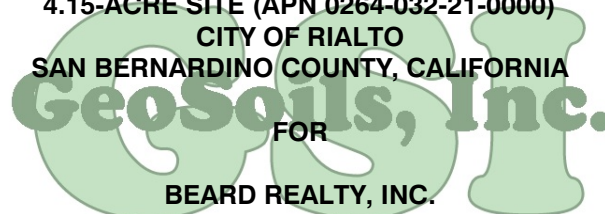


**PRELIMINARY GEOTECHNICAL
AND FAULT FINDING INVESTIGATION
4.15-ACRE SITE (APN 0264-032-21-0000)
CITY OF RIALTO
SAN BERNARDINO COUNTY, CALIFORNIA**



FOR

**BEARD REALTY, INC.
285 W. RIALTO AVENUE
RIALTO, CALIFORNIA 92376**

W.O. 8377-A-SC

AUGUST 18, 2022



Geotechnical • Geologic • Coastal • Environmental

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August 18, 2022

W.O. 8377-A-SC

Beard Realty, Inc.

285 W. Rialto Avenue
Rialto, California 92376

Attention: Mr. Scott Beard

Subject: Preliminary Geotechnical and Fault Finding Investigation, 4.15-Acre Site
(APN 0264-032-21-0000), City of Rialto, San Bernardino County, California

Dear Mr. Beard:

In accordance with your request and authorization, GeoSoils, Inc. (GSI) is presenting the results of our preliminary geotechnical and fault finding investigation of the 4.15-acre site in the City of Rialto, San Bernardino County, California. The primary purpose of our study was to evaluate the potential for active faulting and slope instability onsite, the nature and extent of the earth materials underlying the property, site seismicity, liquefaction potential, settlement potential, groundwater conditions, etc., and their effects on the proposed residential development, from a geotechnical viewpoint, and to provide code-compliant conclusions and recommendations for future residential development of the property.

EXECUTIVE SUMMARY

Based on our recent field and fault finding exploration, geologic, and geotechnical engineering and slope stability analysis, and review of previous geotechnical studies for the adjoining and nearby properties (see Appendix A), residential development within the proposed project site appears feasible from a geotechnical viewpoint, provided the recommendations presented in the text of this report are appropriately implemented. The primary developmental considerations are summarized below:

- Based on our review, the site is located within an Alquist-Priolo Earthquake Fault Zone (APEFZ), and active strands of the San Bernardino segment of the San Jacinto fault zone have been previously identified on the adjoining property. The adjoining property was previously geotechnically investigated by GSI (2018, 207b, and 2006), and Gary S. Rasmussen & Associates, Inc. (GSRAI; 1994b, 1993, 1982a, 1982b), as referenced in Appendix A. These early studies by GSRAI included detailed fault finding studies, and two (2) of the fault finding trenches advanced by GSRAI (1982a, Trench No. 3 and No. 4) are mapped on the southern portion of the property.

- Our current field study included the advancement of one (1) fault finding trench, approximately 80 feet in length, extending across the property. Based on our current geological assessments, GSI concludes that sufficiently active Holocene faults likely do not exist onsite.
- The potential for surface fault rupture, seismic slope instability, and subsidence to affect the site is considered low.
- Removal of all undocumented artificial fill, surficial colluvium/topsoil, and near-surface weathered young alluvial fan deposits will be necessary prior to fill placement, in areas proposed for settlement-sensitive improvements. For preliminary planning purposes, these depths are estimated to be on the order of 5 to 7 feet across the majority of the site, with localized deeper removals on the order of 15 to 24 feet, or more where fault trenching was performed onsite. Variations from these thicknesses should be anticipated. Actual depths of removals will be evaluated in the field during grading by the geotechnical consultant.
- Due to the nature of the earth materials encountered onsite, localized oversized rock materials (i.e., greater than 12 inches in diameter) are anticipated be generated during rough grading, which will require specialized placement techniques. Per the San Bernardino County guidelines, oversized materials greater than 12 inches, should not be placed within 10 feet of finish pad grades. Oversized rock materials should be placed in accordance with the Rock Placement Guidelines section of this report.
- Due to the potential for low cohesion materials and surficial instability, the outer 15 feet of cut slopes (as measured horizontally from the slope face), if constructed, may need to be stabilized and replaced with compacted fill. Evaluation of cut slopes during grading would be necessary in order to identify any areas of non-cohesive sands/gravels. General guidelines for slope construction are presented in Appendix E.
- Adverse geologic features that would preclude project feasibility (e.g., landslides, active faulting, shallow groundwater, etc.) were not encountered.
- The recommendations presented in this report should be incorporated into the design and construction considerations of the project.

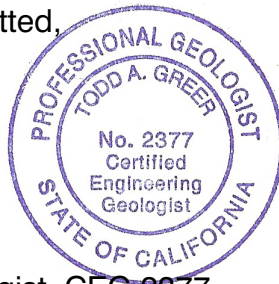
The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to contact our office.

Respectfully submitted,

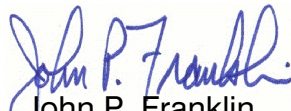
GeoSoils, Inc.



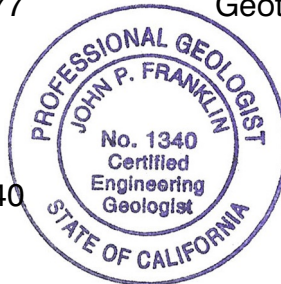
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TAG/JPF/SJC/sh

Distribution: (1) Addressee (PDF by email)

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ATTACHMENTS:

Appendix A - References.	Rear of Text
Appendix B - Seismicity Data	Rear of Text
Appendix C - Laboratory Test Results	Rear of Text
Appendix D - Slope Stability Analysis.	Rear of Text
Appendix E - - General Earthwork, Grading Guidelines, and Preliminary Criteria.	Rear of Text
Plate 1 - Geotechnical Map.	Rear of Text in Folder
Plate 2 - Fault Trench Log.	Rear of Text in Folder

