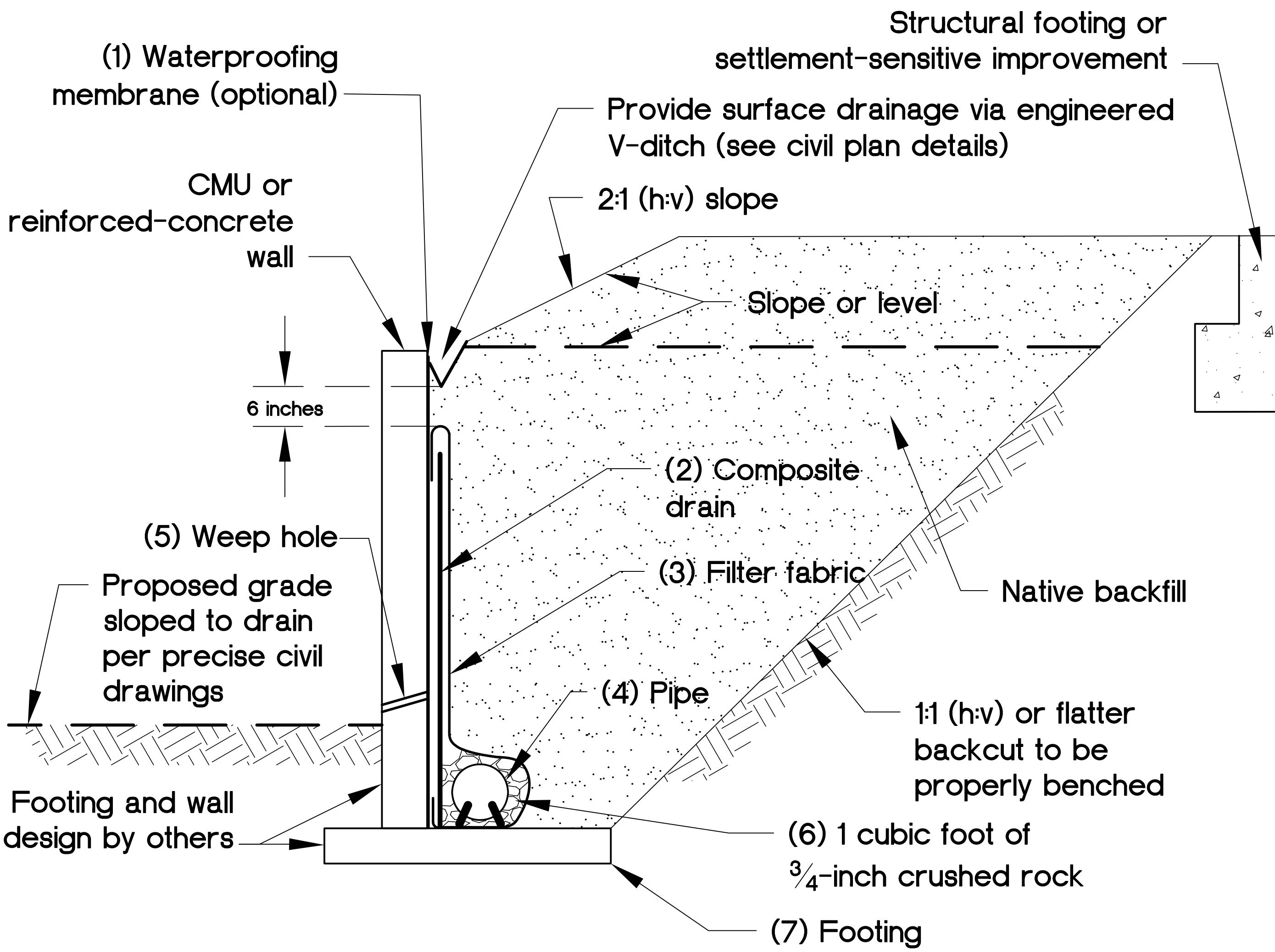
(3) Filter fabric: Mirafi 140N or approved equivalent.

(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.

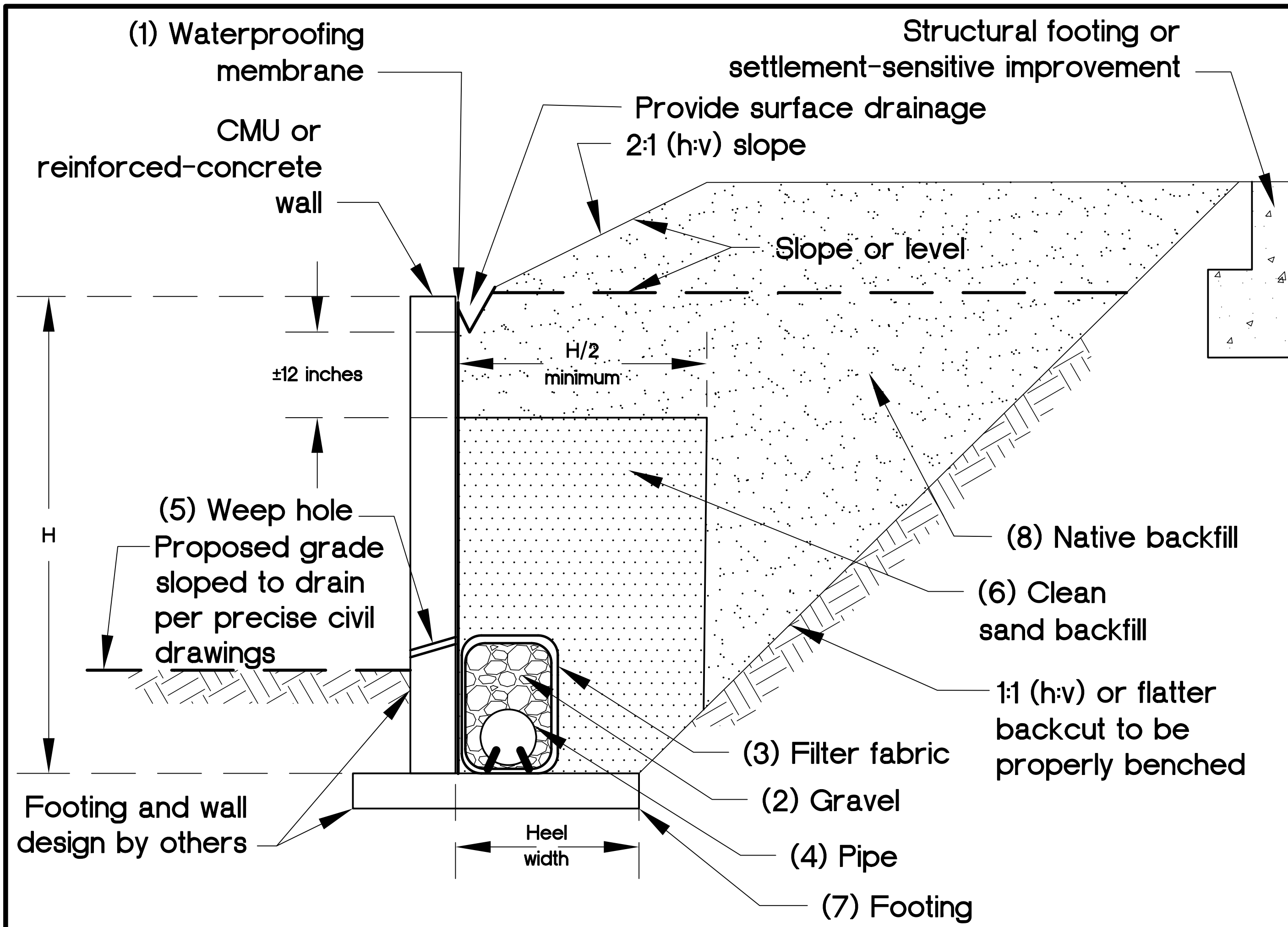
(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

(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 $1\frac{1}{2}$ 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. ≥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

with the enclosed Detail 1 (Typical Retaining Wall Backfill and Drainage Detail). For limited access and confined areas, (panel) drainage behind the wall may be constructed in accordance with Detail 2 (Retaining Wall Backfill and Subdrain Detail Geotextile Drain). Materials with an expansion index (E.I.) potential of greater than 50 and/or P.I. > 15 should not be used as backfill for retaining walls. Retaining wall backfill materials should be moisture conditioned and mixed to achieve the soil's optimum moisture content, placed in relatively thin lifts (6 to 10 inches) with relatively light equipment, and compacted to at least 90 percent relative compaction. 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 and P.I. < 15). 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 structural consultant/wall designer may specify either:

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

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

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

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

Slope Setback Considerations for Footings

Footings should maintain a horizontal distance, X, between any adjacent descending slope face and the bottom outer edge of the footing, and minimally comply with the guidelines depicted on Figure 1808.7.1 of the 2019 CBC. 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.

DRIVEWAY, FLATWORK, AND OTHER IMPROVEMENTS

Some of the site soil materials on site may be expansive. 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 reduce the likelihood of distress, the following recommendations are presented for all exterior flatwork:

1. The subgrade area for exterior 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 125 percent of) the soils' optimum moisture content, to a depth of 18 inches below subgrade elevation. If very low expansive soils are present, only optimum moisture content, or greater, is required and specific presoaking is not warranted. The moisture content of the subgrade should be proof tested within 72 hours prior to pouring concrete.

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

3. Exterior slabs should be a minimum of 4 inches thick. Driveway slabs and approaches should additionally have a thickened edge (12 inches) adjacent to all landscape areas, to help impede infiltration of landscape water under the slab.

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

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

6. Driveways, sidewalks, and patio slabs adjacent to the house should be separated from the house 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.

7. Planters and walls should not be tied to the house.

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

9. 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.

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

11. Positive site drainage should be maintained at all times. Finish grades on the building pads should provide a minimum of 1 to 2 percent fall to the street, as indicated herein. It should be kept in mind that drainage reversals could occur, including post-construction settlement, if relatively flat yard drainage gradients are not periodically maintained by the owner.

12. 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.

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

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.

- 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).

- Impermeable liners used in conjunction with 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, at a maximum inclination of 4:1 (h:v), and meets the following minimum specifications:

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

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

- 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 (horizontal to vertical [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 fines into 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 Deformation

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

It is generally not practical to attempt to eliminate the effects of either slope creep or LFE. Suitable mitigative measures to reduce the potential of lateral deformation typically include:

setback of improvements from the slope faces (per the adopted California Building Code), positive structural separations (i.e., joints) between improvements, and stiffening and deepening of foundations. Expansion joints in walls should be placed no greater than 20 feet on-center, and in accordance with the structural engineer's recommendations. All of these measures are recommended for design of structures and improvements. The ramifications of the above conditions, and recommendations for mitigation, should be provided to each owner and/or any owners association.

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 owners. Over-steepening of slopes should be avoided during building construction activities and landscaping.

Drainage

Adequate building pad 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 building pad, and especially near structures and tops of slopes. Building 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 building pads 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.

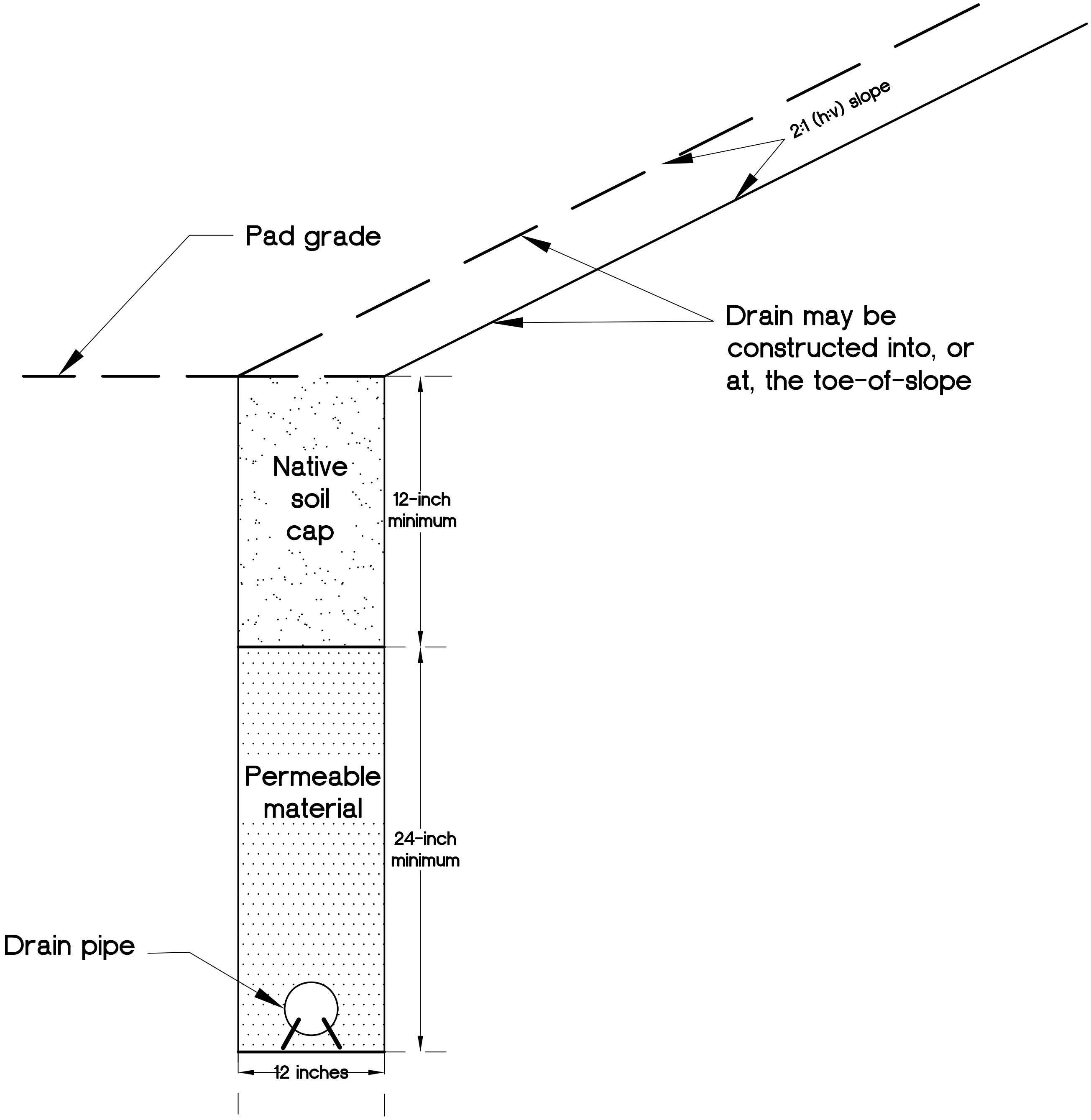
Consideration should be given to avoiding construction of planters adjacent to structures. Pad drainage should be directed toward the street or other approved area(s). Although not a geotechnical requirement, roof gutters, downspouts, or other appropriate, means may be utilized to control roof drainage. Downspouts, 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.)?
- Are there cut-fill transitions (i.e., fill over bedrock), within the slope?
- Was the building pad at the base of the slope overexcavated or is it proposed to be overexcavated? Overexcavated building pads 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” onto a building pad area or a building pad 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 building pad areas. Following are Detail 4 (Schematic Toe Drain Detail) and



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

Detail 5 (Subdrain Along Retaining Wall Detail). Other drains may be warranted due to unforeseen conditions, 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 owners and any owners association.

Erosion Control

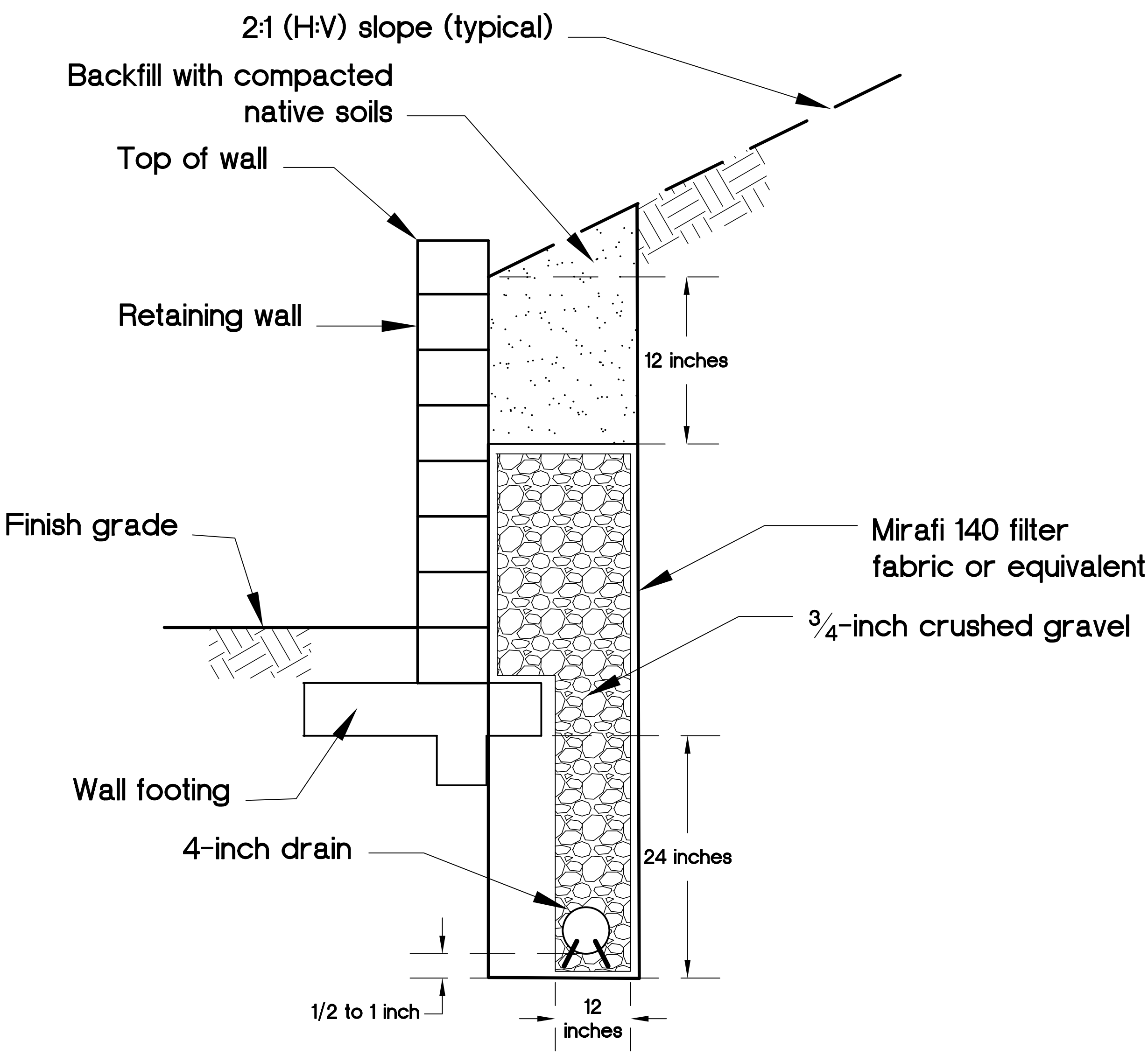
Cut and fill slopes will be subject to surficial erosion during and after grading. Onsite earth materials have a moderate to high erosion potential. Consideration should be given to providing hay bales and silt fences for the temporary control of surface water, from a geotechnical viewpoint.

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.

Gutters and Downspouts

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



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

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 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 construction recommendation should be provided to owners, any owners association, and/or other interested parties. This office should be notified in advance of any fill placement, grading of the site, or trench backfilling after rough grading has been completed. This includes any grading, utility trench and retaining wall backfills, flatwork, etc.

Tile Flooring

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

Additional Grading

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

Footing Trench Excavation

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

Trenching/Temporary Construction Backcuts

Considering the nature of the onsite earth materials, it should be anticipated that caving or sloughing could be a factor in subsurface excavations and trenching. Shoring or excavating the trench walls/backcuts at the angle of repose (typically 25 to 45 degrees [except as specifically superceded within the text of this report]), should be anticipated. All excavations should be observed by an engineering geologist or soil engineer from GSI, prior to workers entering the excavation or trench, and minimally conform to CAL-OSHA, state, and local safety codes. Should adverse conditions exist, appropriate recommendations would be offered at that time. The above recommendations should be provided to any contractors and/or subcontractors, or owners, etc., that may perform such work.

Utility Trench Backfill

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

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

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

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

SUMMARY OF RECOMMENDATIONS REGARDING GEOTECHNICAL OBSERVATION AND TESTING

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

- During grading/recertification.
- During excavation.
- During placement of subdrains, toe drains, or other subdrainage devices, prior to placing fill and/or backfill.
- After excavation of building footings, retaining wall footings, and free standing walls footings, prior to the placement of reinforcing steel or concrete.
- Prior to pouring any slabs or flatwork, after presoaking/presaturation of building pads and other flatwork subgrade, before the placement of concrete, reinforcing steel, capillary break (i.e., sand, pea-gravel, etc.), or vapor barriers (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 developer or owner improvements, such as flatwork, walls, etc., are constructed, prior to construction.
- A report of geotechnical observation and testing should be provided at the conclusion of each of the above stages, in order to provide concise and clear documentation of site work, and/or to comply with code requirements.

OTHER DESIGN PROFESSIONALS/CONSULTANTS

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

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

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

PLAN REVIEW

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

LIMITATIONS

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

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

APPENDIX A

REFERENCES

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

REFERENCES

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APPENDIX B

BORING LOGS
AND CPT DATA

GeoSoils, Inc.

BORING LOG

PROJECT: GLEN IVY PROPERTIES, LLC
SWC of Temescal Canyon Road and Trilogy Parkway
Temescal Valley

W.O. 7731-A-SC BORING B-1 SHEET 1 OF 2

DATE EXCAVATED 1-23-20 LOGGED BY: TAG APPROX. ELEV.: 1,088'

SAMPLE METHOD: 8" HSA - 140 lbs @ 30" Drop, Cal-Sampler & SPT

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
0				SM				**COLLUVIUM/TOPSOIL:** @ 0' SILTY SAND, very dark brown, moist, loose; few roots/rootlets.
				SM				**QUATERNARY MARSH DEPOSITS (Qm):**
5			3	ML/ SM	69.9	33.1	64.1	@ 5' SILT to SILTY SAND, black to yellowish red, moist, soft to very loose; interbedded.
10			6					@ 10' CLAYEY SILT (black) to SILTY SAND (yellowish brown), moist, medium stiff to loose; few peat (pt) layers, interbedded.
15			21		115.1	11.2	67.6	@ 15' CLAYEY SILT to SILTY SAND, black to dark gray (gleyed), moist, very stiff to medium dense.
20			12	SM				@ 20' SILTY SAND, yellowish brown, damp, medium dense; fine to coarse grained
25			48	SM/ GM	121.3	10.5	76.6	@ 25' SILTY SAND to SANDY GRAVEL, dark reddish yellow, wet, dense; seepage encountered along gravels.
30			14	ML/ SM				@ 30' CLAYEY SILT to few SILTY SAND interbeds, very dark gray, wet, stiff to medium dense. @ 32' Groundwater encountered.

Standard Penetration Test
Undisturbed, Ring Sample
Groundwater
Seepage

GeoSoils, Inc.

BORING LOG

PROJECT: GLEN IVY PROPERTIES, LLC
SWC of Temescal Canyon Road and Trilogy Parkway
Temescal Valley

W.O. 7731-A-SC BORING B-1 SHEET 2 OF 2

DATE EXCAVATED 1-23-20 LOGGED BY: TAG APPROX. ELEV.: 1,088'

SAMPLE METHOD: 8" HSA - 140 lbs @ 30" Drop, Cal-Sampler & SPT

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
35		Undisturbed, Ring Sample	25	ML	94.3	26.8	94.2	@ 35' CLAYEY SILT to SILT, blue gray, wet, very stiff; gleyed.
40		Standard Penetration Test	27	SM				@ 40' SILTY SAND, yellowish brown, wet, medium dense; fine to very coarse grained.
45		Undisturbed, Ring Sample	22/ 50-5"	GM	137.8	5.6	73.2	@ 45' SANDY GRAVEL, grayish brown, moist, dense.
								@ 47' Drill rig chatter; Practical refusal @ 47' on cobbles or boulder. Total Depth = 47' Groundwater Encountered @ 32' No Caving Encountered Backfilled 1-23-20
50								
55								
60								
65								

Standard Penetration Test
Undisturbed, Ring Sample
Groundwater
Seepage

GeoSoils, Inc. — BORING LOG

PROJECT: GLEN IVY PROPERTIES, LLC
SWC of Temescal Canyon Road and Trilogy Parkway
Temescal Valley

W.O. 7731-A-SC *BORING* B-2 *SHEET* 1 *OF* 1

DATE EXCAVATED 1-23-20 *LOGGED BY:* TAG *APPROX. ELEV.:* 1,092 MSL

SAMPLE METHOD: 8" HSA - 140 lbs @ 30" Drop, Cal-Sampler & SPT

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
0				SM				**COLLUVIUM/TOPSOIL:** @ 0' SILTY SAND with GRAVEL, very dark brown, damp, loose.
				SM				**QUATERNARY MARSH DEPOSITS (Qm):**
5		Ring	5		63.0	9.5	15.5	@ 5' SILTY SAND, reddish yellow, dry, loose.
10		Ring	31	SM/ ML	86.2	8.6	24.8	@ 10' SILTY SAND to SANDY SILT (interbedded), dark gray to pale brown, dry, medium dense to very stiff.
15		SPT	27	SM/ SC				@ 15' SILTY to CLAYEY SAND (interbedded), black, medium dense, dry; fine to coarse grained, few organics.
20		Ring	49	GM	117.3	8.3	53.6	@ 20' SANDY GRAVEL, dark grayish brown, moist, dense; fine to coarse grained sands.
								Drill rig chatter.
25		SPT	81	SM				SILTY SAND with GRAVEL, light brown, dry, very dense.
								Practical Refusal @ 26½' Total Depth = 26½ No Groundwater or Caving Encountered Backfilled 1-23-20
30								

Standard Penetration Test
Undisturbed, Ring Sample
Groundwater
Seepage

GeoSoils, Inc.

BORING LOG

PROJECT: GLEN IVY PROPERTIES, LLC
SWC of Temescal Canyon Road and Trilogy Parkway
Temescal Valley

W.O. 7731-A-SC *BORING* B-3 *SHEET* 1 *OF* 1

DATE EXCAVATED 1-23-20 *LOGGED BY:* TAG *APPROX. ELEV.:* 1,090'

SAMPLE METHOD: 8" HSA - 140 lbs @ 30" Drop, Cal-Sampler & SPT

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
0				SC				**COLLUVIUM/TOPSOIL:** @ 0' CLAYEY SAND, light to medium brown, damp, loose.
				ML/ SM				**QUATERNARY MARSH DEPOSITS (Qm):**
5		SPT	3					@ 5' SILTY SAND, interbedded, black to gray, dry, very loose; few peat (pt) interbeds, fine grained.
10		Ring	39		96.8	20.1	74.9	@ 10' CLAYEY SILT (black), to SILTY SAND (dark gray), interbedded, moist, hard to dense.
15		SPT	11					@ 15' SILTY SAND (pale brown) to SILT (very dark gray), interbedded, moist, medium dense to stiff.
20		Ring	77	SM	118.0	10.7	70.6	@ 20' SILTY SAND, dark gray, moist, very dense.
								Total Depth = 21½' No Groundwater or Caving Encountered Backfilled 1-23-20
25								
30								

Standard Penetration Test
Undisturbed, Ring Sample
Groundwater
Seepage

GeoSoils, Inc.

BORING LOG

PROJECT: GLEN IVY PROPERTIES, LLC
SWC of Temescal Canyon Road and Trilogy Parkway
Temescal Valley

W.O. 7731-A-SC *BORING* B-4 *SHEET* 1 *OF* 1

DATE EXCAVATED 2-7-20 *LOGGED BY:* RGC *APPROX. ELEV.:* 1088' MSL

SAMPLE METHOD: Track Mounted, Hollow Stem Auger

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
0				SM				**COLLUVIUM/TOPSOIL:** @ 0' SAND, brown, moist, loose; few roots, bioturbated.
				SM				**QUATERNARY ALLUVIAL FAN DEPOSITS (Qf):** @ 2' Gravelly SAND, light brown, moist, loose; few subangular gravels.
				GW				@ 4' Gravelly SAND, brown and dark gray, damp, dense.
5			37					@ 5' As per 4'.
				SM	108.3	5.2	26.4	@ 6' SILTY SAND, light olive brown, damp, dense.
				GW				@ 8' Gravelly SAND, dark yellowish brown, damp, medium dense.
10			26	SM				@ 10' SILTY SAND, with gravel, dark yellowish brown, damp, medium dense.
15			57	GM				@ 15' Gravelly SAND with SILT, light yellowish brown, damp, dense.
				SM	106.3	7.7	36.8	@ 16' SILTY SAND, pale brown, damp, dense.
20			33/ 50-6"	GW				@ 20' Gravelly SAND, brown, dry, dense; 30-40% subangular to subrounded gravels.
								Total Depth = 21½' No Groundwater or Caving Encountered. Backfilled 2-7-20.
25								
30								

Standard Penetration Test
Undisturbed, Ring Sample
Groundwater
Seepage

GeoSoils, Inc. — BORING LOG

PROJECT: GLEN IVY PROPERTIES, LLC
SWC of Temescal Canyon Road and Trilogy Parkway
Temescal Valley

W.O. 7731-A-SC BORING B-5 SHEET 1 OF 1

DATE EXCAVATED 2-7-20 LOGGED BY: RGC APPROX. ELEV.: 1092' MSL

SAMPLE METHOD: Track Mounted, Hollow Stem Auger

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
0				SM				**COLLUVIUM/TOPSOIL:** @ 0' SILTY SAND, dark gray brown, moist, loose; few roots, bioturbates.
2½				SM				**QUATERNARY ALLUVIUM - YOUNGER (Qal):** @ 2½' SILTY SAND with gravel, brown, damp, loose to medium dense.
5	X	SPT	11					@ 5' As per 2½', gravels < 5%, medium dense.
10		Ring	24 50-6"	GM	97.1	4.6	17.2	**QUATERNARY ALLUVIAL FAN DEPOSITS (Qf):** @ 10' Gravelly SAND with SILT, yellowish brown, dry, dense; gravels are subrounded to subangular.
14				GW				@ 14' Gravel layer.
15		SPT	70	GM				@ 15' Gravelly SILTY SAND, as per 10'.
20		Ring	50-5"		118.3	4.0	26.5	@ 20' As per 15', damp, difficult drilling in gravel.

Total Depth = 20½'
No Groundwater or Caving Encountered
Backfilled 2-7-20

Standard Penetration Test
Undisturbed, Ring Sample
Groundwater
Seepage

GeoSoils, Inc.

BORING LOG

PROJECT: GLEN IVY PROPERTIES, LLC SWC of Temescal Canyon Road and Trilogy Parkway Temescal Valley	*W.O.* 7731-A-SC *BORING* B-6 *SHEET* 1 *OF* 2 *DATE EXCAVATED* 2-7-20 *LOGGED BY:* RGC *APPROX. ELEV.:* 1094' MSL *SAMPLE METHOD:* Track Mounted, Hollow Stem Auger

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
0				SM				**COLLUVIUM/TOPSOIL:** @ 0' SILTY SAND, very dark gray brown, moist, loose; bioturbated, trace gravels.
				SP-SM				**QUATERNARY MARSH DEPOSITS (Qm):** @ 2' Fine SAND, light gray, dry, loose; interlayered with SILTY fine SAND, dark brown, loose.
5			6		78.0	7.5	17.8	@ 5' As per 2'.
				GW				@ 9' Gravel layer.
10			11	ML/SM				@ 10' Interlayered SANDY SILT, very dark gray brown, moist, stiff; and SILTY fine SAND, yellowish brown, slightly moist, medium dense.
15			39	SP-SM	106.7	8.6	41.2	@ 15' Interlayered SILTY fine SAND, light gray, damp, dense; and SILTY SAND, yellowish brown, damp, dense; trace fine subangular gravel.
20			35	SP				@ 20' SAND, gray and yellowish brown (mottled), damp, dense; 3" layer of gravelly SAND at 21'.
25			50-5"	SW	100.5	8.5	34.1	@ 25' SAND, brown, damp, dense; few fine subangular gravels, some SILT.
30			50					@ 30' SAND, mottled brown and yellowish brown, moist, dense.
								@ 31½' Groundwater seepage.

Standard Penetration Test
Undisturbed, Ring Sample
Groundwater
Seepage

GeoSoils, Inc.

BORING LOG

PROJECT: GLEN IVY PROPERTIES, LLC
SWC of Temescal Canyon Road and Trilogy Parkway
Temescal Valley

W.O. 7731-A-SC BORING B-6 SHEET 2 OF 2

DATE EXCAVATED 2-7-20 LOGGED BY: RGC APPROX. ELEV.: 1094' MSL

SAMPLE METHOD: Track Mounted, Hollow Stem Auger

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
35		SPT	30	SP				**QUATERNARY MARSH DEPOSITS (Qm):** @ 35' SAND, dark gray, saturated, medium dense.
				ML				@ 36' SILT with SAND, very dark gray, wet, very stiff.
40		SPT	13					@ 40' SILT with SAND, dark gray and very dark gray (banded color - organics?), wet, stiff; thinly laminated, no free water.
45		SPT	22					@ 45' SILT with SAND as per 40', becomes moist, very stiff, no free water.
50		SPT	77	SM				@ 50' SILTY SAND with GRAVEL, very dark gray brown, moist, very dense; no free water.
				SP				@ 51' SAND, yellowish brown, moist, very dense; few subangular gravels.
								Total Depth = 51½' Groundwater Seepage at 31½', Then Saturated to About 36'. Perched Water Table No Caving Encountered Backfilled 2-7-20
55								
60								
65								

Standard Penetration Test
Undisturbed, Ring Sample
Groundwater
Seepage

SUMMAR□

OF

CONE PENETRATION TEST DATA

Project:

Temescal Canyon Road □ Trilogy Park□ay
Temescal □alley□CA
□anuary 27□2020

Prepared for:

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TABLE OF CONTENTS

SUMMAR□

OF

CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the project located at Temescal Canyon Road & Trilogy Parkway in Temescal Valley, California. The work was performed by Kehoe Testing & Engineering (KTE) on January 27, 2020. The scope of work was performed as directed by GeoSoils, Inc. (GSI) personnel.

2. SUMMAR□ OF FIELD WORK

The fieldwork consisted of performing CPT soundings at six locations to determine the soil lithology. A summary is provided in **TABLE 2.1**.

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	17	Refusal
CPT-2	17	Refusal
CPT-3	17	Refusal
CPT-4	16	Refusal
CPT-5	50	
CPT-6	35	Refusal

TABLE 2.1 - Summary of CPT Soundings

3. FIELD EQUIPMENT □ PROCEDURES

The CPT soundings were carried out by **KTE** using an integrated electronic cone system manufactured by Vertek. The CPT soundings were performed in accordance with ASTM standards (D5778). The cone penetrometers were pushed using a 30-ton CPT rig. The cone used during the program was a 15 cm^2 cone and recorded the following parameters at approximately 2.5 cm depth intervals:

- Cone Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (u)
- Inclination
- Penetration Speed

The above parameters were recorded and viewed in real time using a laptop computer. Data is stored at the KTE office for up to 2 years for future analysis and reference. A complete set of baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly.

4. CONE PENETRATION TEST DATA INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. These plots were generated using the CPeT-IT program. Penetration depths are referenced to ground surface. The soil behavior type on the CPT plots is derived from the attached CPT SBT plot (Robertson, “Interpretation of Cone Penetration Test…”, 2009) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (qc), sleeve friction (fs), and penetration pore pressure (u). The friction ratio (Rf), which is sleeve friction divided by cone resistance, is a calculated parameter that is used along with cone resistance to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures.

The CPT data files have also been provided. These files can be imported in CPeT-IT (software by GeoLogismiki) and other programs to calculate various geotechnical parameters.

It should be noted that it is not always possible to clearly identify a soil type based on qc, fs and u. In these situations, experience, judgement and an assessment of the pore pressure data should be used to infer the soil behavior type.

If you have any questions regarding this information, please do not hesitate to call our office at (714) 901-7270.