**PRELIMINARY GEOTECHNICAL
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4.15-ACRE SITE (APN 0264-032-21-0000)
CITY OF RIALTO
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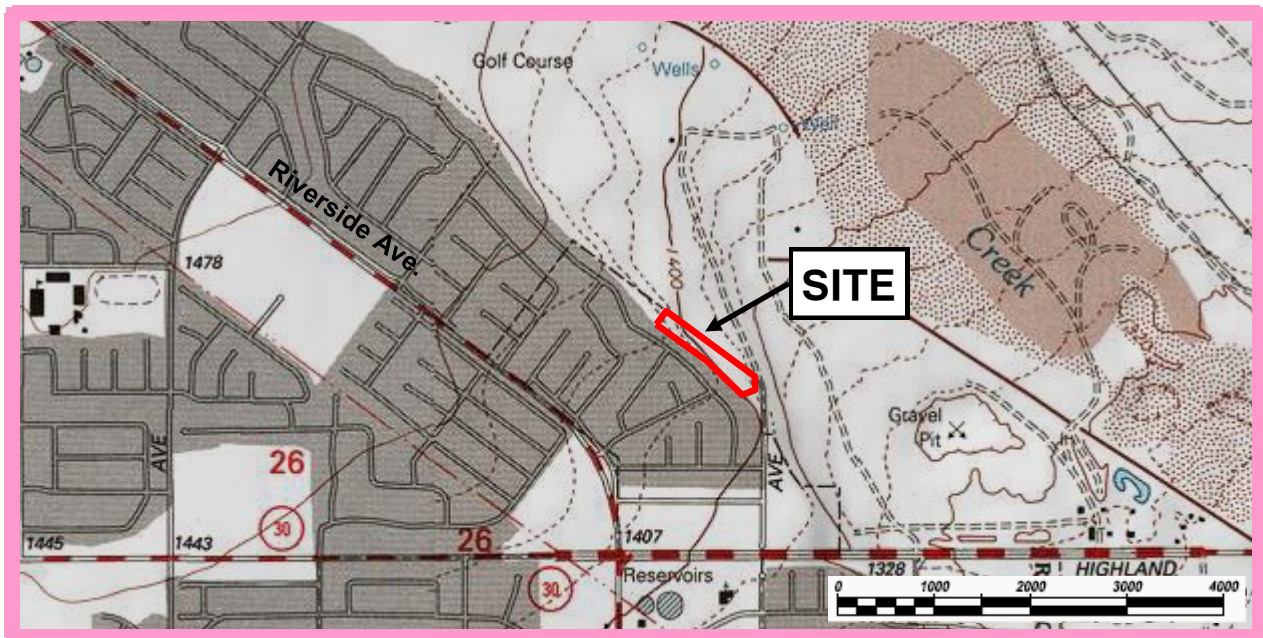
SCOPE OF SERVICES

The scope of our services has included the following:

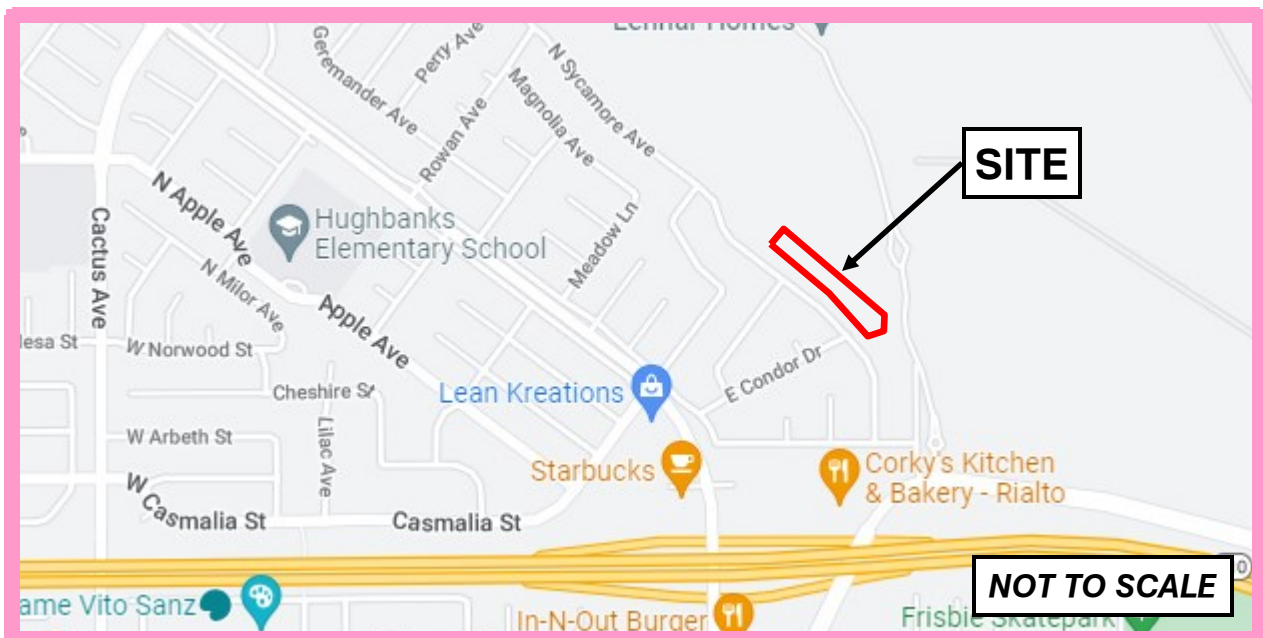
1. Review of available soils and geologic data for the site area, including previous geotechnical and fault finding reports for the adjoining site (Appendix A).
2. Geologic site reconnaissance mapping of significant geologic structures and mapping of surficial deposits.
3. Subsurface exploration consisting of the advancement of one (1) fault finding trench across the property for geologic logging, and geotechnical sampling.
4. Updated general areal seismicity evaluation (Appendix B).
5. Pertinent laboratory testing of representative soil samples collected during our subsurface exploration program. Testing included in-situ moisture and density, maximum density testing, expansion index, direct shear, sieve analysis, resistance value (R-value), and sulfate/corrosion testing of the materials encountered during our subsurface investigation. Results of our laboratory testing are provided in Appendix C.
6. Slope stability analysis for the existing western ascending slope area, designated by the County of San Bernardino as having a low to moderate potential for landslide susceptibility (Appendix D).
7. Appropriate engineering and geologic analyses of data collected, and preparation of this geotechnical report, including a log of the fault finding trench advanced onsite, laboratory test results, regional seismic data, slope stability analysis, general earthwork factors, recommendations for site grading, and preliminary foundation design recommendations.

SITE DESCRIPTION

The 4.15-acre property is located northwest of River Ranch Parkway, and north of East Highland Avenue, in the City of Rialto, San Bernardino County, California (see Figure 1 - Site Location Map). The site is bounded on the west- southwest by residential tract housing, on the southeast by an active gravel quarry (the Volcan Materials Company), and on the east- northeast by partially developed portions of the single-family



Base Map: TOPO! Copyright 2003 National Geographic, USGS San Bernardino North Quadrangle, California -- San Bernardino Co., 7.5 Minute, dated 1967.



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	W.O. 8377-A-SC
SITE LOCATION MAP Figure 1	

residential River Ranch project. In general, the site may be described as generally flat-lying with an ascending natural slope on the west margin. This slope is inclined at about 1.9:1 (h:v) to 2.22:1, or about 24 to 27½ degrees from the horizontal. Existing residential tract housing occupies the top, and offsite areas of the natural ascending slope to the west. The site is covered with native weeds and grasses with numerous olive trees (old grove area) located across the site. Drainage is generally directed to the east- southeast, via sheetflow. According to our review of Google Earth Pro imagery (GEP, 2022), elevations across the site range from a high of approximately 1,420 feet Mean Sea Level (MSL) at the northwest portion of the site to approximately 1,380 feet MSL on the southern portion of the project, near River Ranch Parkway. Therefore, overall relief across the property is on the order of approximately 40 feet.

PROPOSED DEVELOPMENT

Based on communication with the Client, GSI understands that the project site is proposed for single-family residential construction, along with construction of associated interior roadways, and underground utility improvements. The exact number of residential homes to be constructed is not known at this time, and no development plans are currently available for our review. We further assume that the buildings are proposed as one- or two-story structures, with slab-on-grade/continuous footings, using typical wood frame-type construction. Building loads are assumed to be typical for this type of relatively light construction. Sewage disposal is anticipated to be accommodated by tying into the regional system. It is also assumed that all cut and fill slopes will be proposed at 2:1 horizontal to vertical (h:v) gradients, or flatter, not exceeding about 10 to 15 feet in overall vertical height.

FIELD STUDIES

As indicated above, field studies conducted during our preliminary evaluation of the property for this investigation consisted of geologic reconnaissance mapping, and the excavation of approximately 80 feet of exploratory fault finding trench across the property, also for the evaluation of near-surface soil and geologic conditions. Field exploration was performed on June 14 and June 15, 2022. The exploratory fault finding trench was logged by an engineering geologist from our firm who also collected representative bulk samples for appropriate laboratory testing. The approximate locations of exploratory fault finding trenches advanced onsite are presented on Plate 1 (Geotechnical Map). The logs of exploratory fault finding trenches are presented on Plate 2 (Fault Trench Log).

REGIONAL AND SITE GEOLOGY

Regional Geologic Setting

The subject property is located on the northeastern fringe of a prominent natural geomorphic province in southwestern California known as the Peninsular Ranges. The Peninsular Ranges are characterized by steep, elongated ranges and valleys that trend northwestward, whereas, the Transverse Ranges trend east-west. The site area is situated within the Rialto-Colton Basin of the Perris block, an eroded mass of Cretaceous-age and older rocks. The Perris Block is defined by the San Jacinto and Elsinore-Chino faults, to the east and west, respectively, and by the Cucamonga fault, to the north. It is located on coalescing alluvial fans derived locally from alluvium and ancient debris flows originating from the San Gabriel Mountains (basement rock uplifted by bounding faults) to the north, and the Jurupa Mountains (an isolated knob of basement rock) to the south.

Local Geology

The site may be characterized as being primarily underlain by late Holocene (less than about 5,000 years ago)- and late- to middle-Holocene-age (about 8,000 to less than 5,000 years ago) young alluvial-fan deposits (Miller, Matti, and Carson, 2001), which emanate from Lytle Creek Wash. The alluvial deposits onsite are locally mantled by relatively thin layers of undocumented artificial fill, and colluvium/topsoil. As described by Miller, Matti, and Carson (2001), the young alluvial-fan deposits (Qyf₄ [late Holocene] and Qyf₃ [middle Holocene]) are generally coarse-grained alluvial fan deposits having slightly dissected surfaces, and can also be distinguished by terrace level. At depth, the young alluvial-fan deposits encountered during this study were generally observed to be varying hues of light grayish brown, light gray, and light yellowish brown silty sands, with gravel- to boulder-sized clasts (see Plate 2).

Site Geologic Units

The site geologic units encountered during our subsurface investigation and site geologic mapping included undocumented artificial fill, colluvium/topsoil, and Quaternary-age young axial fan deposits (Units 3 and 4). The distribution of mappable units are shown on Plate 1. Supplemental descriptions for the units are shown in the Fault Trench Log on Plate 2. The major geologic units are generally described below (from youngest to oldest):

Artificial Fill - Undocumented (Map Symbol - Afu)

Localized areas of undocumented fill were observed at several locations across the site. The undocumented fill is generally associated with soils stockpiles from trenching of the perimeter wall footings on the adjoining site, dumped rock/concrete debris, and the previous and recent fault finding investigations conducted. Although not encountered during our site investigation, artificial fill is also present at the location of the previous fault

finding trench by GSRAI (1982a). Where observed, the artificial fill consists of dark brown, to light grayish brown, silty sand with locally abundant, sub-rounded gravel- to boulder-sized clasts. Based on our review, the undocumented fill ranges in thickness from 1 to 4 feet or more, with localized areas on the order of approximately 15 to 25 feet or more where previous fault trenching was performed onsite. All undocumented fill is considered compressible and may settle appreciably under additional loading and improvements.

Colluvium/Topsoil and Surficial Fill (Unmapped)

Colluvium/topsoil and surficial fill materials were observed in our subsurface investigation locally mantling the young alluvial-fan deposits onsite. These surficial soils were generally observed to be about 1 to 2 feet in thickness, with localized areas up to 5 to 7 feet in thickness. As encountered onsite, the colluvium/topsoil is typically light to dark grayish brown, fine- to coarse-grained silty sands with localized roots and rootlets. The colluvium/topsoil and surficial fill materials were generally dry and loose to medium dense with depth. The colluvium is likely less than 500 years old, and perhaps much younger. Due to the potentially compressible nature of these surficial soils, they are considered unsuitable for support of structures or improvements in their existing state.

Quaternary Young Alluvial-Fan Deposits - Unit 4 (Map Symbol - Qyf₄)

Young alluvial fan deposits (Unit 4) were present throughout the lower portions of the development area. These deposits generally consist of light gray to very pale brown fine to coarse-grained sands and sandy gravels, with few to locally abundant well- to sub-rounded cobble to boulder sized clasts. These deposits were generally dry to slightly damp and loose to medium dense with depth. As indicated previously, these sediments are likely of late-Holocene age, and are in part contemporaneous and interfinger with the older, Unit 3 alluvial fan deposits. On a preliminary basis, these deposits generally have a very low expansion potential. Where unweathered, these deposits appear suitable for the support of settlement-sensitive improvements or engineered fill. The upper weathered portions (upper 5 feet) should be removed to competent/dense young alluvial-fan deposits in areas of proposed development, if not removed during planned excavation.

Quaternary Young Alluvial-Fan Deposits - Unit 3 (Map Symbol - Qyf₃)

Young alluvial-fan deposits (Unit 3) were present above (i.e., higher elevations of the property), and mapped near the toe of the natural ascending slope onsite. This mapped unit can be distinguished by terrace level and has been previously identified as the Rialto Bench. These deposits generally consist of light brownish gray to pale brown gravelly sands with silt, with few to locally abundant well-rounded gravel-sized clasts. These deposits were generally dry to slightly damp and loose to medium dense with depth. As also indicated previously, these sediments are likely of middle -Holocene age, and are in part contemporaneous and interfinger with the younger Unit 4 alluvial fan deposits. On a preliminary basis, these deposits generally have a very low expansion potential. Where

unweathered, these deposits appear suitable for the support of settlement-sensitive improvements or engineered fill.