Sincerely,

KEHOE TESTING & ENGINEERING

Steven P. Kehoe
President

01/29/20-hh-1398

APPENDIX

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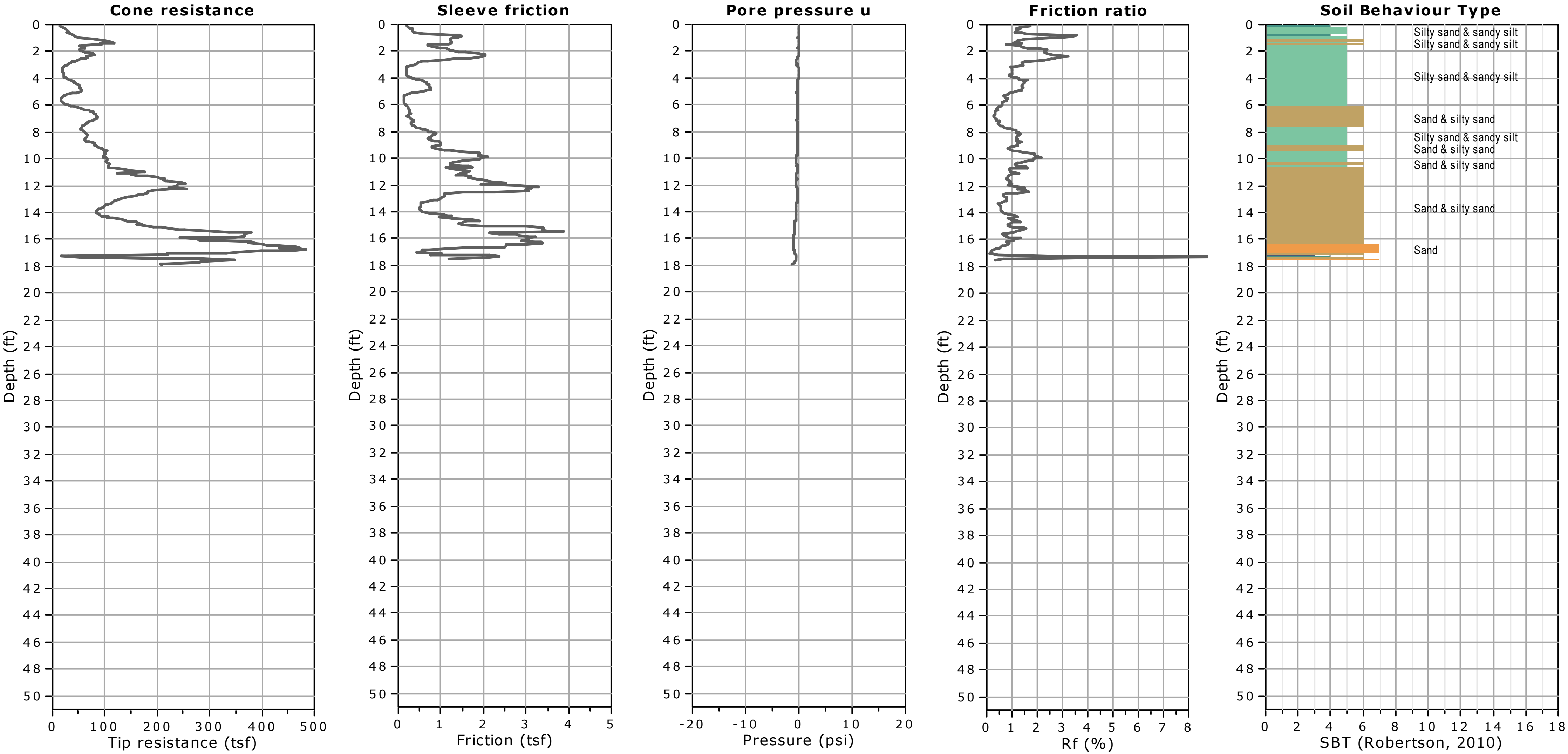
www.kehoetesting.com

Project: GeoSoils **CPT-1**

Location: Temescal Canyon Rd & Trilogy Pkwy, Temescal Valley, CA

Total depth: 17.93 ft, Date: 1/27/2020



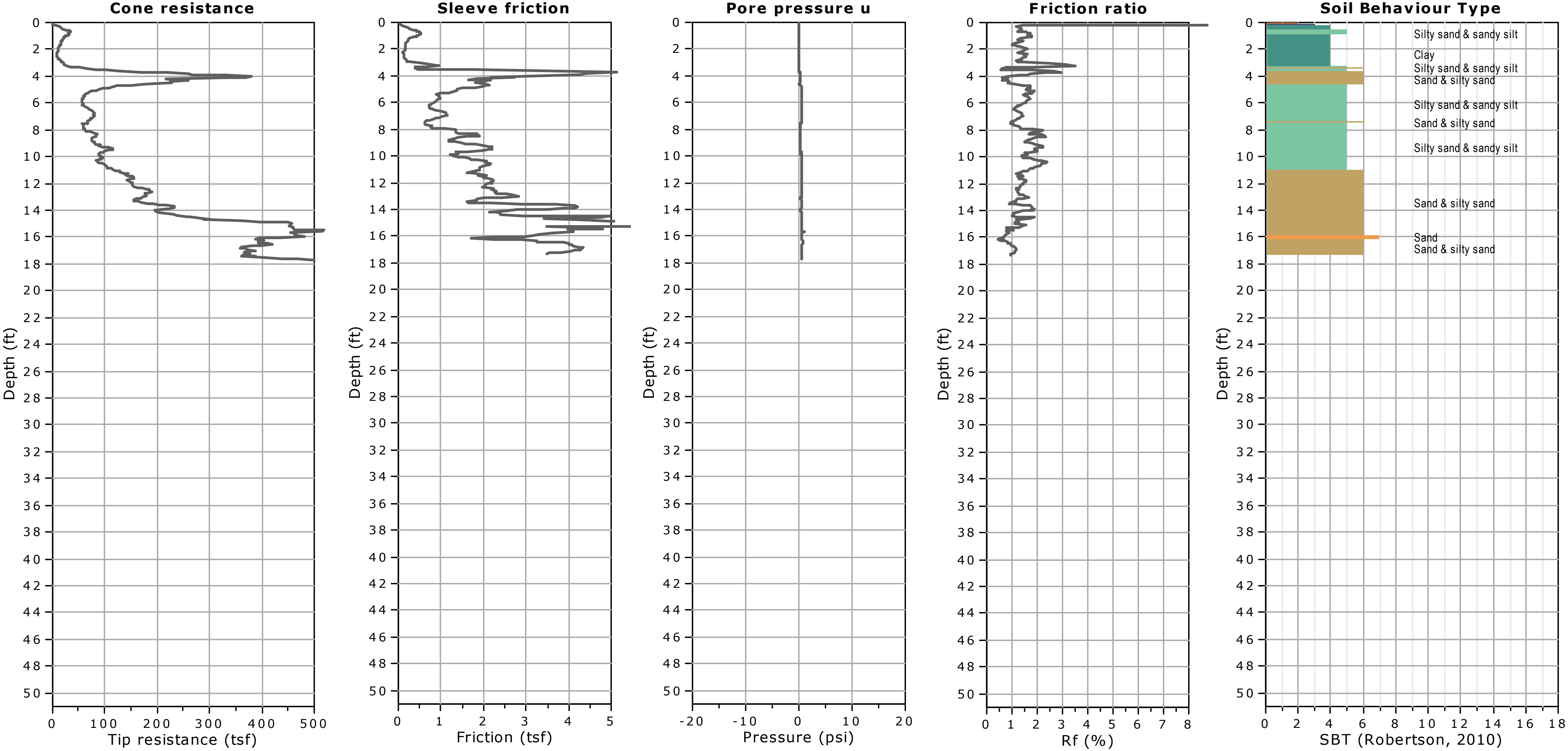
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: GeoSoils

Location: Temescal Canyon Rd & Trilogy Pkwy, Temescal Valley, CA

CPT-2

Total depth: 17.72 ft, Date: 1/27/2020

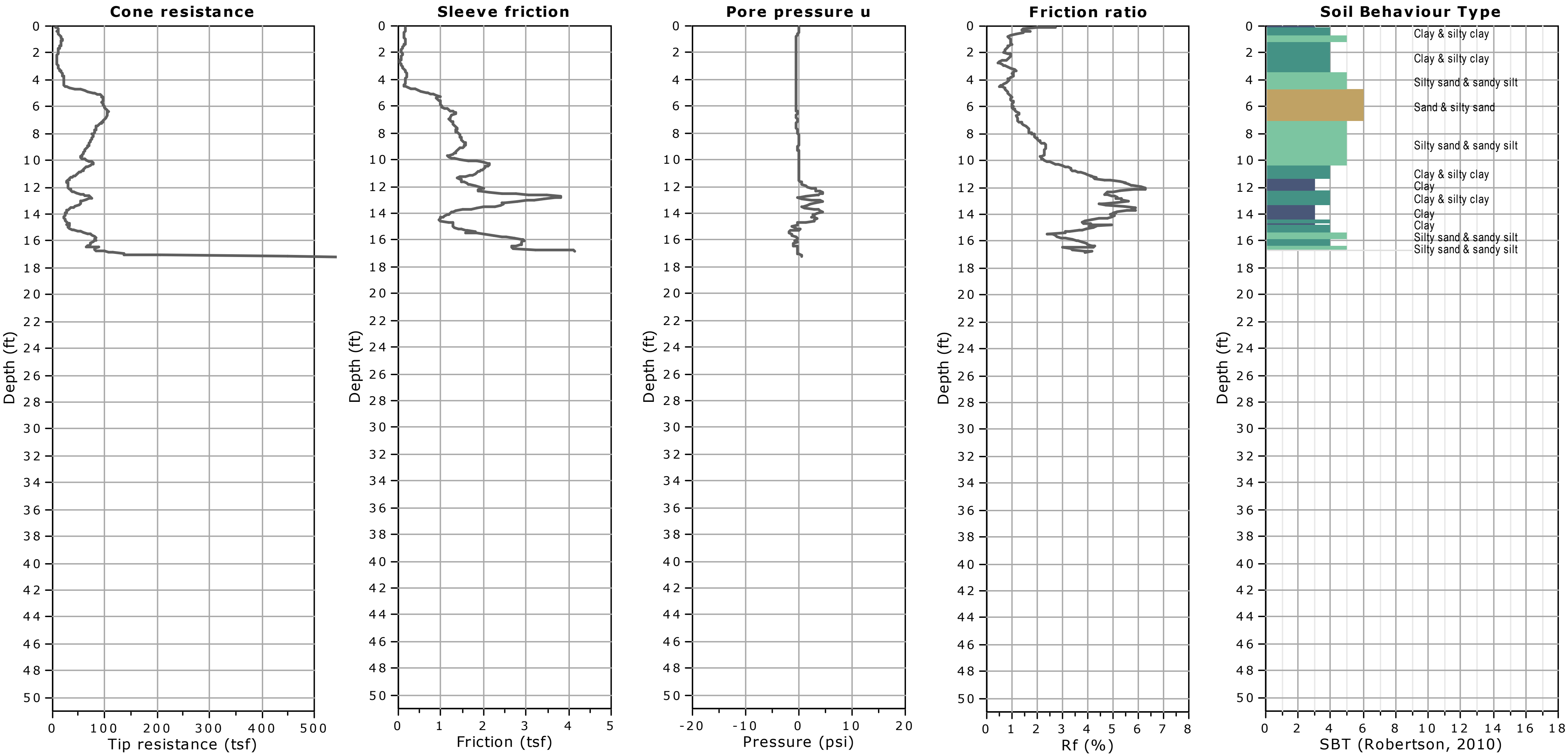


Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: GeoSoils **CPT-3**

Location: Temescal Canyon Rd & Trilogy Pkwy, Temescal Valley, CA

Total depth: 17.20 ft, Date: 1/27/2020



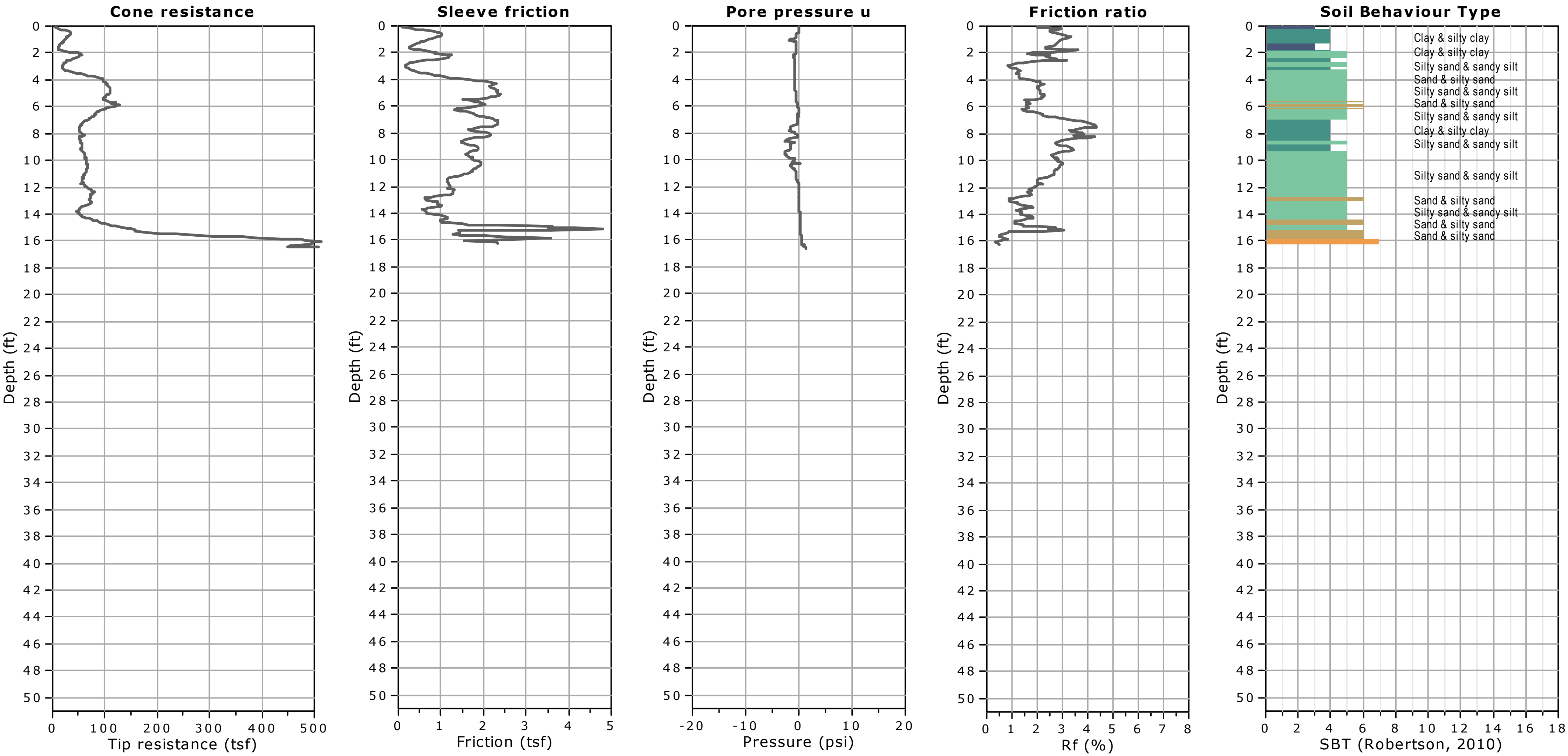
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: **GeoSoils**

Location: **Temescal Canyon Rd & Trilogy Pkwy, Temescal Valley, CA**

CPT-4

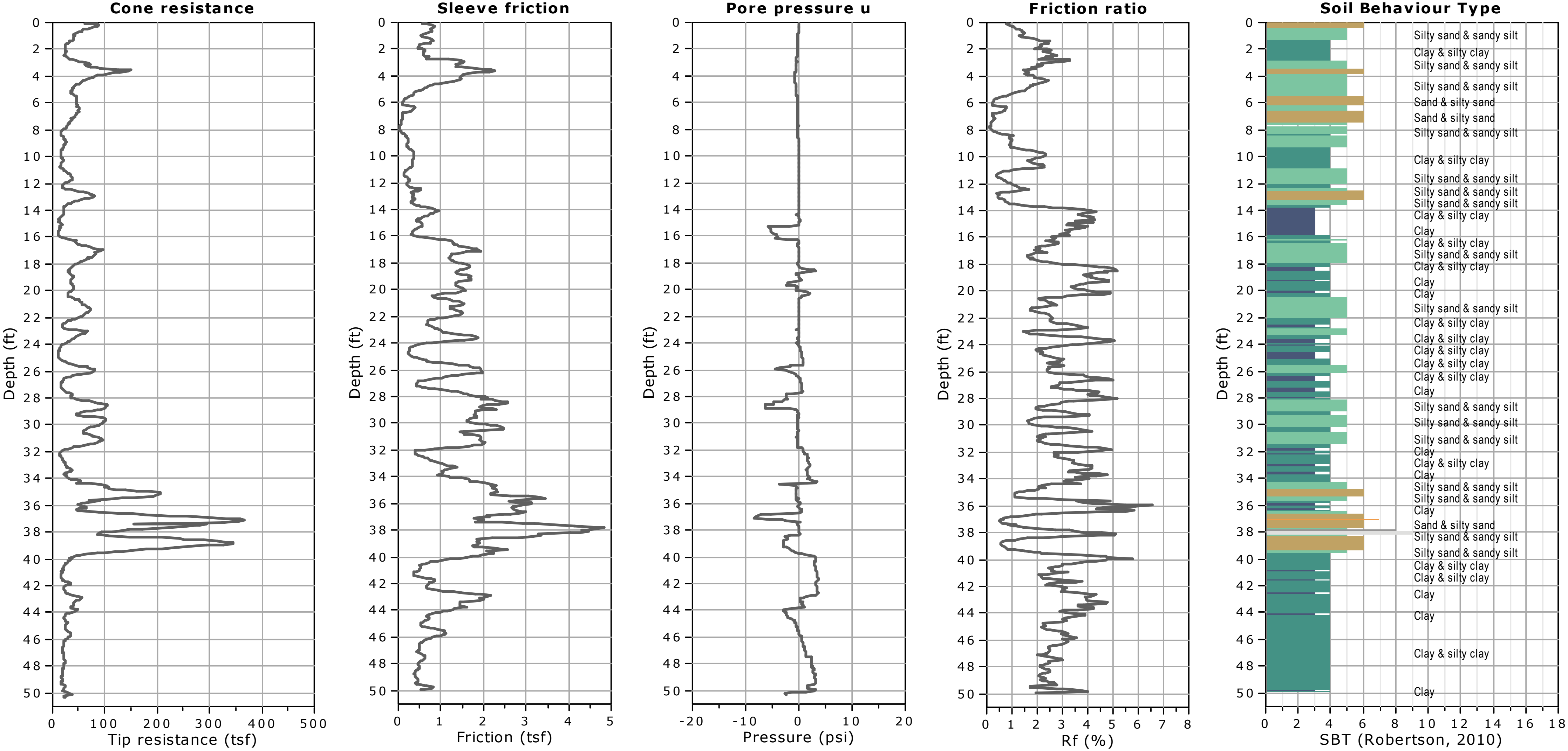
Total depth: 16.60 ft, Date: 1/27/2020



Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: GeoSoils
Location: Temescal Canyon Rd & Trilogy Pkwy, Temescal Valley, CA

CPT-5
Total depth: 50.34 ft, Date: 1/27/2020



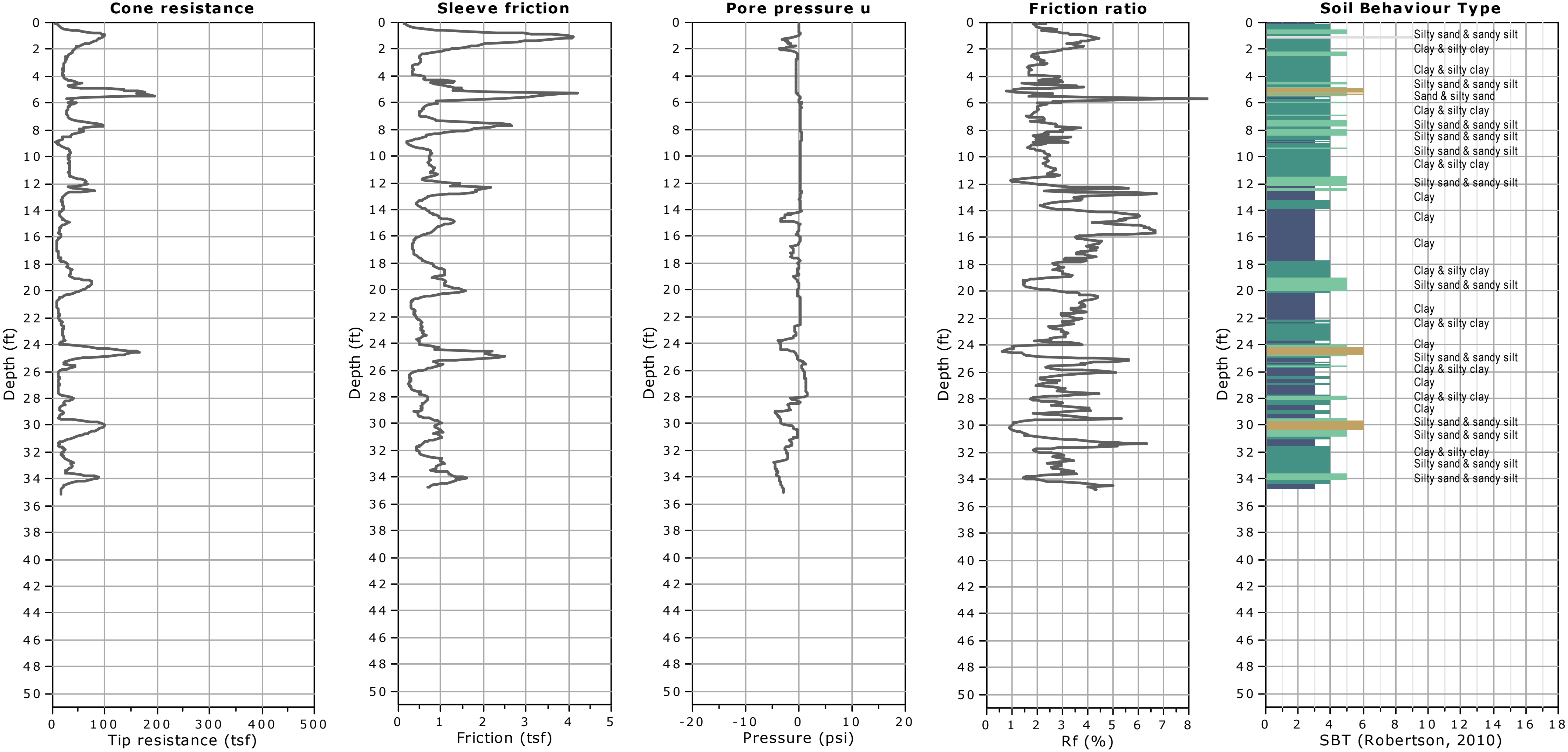
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: GeoSoils

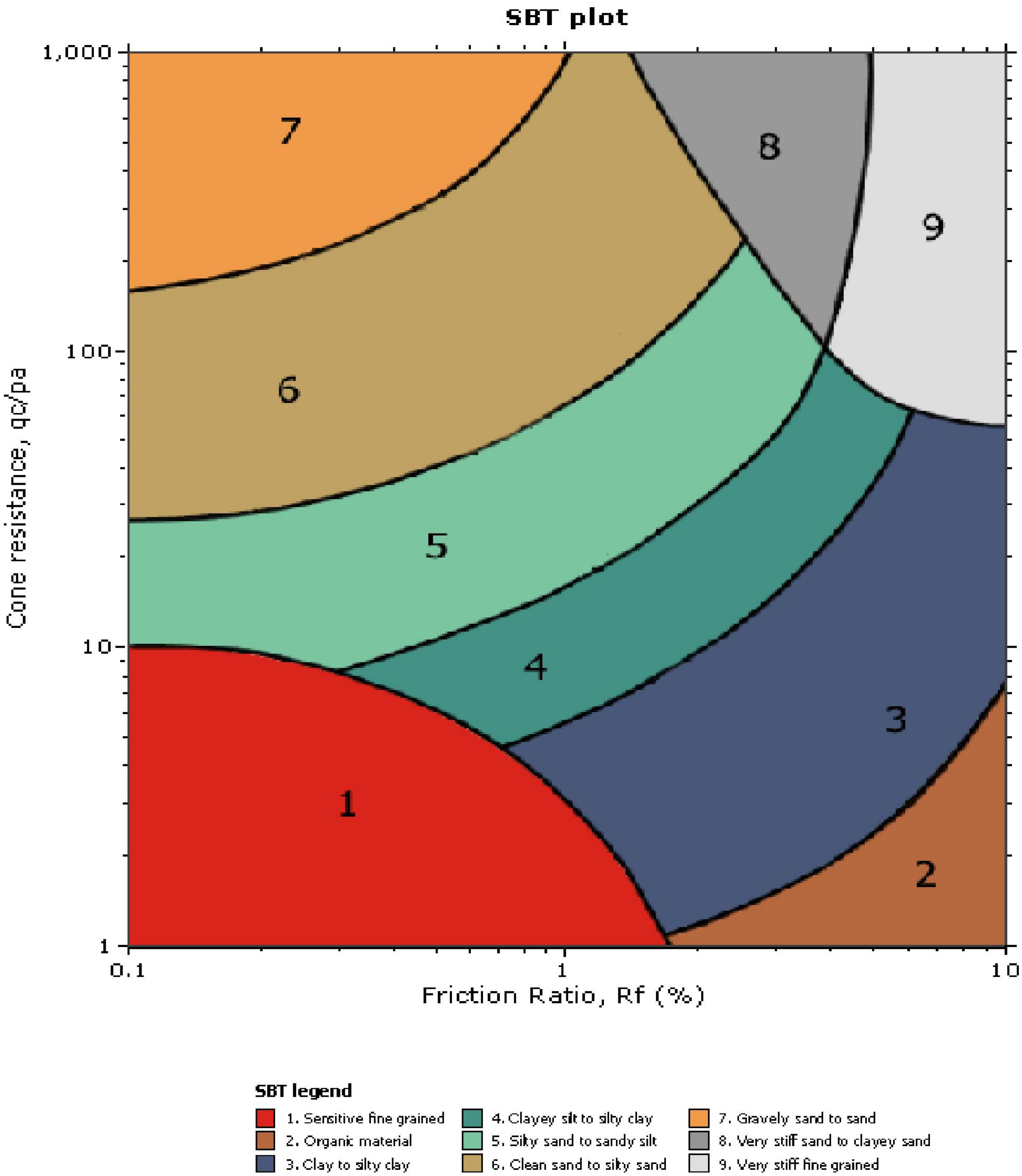
Location: Temescal Canyon Rd & Trilogy Pkwy, Temescal Valley, CA

CPT-6

Total depth: 35.18 ft, Date: 1/27/2020



Kehoe Testing & Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com



APPENDIX C

SEISMICITY DATA

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

JOB NUMBER: 7731-A-SC

DATE: 02-20-2020

JOB NAME: Glen Ivy Properties, LLC

CALCULATION NAME: Test Run Analysis

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

SITE COORDINATES:
   SITE LATITUDE: 33.7654
   SITE LONGITUDE: 117.4879

SEARCH RADIUS: 62.2 mi

ATTENUATION RELATION: 10) Bozorgnia Campbell Niazi (1999) Hor.-Holocene Soil-Cor.
   UNCERTAINTY (M=Median, S=Sigma): S   Number of Sigmas: 1.0
   DISTANCE MEASURE: cdist
   SCOND: 0
   Basement Depth: 5.00 km   Campbell SSR: 0   Campbell SHR: 0
   COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: C:\Program Files\EQFAULT1\CGSFLTErev.DAT

MINIMUM DEPTH VALUE (km): 3.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

W.O. 6973-A-SC
PLATE C-1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
ELSINORE (GLEN IVY)	0.0(0.0)	6.8	0.832	XI
CHINO-CENTRAL AVE. (Elsinore)	6.8(10.9)	6.7	0.657	X
WHITTIER	10.8(17.4)	6.8	0.339	IX
ELSINORE (TEMECULA)	11.8(19.0)	6.8	0.314	IX
SAN JOAQUIN HILLS	17.3(27.9)	6.6	0.274	IX
SAN JACINTO-SAN J.VLY-CASA LOMA	22.6(36.4)	6.9	0.180	VIII
SAN JACINTO-SAN BERNARDINO	22.7(36.5)	6.7	0.157	VIII
PUENTE HILLS BLIND THRUST	24.7(39.7)	7.1	0.267	IX
NEWPORT-INGLEWOOD (Offshore)	25.3(40.7)	7.1	0.183	VIII
SAN JOSE	26.7(42.9)	6.4	0.154	VIII
NEWPORT-INGLEWOOD (L.A.Basin)	27.5(44.2)	7.1	0.169	VIII
CUCAMONGA	29.7(47.8)	6.9	0.192	VIII
SIERRA MADRE	30.0(48.2)	7.2	0.233	IX
SAN ANDREAS - SB-Coach. M-1b-2	30.5(49.1)	7.7	0.229	IX
SAN ANDREAS - Whole M-1a	30.5(49.1)	8.0	0.281	IX
SAN ANDREAS - SB-Coach. M-2b	30.5(49.1)	7.7	0.229	IX
SAN ANDREAS - San Bernardino M-1	30.5(49.1)	7.5	0.200	VIII
SAN JACINTO-ANZA	32.9(52.9)	7.2	0.151	VIII
SAN ANDREAS - Cho-Moj M-1b-1	36.3(58.4)	7.8	0.208	VIII
SAN ANDREAS - 1857 Rupture M-2a	36.3(58.4)	7.8	0.208	VIII
SAN ANDREAS - Mojave M-1c-3	36.3(58.4)	7.4	0.157	VIII
CLEGHORN	36.5(58.7)	6.5	0.084	VII
NORTH FRONTAL FAULT ZONE (West)	37.2(59.9)	7.2	0.187	VIII
ELSINORE (JULIAN)	38.3(61.7)	7.1	0.120	VII
PALOS VERDES	39.1(63.0)	7.3	0.135	VIII
RAYMOND	40.3(64.8)	6.5	0.107	VII
CLAMSHELL-SAWPIT	40.6(65.4)	6.5	0.106	VII
UPPER ELYSIAN PARK BLIND THRUST	40.9(65.9)	6.4	0.099	VII
CORONADO BANK	42.6(68.6)	7.6	0.153	VIII
ROSE CANYON	44.6(71.8)	7.2	0.110	VII
VERDUGO	45.9(73.9)	6.9	0.122	VII
PINTO MOUNTAIN	48.5(78.0)	7.2	0.101	VII
HOLLYWOOD	49.6(79.8)	6.4	0.080	VII
NORTH FRONTAL FAULT ZONE (East)	50.6(81.5)	6.7	0.096	VII
HELENDALE - S. LOCKHARDT	54.9(88.3)	7.3	0.095	VII
SANTA MONICA	57.8(93.0)	6.6	0.078	VII
SAN GABRIEL	59.4(95.6)	7.2	0.082	VII
SIERRA MADRE (San Fernando)	59.5(95.7)	6.7	0.081	VII
SAN ANDREAS - Coachella M-1c-5	59.5(95.8)	7.2	0.081	VII
SAN JACINTO-COYOTE CREEK	60.2(96.9)	6.6	0.053	VI

DETERMINISTIC SITE PARAMETERS

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.