FAULTING AND REGIONAL SEISMICITY

The seismicity and seismic history of the project site area have been previously described in GSI (2018 and 2008). The seismicity of the region has not changed appreciably since the issuance of those reports. Those studies also provided detailed information on previous fault finding investigations on the adjoining site, which identified active faulting associated with the San Bernardino segment of the San Jacinto fault zone. The active faulting previously identified on the adjoining site (GSRAI, 1994b and 1982a; GSI 2006) is somewhat isolated to the central portion of the adjoining site, extending to the west-northwest. As such, appropriate structural setbacks were provided by (GSRAI, 1994a and 1994b), and apparently approved by representatives of the City of Rialto for development of the adjoining River Ranch project. Based on our review (GSRAI, 1994a and 1994b), structural setbacks are mapped in close proximity to the southeast corner of the site; however, the structural setback is associated with a concealed (buried) fault trace in this area, and faulting was not encountered in either of the early fault finding trenches in the southern portion of the site by GSRAI (1994b [see Plates 1 and 2]), nor during our recent subsurface investigation. Therefore, structural setbacks are not considered warranted.

Seismicity

Deterministic Maximum Credible Site Acceleration

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 ("maximum credible") earthquake on that 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 San Bernardino segment of the San Jacinto fault may be on the order of 0.765g. The computer printouts of pertinent portions of the EQFAULT program are included within Appendix B.

Historical Site Acceleration

Historical site seismicity was evaluated with the acceleration-attenuation relation of Bozorgnia, Campbell, and Niazi (1999), and the computer program EQSEARCH

(Blake, 2000b, updated through 2021). 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 May 2021. 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 May 2021 was about 0.336g. 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 B.

Airphoto Lineament Analysis

In order to identify possible unmapped faults and fissures, and to further evaluate the active strands of the San Jacinto fault mapped offsite, a lineament analysis was performed. Stereoscopic "false-color" infrared aerial photographs (United States Department of Agriculture [USDA], 1980) at a scale of approximately 1:3,333 were used in our lineament analysis. Lineaments are classified according to their development as strong, moderate, or weak. A strong lineament is a well defined feature which can be continuously traced several hundred feet to a few thousand feet. A moderate lineament is less well defined, somewhat discontinuous, and can be traced for only a few hundred feet. A weak lineament is discontinuous, poorly defined, and can be traced for a few hundred feet or less.

A moderately developed lineament transects the site in northwest-southeast direction and is coincident with the toe of the aforementioned ascending onsite slope to the west, identified as the Rialto Bench. The lineament was intercepted by our trench as well as a previous trench by GSRAl (1982b in 1994). These mid- to late- Holocene sediments were not offset, as discussed below.

Onsite Fault Finding Investigation and Fault Activity Review

As indicated above, the site is located within an Alquist-Priolo Earthquake Fault Zone (APEFZ), and active strands of the San Bernardino segment of the San Jacinto fault zone have been previously identified on the adjoining property (GSRAl, 1994b and GSI, 2006). Based on our review, active strands of the San Jacinto fault zone are located approximately 60 feet from the southeast corner of the property, and a concealed (buried) fault is mapped offsite, approximately 40 feet from that property boundary. In addition, and as discussed above, a moderately developed lineament transects the site in northwest-southeast direction and is coincident with the toe of the aforementioned Rialto Bench.

The lineament was intercepted by both the GSI FT-1 (this study), and GSRAl (1982b in 1994) Trench T-4, which both exposed sediments conservatively estimated to be at least several thousand years old (less than about 5,000 ybp or mid-Holocene), that were not

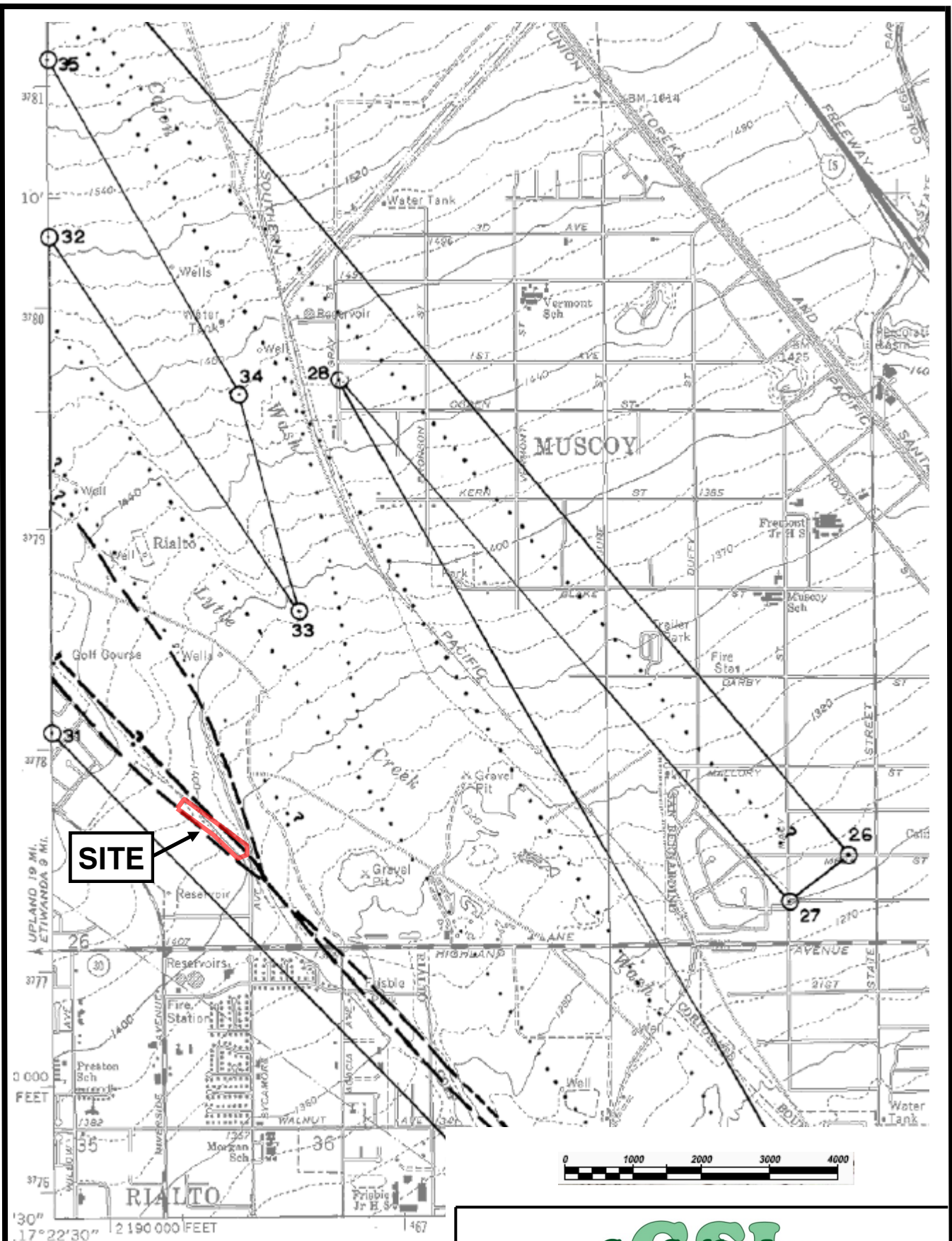
displaced. When contrasted to the recurrence interval of about 107 years (Treiman, and Lundberg, 1999) for the active San Jacinto fault, or even secondary faults associated with the SJFZ (GSI, 2006, and 2007b), of about 1,000-2,000 years, multiple events would have been apparent if there was a sufficiently active fault present.

In addition, the existing and adjoining residential developments to the west (Tracts 8612, 9147, and 9148) were initially investigated by F. Beach Leighton & Associates (FBLA) in 1972, and then by Leighton and Associates (LA) in 1975. The LA (1975) supplementary fault investigation included the advancement of two (2) offsite fault finding trenches near the top of the slope (see Plate 1). Trench 2 (LA, 1975) extended to the western property boundary, and intercepted the western fault trace from the Alquist-Priolo Special Studies Zone (CDMG, 1974) mapping (see Figure 2). The LA (1975) supplementary fault investigation report concluded that no fault related features were exposed in the subsurface probes that may affect the current study area, and our investigation also disclosed no evidence of surface faulting/displacement of Holocene sediments. Accordingly, based on all of the above, it is reasonably judged that sufficiently active Holocene faults are not present onsite, and lie offsite and further east. As such, the potential for surface rupture onsite is considered low.

Seismic Design Parameters

Based on the site conditions, the following table summarizes the site-specific design criteria obtained from the 2019 CBC (CBSC, 2019a), Chapter 16 Structural Design, Section 1613, Earthquake Loads. The computer program “Seismic Design Maps,” provided by the California Office of Statewide Health Planning and Development (OSHPD, 2022) was used to aid in design (<https://seismicmaps.org/>). The short spectral response uses a period of 0.2 seconds.

2019 CBC SEISMIC DESIGN PARAMETERS		
PARAMETER	SITE SPECIFIC VALUE PER ASCE 7-16	2019 CBC OR REFERENCE
Risk Category	I, II, or III	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	1.44 g (Section 21.3)	Section 1613.2.1 Figure 1613.2.1(1)
Spectral Response - (1 sec), S_1	1.63 g (Section 21.3)	Section 1613.2.1 Figure 1613.2.1(2)
Site Coefficient, F_a	1.0	Table 1613.2.3(1)
Site Coefficient, F_v	2.5 (Section 21.3)	Table 1613.2.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec), S_{MS}	2.601 g (Section 21.4)	Section 1613.2.3 (Eqn 16-36)



Base Map: State of California Special Studies Zones, CDMG - Effective July 1, 1974, USGS San Bernardino North Quadrangle, California -- San Bernardino Co., 7.5 Minute, dated 1967.

2019 CBC SEISMIC DESIGN PARAMETERS		
PARAMETER	SITE SPECIFIC VALUE PER ASCE 7-16	2019 CBC OR REFERENCE
Maximum Considered Earthquake Spectral Response Acceleration (1 sec), S_{M1}	3.65 g (Section 21.4)	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	1.734 g (Section 21.4)	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	2.437 g (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.	1.099 g (Section 21.5.3)	ASCE 7-16 (Eqn 11.8.1)
Seismic Design Category	E (Section 11.6)	Section 1613.2.5/ASCE 7-16 (p. 85: Table 11.6-1 or 11.6-2)

GENERAL SEISMIC PARAMETERS	
PARAMETER	VALUE
Distance to Seismic Source (A fault) ⁽¹⁾	<0.1 mi (0.16 km)
Upper Bound Earthquake San Jacinto Fault (San Bernardino) Segment ⁽²⁾	$M_w = 6.7^{(1)}$
⁽¹⁾ - Cao, et al. (2003)	
⁽²⁾ - Blake (2000a)	

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

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.

GROUNDWATER

Regional or perched groundwater was not encountered within the subsurface excavation advanced during our field work. Likewise, seeps, springs, or other indications of a high groundwater level were not noted on the subject property during the time of our field

investigation; however, artesian conditions have been known to occur in the past in low-lying areas of Lytle Wash (USGS, 1963). Since this groundwater basin is adjudicated (see below), it is unlikely that the Watermaster would allow groundwater to increase to the point that groundwater levels and conditions of the past would be duplicated, leading to artesian conditions in the future.

Based on our review of the California Department of Water Resources (CDWR) Water Data Library (2022), the depth to groundwater in wells on the adjoining property ranges from approximately 242 feet below the ground surface (bgs) to approximately 351 bgs. Groundwater, at depth, is likely contained in relatively permeable zones within the underlying alluvial units, and in a general sense perched on the younger to older alluvium or at depth or along unconformable contacts with permeability contrasts (i.e., relatively permeable sediments underlain by aquitards). Due to the observed site conditions, groundwater is not anticipated be encountered during grading; however, the possibility of transient, localized perched conditions cannot be precluded within existing drainage areas, and impermeable zones. Based on our review, GSI estimates that the down-groundwater gradient is in a southeasterly direction; however, it may be influenced by local groundwater withdrawal or recharge. These observations reflect site conditions at the time of our field work and do not prevent future changes in local groundwater conditions from excessive irrigation, precipitation, or that were not evident at the time of our investigation. Due to the proximity of the site to the Lytle Creek wash, the potential for flooding should be further evaluated by the design civil engineer.

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.

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. Typically, liquefaction has a relatively low potential at depths greater than 45 feet and is virtually unknown below a depth of 60 feet.

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 three or perhaps four of the necessary five concurrent conditions have the potential to affect the site, after remedial grading.

Based on our review of data from previous investigations conducted for the adjoining property (GSI, 2018, 2007b, 2006, and GSRAI, 1994b), we found there is a potential for seismic activity; however, based on the generally granular and coarse nature of onsite soils, the anticipated removal of low density near surface soils and compacted fill placement, appropriate foundation design, and the depth to regional groundwater, the potential for liquefaction to affect the site is considered low.

SUBSIDENCE

Our experience in the site vicinity and review of readily available data indicate that the site area may be subsiding owing to down-faulting along bordering fault zones, or regional groundwater withdrawal. The scope of this potential for affecting the subject site is beyond the scope of this current study. However, based on our review of Fife, et al. (1976) aerial subsidence within the generalized area may be on the order of 0.8 feet. The main subsidence zone is located near San Bernardino, which is approximately 6 miles southeast of the site. Areal subsidence generally occurs at the transition condition between materials of substantially different engineering properties (i.e., bedrock and alluvium), or active faults. At the site, no potential for this transition condition appears to exist; however, active faults transect the adjoining site. Our review of available stereoscopic aerial photographs (USDA, 1980) also showed no features generally associated with areal subsidence (i.e., radially-directed drainages flowing into a depression(s), linear depressions associated with mountain fronts, etc.).

Furthermore, the Rialto-Colton Basin is an adjudicated water basin. Past overdraft had resulted in subsidence as early as 1971, and resulted in damage to existing infrastructure. Groundwater pumping induced drawdown and aquifer-system compaction were identified as the most likely cause of the land subsidence in the affected area. The main areas of known subsidence lie several miles to the southeast of the subject site (Fife, et al., 1976). Since the groundwater basin is now adjudicated, it is unlikely that the Watermaster would allow groundwater extraction to increase to the point that significant subsidence would occur onsite. Regardless, the potential for subsidence to affect the site is considered low to perhaps moderate.