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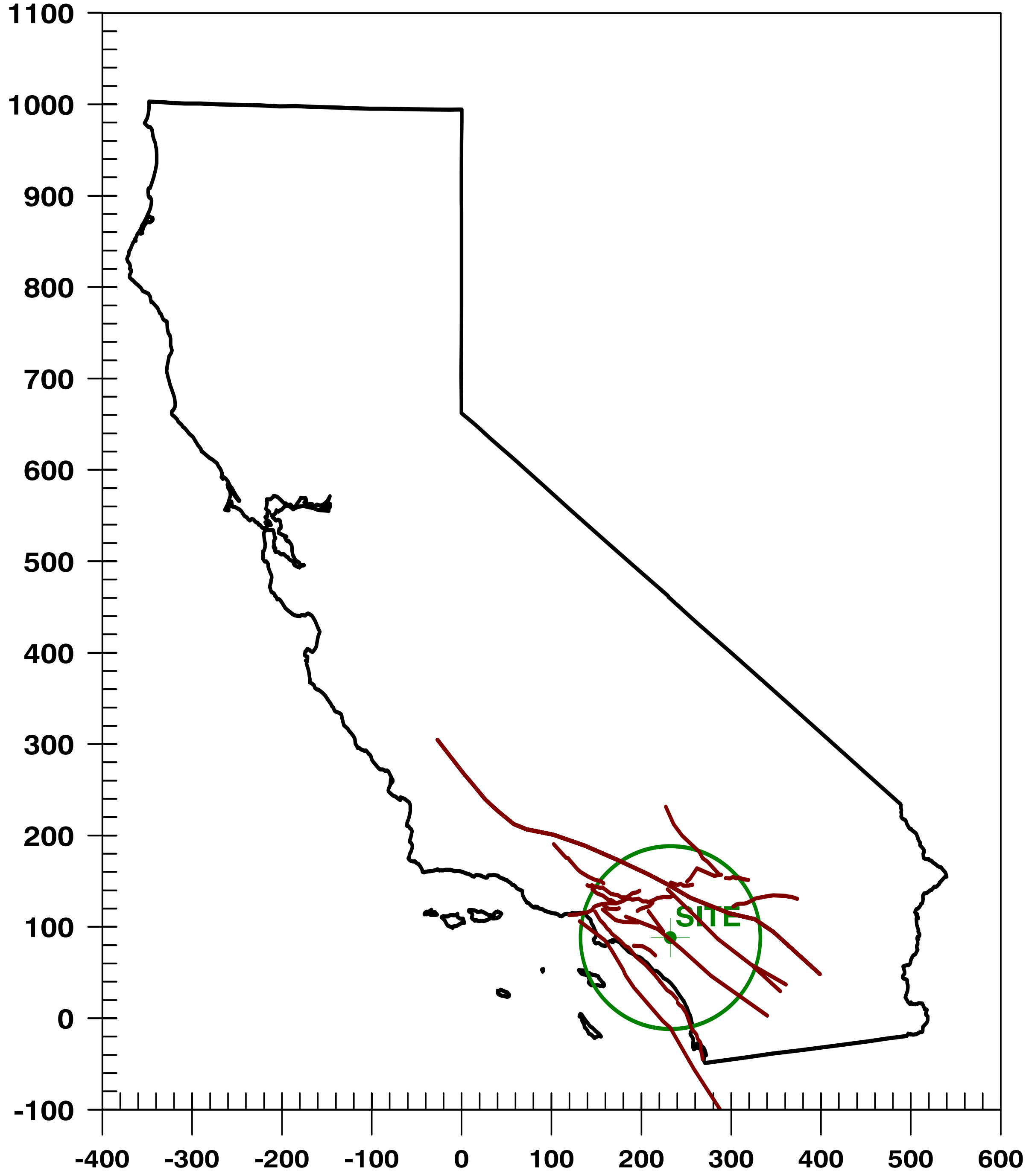
-END OF SEARCH- 40 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ELSINORE (GLEN IVY) FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 0.0 MILES (0.0 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.8318 g

CALIFORNIA FAULT MAP

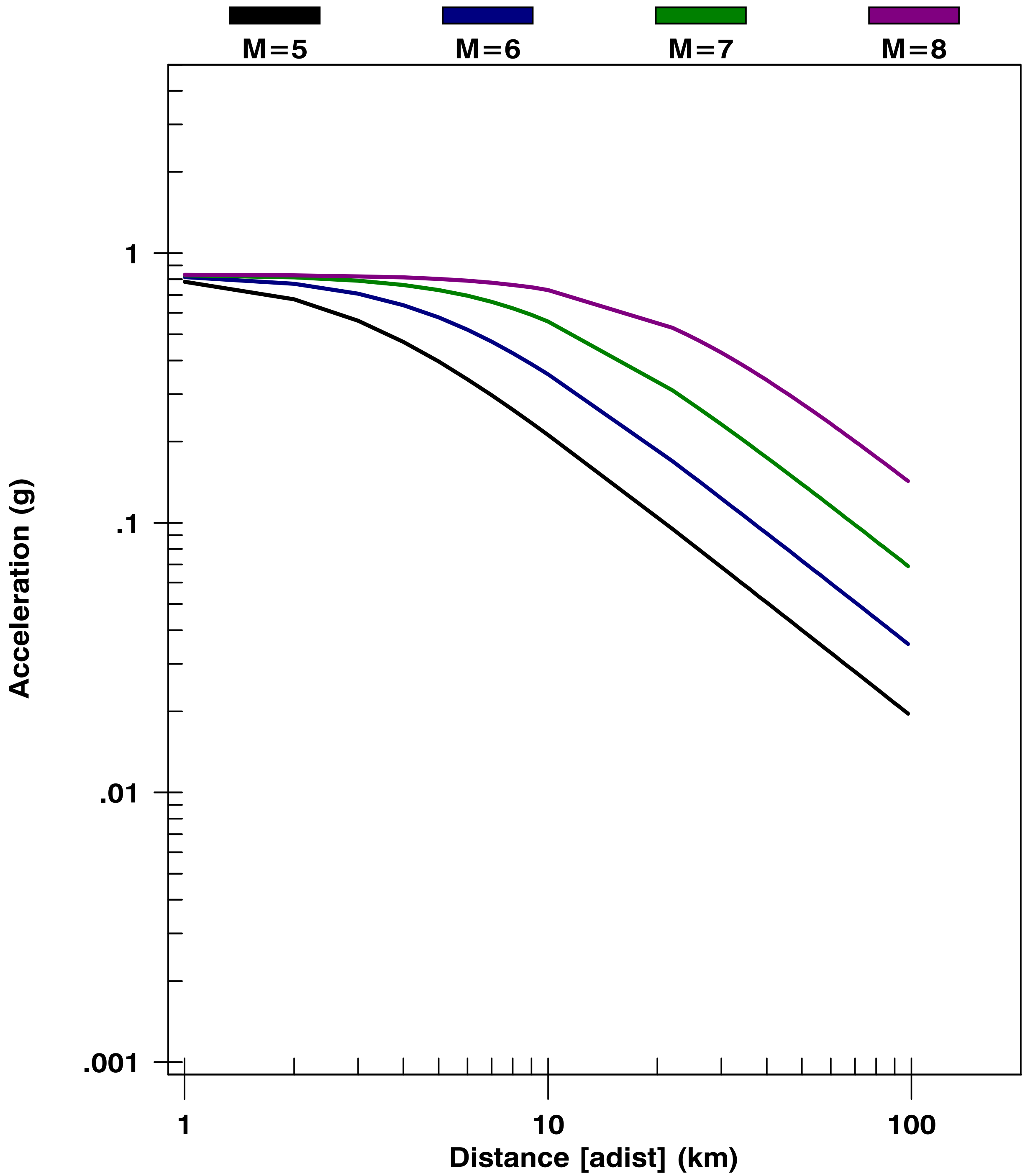
Glen Ivy Properties, LLC



W.O. 6973-A-SC
PLATE C-3

STRIKE-SLIP FAULTS

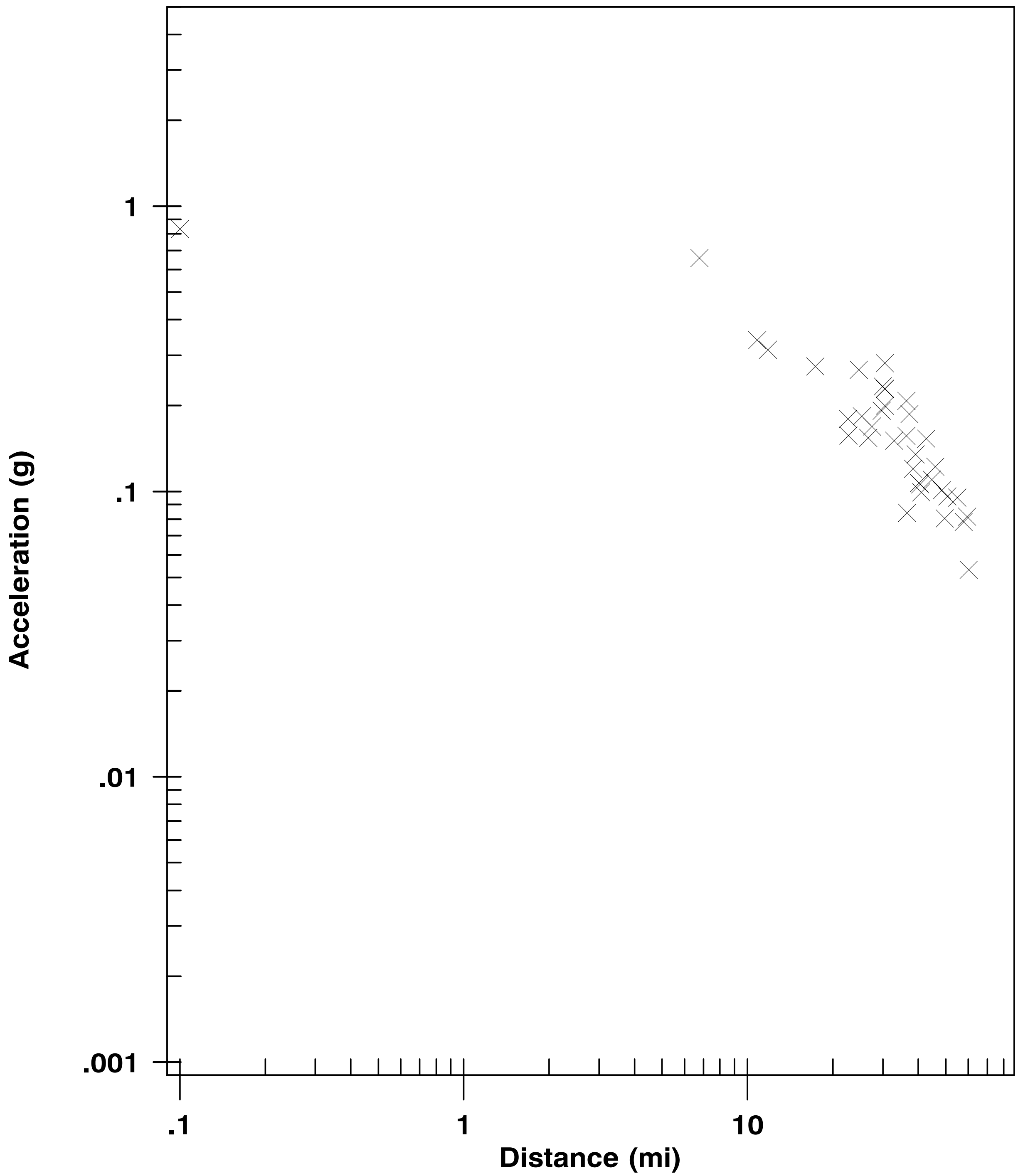
10) Bozorgnia Campbell Niazi (1999) Hor.-Holocene Soil-Cor.



W.O. 6973-A-SC
PLATE C-4

MAXIMUM EARTHQUAKES

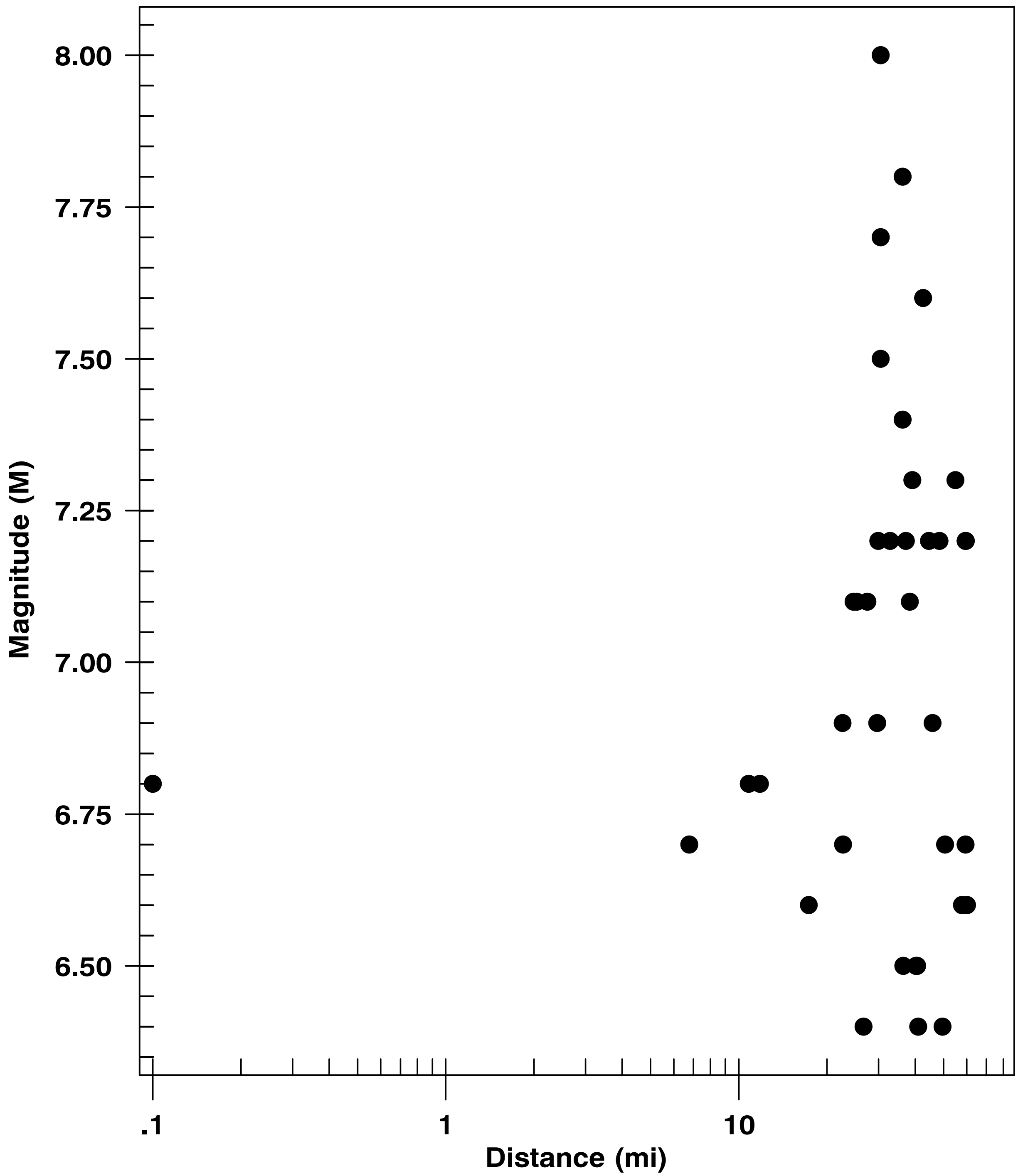
Glen Ivy Properties, LLC



W.O. 6973-A-SC
PLATE C-5

EARTHQUAKE MAGNITUDES & DISTANCES

Glen Ivy Properties, LLC



W.O. 6973-A-SC
PLATE C-6

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

JOB NUMBER: 7731-A-SC

DATE: 02-20-2020

JOB NAME: Glen Ivy Properties, LLC

EARTHQUAKE-CATALOG-FILE NAME: C:\Program Files\EQSEARCH\ALLQUAKE.DAT

MAGNITUDE RANGE:
   MINIMUM MAGNITUDE: 5.00
   MAXIMUM MAGNITUDE: 9.00

SITE COORDINATES:
   SITE LATITUDE: 33.7654
   SITE LONGITUDE: 117.4879

SEARCH DATES:
   START DATE: 1800
   END DATE: 2019

SEARCH RADIUS:
   62.2 mi
   100.1 km

ATTENUATION RELATION: 10) Bozorgnia Campbell Niazi (1999) Hor.-Holocene 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: 0 Depth Source: A
   Basement Depth: 5.00 km   Campbell SSR: 0   Campbell SHR: 0
   COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km): 3.0

EARTHQUAKE SEARCH RESULTS

W.O. 6973-A-SC
PLATE C-7

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	33.6990	117.5110	05/31/1938	83455.4	10.0	5.50	0.285	IX	4.8(7.7)
DMG	33.7000	117.4000	05/13/1910	620 0.0	0.0	5.00	0.162	VIII	6.8(10.9)
DMG	33.7000	117.4000	05/15/1910	1547 0.0	0.0	6.00	0.317	IX	6.8(10.9)
DMG	33.7000	117.4000	04/11/1910	757 0.0	0.0	5.00	0.162	VIII	6.8(10.9)
MGI	33.8000	117.6000	04/22/1918	2115 0.0	0.0	5.00	0.160	VIII	6.9(11.0)
MGI	34.0000	117.5000	12/16/1858	10 0 0.0	0.0	7.00	0.263	IX	16.2(26.1)
DMG	33.9000	117.2000	12/19/1880	0 0 0.0	0.0	6.00	0.120	VII	18.9(30.5)
GSG	33.9530	117.7610	07/29/2008	184215.7	14.0	5.30	0.073	VII	20.3(32.7)
DMG	34.0000	117.2500	07/23/1923	73026.0	0.0	6.25	0.126	VIII	21.2(34.1)
MGI	34.1000	117.3000	07/15/1905	2041 0.0	0.0	5.30	0.058	VI	25.5(41.0)
GSP	33.9325	117.9158	03/29/2014	040942.2	5.1	5.10	0.048	VI	27.1(43.6)
DMG	33.7500	117.0000	04/21/1918	223225.0	0.0	6.80	0.135	VIII	28.0(45.1)
DMG	33.7500	117.0000	06/06/1918	2232 0.0	0.0	5.00	0.044	VI	28.0(45.1)
DMG	33.8000	117.0000	12/25/1899	1225 0.0	0.0	6.40	0.103	VII	28.1(45.2)
GSP	34.1400	117.7000	02/28/1990	234336.6	5.0	5.20	0.048	VI	28.6(46.0)
DMG	33.6170	117.9670	03/11/1933	154 7.8	0.0	6.30	0.093	VII	29.4(47.3)
DMG	34.2000	117.4000	07/22/1899	046 0.0	0.0	5.50	0.054	VI	30.4(49.0)
DMG	33.5750	117.9830	03/11/1933	518 4.0	0.0	5.20	0.044	VI	31.3(50.4)
DMG	33.6170	118.0170	03/14/1933	19 150.0	0.0	5.10	0.040	V	32.1(51.6)
DMG	33.7100	116.9250	09/23/1963	144152.6	16.5	5.00	0.038	V	32.5(52.4)
DMG	33.6830	118.0500	03/11/1933	658 3.0	0.0	5.50	0.050	VI	32.8(52.7)
MGI	34.0000	118.0000	12/25/1903	1745 0.0	0.0	5.00	0.037	V	33.5(53.9)
DMG	33.7000	118.0670	03/11/1933	51022.0	0.0	5.10	0.039	V	33.6(54.0)
DMG	33.7000	118.0670	03/11/1933	85457.0	0.0	5.10	0.039	V	33.6(54.0)
DMG	33.7500	118.0830	03/13/1933	131828.0	0.0	5.30	0.043	VI	34.2(55.0)
DMG	33.7500	118.0830	03/11/1933	323 0.0	0.0	5.00	0.036	V	34.2(55.0)
DMG	33.7500	118.0830	03/11/1933	230 0.0	0.0	5.10	0.038	V	34.2(55.0)
DMG	33.7500	118.0830	03/11/1933	910 0.0	0.0	5.10	0.038	V	34.2(55.0)
DMG	33.7500	118.0830	03/11/1933	2 9 0.0	0.0	5.00	0.036	V	34.2(55.0)
DMG	34.2700	117.5400	09/12/1970	143053.0	8.0	5.40	0.044	VI	35.0(56.3)
DMG	34.3000	117.5000	07/22/1899	2032 0.0	0.0	6.50	0.083	VII	36.9(59.4)
DMG	33.7830	118.1330	10/02/1933	91017.6	0.0	5.40	0.042	V	37.0(59.6)
DMG	34.2000	117.1000	09/20/1907	154 0.0	0.0	6.00	0.059	VI	37.3(60.1)
DMG	34.3000	117.6000	07/30/1894	512 0.0	0.0	6.00	0.059	VI	37.5(60.3)
DMG	34.2000	117.9000	08/28/1889	215 0.0	0.0	5.50	0.043	VI	38.2(61.4)
DMG	33.9500	116.8500	09/28/1946	719 9.0	0.0	5.00	0.031	V	38.7(62.3)
PAS	34.0610	118.0790	10/01/1987	144220.0	9.5	5.90	0.053	VI	39.5(63.6)
PAS	34.0730	118.0980	10/04/1987	105938.2	8.2	5.30	0.035	V	40.9(65.8)
MGI	34.1000	118.1000	07/11/1855	415 0.0	0.0	6.30	0.064	VI	42.0(67.6)
DMG	34.3700	117.6500	12/08/1812	15 0 0.0	0.0	7.00	0.100	VII	42.8(68.8)
DMG	34.1800	116.9200	01/16/1930	02433.9	0.0	5.20	0.031	V	43.3(69.7)
DMG	34.1800	116.9200	01/16/1930	034 3.6	0.0	5.10	0.030	V	43.3(69.7)
DMG	33.7830	118.2500	11/14/1941	84136.3	0.0	5.40	0.035	V	43.8(70.4)
DMG	33.8500	118.2670	03/11/1933	1425 0.0	0.0	5.00	0.027	V	45.1(72.5)
GSP	34.2620	118.0020	06/28/1991	144354.5	11.0	5.40	0.034	V	45.2(72.7)
GSP	34.1630	116.8550	06/28/1992	144321.0	6.0	5.30	0.032	V	45.5(73.2)
DMG	34.1000	116.8000	10/24/1935	1448 7.6	0.0	5.10	0.028	V	45.7(73.5)
DMG	34.2670	116.9670	08/29/1943	34513.0	0.0	5.50	0.035	V	45.7(73.5)
DMG	33.9760	116.7210	06/12/1944	104534.7	10.0	5.10	0.028	V	46.3(74.5)
GSP	34.1950	116.8620	08/17/1992	204152.1	11.0	5.30	0.031	V	46.5(74.9)
T-A	34.0000	118.2500	03/26/1860	0 0 0.0	0.0	5.00	0.026	V	46.6(75.0)
T-A	34.0000	118.2500	01/10/1856	0 0 0.0	0.0	5.00	0.026	V	46.6(75.0)
T-A	34.0000	118.2500	09/23/1827	0 0 0.0	0.0	5.00	0.026	V	46.6(75.0)

EARTHQUAKE SEARCH RESULTS

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	33.9940	116.7120	06/12/1944	111636.0	10.0	5.30	0.030	V	47.2(75.9)
GSP	34.2900	116.9460	02/10/2001	210505.8	9.0	5.10	0.027	V	47.7(76.7)

W.O. 6973-A-SC
PLATE C-8

GSN	34.2030	116.8270	06/28/1992	150530.7	5.0	6.70	0.071	VI	48.4(77.9)
MGI	34.0800	118.2600	07/16/1920	18 8 0.0	0.0	5.00	0.024	V	49.3(79.3)
MGI	34.0000	118.3000	09/03/1905	540 0.0	0.0	5.30	0.029	V	49.3(79.3)
GSP	34.2390	116.8370	07/09/1992	014357.6	0.0	5.30	0.029	V	49.6(79.8)
DMG	34.1000	116.7000	02/07/1889	520 0.0	0.0	5.30	0.028	V	50.7(81.6)
GSP	34.3400	116.9000	11/27/1992	160057.5	1.0	5.30	0.027	V	52.0(83.7)
GSG	34.3100	116.8480	02/22/2003	121910.6	1.0	5.20	0.026	V	52.5(84.5)
PAS	33.9980	116.6060	07/08/1986	92044.5	11.7	5.60	0.032	V	53.0(85.3)
GSP	34.3690	116.8970	12/04/1992	020857.5	3.0	5.30	0.027	V	53.7(86.3)
DMG	33.0000	117.3000	11/22/1800	2130 0.0	0.0	6.50	0.056	VI	53.9(86.8)
GSP	33.5290	116.5720	06/12/2005	154146.5	14.0	5.20	0.024	V	55.1(88.7)
GSP	33.5080	116.5140	10/31/2001	075616.6	15.0	5.10	0.021	IV	58.7(94.5)
PAS	33.5010	116.5130	02/25/1980	104738.5	13.6	5.50	0.027	V	58.9(94.8)
PAS	32.9710	117.8700	07/13/1986	1347 8.2	6.0	5.30	0.024	V	59.1(95.1)
DMG	34.0170	116.5000	07/25/1947	61949.0	0.0	5.20	0.023	IV	59.2(95.3)
DMG	34.0170	116.5000	07/26/1947	24941.0	0.0	5.10	0.021	IV	59.2(95.3)
DMG	34.0170	116.5000	07/24/1947	221046.0	0.0	5.50	0.027	V	59.2(95.3)
DMG	34.0170	116.5000	07/25/1947	04631.0	0.0	5.00	0.020	IV	59.2(95.3)
DMG	33.5000	116.5000	09/30/1916	211 0.0	0.0	5.00	0.020	IV	59.7(96.0)
DMG	33.2000	116.7000	01/01/1920	235 0.0	0.0	5.00	0.020	IV	59.9(96.3)
MGI	33.0000	117.0000	09/21/1856	730 0.0	0.0	5.00	0.020	IV	59.9(96.3)
MGI	34.0000	118.5000	11/19/1918	2018 0.0	0.0	5.00	0.020	IV	60.2(96.9)
DMG	34.0000	118.5000	08/04/1927	1224 0.0	0.0	5.00	0.020	IV	60.2(96.9)
GSG	33.4200	116.4890	07/07/2010	235333.5	14.0	5.50	0.026	V	62.2(100.1)

****

-END OF SEARCH- 79 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 2019

LENGTH OF SEARCH TIME: 220 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 4.8 MILES (7.7 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.317 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:

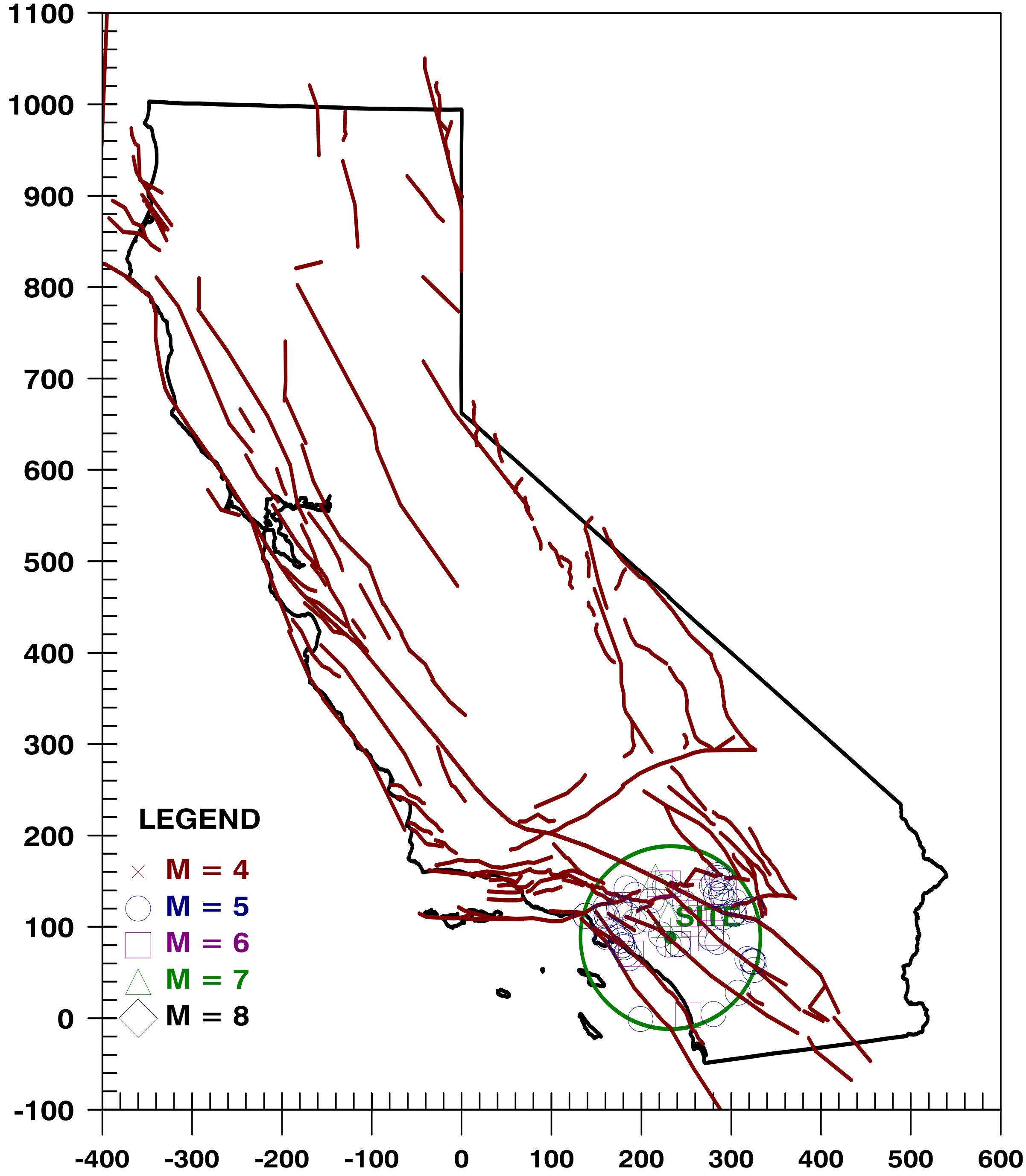
a-value= 1.322
b-value= 0.404
beta-value= 0.930

TABLE OF MAGNITUDES AND EXCEEDANCES:

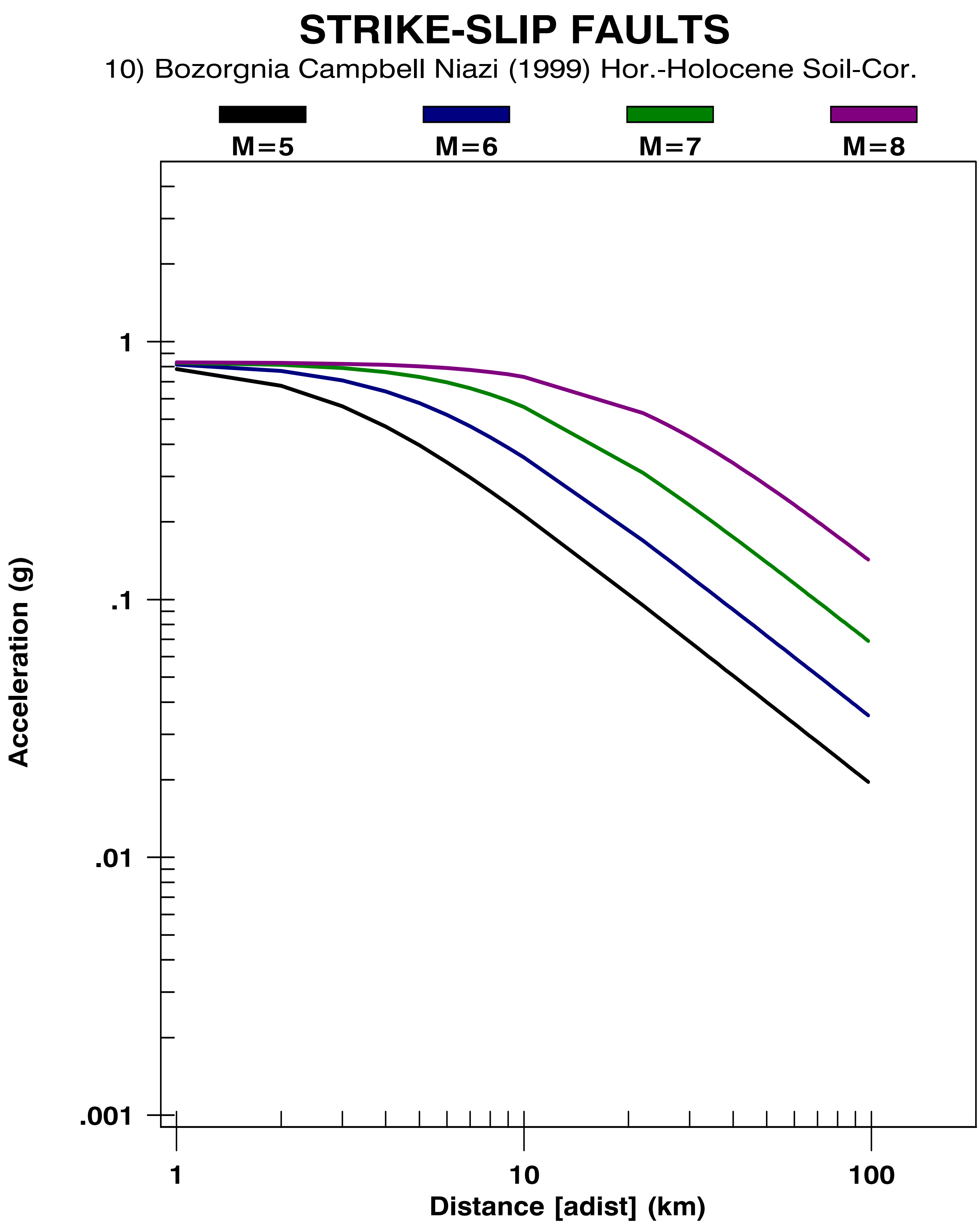
Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	79	0.35909
4.5	79	0.35909
5.0	79	0.35909
5.5	24	0.10909
6.0	14	0.06364
6.5	6	0.02727
7.0	2	0.00909

EARTHQUAKE EPICENTER MAP

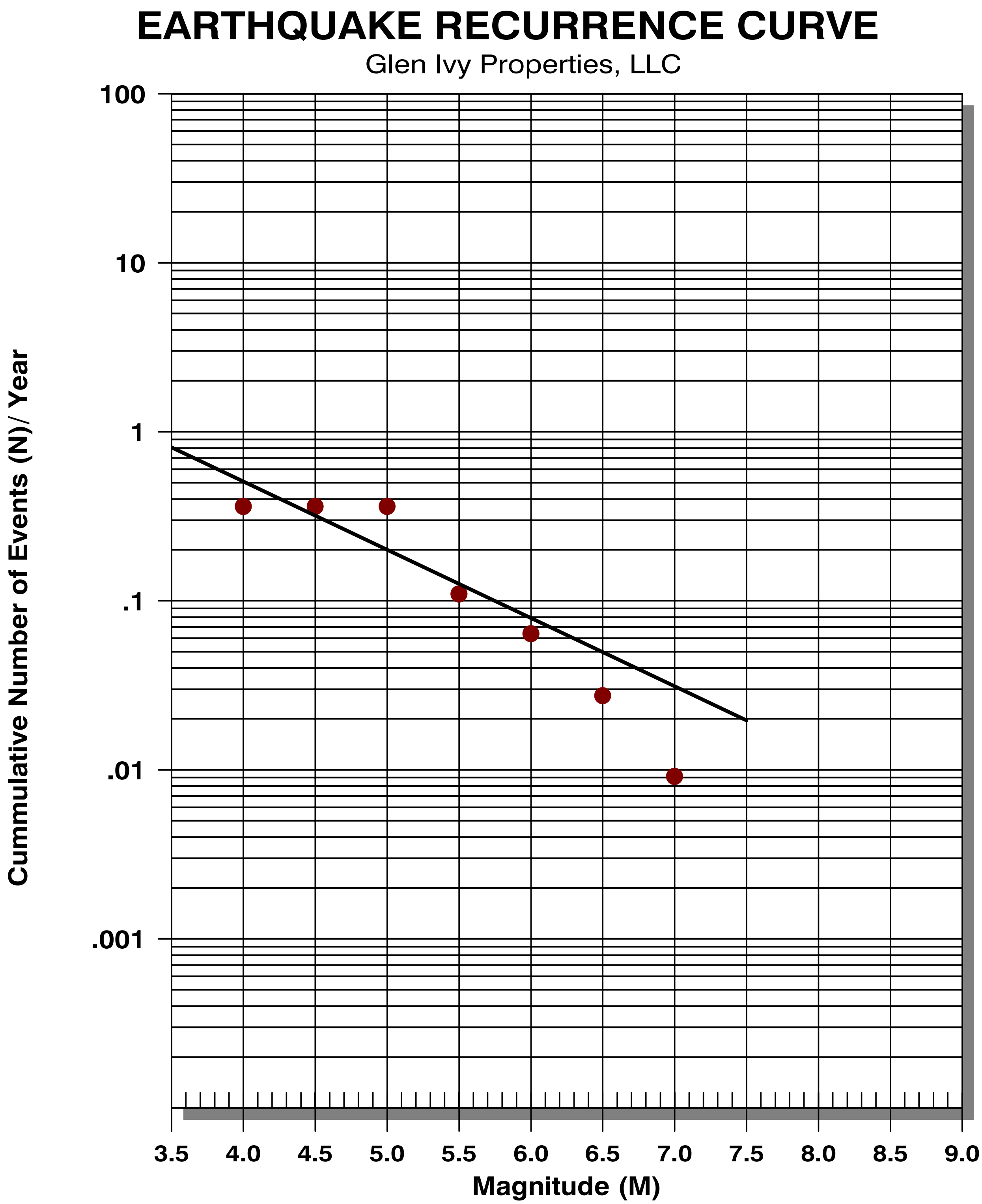
Glen Ivy Properties, LLC



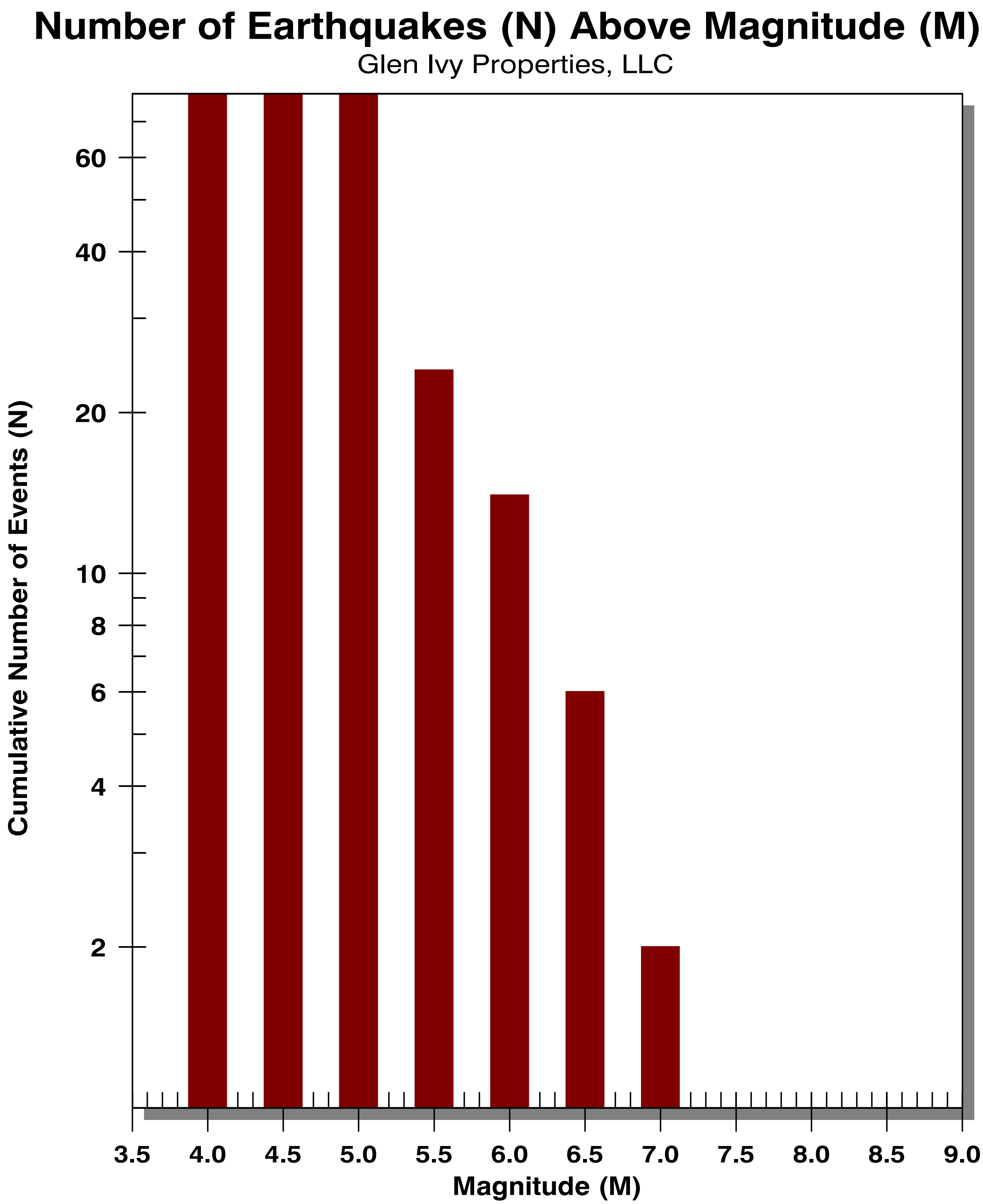
W.O. 6973-A-SC
PLATE C-10



W.O. 6973-A-SC
PLATE C-11



W.O. 6973-A-SC
PLATE C-12

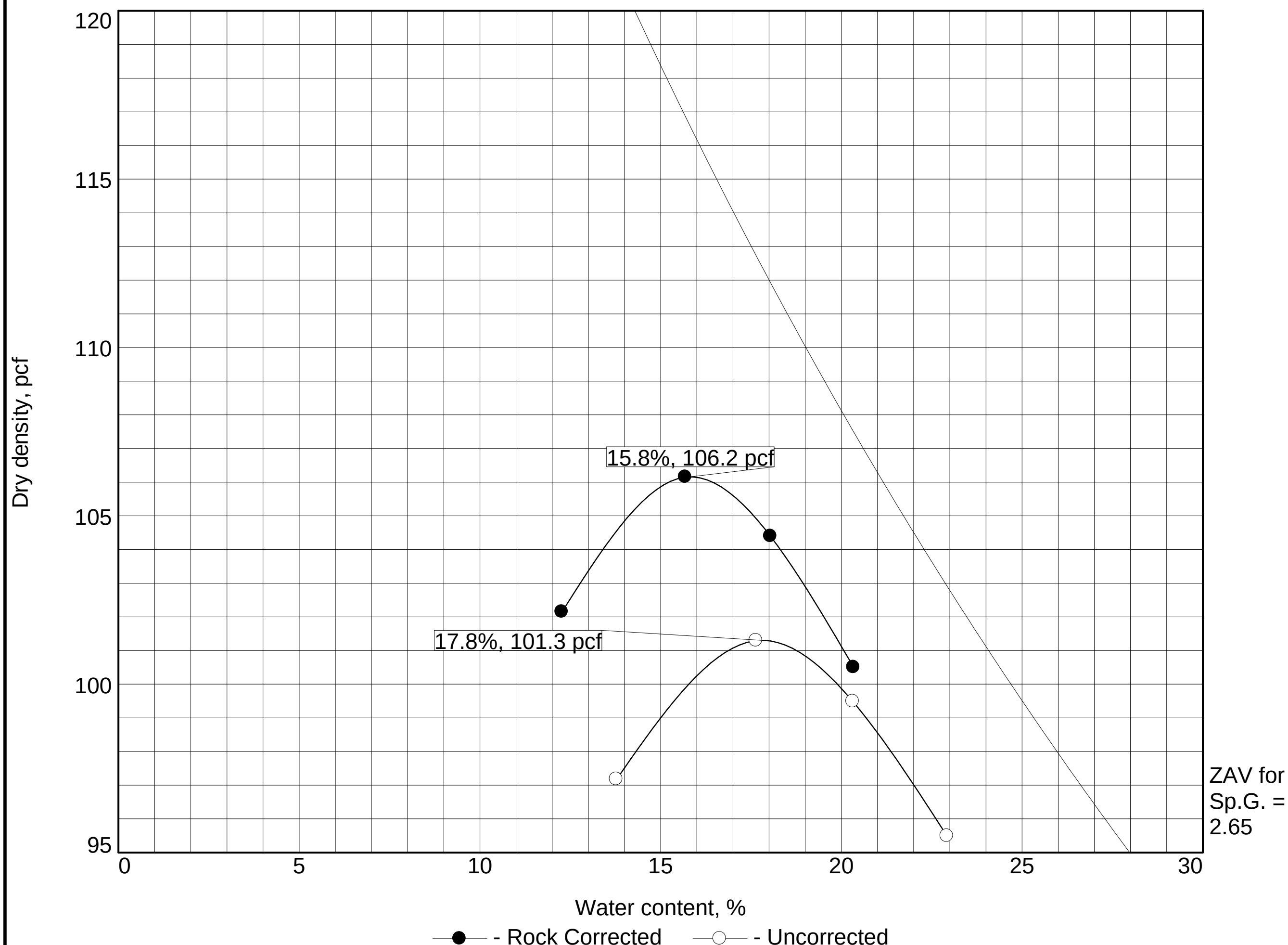


W.O. 6973-A-SC
PLATE C-13

APPENDIX D

LABORATORY TEST RESULTS

COMPACTION TEST REPORT For Curve No. B-1, 0-5ft



Test specification: ASTM D 1557-12 Method B Modified
ASTM D4718-15 Oversize Corr. Applied to Each Test Point

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/8 in.	% < No.200
	USCS	AASHTO						
0-5	OH			2.65			11.8	

ROCK CORRECTED TEST RESULTS	UNCORRECTED	MATERIAL DESCRIPTION
Maximum dry density = 106.2 pcf	101.3 pcf	Black Sandy Organic Soil
Optimum moisture = 15.8 %	17.8 %	

Project No. 7731-A-SC **Client:** Glen Ivy Properties
Project: Retail & Senior Living
Date: 1-30-20
Source of Sample: B-1 **Sample Number:** B-1

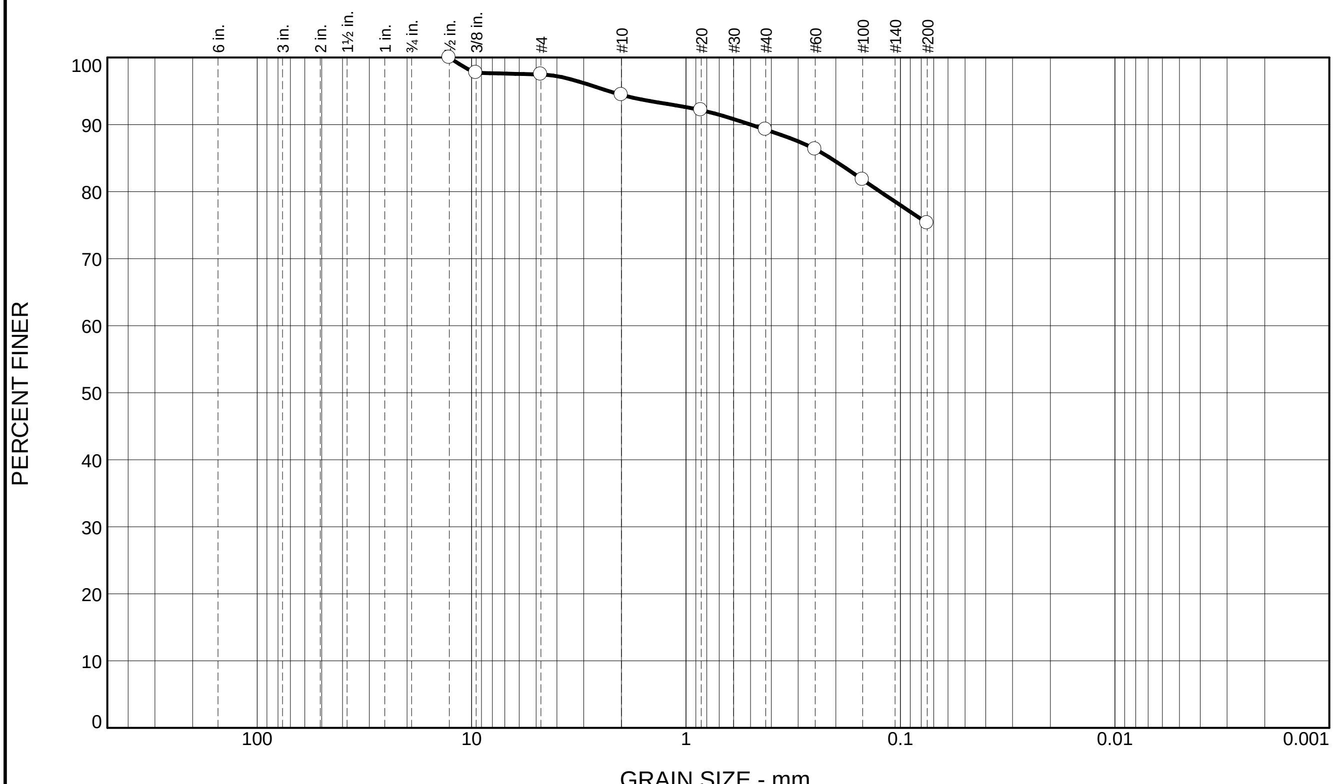
GeoSoils, Inc.

Remarks:

Plate

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

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	2.5	3.1	5.2	13.9	75.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.5	100.0		
.375	97.7		
#4	97.5		
#10	94.4		
#20	92.2		
#40	89.2		
#60	86.3		
#100	81.8		
#200	75.3		

* (no specification provided)

Soil Description

Black Sandy Organic Soil

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.5003 D_{85}= 0.2118 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= OL AASHTO=

Remarks

Source of Sample: B-6 **Depth:** 10.0
Sample Number: B-6

Date: 3-6-20

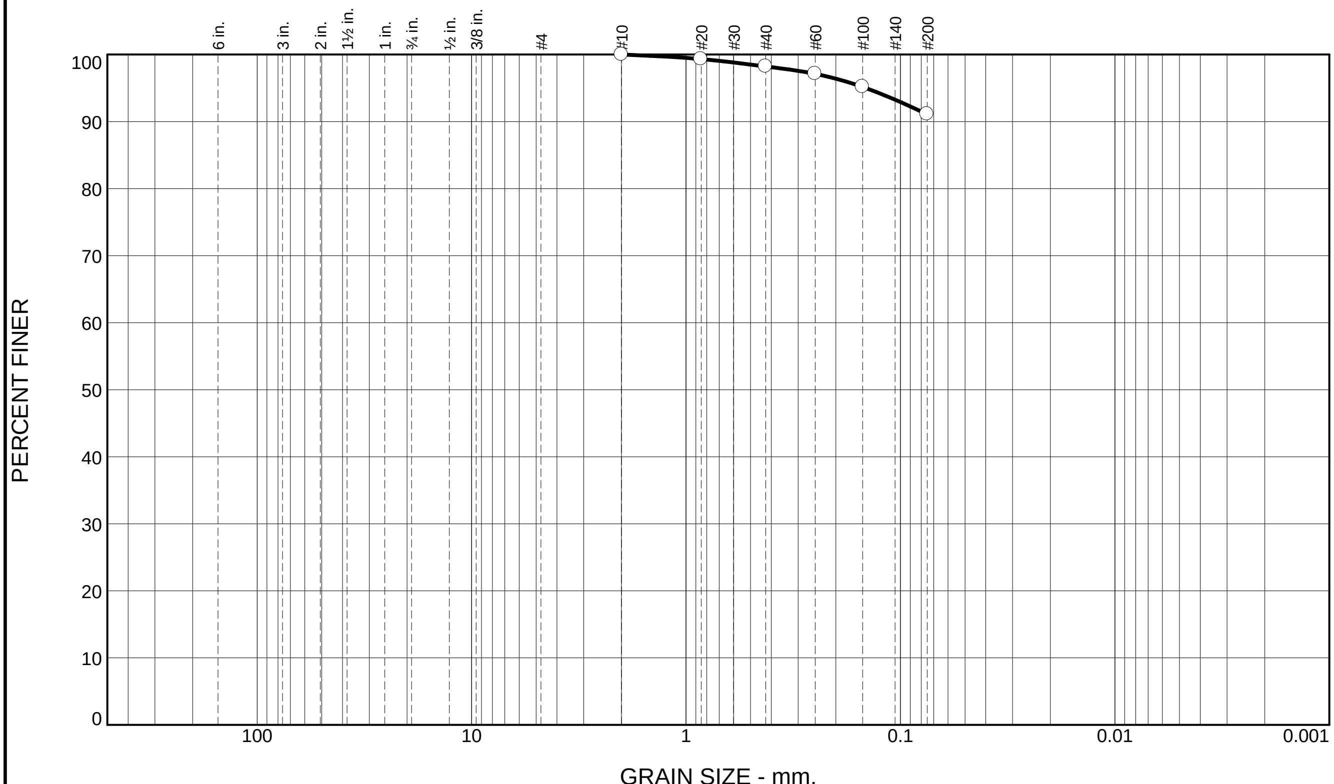
GeoSoils, Inc.

Client: Glen Ivy Properties
Project: Retail & Senior Living

Project No: 7731-A-SC **Figure**

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

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	1.8	7.1	91.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	99.3		
#40	98.2		
#60	97.1		
#100	95.2		
#200	91.1		

* (no specification provided)

Soil Description

Greenish Gray Sandy Silt

Atterberg Limits

PL= LL= PI=

Coefficients

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

Classification

USCS= ML AASHTO=

Remarks

Source of Sample: B-6 **Depth:** 40.0
Sample Number: B-6

Date: 3-6-20

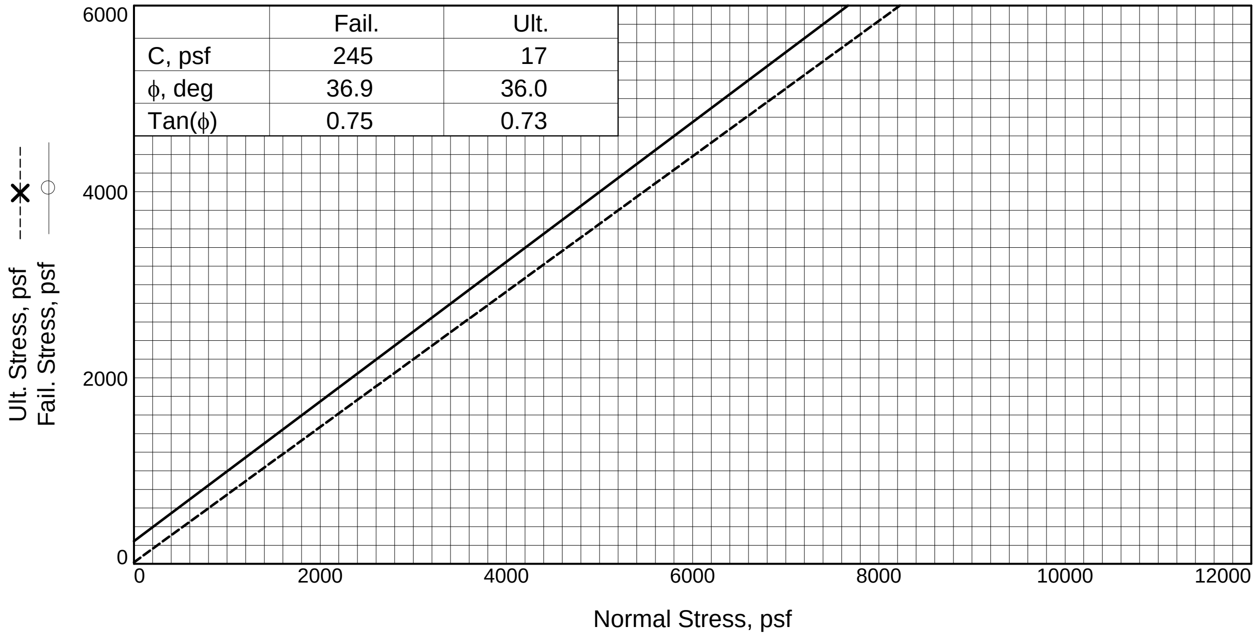
GeoSoils, Inc.

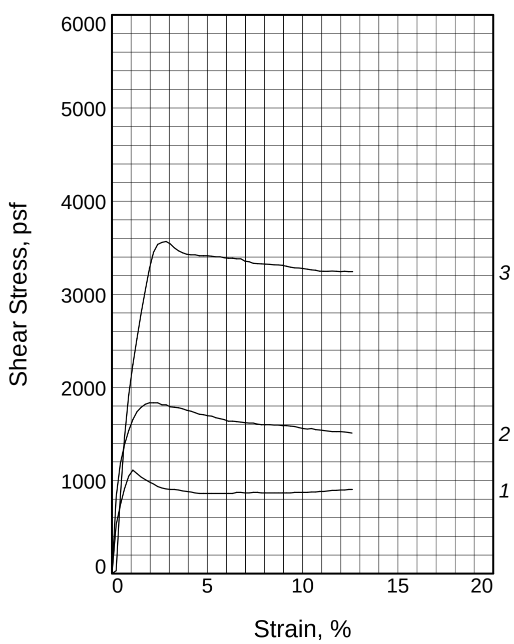
Client: Glen Ivy Properties
Project: Retail & Senior Living

Project No: 7731-A-SC **Figure**

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

	Fail.	Ult.
C, psf	245	17
ϕ, deg	36.9	36.0
Tan(ϕ)	0.75	0.73





	Sample No.	1	2	3
Initial	Water Content, %	8.6	8.6	8.6
	Dry Density, pcf	91.8	82.7	89.0
	Saturation, %	28.4	22.8	26.5
	Void Ratio	0.8027	1.0012	0.8588
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	24.7	27.2	23.0
	Dry Density, pcf	92.3	83.6	90.6
	Saturation, %	82.6	73.7	73.9
	Void Ratio	0.7919	0.9792	0.8253
	Diameter, in.	2.38	2.38	2.38
	Height, in.	0.99	0.99	0.98
	Normal Stress, psf	1100	2200	4400
	Fail. Stress, psf	1111	1834	3568
	Strain, %	1.1	2.2	2.8
	Ult. Stress, psf	877	1526	3248
	Strain, %	4.1	11.8	10.9
	Strain rate, in./min.	0.001	0.001	0.001

Sample Type: Undisturbed
Description: Black Sandy Organic Soil

Specific Gravity= 2.65
Remarks:

Plate ________

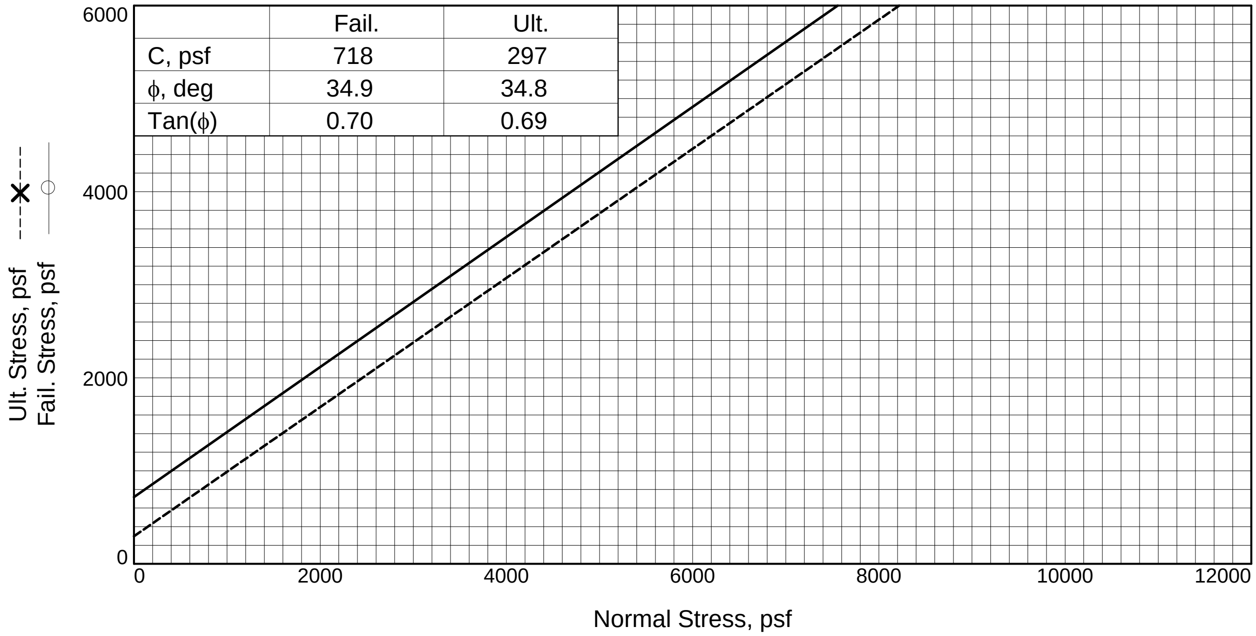
Client: Glen Ivy Properties

Project: Retail & Senior Living

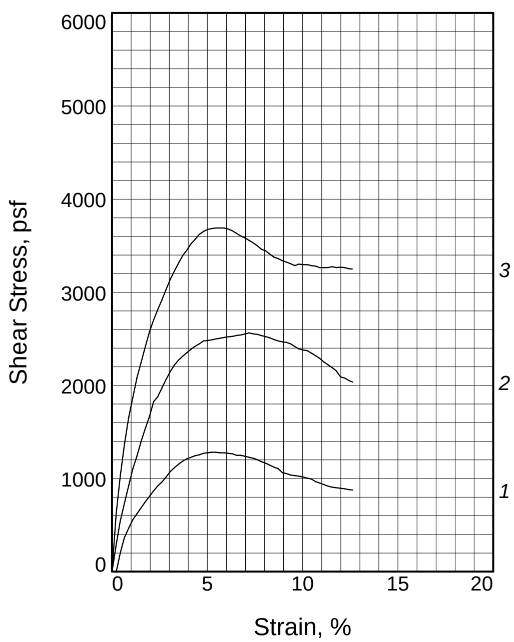
Source of Sample: B-2 **Depth:** 10.0
Sample Number: B-2
Proj. No.: 7731-A-SC **Date Sampled:**

GeoSoils, Inc.

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



	Fail.	Ult.
C, psf	718	297
ϕ, deg	34.9	34.8
Tan(ϕ)	0.70	0.69



	Sample No.	1	2	3
Initial	Water Content, %	5.2	5.2	5.2
	Dry Density, pcf	98.9	103.4	100.5
	Saturation, %	20.5	23.0	21.4
	Void Ratio	0.6726	0.5999	0.6454
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	21.6	19.4	21.2
	Dry Density, pcf	99.4	104.1	102.4
	Saturation, %	86.1	87.3	91.3
	Void Ratio	0.6643	0.5887	0.6158
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	0.99	0.98
	Normal Stress, psf	1100	2200	4400
	Fail. Stress, psf	1281	2563	3690
	Strain, %	5.2	7.2	5.7
	Ult. Stress, psf	889	2080	3265
	Strain, %	12.2	12.2	11.1
	Strain rate, in./min.	0.002	0.002	0.002

Sample Type: Undisturbed
Description: Dark Brown Silty Sand

Specific Gravity= 2.65
Remarks:

Plate ________

Client: Glen Ivy Properties

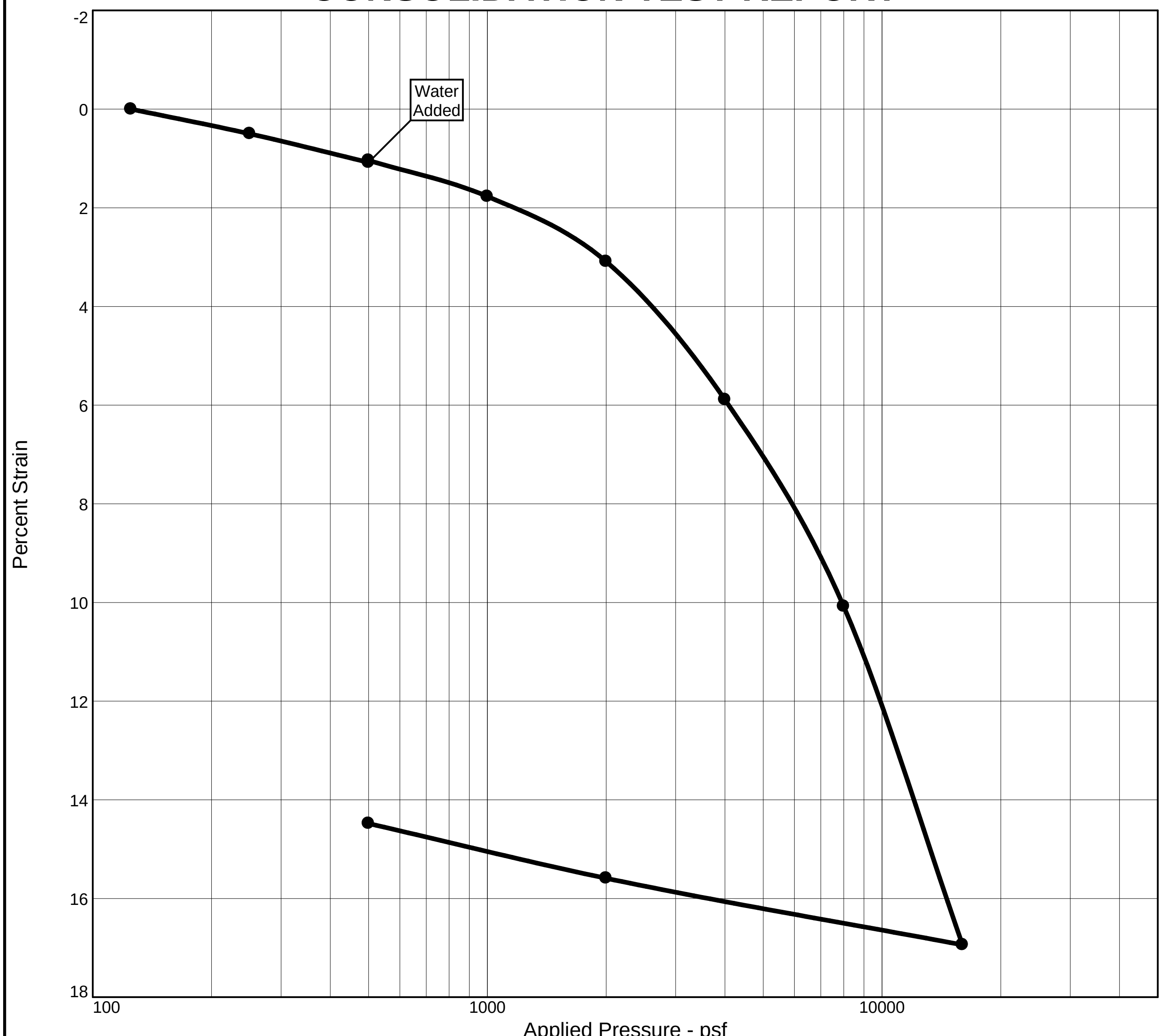
Project: Retail & Senior Living

Source of Sample: B-4 **Depth:** 5.0
Sample Number: B-4
Proj. No.: 7731-A-SC **Date Sampled:**

GeoSoils, Inc.

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

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_s	Swell Press. (psf)	Swell %	e_o
Sat.	Moist.											
60.0 %	33.1 %	67.2			2.65	500	4591	0.57	0.04	520	0.0	1.461

MATERIAL DESCRIPTION	USCS	AASHTO
Black Sandy Organic Soil	OL	

Project No. 7731-A-SC **Client:** Glen Ivy Properties

Project: Retail & Senior Living

Source of Sample: B-1 **Depth:** 5.0 **Sample Number:** B-1

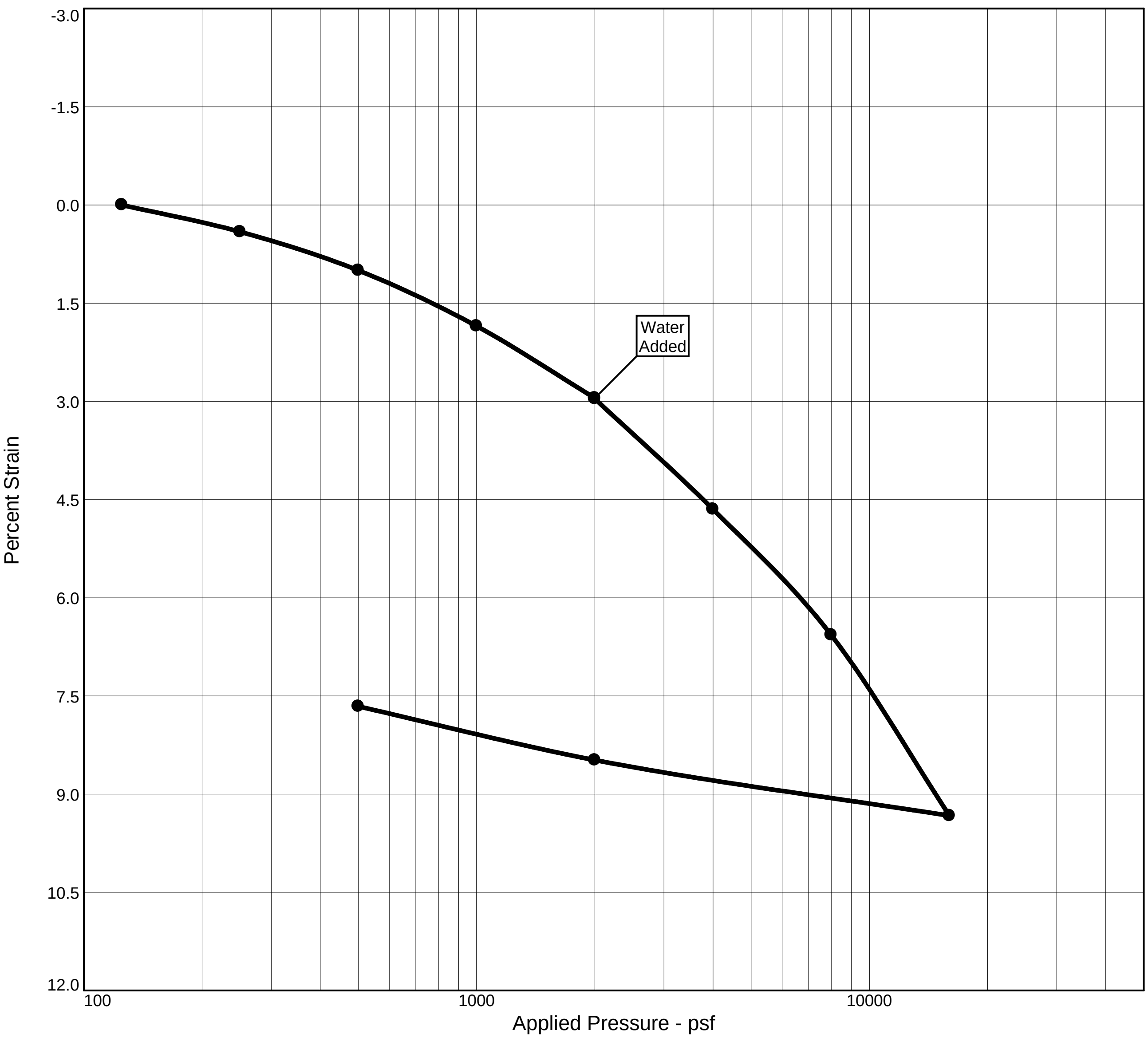
GeoSoils, Inc.

Remarks:

Figure

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

CONSOLIDATION TEST REPORT



Natural Sat.	Natural Moist.	Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_s	Swell Press. (psf)	Clpse. %	e_o
61.9 %	11.2 %	111.9			2.65	2000	8245	0.14	0.02		0.0	0.478

MATERIAL DESCRIPTION	USCS	AASHTO
Black Silty Sand	SM	

Project No. 7731-A-SC **Client:** Glen Ivy Properties

Project: Retail & Senior Living

Source of Sample: B-1 **Depth:** 15.0 **Sample Number:** B-1

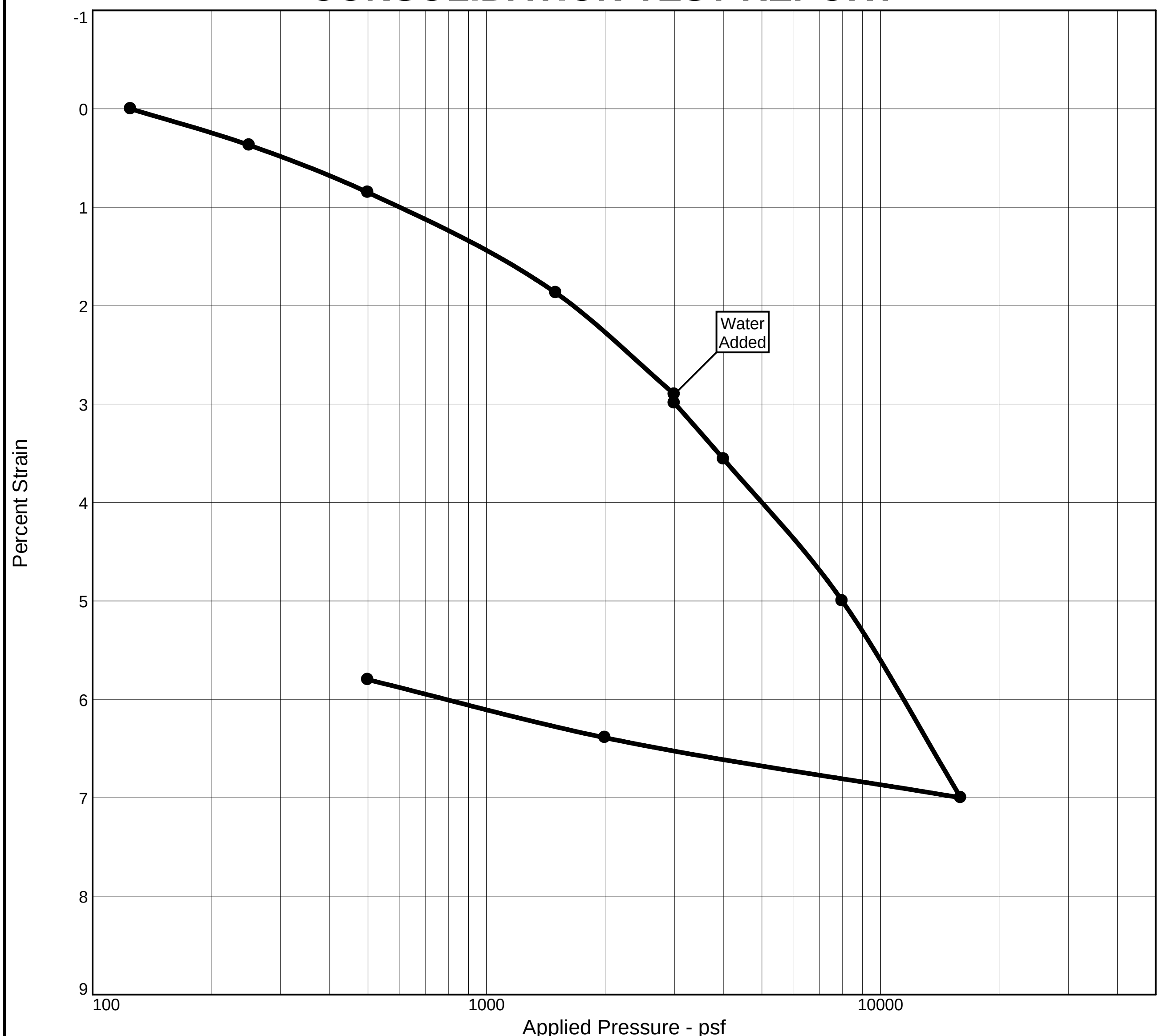
GeoSoils, Inc.