OTHER GEOLOGIC/SECONDARY SEISMIC HAZARDS

The following list includes other geologic/seismic related hazards that have been considered during our evaluation of the site. The hazards listed are considered negligible

or mitigated as a result of site location, soil characteristics, and typical site development procedures:

- Dynamic Settlement
- Ground Lurching or Shallow Ground Rupture
- Shallow Groundwater
- Tsunami
- Seiche

LABORATORY TESTING

Classification

Soils were classified visually according to the Unified Soils Classification System. The soil classifications are shown on the Fault Trench Logs, Plate 2.

Moisture Density

The field moisture contents and dry unit weights were determined for the onsite soils encountered in the fault finding trench. Field moisture and density tests were performed using the nuclear (densometer) method ASTM D 6938. The results of the field moisture and density testing are presented below.

LOCATION & DEPTH (ft.)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)
SW Corner FT-1 @ 0'	91.5	2.9
SW Corner FT-1 @ 5'	124.6	3.0
SW Corner FT-1 @ 10'	107.1	3.6
S Corner FT-1 @ 0'	90.2	4.4
N Corner FT-1 @ 1.5'	88.4	5.0
N Corner FT-1 @ 5'	121.2	3.3

Laboratory Standard

The maximum density and optimum moisture content was determined for the major soil types encountered within the exploratory fault finding trench. A rock correction was determined and the maximum density values listed below reflect the "rock corrected" test results. The laboratory standard used was ASTM D 1557. The moisture-density relationships obtained for the site soils are shown below, and in Appendix C:

SOIL TYPE	LOCATION & DEPTH (ft.)	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
Silty SAND, Dark Gray	SW Corner FT-1 @ 0'-4'	119.0	9.7
Gravely SAND, Gray	N Corner FT-1 @ 7'-15'	129.7	9.3

Expansion Potential

Expansion Index (E.I.) testing and expansion potential classification were performed in general accordance with ASTM Standard D 4829 on a representative sample of the onsite soils collected from the field exploration. Based on the preliminary test results obtained, the soils exhibit a very low expansion potential (E.I. 0-20). Additional E.I. testing should be conducted at the conclusion of site grading to further evaluate the preliminary test results obtained. The results of the expansion index testing are presented in the following table.

SAMPLE LOCATION AND DEPTH (FT)	EXPANSION INDEX	EXPANSION POTENTIAL
SW Corner FT-1 @ 0'-4'	< 5	Very Low

Sieve Analysis Testing

Sieve analysis testing was performed on collected soil samples in general accordance with ASTM Test Method D 422. The sieve test results are presented in Appendix C.

Direct Shear Test

Shear tests were performed on remolded 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 summarized below and presented in Appendix C.

SAMPLE LOCATION AND DEPTH (FT)	PRIMARY		RESIDUAL	
	COHESION (PSF)	FRICTION ANGLE (DEGREES)	COHESION (PSF)	FRICTION ANGLE (DEGREES)
SW Corner FT-1 @ 0'-4'	247	35	172	35
N Corner FT-1 @ 7.5'-15'	82	42	34	38

R-Value Test

An evaluation of the R-value for a representative sample of the site soils was performed in general accordance with the latest revisions to the Department of Transportation, State of California, Material & Research Test Method No. 301. Test results obtained indicated an R-value of 72. Based on the test results obtained, preliminary pavement designs for the project have been developed, and are provided in the Preliminary Pavement Design Recommendations section of this report. The R-value test results obtained are presented in Appendix C.

Soluble Sulfates/Corrosion

A typical sample of the site soil 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	RESISTIVITY (OHMS-CM)
SW Corner FT-1 @ 0'-4'	0.006	53	6.0	11,000

For preliminary planning purposes, and based upon the soluble sulfate test results and the latest edition of the 2019 CBC (CBSC, 2019), the soluble sulfate content is classified as "S0" per the ACI (2014a), and sulfate-resistant concrete is currently not anticipated. Based on the results of the resistivity and pH testing, the onsite soils are generally moderately acidic (a pH of 6.1 to 6.5 is considered moderately acidic) and are considered mildly corrosive to ferrous metals in a saturated state (over 10,000 ohm-cm is considered mildly corrosive). Chloride levels are generally low.

Although the site soils are categorized as mildly corrosive to ferrous metals, other than Exposure Classes S0, W0, and C1, no exposure conditions indicated in Table 19.3.1.1 of the ACI (2014a) were considered warranted based on our preliminary laboratory testing, as the footings would likely be exposed to moisture. It is our understanding that ferrous metals embedded in properly poured and formed concrete with the proper mix should be adequately protected from these conditions. Based upon the preliminary laboratory test results obtained, a consulting corrosion engineer should be retained to provide specific recommendations for foundations, utility piping, etc, as warranted.

SLOPE STABILITY

As indicated previously, our review of Geologic Hazard Overlay maps in the San Bernardino County Land Use Plan (County of San Bernardino, 2010a and 2010b), indicates that localized areas of the ascending onsite slope to the west, identified as the

Rialto Bench, are mapped to have a low to moderate generalized landslide susceptibility. As such, our study has included a slope stability analysis of the existing onsite natural slope configuration.

Conventional slope stability analyses have been performed for this study, based on the site topography presented in the Tentative Tract Map 20092 for the adjoining project, by Otte-Berkeley Groupe, Inc., (OBGI, 2018), and our laboratory testing. The existing natural slope onsite has been evaluated using version number two of the computer program GSTABL7 v.2. The GSTABL7 v.2 program performs a 2-dimensional limit equilibrium analysis to compute the factor of safety for a layered slope using the simplified Bishop or Janbu (Block) methods. A representative geologic cross-section was prepared for this analysis, using the tentative tract map by OBGI (2018), as well as field and laboratory data from our recent investigation. The results of the analysis are discussed below and included in Appendix D.

Natural Slope Stability

A calculated gross factor-of-safety greater than 1.5 or 1:1 has been obtained for the ascending natural slope to the west (up to approximately 30 feet in vertical height), when analyzed from a static or pseudo-static (seismic) viewpoint, respectively. Based on our review and analyses, it is our opinion that construction below the natural slope is feasible from a geotechnical viewpoint, provided the following recommendations are incorporated into the design and construction of the project:

1. Approximately 2 to 7 feet of loose surficial soils (colluvium/topsoil deposits) exist near the toe of the natural ascending slope, above or near anticipated retaining walls, roadway, or building pads areas. Extra care should be used when grading or cutting any natural slope area, such that adverse slope stability and drainage is not created.
2. Concrete v-ditches should be considered near the top of the natural slope, and constructed near the tops of any retaining walls to mitigate the potential for water to flow over, or over-top the retaining walls and cause additional erosion or potential surficial failures.
3. Concentrated slope drainage should be directed away from slopes and structures, in accordance with the recommendations of the design engineer.
4. All other drainage, slope maintenance, and planting/landscape recommendations contained herein should be adhered to.