Remarks:

Figure

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

CONSOLIDATION TEST REPORT



Natural Sat.	Natural Moist.	Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_s	Swell Press. (psf)	Clpse. %	e_o
98.6 %	26.8 %	96.1			2.65	3000	8233	0.12	0.01		0.1	0.721

MATERIAL DESCRIPTION	USCS	AASHTO
Dark Greenish Gray Sandy Silt	ML	

Project No. 7731-A-SC **Client:** Glen Ivy Properties

Project: Retail & Senior Living

Source of Sample: B-1 **Depth:** 35.0 **Sample Number:** B-1

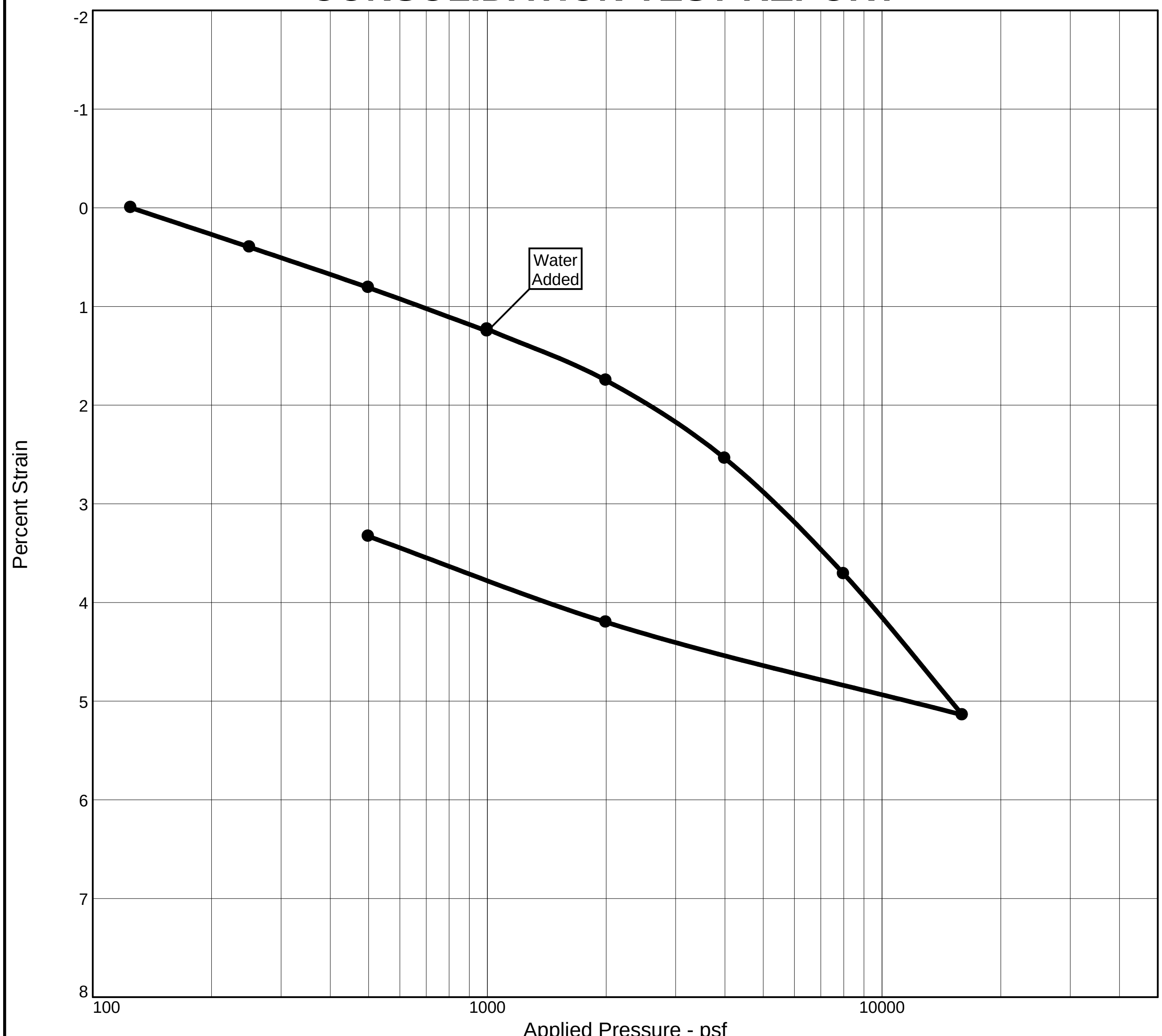
GeoSoils, Inc.

Remarks:

Figure

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

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_s	Swell Press. (psf)	Swell %	e_o
Sat.	Moist.											
77.3 %	20.1 %	97.9			2.65	1000	3568	0.08	0.02	1027	0.0	0.690

MATERIAL DESCRIPTION	USCS	AASHTO
Black Sandy Organic Soil	OL	

Project No. 7731-A-SC **Client:** Glen Ivy Properties

Project: Retail & Senior Living

Source of Sample: B-3 **Depth:** 10.0 **Sample Number:** B-3

GeoSoils, Inc.

Remarks:

Figure

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

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

Work Order No.: 20B1141
Client: GeoSoils, Inc.
Project No.: 7731-A-SC
Project Name: Glen Ivy Properties
Report Date: February 21, 2020

Laboratory Test(s) Results Summary

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

Sample Identification	pH (H+)	As Rec'd Resistivity (ohm-cm)	Saturated Resistivity (ohm-cm)	Sulfate Content (mg/L)	Chloride Content (mg/L)
B-1 @ 0-5 feet	4.1	1,700	930	620	30

*ND=No Detection

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

Ahmet K. Kaya, Laboratory Manager

ASTM INTERNATIONAL ORGANIZATIONAL MEMBER

APPENDIX E

LIQUEFACTION ANALYSIS

APPENDIX E

LIQUEFACTION ANALYSES

CLiq (Version 1.7.6.34) Analyses

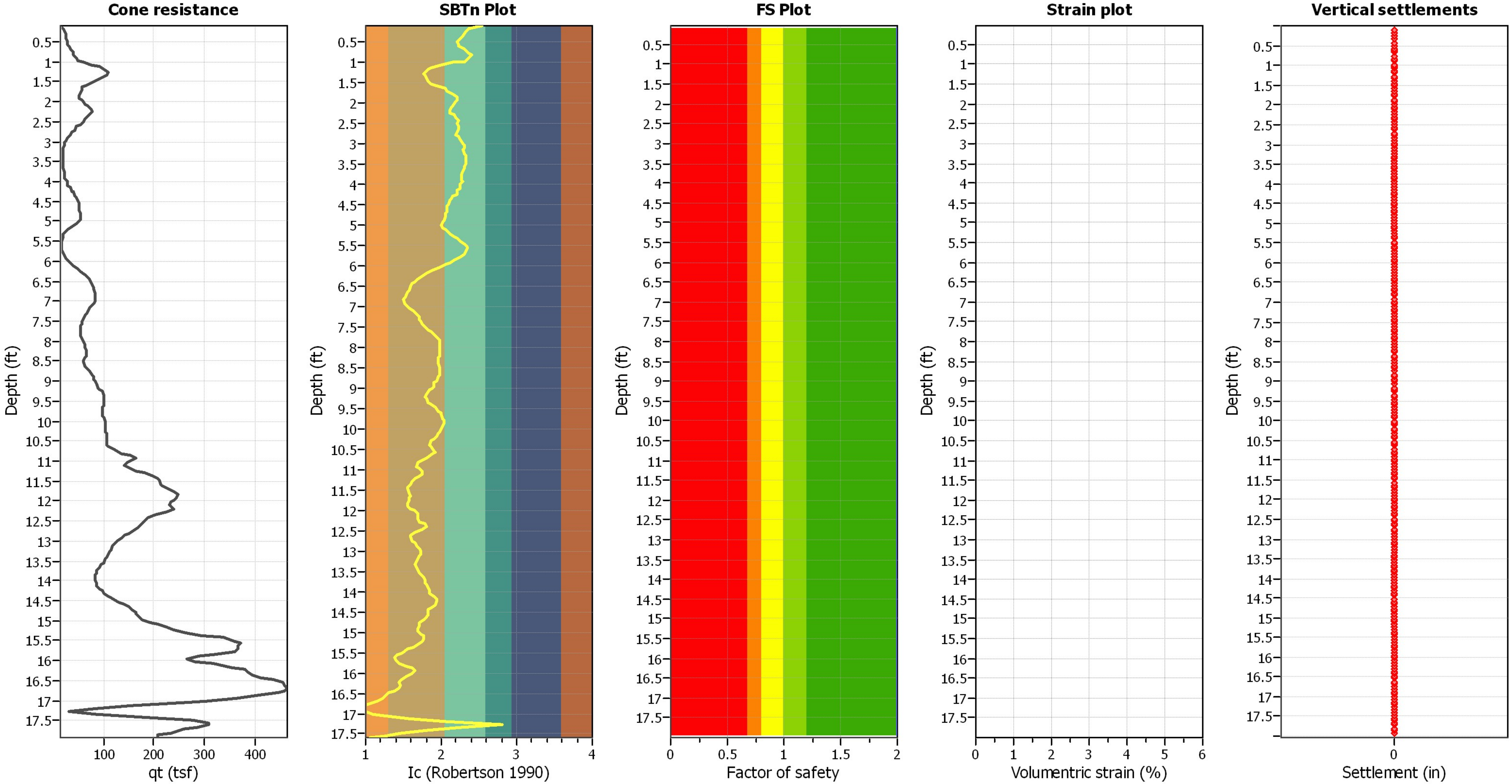
The liquefaction analyses for this project was performed using Borings B-1 through B-6 in conjunction with the correlative Cone Penetration Tests (CPTs) CPT-1 through CPT-6 and were evaluated to be representative of the site subsurface conditions given the available body of data on this site, at the start of our evaluation.

Our analyses utilized the CLiq computer program, Version 1.7.6.34. This software was written by GeoLogismiki Geotechnical Software Company (GGSC). A registered copy of this software is in use in our Carlsbad, California office and documentation of its authenticity is available either through GeoSoils, Inc. (GSI) or GGSC. CLiq is a software that evaluates the liquefaction potential of earth materials and calculates the potential settlement of soil deposits due to seismic loading. The software is developed in collaboration with Gregg Drilling & Testing Inc. (GDTI), and Professor Peter Robertson, author of the method utilized in the software. Cliq performs analyses by applying state-of-the-art methods (e.g. Youd, et al, 2001) along with the calibrated procedures for post-earthquake displacements (e.g. Zhang, et al, 2002 and 2004).

GSI has assumed that site elevations may be raised on the order of ±3 to ±5 feet above existing grades (in the areas of Borings B-1 through B-6). During our field study perched groundwater was encountered in the marsh deposits at a depth of approximately ±32 feet below the ground surface (b.g.s.). Groundwater levels in other nearby wells were previously measured at depths ranging from ±22 feet (Well No. 337430N1174280W001 - November 13, 2018) to ±53 feet (Well No. 338227N1175072W001 - November 13, 2016) below the ground surface (CDWR, 2020). However, it should be noted that these wells lie within alluvial valley areas. For conservatism, a groundwater depth of 30 feet was utilized in our analysis. Our analyses utilized a Mean Peak Ground Acceleration (PGA_M) of 1.139g and an earthquake magnitude (Mw) of 6.8.

Our analyses evaluated Borings B-1 through B-6 and CPT-1 through CPT-6 that were advanced onsite by GSI. Borings B-1 and B-6 were advanced to a depth of approximately ±51½ feet. The following printouts (Plates E-1 through E-6) present the results of our liquefaction analyses:

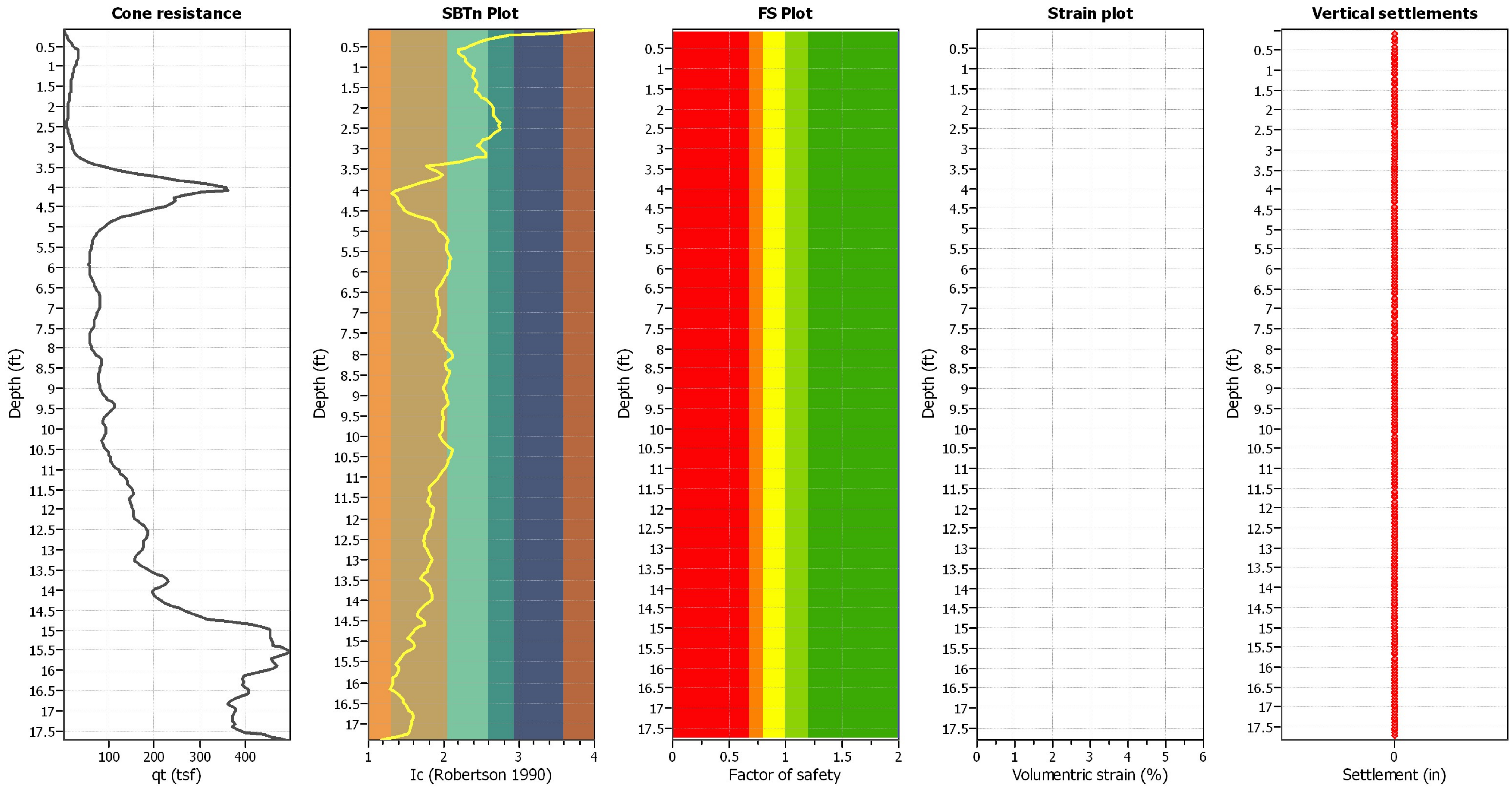
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

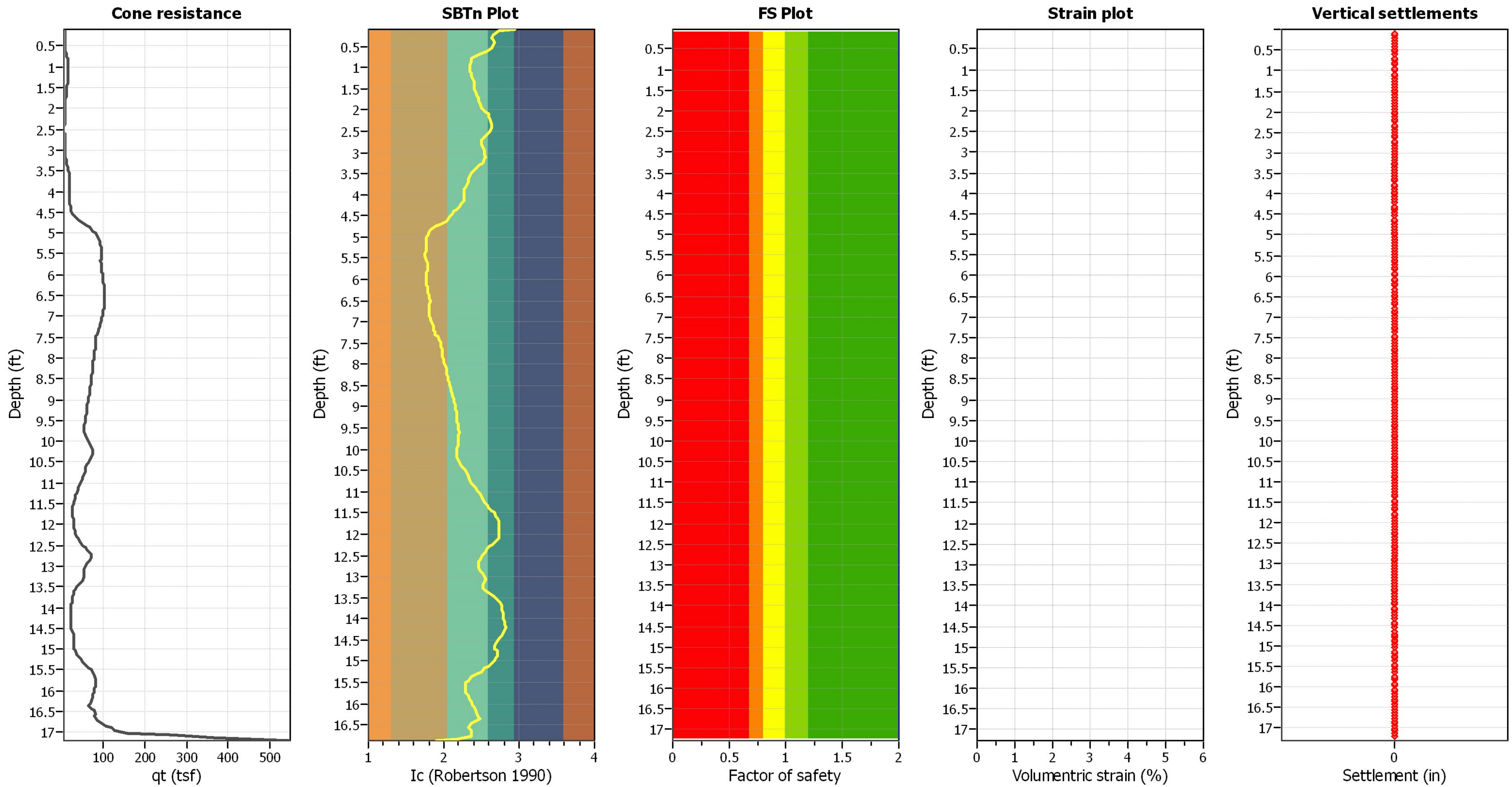
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake settlements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
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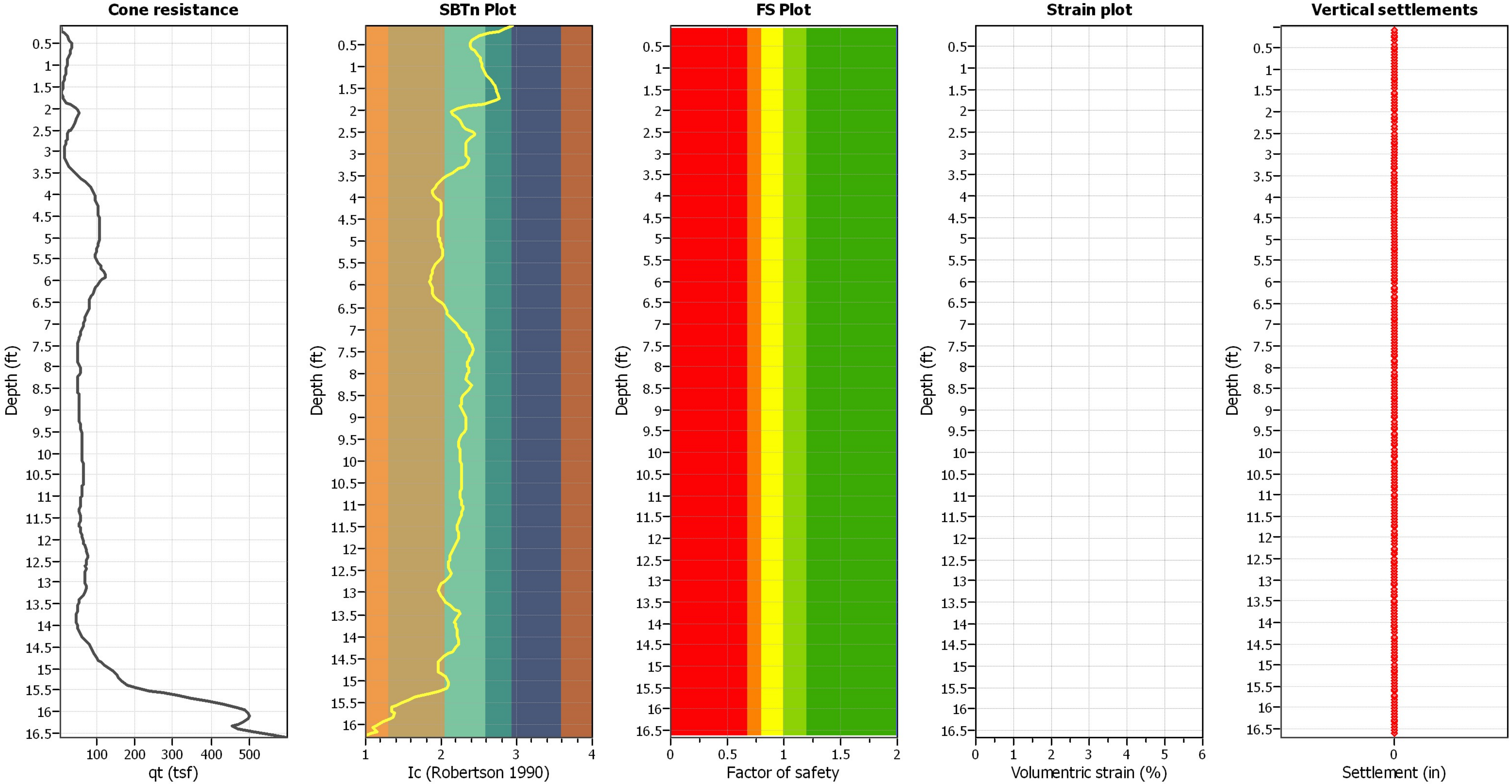
Estimation of post-earthquake settlements



Abbreviations

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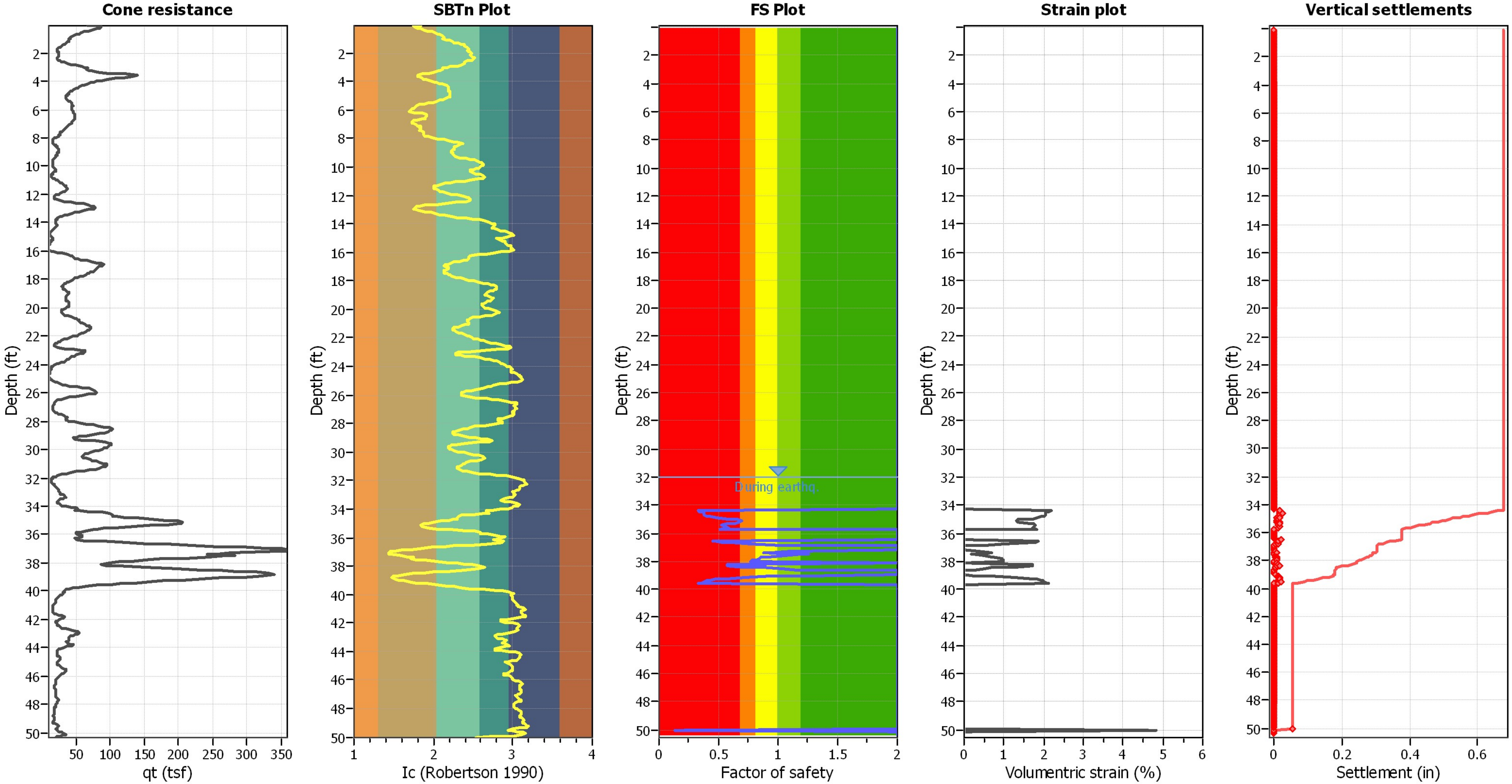
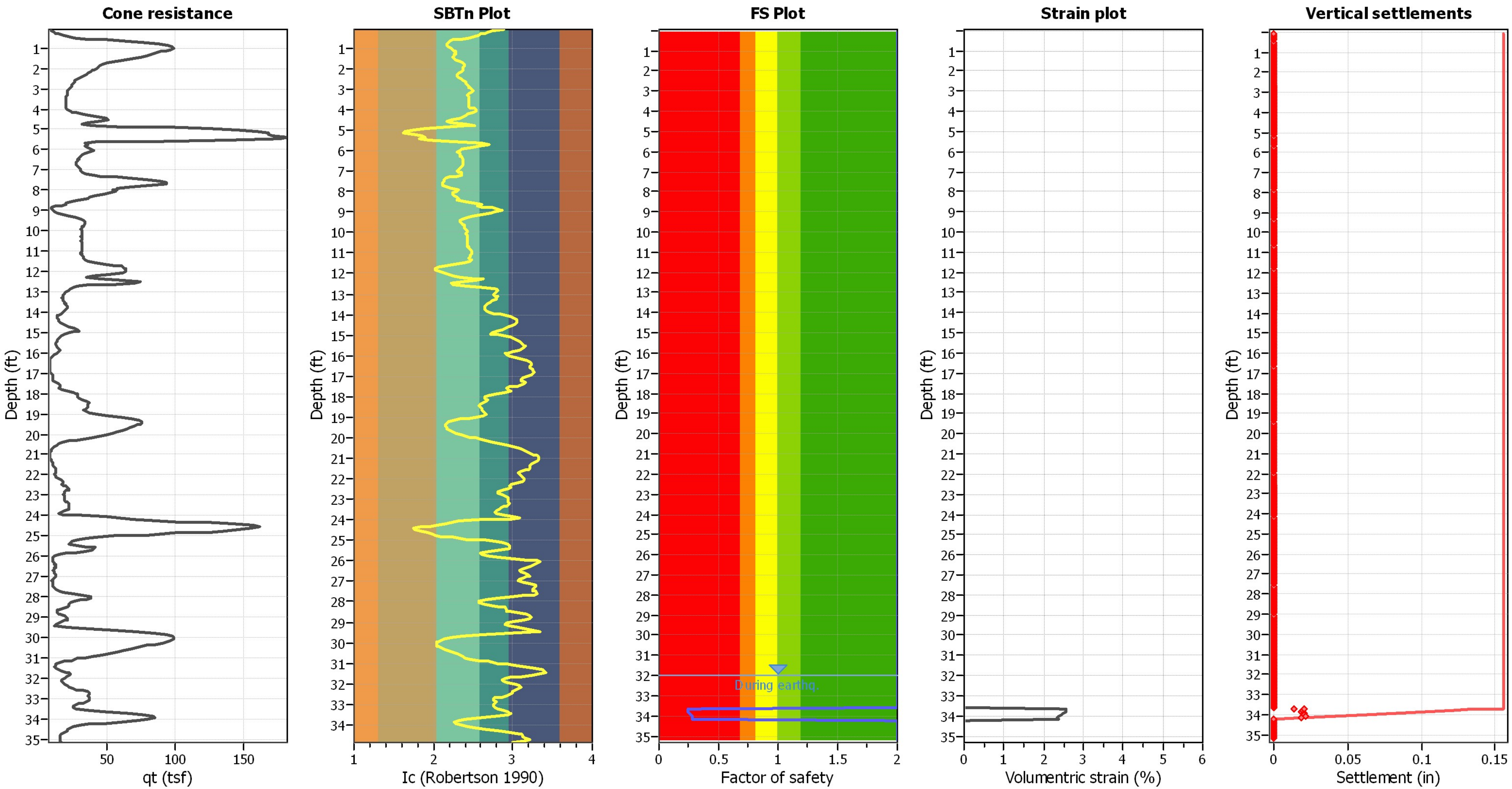
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

APPENDIX F

GENERAL EARTHWORK, GRADING GUIDELINES AND PRELIMINARY CRITERIA

GENERAL EARTHWORK, GRADING GUIDELINES, AND PRELIMINARY CRITERIA

General

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

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

EARTHWORK OBSERVATIONS AND TESTING

Geotechnical Consultant

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

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

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

Laboratory and Field Tests

Maximum dry density tests to determine the degree of compaction should be performed in accordance with American Standard Testing Materials test method ASTM designation

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

Contractor's Responsibility

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

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

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

SITE PREPARATION

All major vegetation, including brush, trees, thick grasses, organic debris, and other deleterious material, should be removed and disposed of off-site. These removals must be 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, etc.), the developer may consider increasing the hold-down depth of any rocky fills to be placed, as appropriate. In addition, some agencies/jurisdictions mandate a specific hold-down depth for oversize materials placed in fills. The hold-down depth, and potential to encounter oversize rock, both within fills, and occurring in cut or natural areas, would need to be disclosed to all interested/affected parties. Once approved by the governing agency, the hold-down depth for oversized rock (i.e., greater than 12 inches) in fills on this project is provided as 10 feet, unless specified differently in the text of this report. The governing agency may require that these materials need to be deeper, crushed, or reduced to less than 12 inches in maximum dimension, at their discretion.

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

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

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

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

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

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

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

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

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

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

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

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

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

SUBDRAIN INSTALLATION

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

EXCAVATIONS

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

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

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

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

COMPLETION

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

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

JOB SAFETY

General

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

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

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

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

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

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

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

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

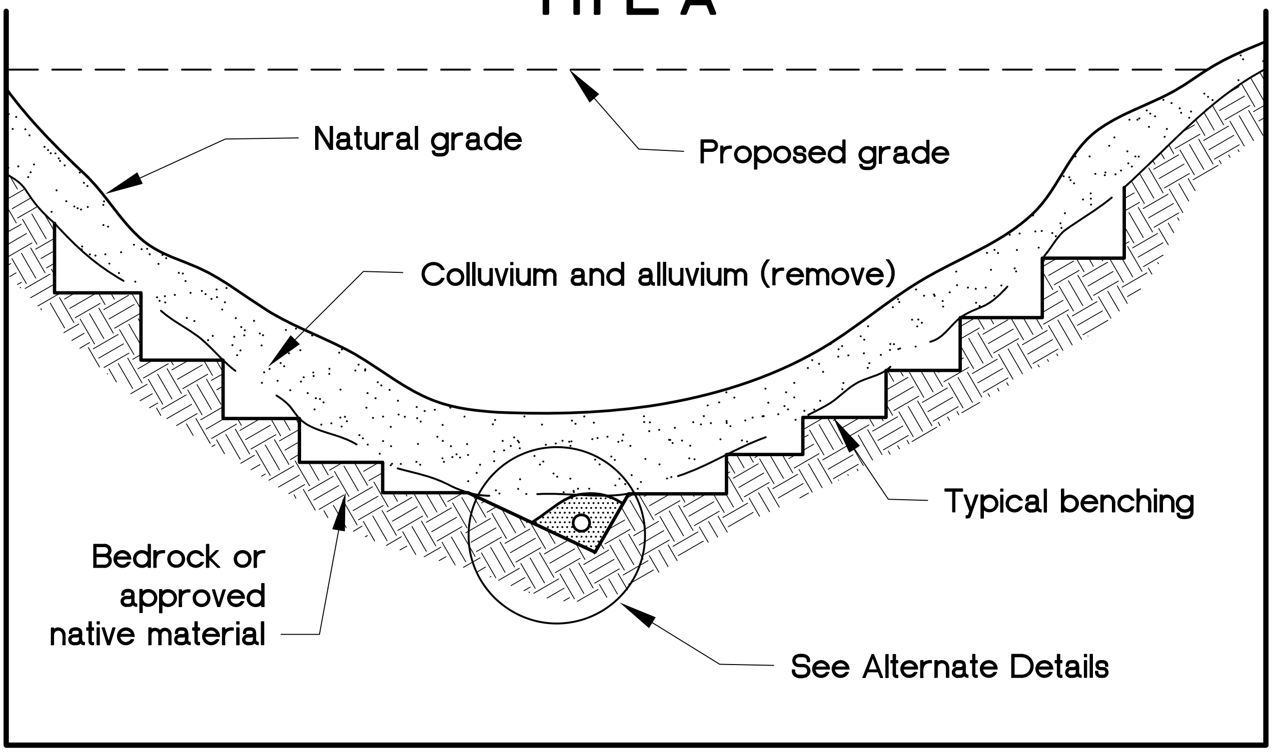
It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Our personnel are directed not to enter any excavation or vertical cut which: 1) is 5 feet or deeper unless shored or laid back; 2) displays any evidence of instability, has any loose rock or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

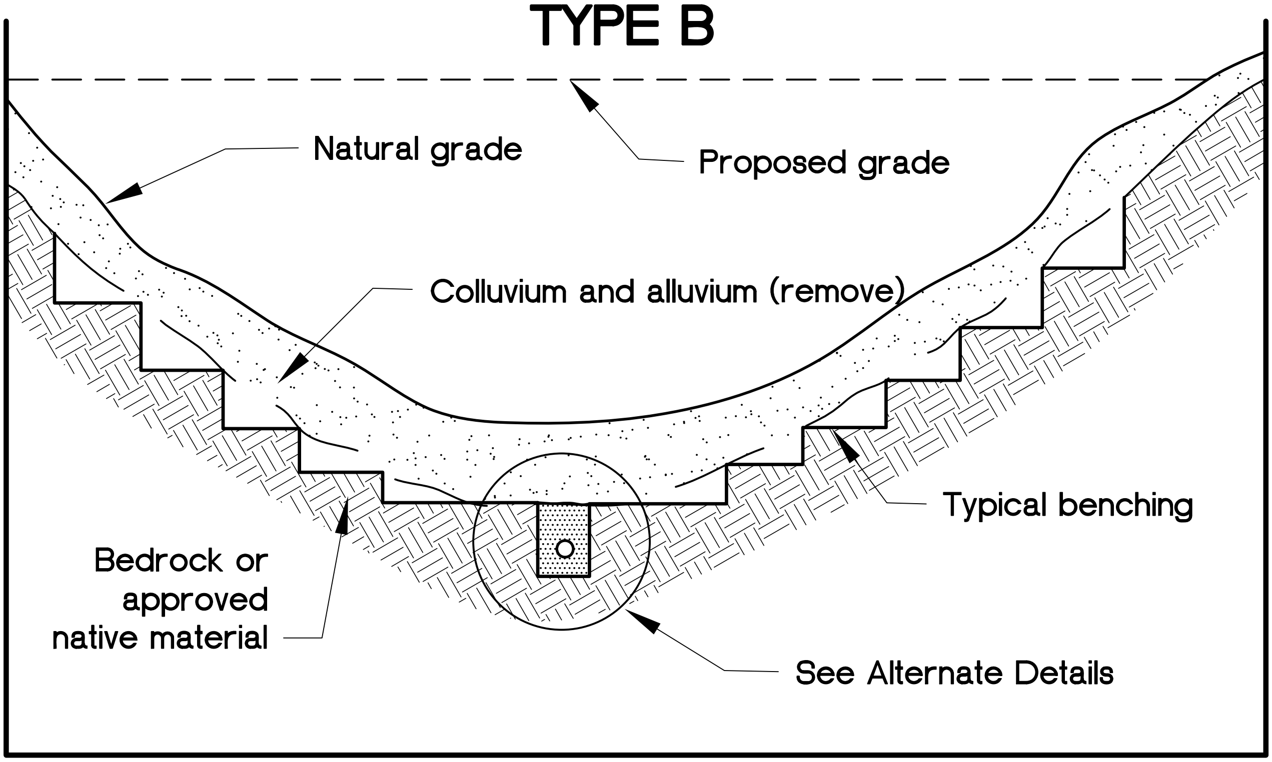
If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraw and notify his/her supervisor. The contractor's representative will be contacted in an effort to affect a solution. All backfill not tested due to safety concerns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

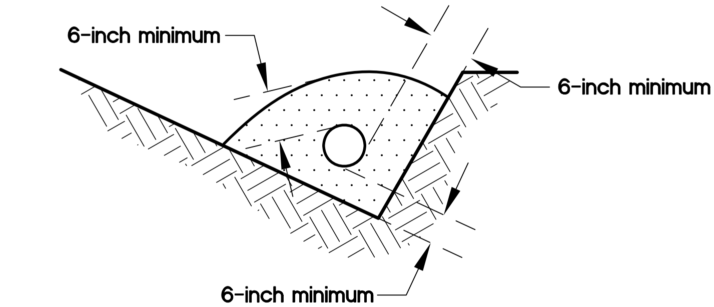


TYPE B

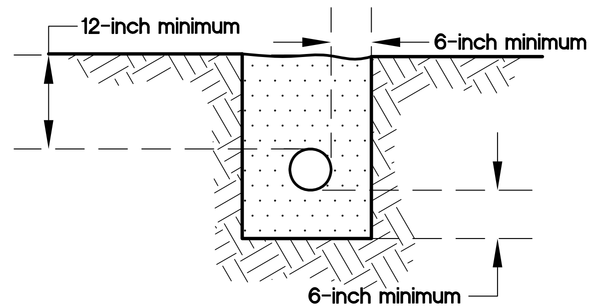


Selection of alternate subdrain details, location, and extent of subdrains should be evaluated by the geotechnical consultant during grading.