Surficial Slope Stability

The surficial stability calculations (Appendix D) indicate a surficial safety factor slightly greater than 1.5 for the upper portions of natural slope (Q_{yf_3} - Unit 3). However, a surficial safety factor of only 0.96 was obtained using the residual shear parameters obtained from Unit 4 (Q_{yf_4}) on the lower portion of the site. Therefore, the potential for minor surficial failures may exist. Accordingly, the drainage, slope maintenance, planting, and landscape recommendations provided herein should be implemented.

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, disking, 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 (excluding the removal of oversize materials):

Undocumented Artificial Fill (A_{fu})	15% to 20% shrinkage
Colluvium/Topsoil	10% to 18% shrinkage
Young Alluvial-Fan Deposits (Q_{yf_4}/Q_{yf_3})	10% to 15% shrinkage

An additional shrinkage factor item would include the removal of existing root systems of individual large plants or trees. These plants or trees vary in size but, when pulled, they may generally result in a loss of $\frac{1}{2}$ to 1 cubic yard 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, but will depend on haul routes, etc.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on our current field exploration, review of previous geotechnical studies for the adjacent property, laboratory testing, and our engineering and geologic analyses, it is our opinion that the project site appears suited for the proposed residential use from a soils engineering and geologic viewpoint, provided that the recommendations presented in the following sections are incorporated into the design and construction phases of site development. The 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 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 factor in development of the site. Due to the sandy and non-cohesive nature of the site materials and terrain, minor seepage may be encountered throughout the site along with seasonal perched water. In addition, seepage may be encountered in "daylighted" discontinuity systems or sandy lenses within the native onsite earth materials. Given the high permeability of onsite soils and lack of significant canyon areas, no canyon subdrain systems are currently anticipated. However, subdrainage systems for the control of localized groundwater seepage, cannot be precluded and should be anticipated.
4. Based upon our field explorations throughout the site, site materials should be ripplable to the depths anticipated for development.
5. Due to the non-cohesive nature of some of the onsite materials, some caving and sloughing may be anticipated to be a factor in subsurface excavations and trenching. Therefore, current local and state/federal safety ordinances for subsurface trenching should be enforced.
6. General Earthwork, Grading Guidelines, and Preliminary Criteria are provided at the end of this report as Appendix E. Specific recommendations are provided below.

Faulting/Seismicity Conclusions and Recommendations

1. Previous studies by GSRAI (1994b, 1993, 1982a, 1982b) indicate that the adjacent offsite zone of faulting, within the San Bernardino segment of the San Jacinto fault zone, is active (i.e., movement within the Holocene epoch, or last 11,700 years), according to the State of California (California Geological Survey, 2018).
2. With regard to the project site, evidence for sufficiently active faulting was not encountered during this study, or within the previous onsite fault finding trenches (T-3 and T4) advanced in the southern portion of the site by GSRAI (1994b). Based on our review of available data and literature, subsurface investigation, and soil stratigraphy, GSI concludes that no sufficiently active faults likely exist onsite.

3. Based on our review (GSRAI, 1994a and 1994b), structural setbacks are mapped in close proximity to the southeast corner of the site; however, the structural setback in this specific area is associated with a concealed fault trace, and faulting was not encountered in either of the early fault finding trenches in the southern portion of the site by GSRAI (see Plates 1 and 2), nor during our recent subsurface investigation. Therefore, structural setbacks are not considered warranted.
4. Moderate to severe seismic shaking may occur throughout the site, should an earthquake occur on one of the nearby active faults.

Demolition/Grubbing

1. Any existing surficial/subsurface structures (i.e., foundations, wells, septic systems, etc.), major vegetation, tree remains, 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 or wells, 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 exploratory fault finding trenches) remaining after demolition and site clearance should be cleaned out, observed by the geotechnical consultant, 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, if not removed by proposed cuts.

Treatment of Existing Ground

1. Removal of all undocumented artificial fill, surficial colluvium/topsoil, and near-surface weathered young alluvial-fan deposits will be necessary prior to fill placement, in areas proposed for settlement-sensitive improvements. Near-surface young alluvial-fan deposits should be removed to competent materials (i.e., greater than or equal to 85 percent saturation, 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, these depths are estimated to be on the order of 5 to 7 feet across the majority of the site, with localized deeper removals on the order of 15 to 24 feet, or more where previous fault trenching by GSI and others was performed onsite. Variations from these thicknesses should be anticipated. Actual depths of removals will be evaluated in the field during grading by the geotechnical consultant.
2. After 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.

3. The existing artificial fill, colluvium/topsoil, wash and axial 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. Oversized rock materials greater than 12 inches in diameter should be placed in accordance with the Rock Placement Guidelines section of this report. Rock materials greater than 12 inches should not be placed within 10 feet of finish grade. The maximum particle size for utilities in this area may be on the order of 4 to 6 inches. The use of particles greater than 6 inches may require screening prior to re-use in utility backfill, and rock fills may not be suitable for the support of swimming pools or spas (disclosure issue).
4. Any import materials should be observed and determined suitable by the geotechnical consultant prior to placement on the site. Foundation designs may be altered if import materials have a greater sulfate or expansion values than the onsite materials encountered during our preliminary investigation.

Slope Considerations and Slope Design

It is assumed that all cut and fill slopes will be proposed at 2:1 (h:v) gradients, or flatter, and not exceeding about 20 to 30 feet in vertical height. Any proposed fill slopes constructed using onsite materials, should be grossly and surficially stable provided the recommendations contained herein are implemented during site development. Due to the potential for low cohesion materials and surficial instability, the outer 15 feet of cut slopes (as measured horizontally from the slope face), if constructed, may need to be stabilized and replaced with compacted fill.

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

1. Fill slopes should be designed and constructed at a 2:1 (h:v) gradient or flatter and should not exceed about 20 to 30 feet in vertical height, without a specific slope stability evaluation. Fill slopes should be properly built and compacted to a minimum relative compaction of 90 percent throughout, including the slope surfaces. General guidelines for slope construction are presented in Appendix E.
2. Due to the granular nature and low cohesion of a majority of the young alluvial-fan deposits encountered onsite, fill slopes may be subject to erosion. This should be disclosed to all interested/affected parties.
3. Cut slopes should be designed at gradients of 2:1 or flatter and should not exceed about 20 to 30 feet in vertical height, without a specific slope stability evaluation. Due to the potential for low cohesion materials and surficial instability, the outer 15 feet of cut slopes (as measured horizontally from the slope face) may need to be stabilized and replaced with compacted fill. Evaluation of cut slopes during grading would be necessary in order to identify any areas of non-cohesive sands/gravels. Should any these materials be exposed during construction, the soils engineer/geologist, would assess 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 observations. A preliminary evaluation of proposed cut slopes can be provided at the 40-scale design stage when a site grading plan and slope configurations are better known. General guidelines for slope construction are presented in Appendix E.
4. Locally adverse geologic conditions (i.e., sandy lenses) may be encountered which may require remedial grading, stabilization, or laying back of slopes to an angle flatter than the adverse geologic condition.
5. GSI recommends using paved brow ditches at the tops of slopes. These devices may need to be maintained periodically, and this should be considered during project planning.
6. Loose rock debris and fines remaining on the face of the cut slopes, if constructed, should be removed during grading. This can be accomplished by the use of a dozer-mounted slope board, as warranted.
7. Where loose materials are exposed on the cut slopes, the project's engineering geologist would require that the slope be cleaned as described above prior to making their final observations. Final approval of the cut slopes can only be made after the slope is fully cut and cleaned.
8. Irrigation of natural (ungraded) slope areas is not recommended. Site drainage should be directed away from the top and bottom (toe) of natural slope areas, and such slopes will also require regular and periodic maintenance.