GeoSoils, Inc.	CANYON SUBDRAIN DETAIL	Plate F-1



A-1



B-1

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

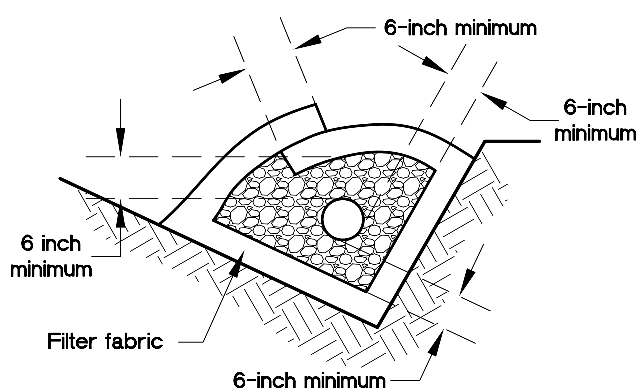
Perforated pipe: 6-inch-diameter ABS or PVC pipe or approved substitute with minimum 8 perforations ($\frac{1}{4}$-inch diameter) per lineal foot in bottom half of pipe (ASTM D-2751, SDR-35, or ASTM D-1527, Schd. 40).

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

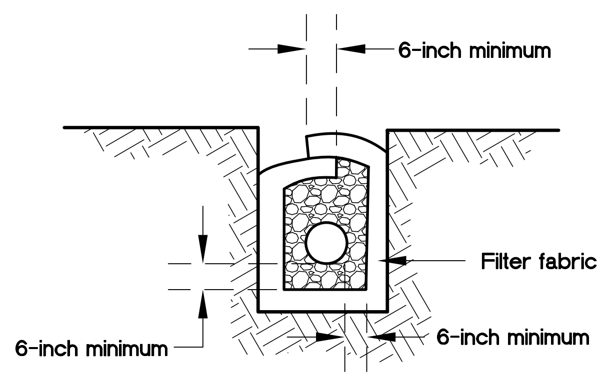
FILTER MATERIAL

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

ALTERNATE 1: PERFORATED PIPE AND FILTER MATERIAL



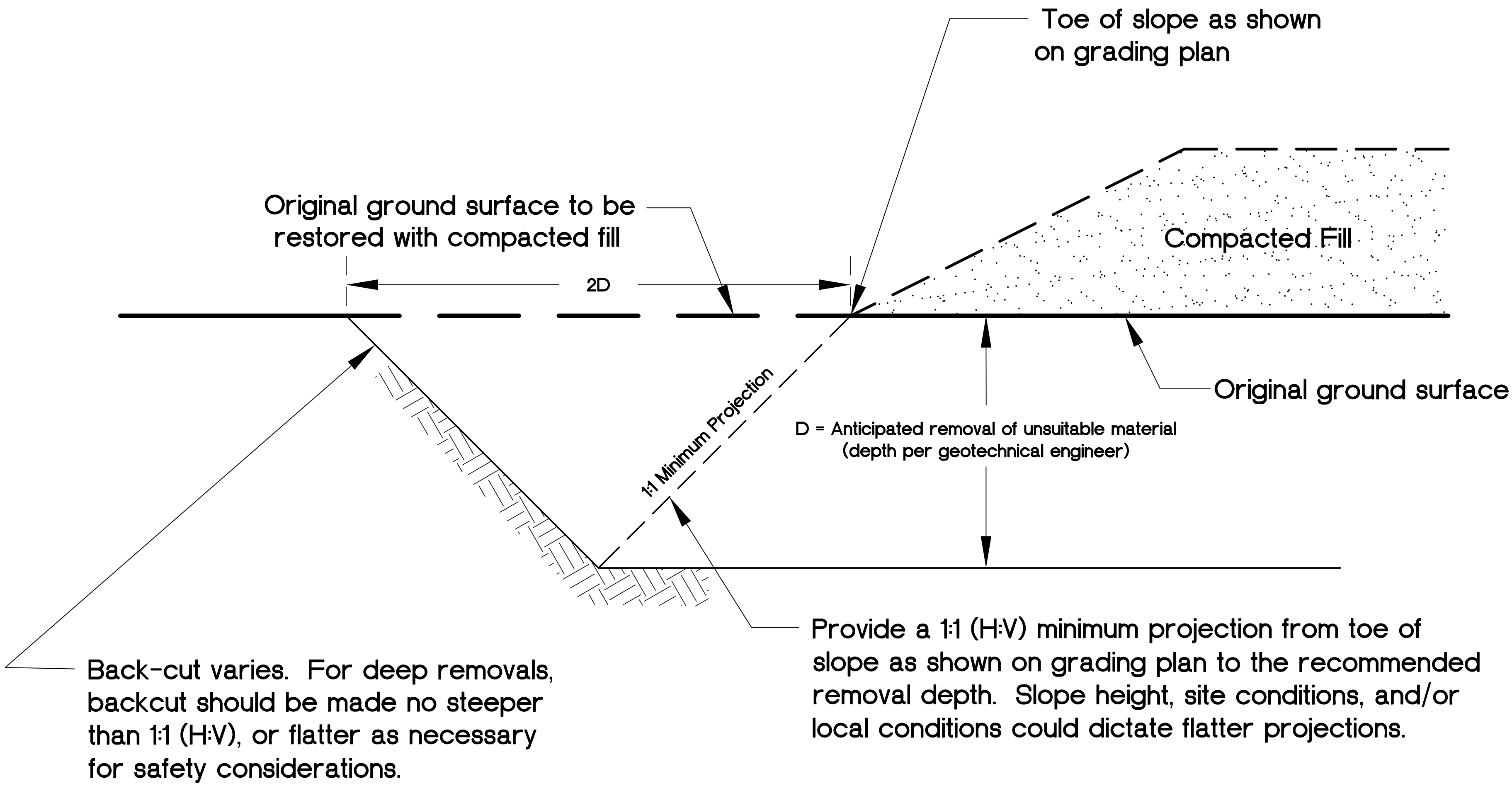
A-2



B-2

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

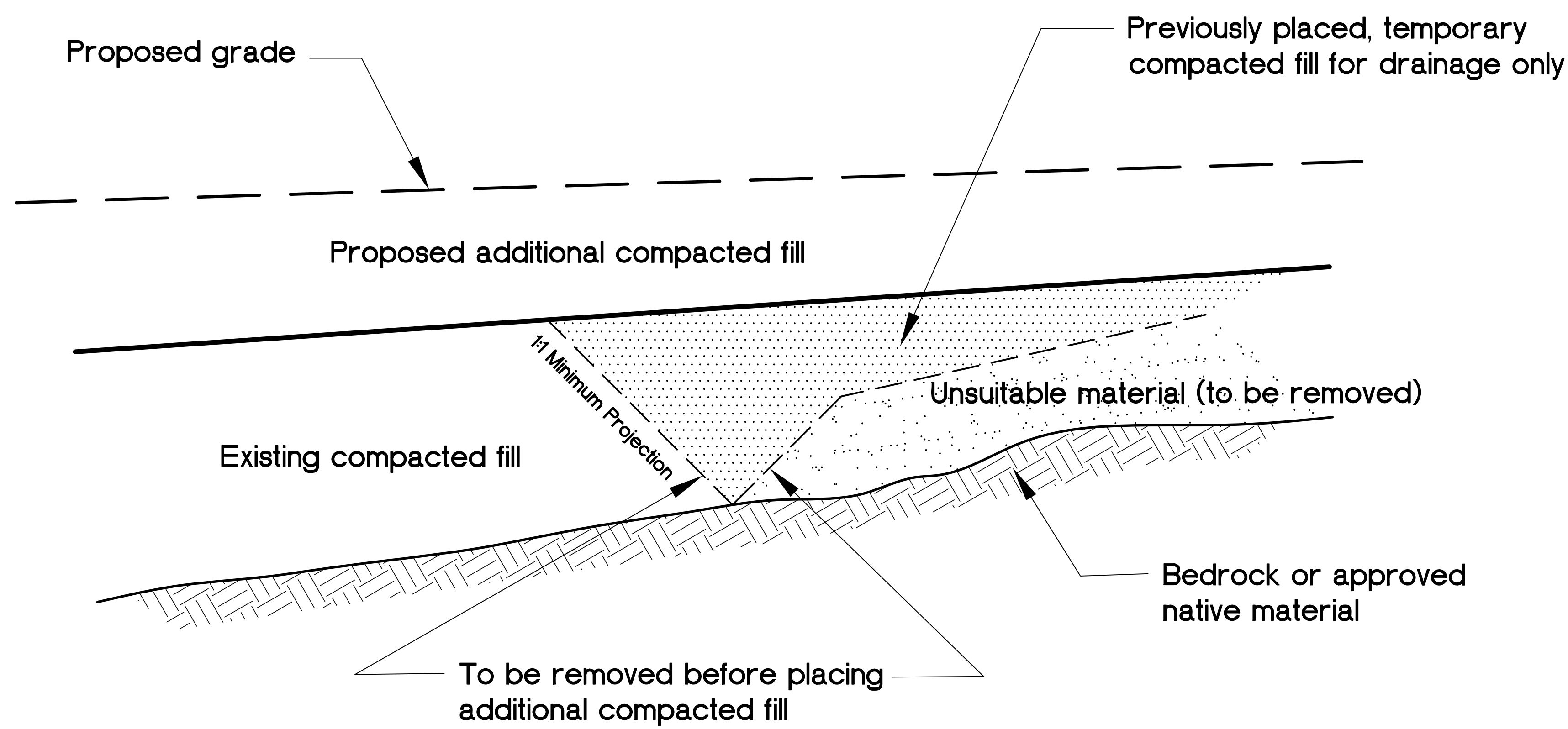
ALTERNATE 2: PERFORATED PIPE, GRAVEL, AND FILTER FABRIC



GeoSoils, Inc.

FILL SLOPE TOEING OUT ON FLAT ALLUVIATED CANYON DETAIL

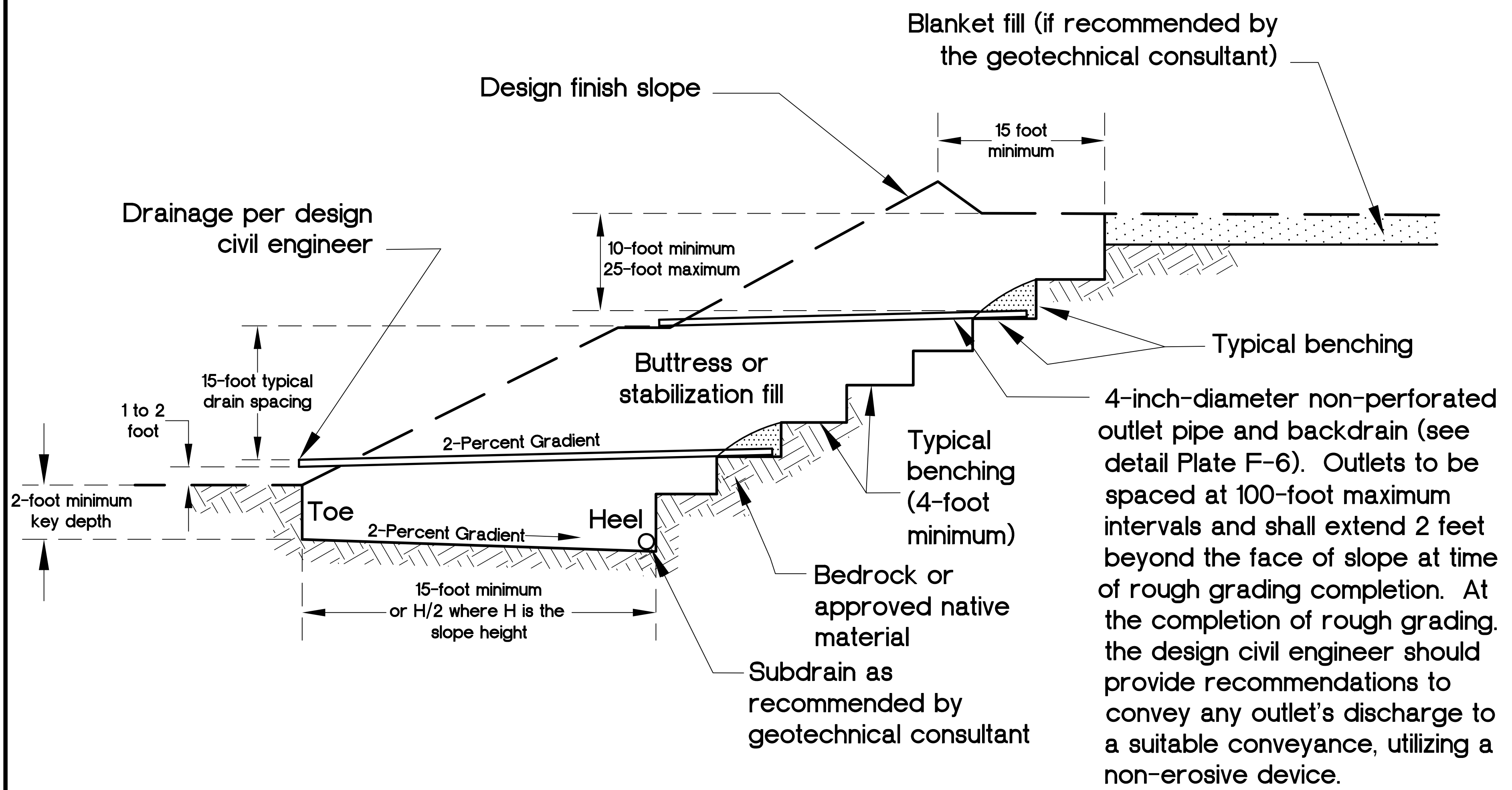
Plate F-3



GeoSoils, Inc.

REMOVAL ADJACENT TO EXISTING FILL ADJOINING CANYON FILL DETAIL

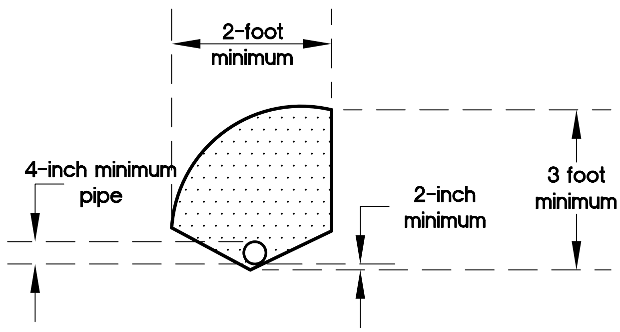
Plate F-4

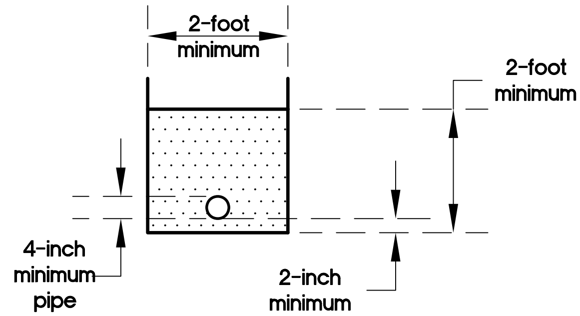


GeoSoils, Inc.

TYPICAL STABILIZATION / BUTTRESS FILL DETAIL

Plate F-5





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

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

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

Notes:

1. Trench for outlet pipes to be backfilled and compacted with onsite soil.

2. Backdrains and lateral drains shall be located at elevation of every bench drain. First drain located at elevation just above lower lot grade. Additional drains may be required at the discretion of the geotechnical consultant.

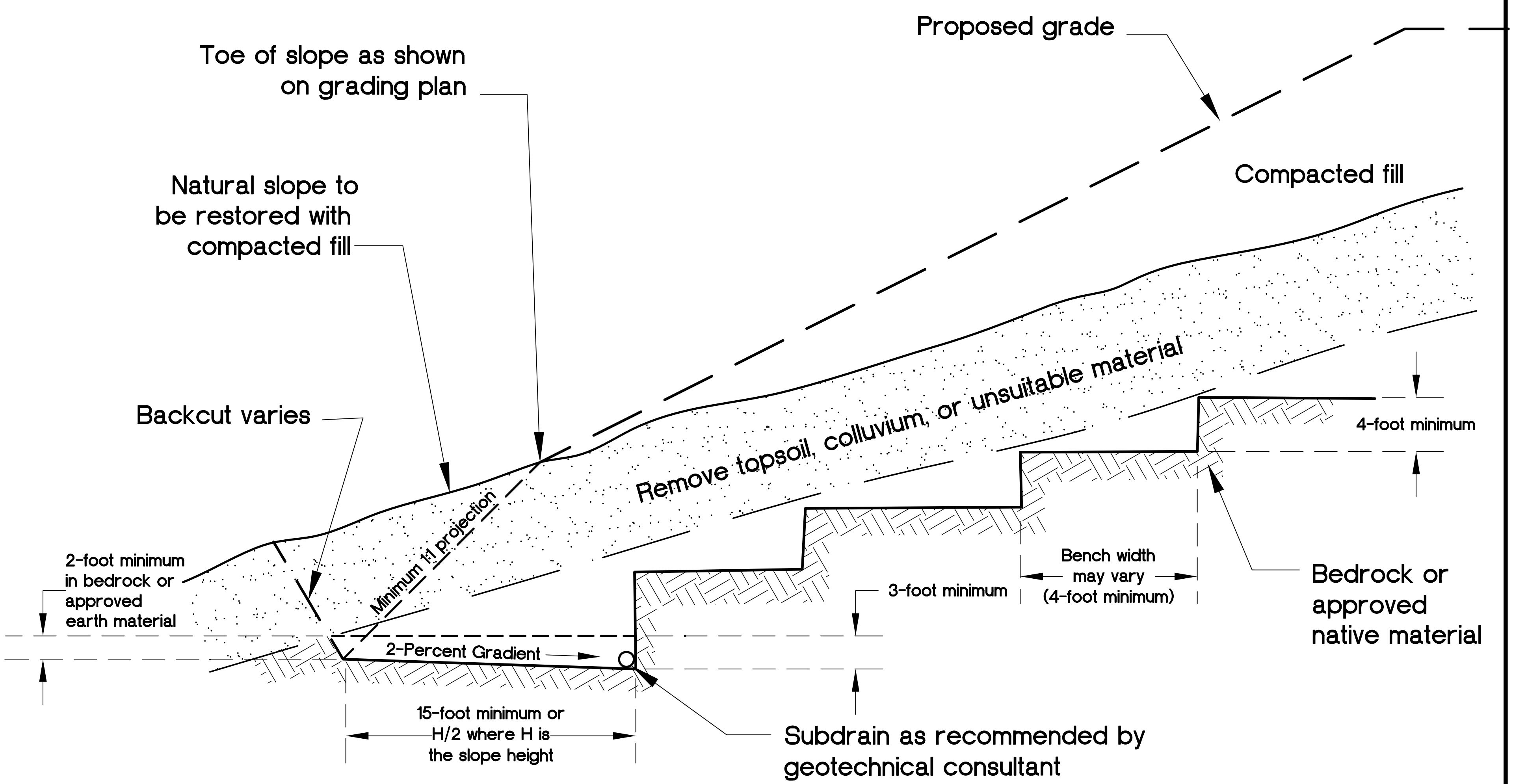
Filter Material shall be of the following specification or an approved equivalent.

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

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

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

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



NOTES:

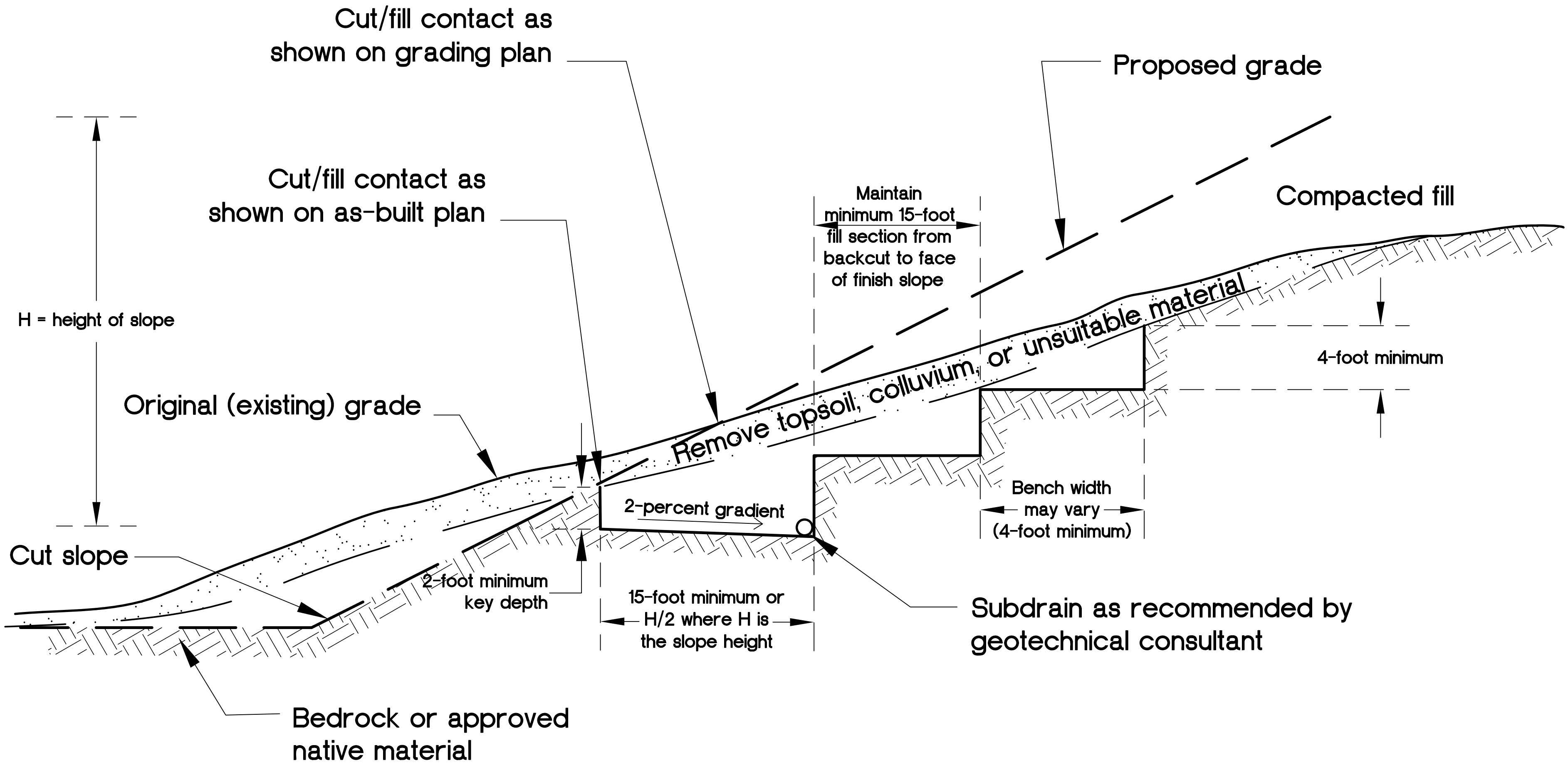
1. Where the natural slope approaches or exceeds the design slope ratio, special recommendations would be provided by the geotechnical consultant.

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

GeoSoils, Inc.

FILL OVER NATURAL (SIDEHILL FILL) DETAIL

Plate F-7

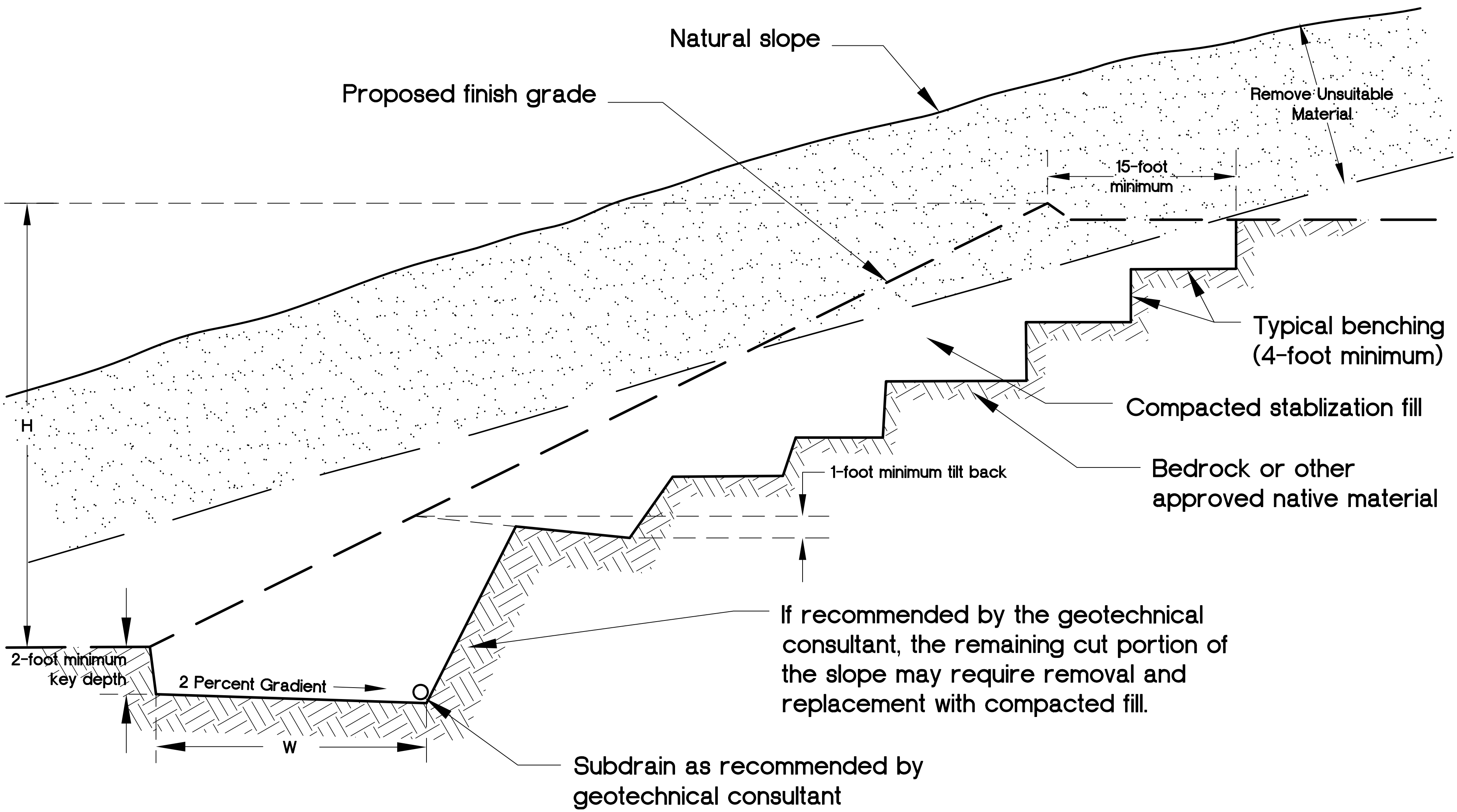


NOTE: The cut portion of the slope should be excavated and evaluated by the geotechnical consultant prior to construction of the fill portion.

GeoSoils, Inc.

FILL OVER CUT DETAIL

Plate F-8

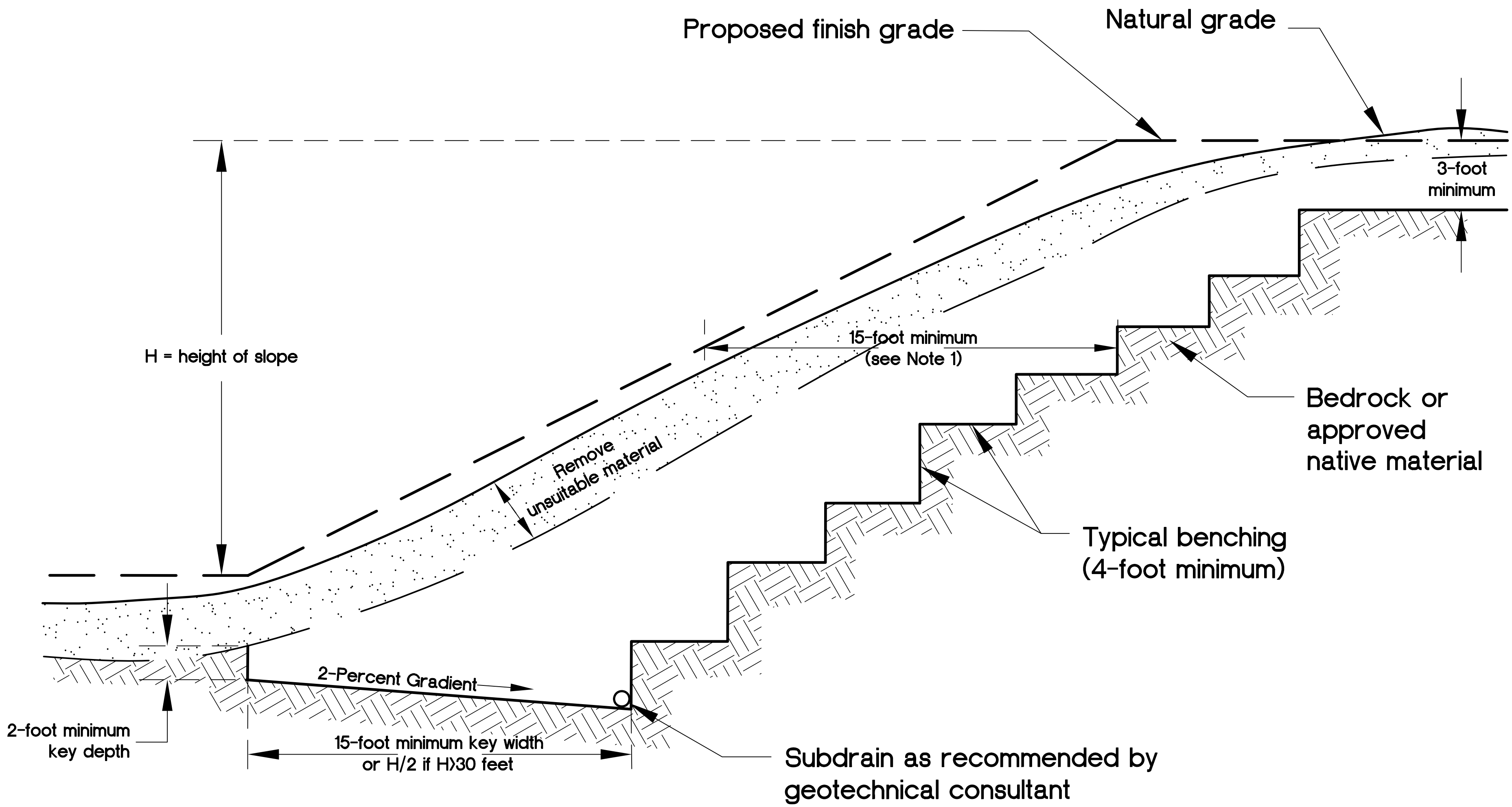


NOTES:

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

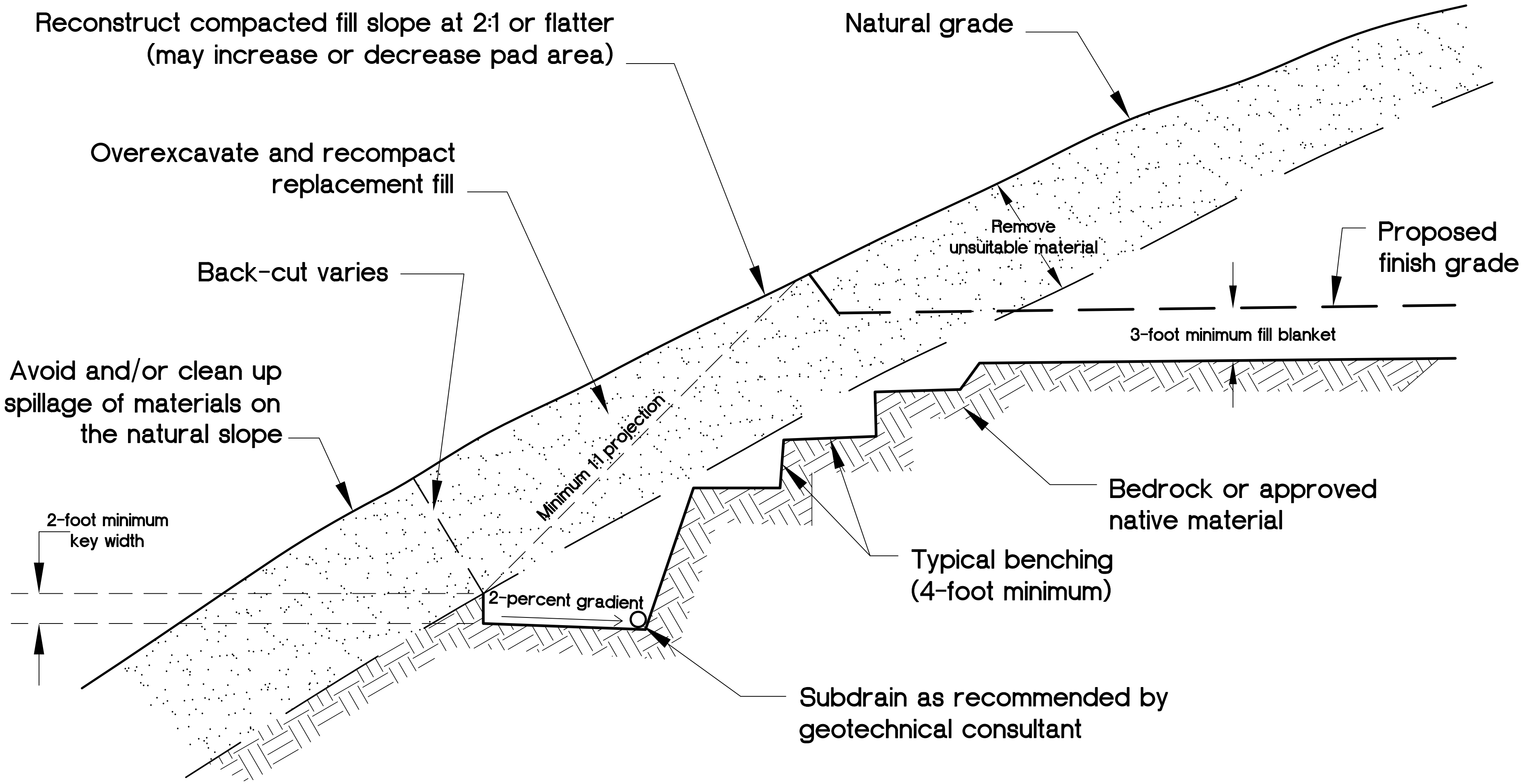
2 W shall be equipment width (15 feet) for slope heights less than 25 feet. For slopes greater than 25 feet, W shall be evaluated by the geotechnical consultant. At no time, shall W be less than H/2, where H is the height of the slope.

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



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

GeoSoils, Inc. | SKIN FILL OF NATURAL GROUND DETAIL | Plate F-10



NOTES:

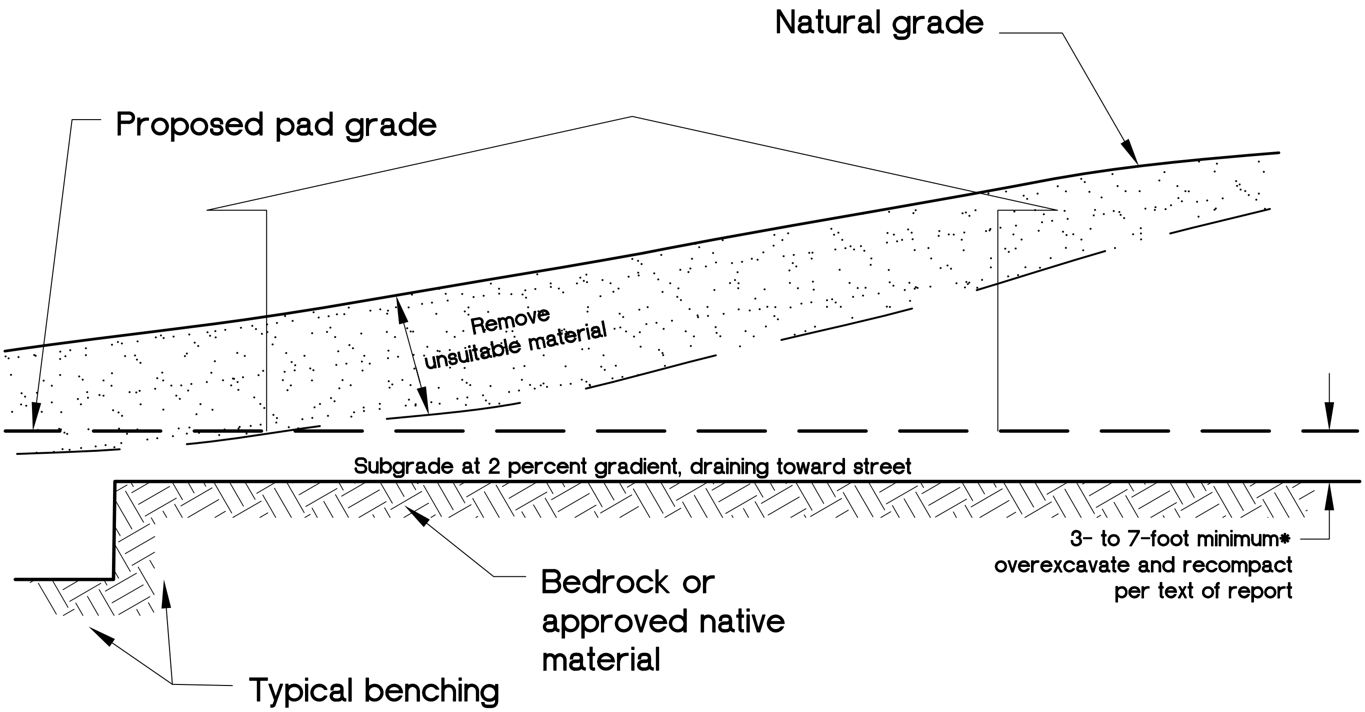
1. Subdrain and key width requirements will be evaluated based on exposed subsurface conditions and thickness of overburden.

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

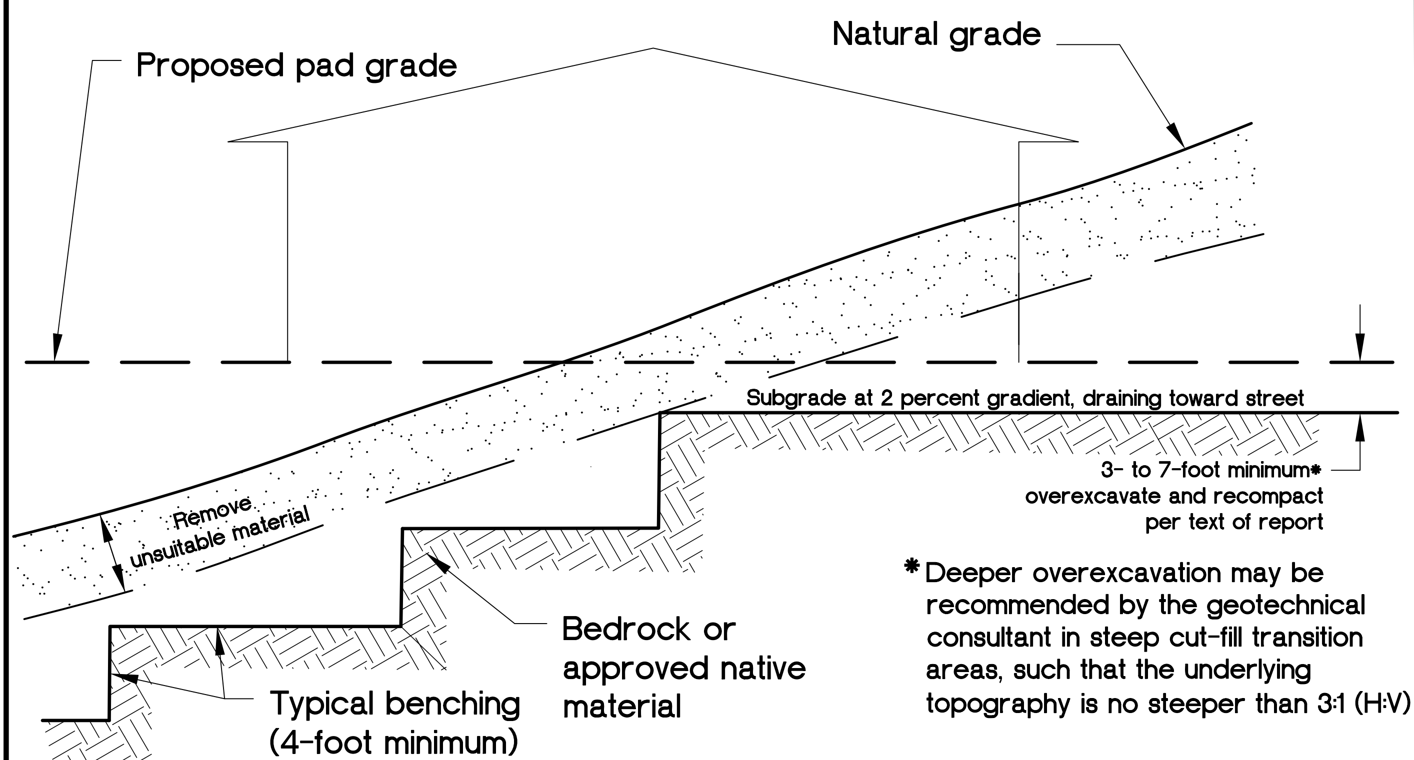
GeoSoils, Inc.

DAYLIGHT CUT LOT DETAIL

Plate F-11



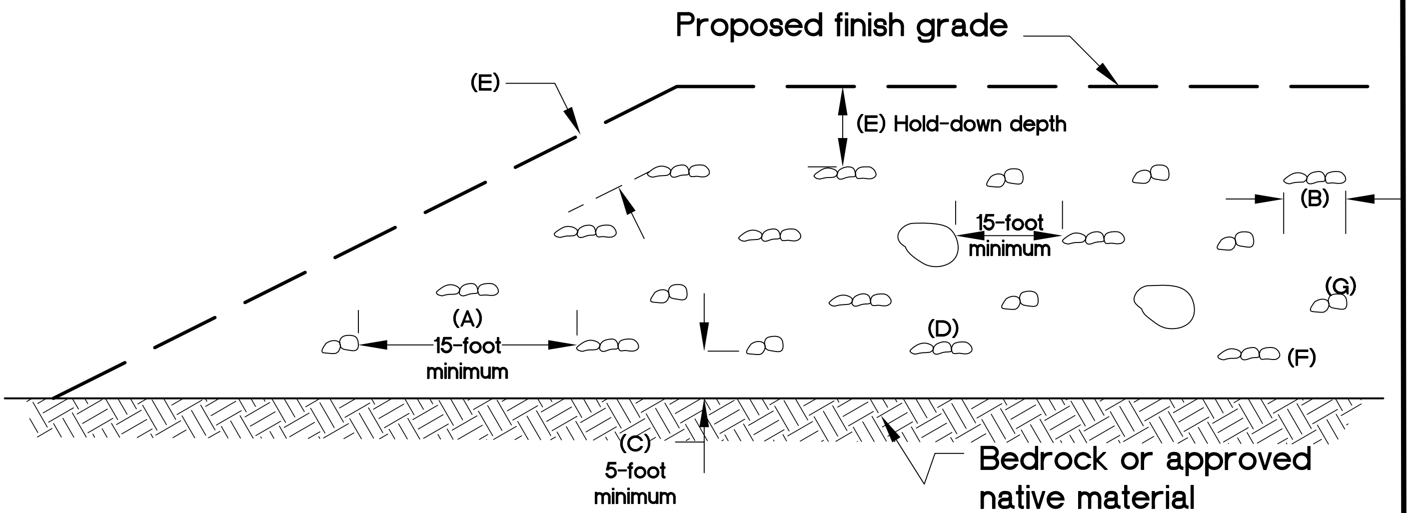
CUT LOT OR MATERIAL-TYPE TRANSITION



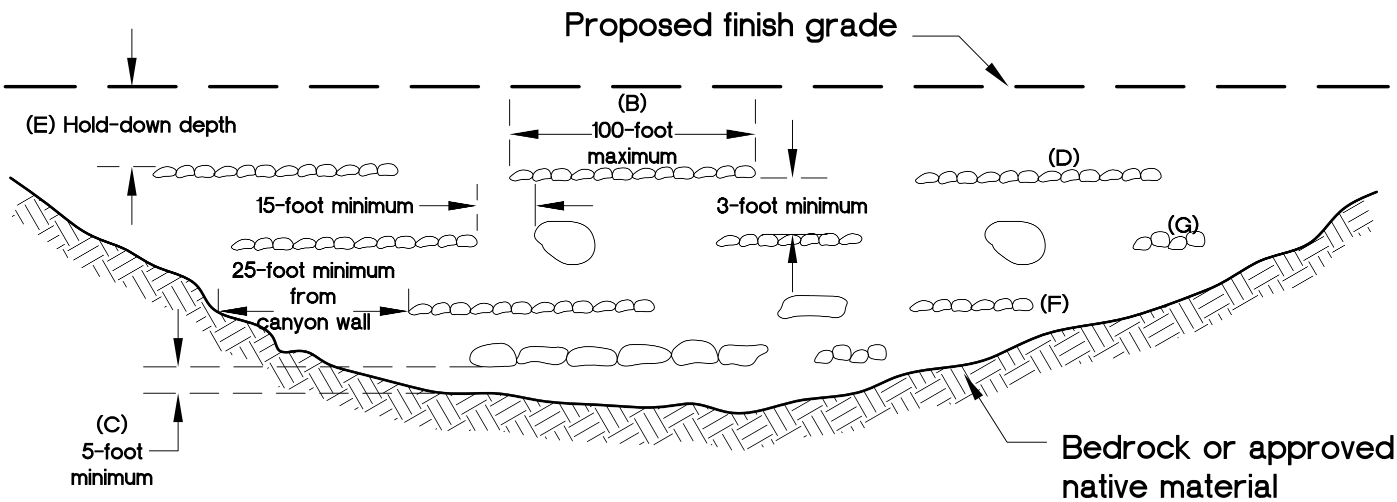
CUT-FILL LOT (DAYLIGHT TRANSITION)

GeoSoils, Inc.	TRANSITION LOT DETAILS	Plate F-12

VIEW NORMAL TO SLOPE FACE



VIEW PARALLEL TO SLOPE FACE



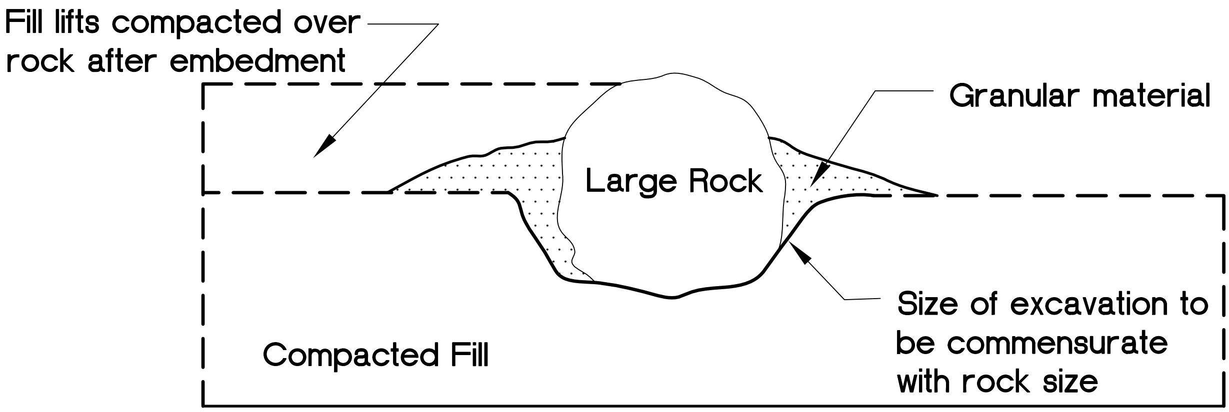
NOTES:

A. One equipment width or a minimum of 15 feet between rows (or windrows).
B. Height and width may vary depending on rock slze and type of equipment. Length of windrow shall be no greater than 100 feet.
C. If approved by the geotechnical consultant, windrows may be placed direclty on competent material or bedrock, provided adequate space is available for compaction.
D. Orientation of windrows may vary but should be as recommended by the geotechnical engineer and/or engineering geologist. Staggering of windrows is not necessary unless recommended.
E. Clear area for utility trenches, foundations, and swimming pools; Hold-down depth as specified in text of report, subject to governing agency approval.
F. All fill over and around rock windrow shall be compacted to at least 90 percent relative compaction or as recommended.
G. After fill between windrows is placed and compacted, with the lift of fill covering windrow, windrow should be proof rolled with a D-9 dozer or equivalent.

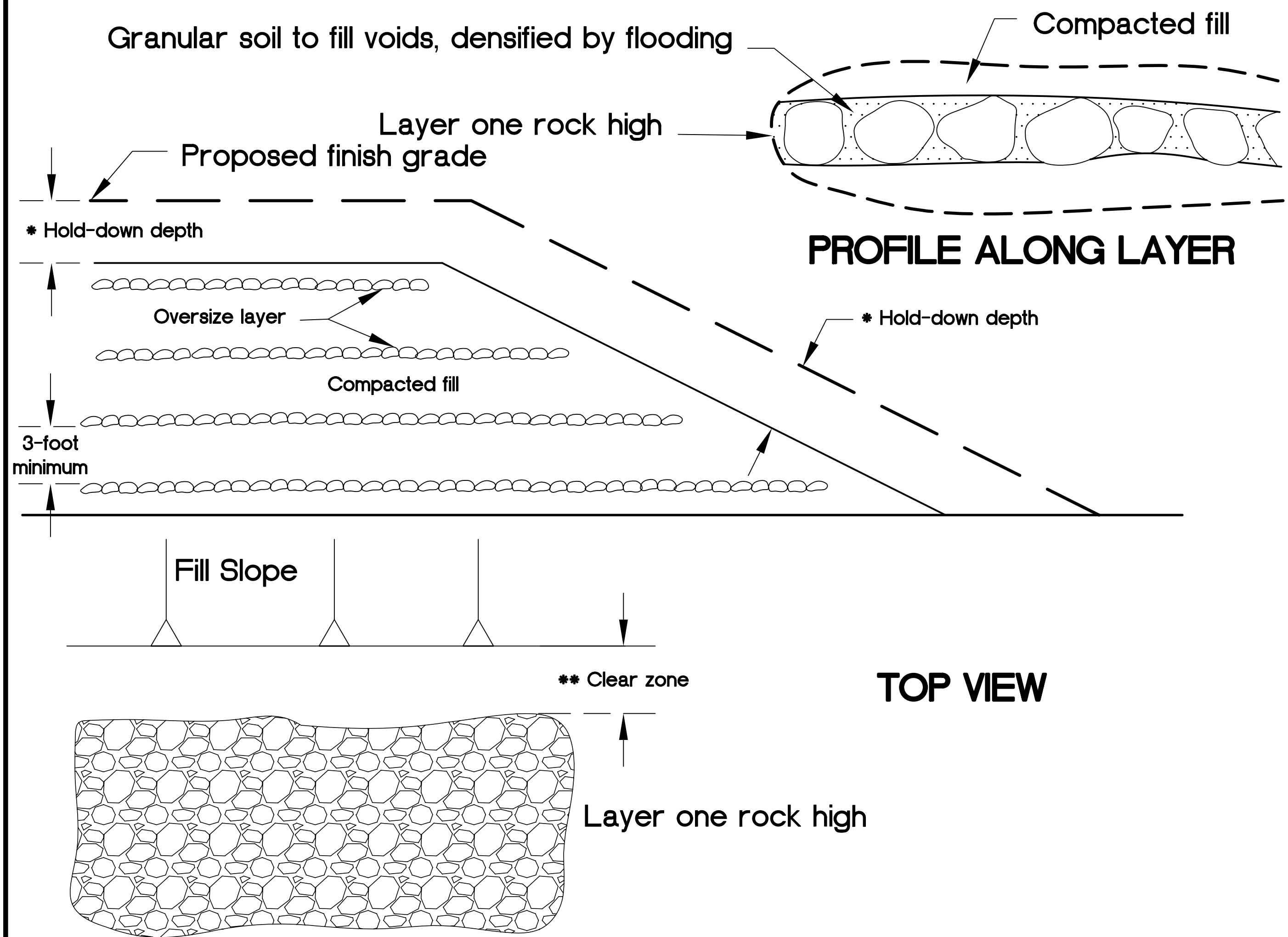
VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE
ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED

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

ROCK DISPOSAL PITS



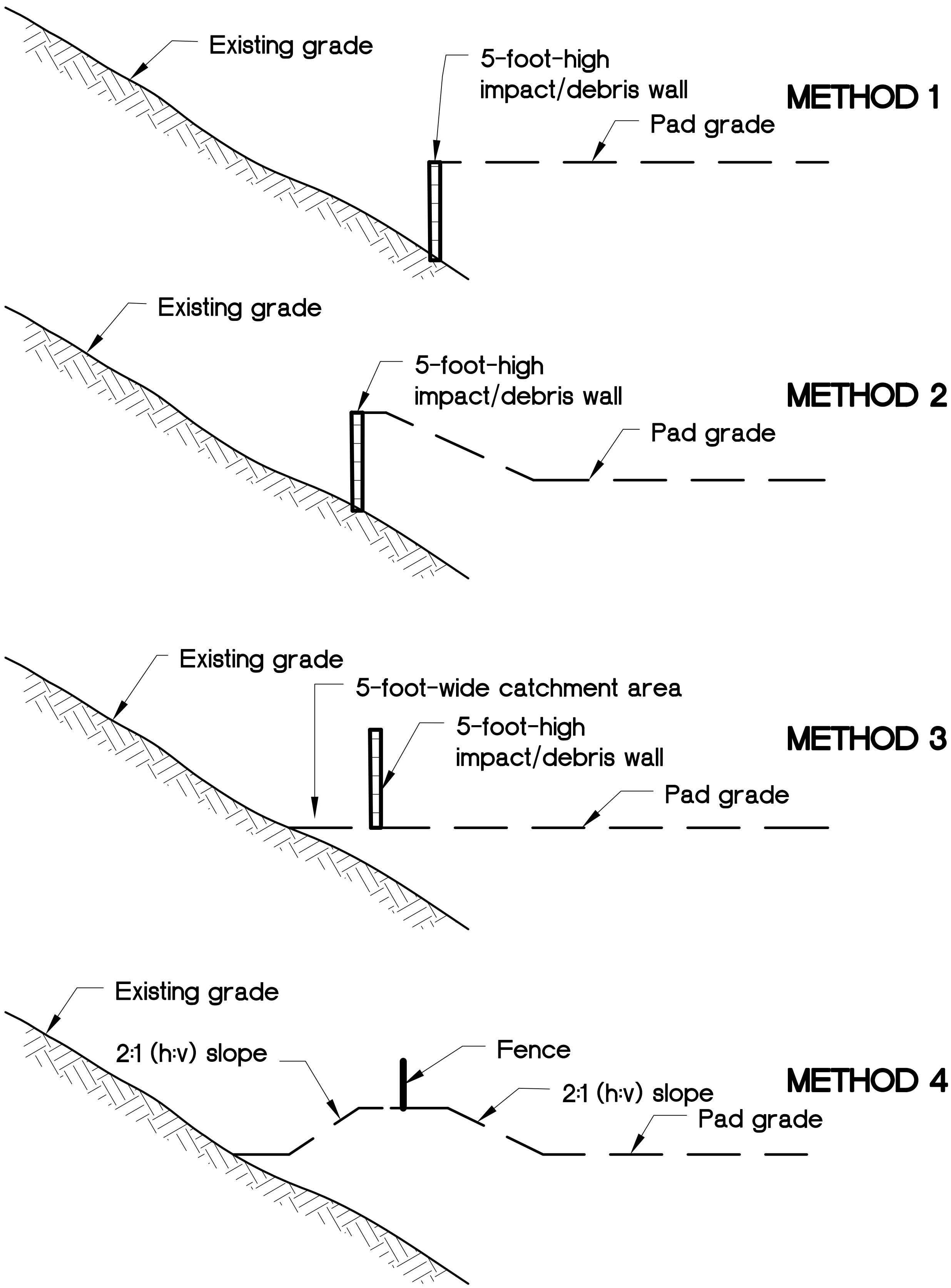
ROCK DISPOSAL LAYERS



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

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

GeoSoils, Inc.	ROCK DISPOSAL DETAIL	Plate F-14

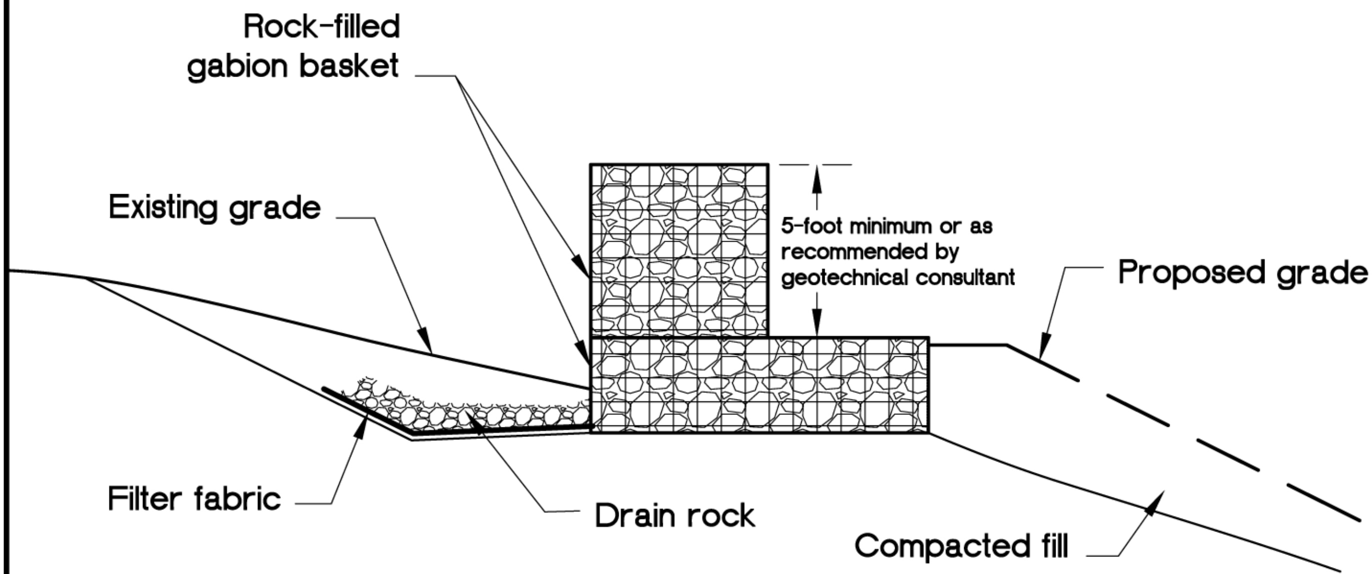


NOT TO SCALE

GeoSoils, Inc.

DEBRIS DEVICE CONTROL METHODS DETAIL

Plate F-15



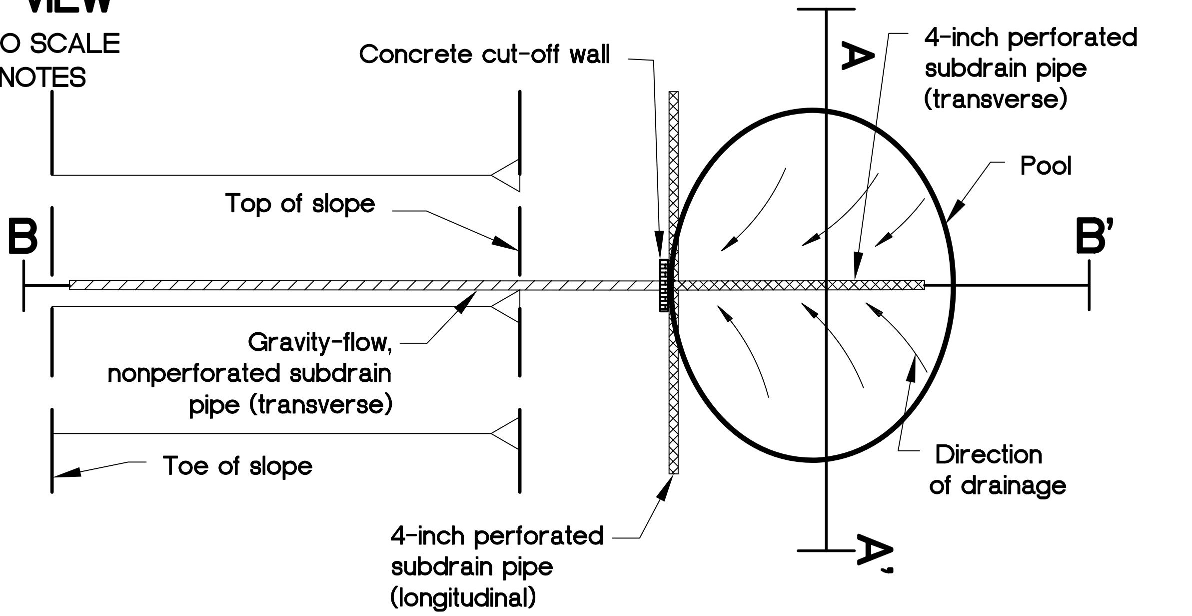
Gabion impact or diversion wall should be constructed at the base of the ascending slope subject to rock fall. Walls need to be constructed with high segments that sustain impact and mitigate potential for overtopping, and low segment that provides channelization of sediments and debris to desired depositional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

From GSA, 1987

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

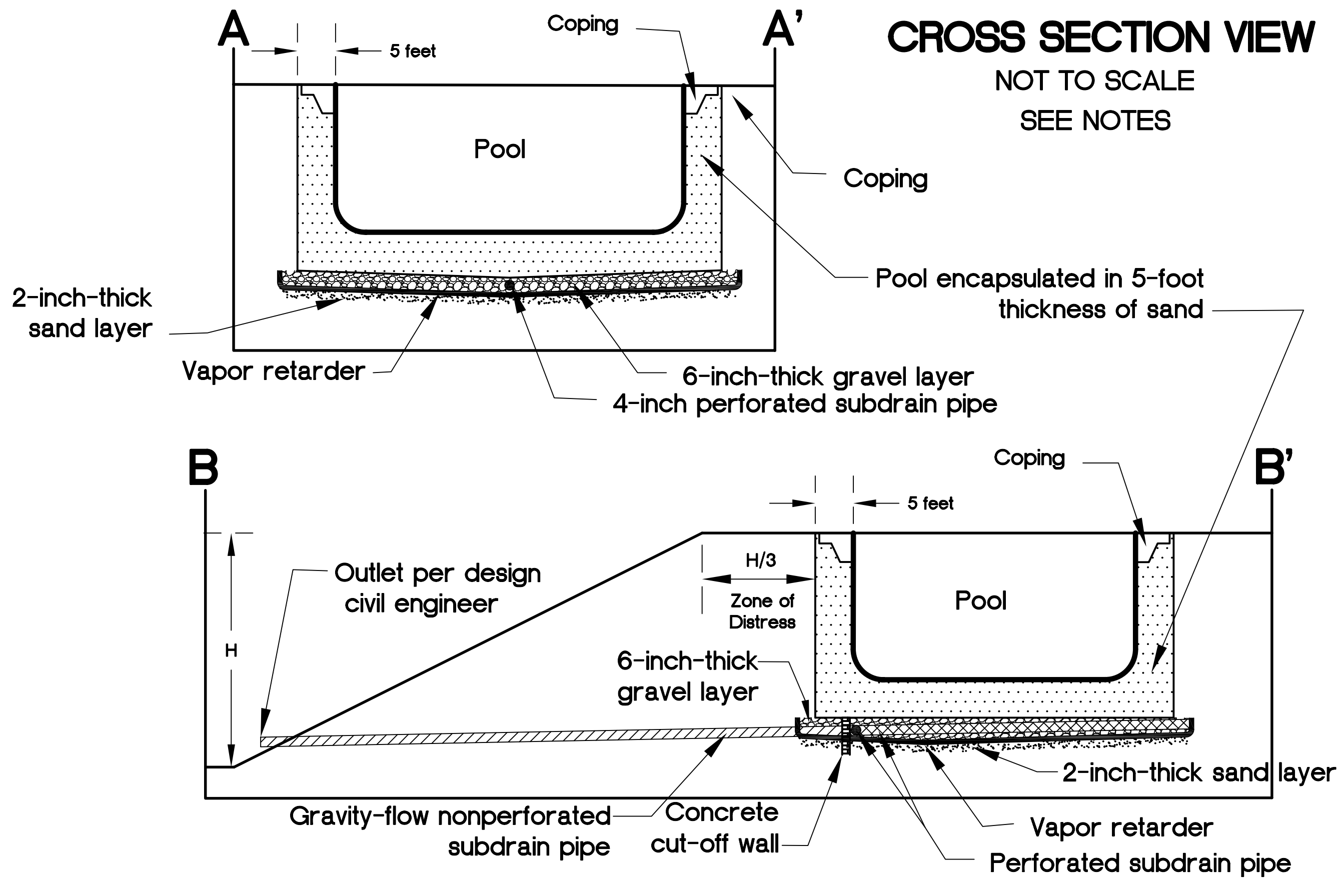
MAP VIEW

NOT TO SCALE
SEE NOTES



CROSS SECTION VIEW

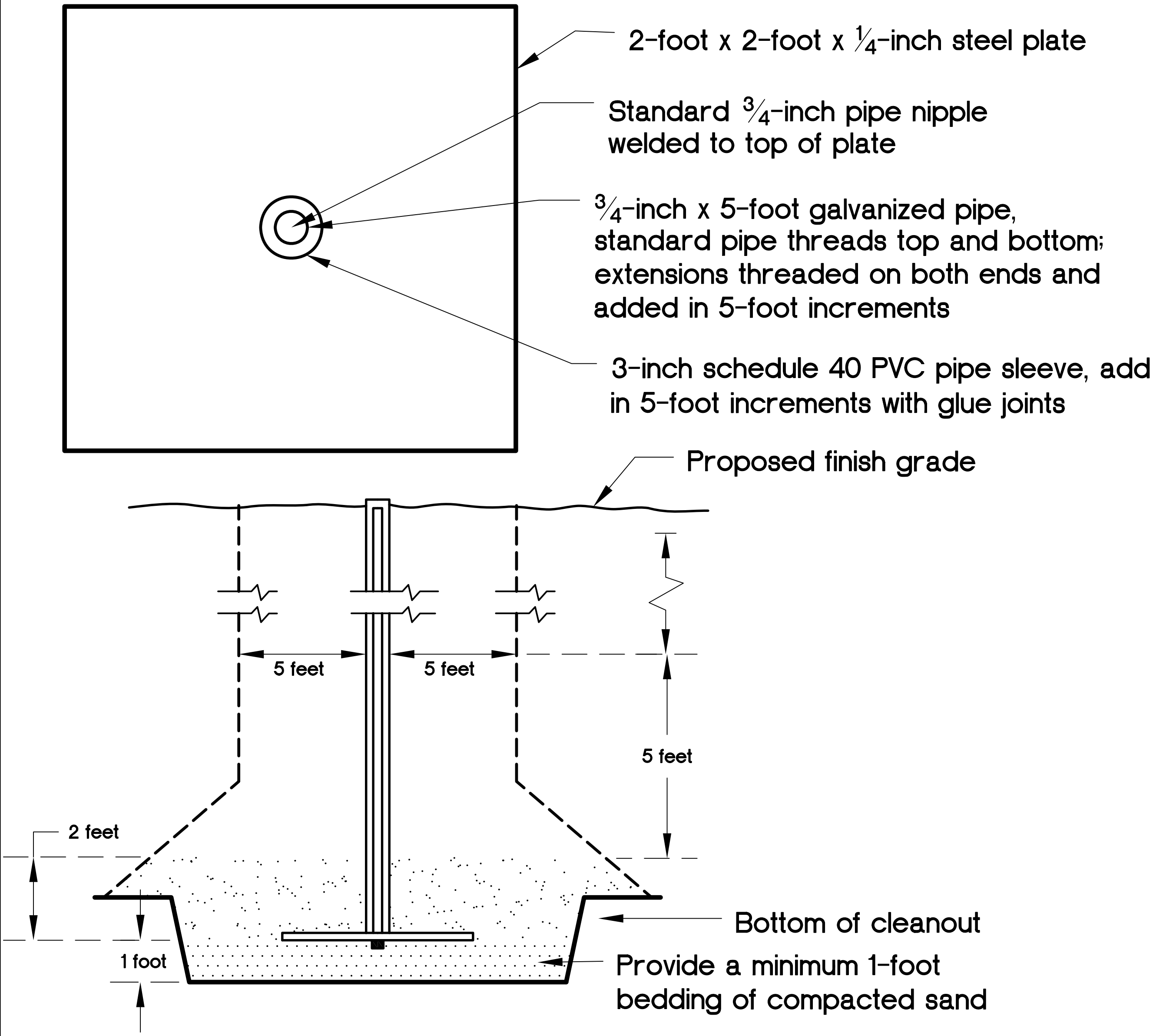
NOT TO SCALE
SEE NOTES



NOTES:

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

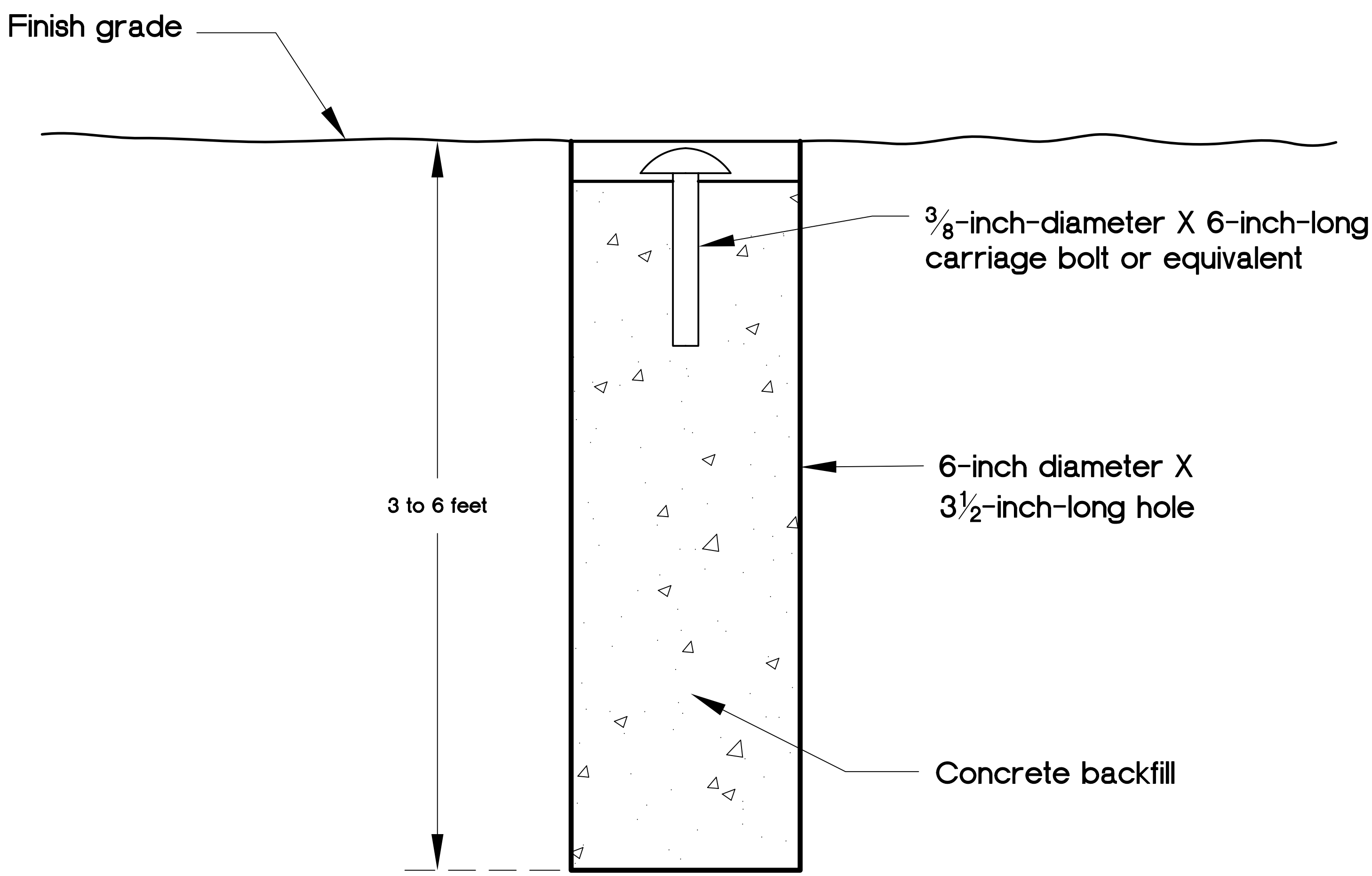
GeoSoils, Inc.	TYPICAL POOL/SPA DETAIL	Plate F-17



NOTES:

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

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

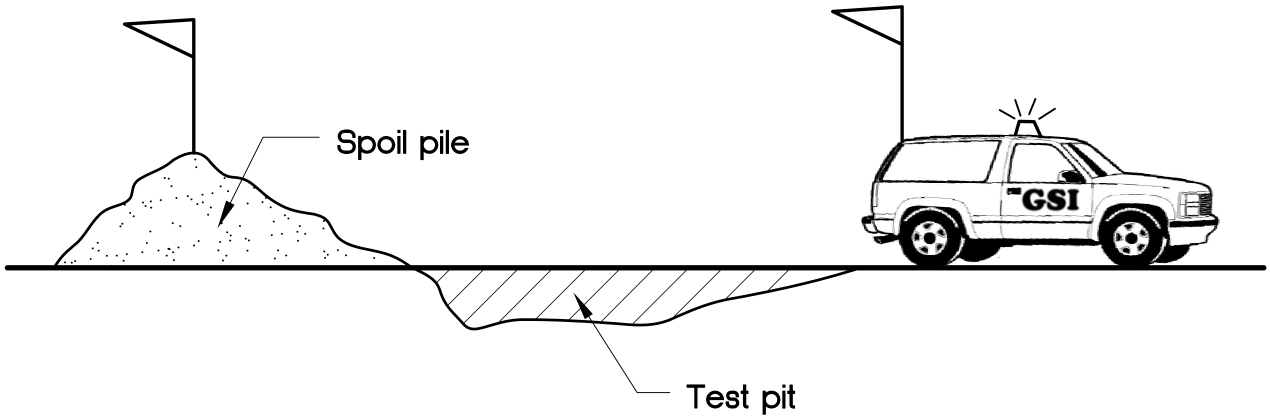


GeoSoils, Inc.

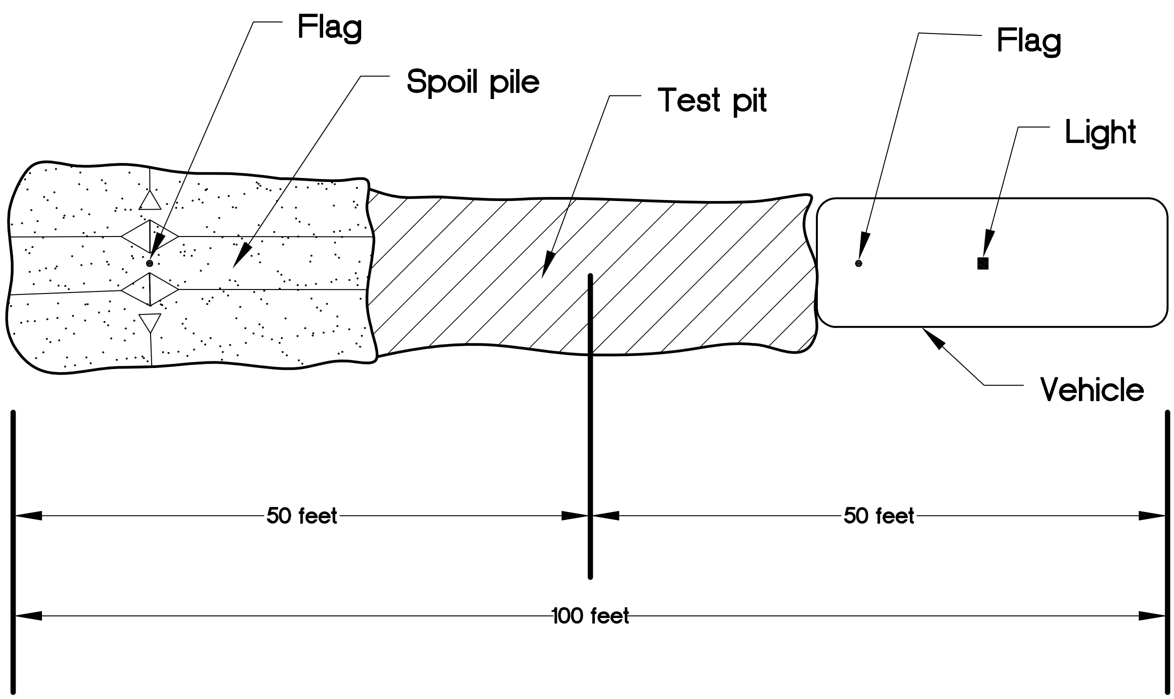
TYPICAL SURFACE SETTLEMENT MONUMENT

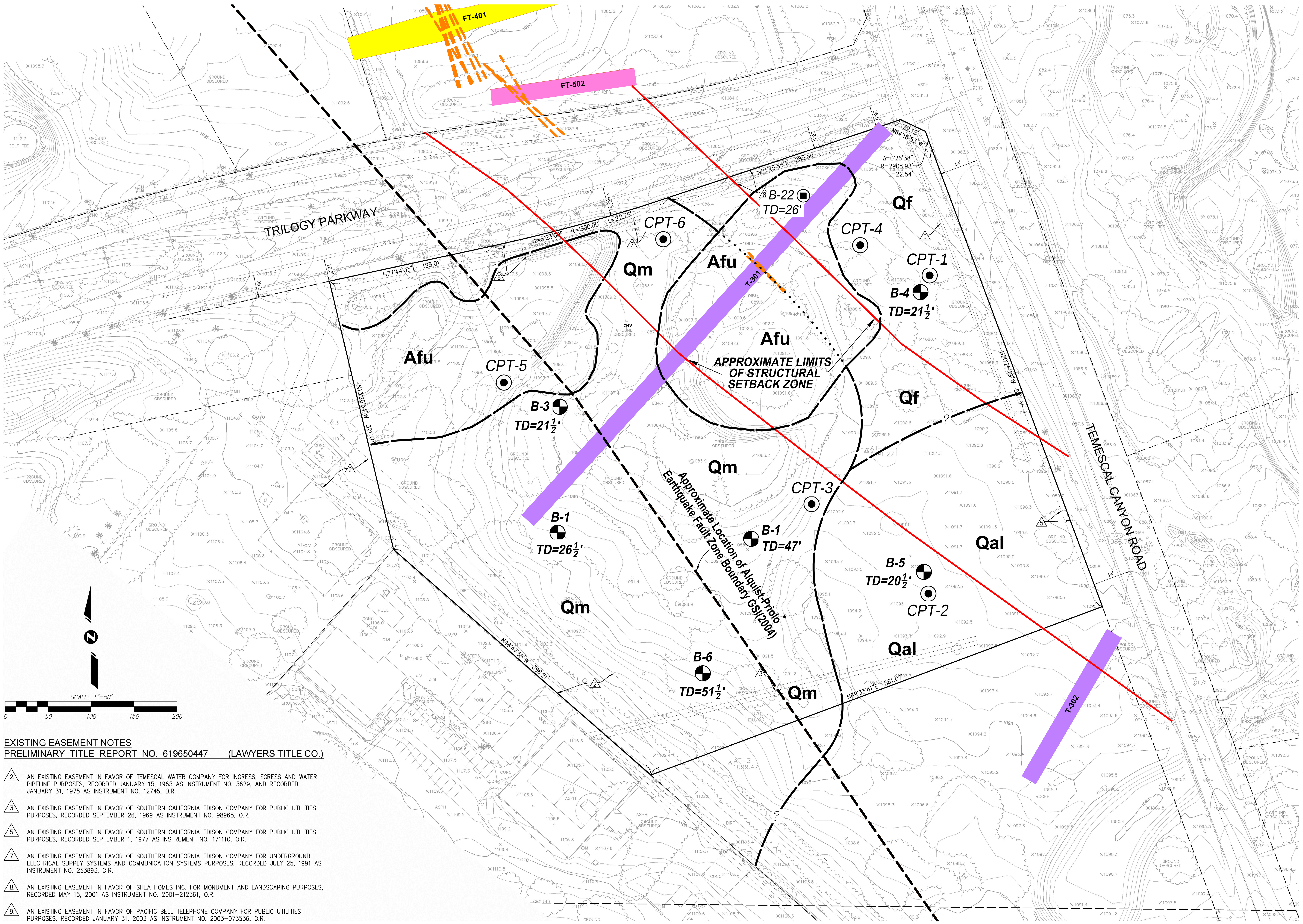
Plate F-19

SIDE VIEW



TOP VIEW





EXISTING EASEMENT NOTES
PRELIMINARY TITLE REPORT NO. 619650447 (LAWYERS TITLE CO.)

2. AN EXISTING EASEMENT IN FAVOR OF TEMESCAL WATER COMPANY FOR INGRESS, EGRESS AND WATER PIPELINE PURPOSES, RECORDED JANUARY 15, 1965 AS INSTRUMENT NO. 5629, AND RECORDED JANUARY 31, 1975 AS INSTRUMENT NO. 12745, O.R.

3. AN EXISTING EASEMENT IN FAVOR OF SOUTHERN CALIFORNIA EDISON COMPANY FOR PUBLIC UTILITIES PURPOSES, RECORDED SEPTEMBER 26, 1969 AS INSTRUMENT NO. 98965, O.R.

5. AN EXISTING EASEMENT IN FAVOR OF SOUTHERN CALIFORNIA EDISON COMPANY FOR PUBLIC UTILITIES PURPOSES, RECORDED SEPTEMBER 1, 1977 AS INSTRUMENT NO. 171110, O.R.

7. AN EXISTING EASEMENT IN FAVOR OF SOUTHERN CALIFORNIA EDISON COMPANY FOR UNDERGROUND ELECTRICAL SUPPLY SYSTEMS AND COMMUNICATION SYSTEMS PURPOSES, RECORDED JULY 25, 1991 AS INSTRUMENT NO. 253893, O.R.

8. AN EXISTING EASEMENT IN FAVOR OF SHEA HOMES INC. FOR MONUMENT AND LANDSCAPING PURPOSES, RECORDED MAY 15, 2001 AS INSTRUMENT NO. 2001-212361, O.R.

9. AN EXISTING EASEMENT IN FAVOR OF PACIFIC BELL TELEPHONE COMPANY FOR PUBLIC UTILITIES PURPOSES, RECORDED JANUARY 31, 2003 AS INSTRUMENT NO. 2003-073536, O.R.

GSI LEGEND

Afu	ARTIFICIAL FILL – UNDOCUMENTED
Qal	QUATERNARY ALLUVIUM, YOUNGER
Qm	QUATERNARY MARSH DEPOSITS
Qf	QUATERNARY ALLUVIAL FAN DEPOSITS, OLDER
? • • •	APPROXIMATE LOCATION OF GEOLOGIC CONTACT, DOTTED WHERE BURIED, QUERIED WHERE UNCERTAIN
	APPROXIMATE LOCATION OF PLOTTED FAULT
	APPROXIMATE LOCATION OF SETBACK LIMIT FOR HABITABLE STRUCTURES
FT-502	APPROXIMATE LOCATION OF FAULT–LOCATING TRENCH (GSI, 2007B)
FT-401	APPROXIMATE LOCATION OF FAULT–LOCATING TRENCH (GSI, 2007C)
T-302	APPROXIMATE LOCATION OF FAULT–LOCATING TRENCH (GSI, 1999A)
B-6 TD=51½'	APPROXIMATE LOCATION OF EXPLORATORY BORING WITH TOTAL DEPTH IN FEET (GSI, THIS STUDY)
CPT-6	APPROXIMATE LOCATION OF CONE PENETRATION TESTS (GSI, THIS STUDY)
B-26 TD=35'	APPROXIMATE LOCATION OF EXPLORATORY BORING WITH TOTAL DEPTH IN FEET (GSI, 1987)

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.

N

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

BASE MAP FROM:

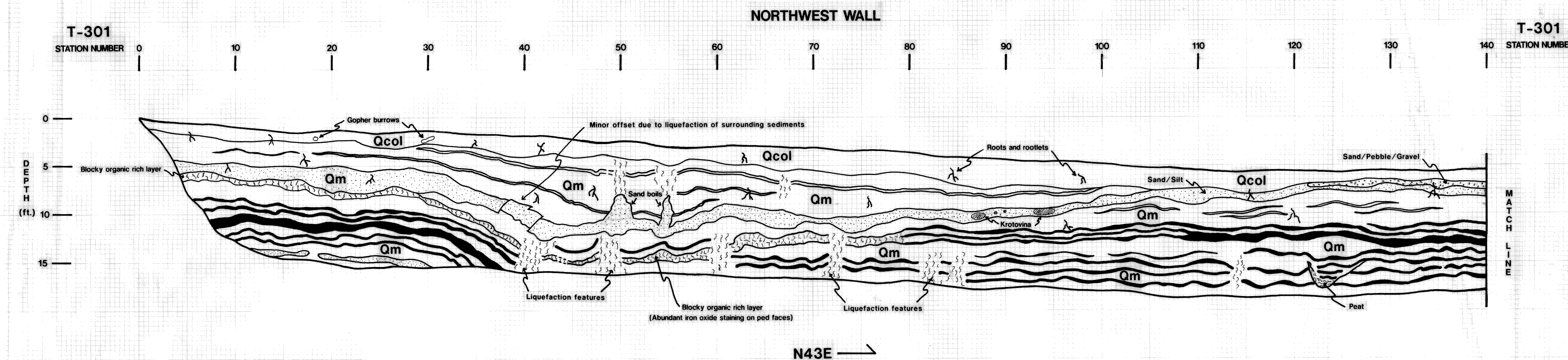
K&A Engineering, Inc.
ENGINEERING
LAND PLANNING
SURVEYING
357 N. SHERIDAN STREET
SUITE 117
CORONA, CALIFORNIA 92880
TEL (951) 279-1800
FAX (951) 279-4380

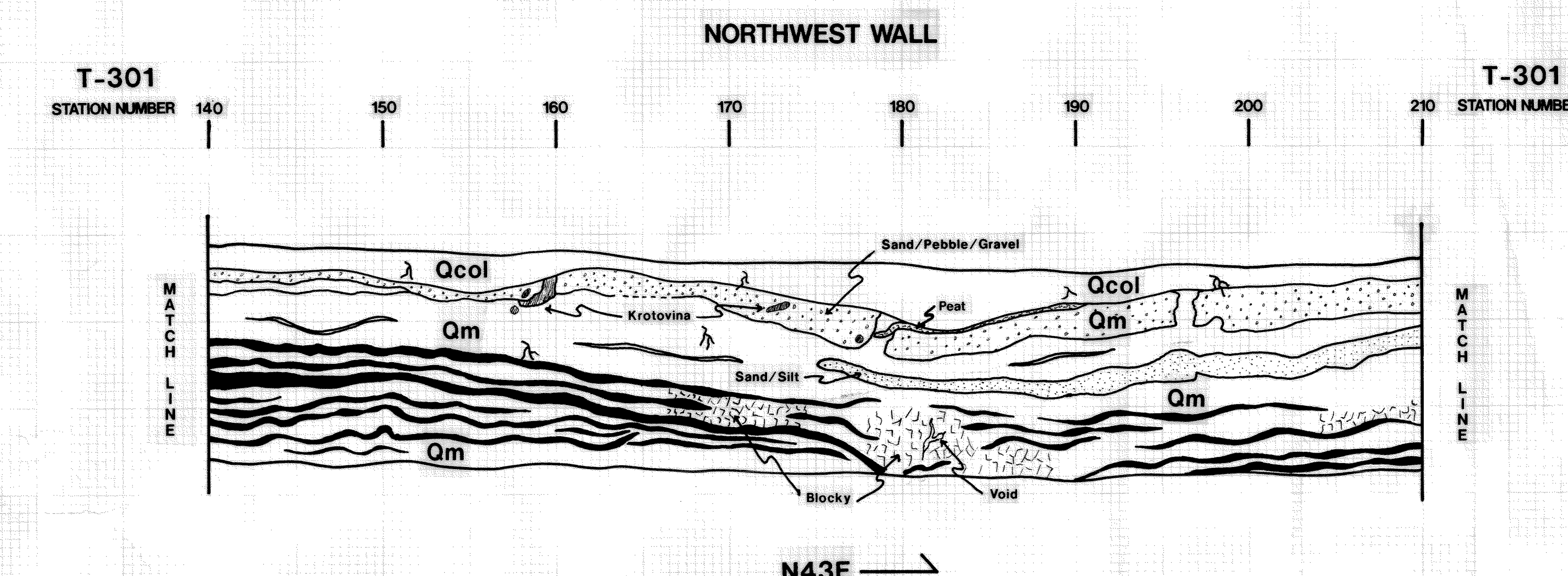
GeoSoils, Inc.

GEOTECHNICAL MAP

Plate 1

W.O. 7731-A-SC	DATE: 03/20	SCALE: 1" = 50'





LEGEND

T-301

Afu- Artificial Fill Undocumented: **Silty Sand,** light yellowish brown 10YR 6/4, dry to damp, loose, abundant asphalt and concrete spoils, minor organics branches/lumber etc.

Qcol- Quaternary Colluvium: **Silt and Silty Sand,** dark gray 10YR 4/1, damp, loose to medium dense, abundant fine grained material, abundant roots and rootlets, abundant bioturbation, worm fecal pellets, and burrows.

Qm- Quaternary Marsh Deposits: **Silt, Silty Sand, Clayey Silt and Organic Detritus,** light gray 10YR 7/1 to brownish yellow 10YR 6/6 (due to iron oxide staining in silty sand lenses) dark gray 10YR 4/1 to dark grayish brown 10YR 4/2 (in silty layers) very dark grayish brown 10YR 3/2 to black 10YR 2/1(in organic rich layers). Damp to moist, medium dense, organic rich layers become blocky in areas with abundant iron oxide staining on ped faces, few voids noted in blocky areas, abundant roots and rootlets, bedding planes are convoluted with relief on the order of 4- to 6-inches (greater in some areas), contains many liquefaction features and structures, abundant peat and organically enriched layers, abundant krotovina and root-cast structures in silty sand layers, silty sand layers represent influx of fluvial derived sediments, cross-bedding noted in areas.

Qf- Quaternary Fanglomerate Deposits: **Silty Sand and Silt with Pebble to Cobble,** very pale brown 10YR 7/4, to strong brown 7.5YR 5/8, in moderately developed silty soil unit stratigraphically on top of older fanglomerate deposits, dry to damp, dense to very dense. Fanglomerate facies contains un-oriented pebble to cobble sized clasts, consisting of highly weathered granitic, volcanic and Bedford Canyon Formation detritus, clasts are angular to sub-rounded, ranging in size from ¼-inch to over 8-inches in diameter. Abundant root cast structures, becomes blocky and fractured in areas adjacent to faulting, however, fractures are not offset. Soil horizon grades from strong brown to dark brown 10YR 3/3 and becomes clayey to the northeast end of the trench.

GSI GeoSoils, Inc.

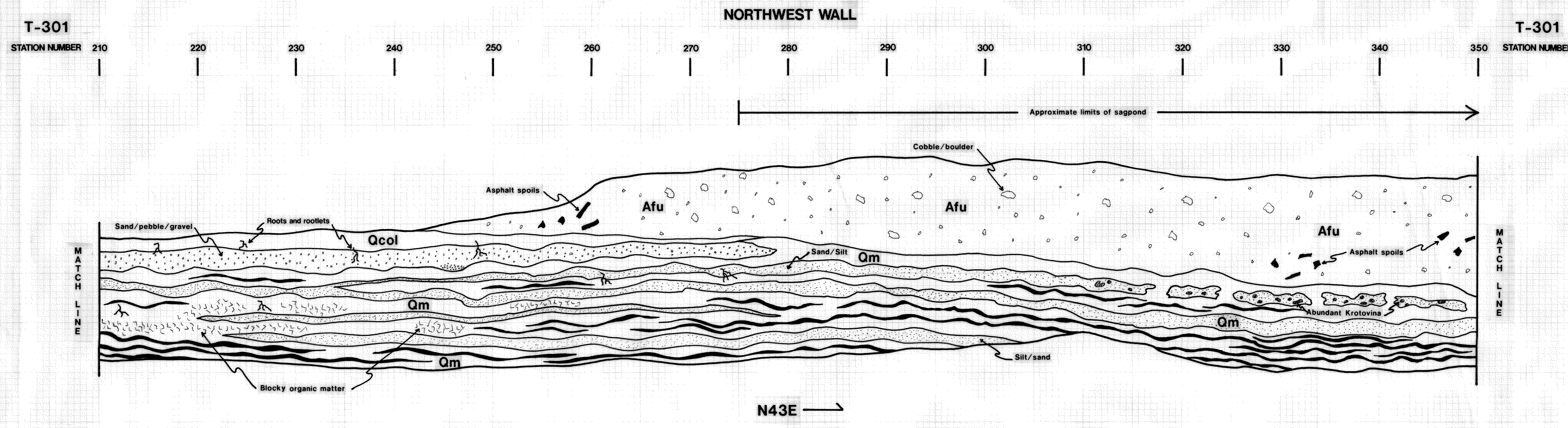
LOS ANGELES CO.
RIVERSIDE CO.
ORANGE CO.
SAN DIEGO CO.

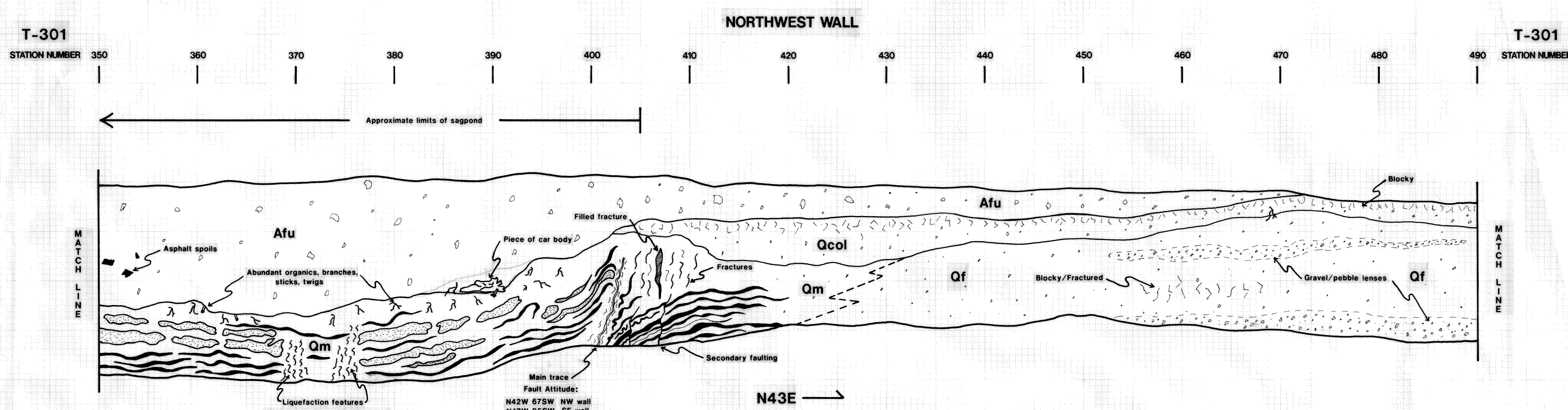
TRENCH LOG T-301

Plate 2

W.O. 2579-A3-SC DATE 4/99 SCALE 1"=5'

2629-A-SC





(See Plate 2 for legend)

GSI GeoSoils, Inc.

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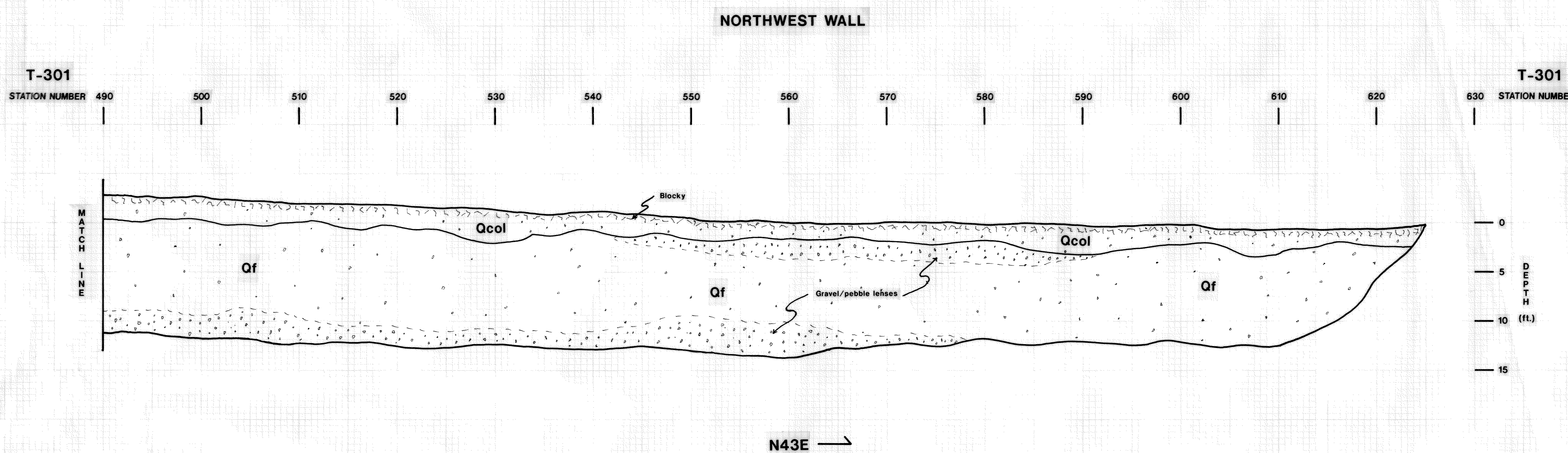
RIVERSIDE CO.
ORANGE CO.
SAN DIEGO CO.

TRENCH LOG T-301

Plate 3

W.O. 2579-A3-SC	DATE 4/99	SCALE 1"=5'

2629-A-SC



(See Plate 2 for legend)

GeoSoils, Inc.

LOS ANGELES CO.
RIVERSIDE CO.
ORANGE CO.
SAN DIEGO CO.

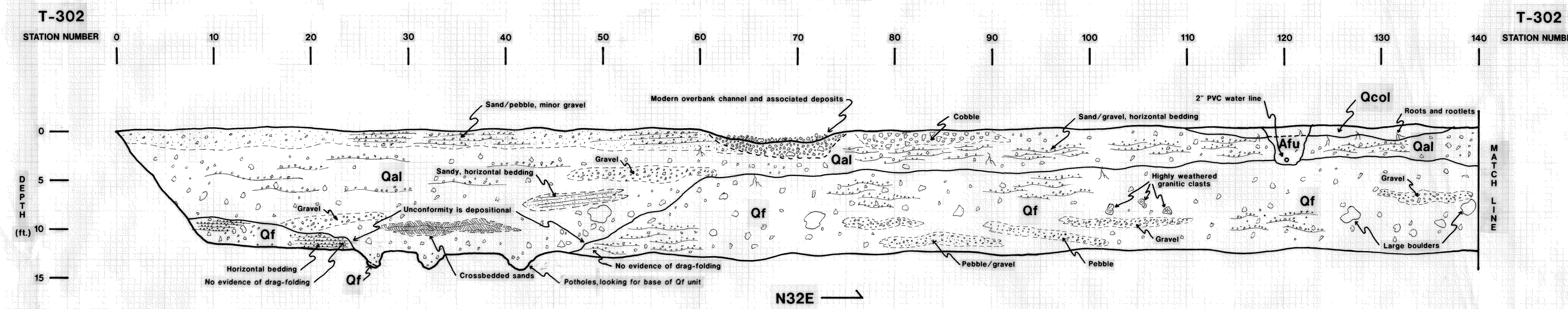
TRENCH LOG T-301

Plate 4

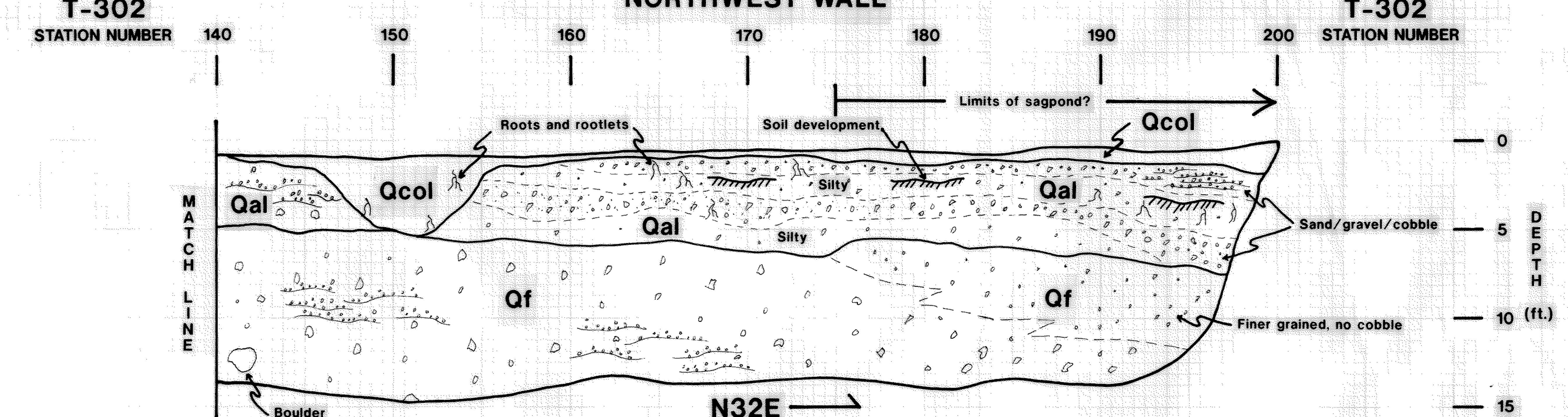
W.O. 2579-A3-SC DATE 4/99 SCALE 1"=5'

2629-A-SC

NORTHWEST WALL



NORTHWEST WALL



LEGEND

T-302

Afu- <u>Artificial Fill-Undocumented:</u> **Silty Sand,** brown 10YR 5/3, dry, loose, trench backfill associated with 2 inch PVC water line.

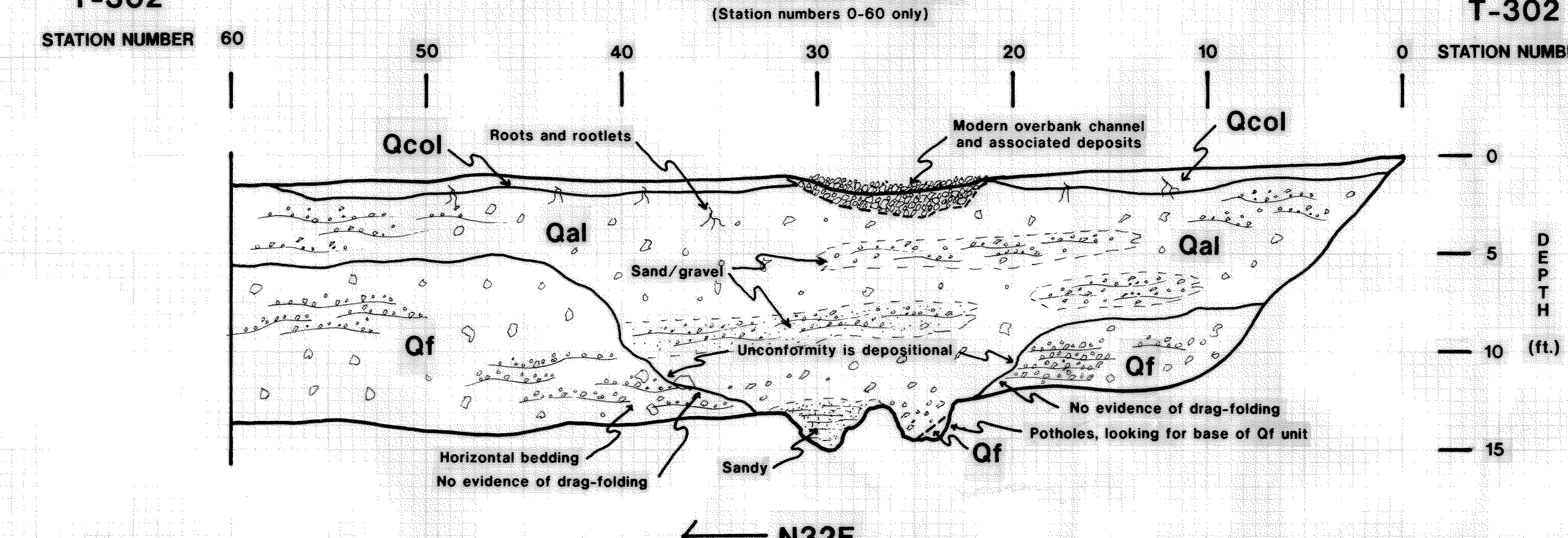
Qcol- <u>Quaternary Colluvium:</u> **Silty Sand and Silt,** grayish brown 10YR 5/2, dry loose to medium dense, minor pebble and gravel sized clasts, abundant roots and rootlets. Colluvium is discontinuous and truncated in areas by modern over-bank channel meanders/deposits.

Qal- <u>Quaternary Alluvium:</u> **Pebble, Gravel and Cobble with Silty Sand Matrix,** light gray 10YR 7/1, dry, loose to medium dense, represents deposition of fluvial stream over-bank and main channel deposits of relatively younger alluvium. Clasts are un-oriented angular to sub-rounded, and range in size from ¼-inch to 8-inches in diameter, minor boulder sized clasts to 14-inches in diameter. Moderately developed soil horizons present at northeastern end of trench in probable sag pond area.

Qf- <u>Quaternary Fanglomerate Deposits:</u> **Pebble, Gravel, Cobble with Silty Sand Matrix,** light yellowish brown 10YR 6/4 to brownish yellow 10YR 6/6, dry, medium dense to dense, abundant pebble and gravel lenses, horizontally bedded. Unit grades to finer grained silty sand material lacking cobble facies at northeast end of trench, abundant rootlets, minor boulders in excess of 24-inches in diameter.

SOUTHEAST WALL

(Station numbers 0-60 only)



GeoSoils, Inc.

LOS ANGELES CO.
RIVERSIDE CO.
ORANGE CO.
SAN DIEGO CO.

TRENCH LOG T-302

Plate 5

W.O. 2579-A3-SC DATE 4/99 SCALE 1"=5'

2629-A-SC