Transition and Overexcavation Areas

In order to reduce the potential for differential settlements between cut and fill materials, or earth materials of differing compositions, the entire cut portion of cut/fill transitions should be overexcavated to a minimum depth of 3 feet below finish grade, or a maximum ratio of fill thickness of 3:1 (maximum to minimum), and replaced with compacted fill. In addition, building pads located entirely in cut areas should be overexcavated and capped with at least 3 feet of fill, or 2 feet below the bottom of the foundation, whichever is greater.

ROCK PLACEMENT GUIDELINES

Rock Disposal

We anticipate that soils to be used as fill material for the subject project will contain some rock. Appropriately, the need for rock disposal will be necessary during grading operations on the site. Generally for the purpose of this report, the materials may be described as either 8 inches or less, greater than 8 and less than 36 inches, and greater than 36 inches. These three categories set the basic dimensions for where and how the materials are to be placed. Consideration should be given to evaluating anticipated oversize volume and comparing that to the available volume for oversize materials.

Materials 8 Inches in Diameter or Less

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

1. Fines and rock fragments 8 inches or less in diameter may be placed as compacted fill cap materials within the building pads, slopes, and street areas as described below. The rock fragments and fines should be brought to at least optimum moisture content and compacted to a minimum relative compaction of 90 percent of the laboratory standard.
2. The purpose for the 8-inch diameter limits is to allow reasonable sized rock fragments into the fill under selected conditions (optimum moisture or above) surrounded with compacted fines. The 8-inch diameter size also allows a greater volume of the rock fragments to be handled during grading, while staying in reasonable limits for later onsite excavation equipment (backhoes and trenchers) to excavate footings and utility lines.
3. Fill materials 8 inches or less in diameter should be placed, but not limited to, within the upper 10 feet of proposed fill pads, the upper 3 feet of overexcavated cut areas of cut/fill transition pads, and the entire street right-of-way width, including the

proposed overexcavated areas and replacement fill areas, from the depth of the lowest utility to subgrade, or to a depth of 10 feet below finish grade.

Materials Greater Than 8 inches and Less Than 36 Inches in Diameter

1. During the process of excavation, a significant amount of rock fragments or constituents larger than 8 inches in diameter may be generated. These significant amounts of oversized materials between greater than 8 and less than 36 inches in diameter may be incorporated into the fills using a series of rock blankets.
2. Each rock blanket should consist of rock fragments of approximately greater than 8 and less than 36 inches in diameter along with fines generated from the proposed cuts and overburden materials from removal areas. The blankets should be limited to 24 to 36 inches in thickness and should be placed with granular fines which are flooded into and around the rock fragments.
3. Rock blankets should be restricted to areas which are at least 1 foot below the lowest utility invert within the street right-of-way, 10 feet below finish grade on the proposed fill lots, and a minimum of 20 horizontal feet from any fill slope surface.
4. Compaction may be achieved by using wheel rolling methods with scrapers and water trucks, track-walking by bulldozers, and sheepfoot tampers.
5. Each rock blanket should be completed with its surface compacted prior to placement of any additional rock blanket or rock windrow.

Materials Greater Than 36 Inches in Diameter

1. Although generally not anticipated, oversize rock greater than 36 inches in diameter should be placed in single rock windrows. The windrows should be at least 15 feet or an equipment width apart, whichever is greatest, and at least 10 feet below final grade.
2. The void spaces between rocks in windrows should be filled with the more granular soils by flooding them into place.
3. A minimum vertical distance of 3 feet between soil fill and rock lift should be maintained. Also, the windrows should be staggered from lift to lift. Rock windrows should not be placed closer than 20 feet from the face of fill slopes.
4. Larger rocks too difficult to be placed into windrows may be individually placed into a dozer trench. Each trench should be excavated into the compacted fill or dense natural ground a minimum of 1 foot deeper than the size of the rock to be buried. After the rocks are placed in the trench (not immediately adjacent to each other), granular fill material should be flooded into the trench to fill the voids.

5. The oversize rock trenches should be no closer together than 15 feet at a particular elevation and at least 20 feet from any slope face. Trenches at higher elevations should be staggered and there should be 4 feet of compacted fill between the top of one trench and the bottom of the next higher trench. Placement of rock into these trenches should be under the full-time inspection of the soils engineer.
6. Consideration should be given to using oversize materials in open space “green belt” areas that would be designated as non-structural fills.

PRELIMINARY 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 (2015 and 2014a), and the differential settlement and expansion potential values anticipated. The onsite soils expansion potentials for the project have been evaluated to be in the very low range (E.I. of 0 to 20), as defined by the 2019 CBC. The site lies in an area of potentially moderate to severe seismic activity, and horizontal seismic accelerations are anticipated to be approximately 1g, should the design earthquake occur. Our evaluation indicates that the potential for seismic activity and associated settlement can be reasonably mitigated by use of properly designed post-tensioned or mat foundations along with anticipated removals of near surface low density soils.

For the purpose of our geotechnical review and analyses, the residential wall loads for one- and two-story structures are anticipated to be 1 to 2 kips per lineal foot of wall and 20 to 50 pounds per square foot (psf) of concrete floor load. Isolated column loads are anticipated to be in the range of 10 to 30 kips. All footings are recommended to be embedded into compacted fill as indicated in this report. When the final foundation loads are completed, GSI should be provided with this information for comment or review, as necessary.

General recommendations for foundations using post-tension 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, and should be incorporated into project plans and construction considerations.

General Foundation Design

1. Geotechnical evaluations of these soils indicate that an allowable bearing value of 1,800 psf for very low expansive soils. These soils should be compacted to a minimum of 90 percent (ASTM D 1557) relative compaction and footings should maintain a minimum width of 12 inches (continuous) and 24 inches square (isolated), and a minimum depth of at least 12 inches into properly compacted fill.

Isolated, square footings should extend a minimum of 18 inches into the compacted fill. The bearing value may be increased by one-third for seismic or other temporary loads. This value may be increased by 20 percent for each additional 12 inches in depth to a maximum of 2,500 psf. No increase in bearing should be used for increases in footing width.

2. For lateral sliding resistance, a 0.35 coefficient of friction may be used for a concrete to soil contact (compacted, documented, very low expansive, artificial fill) when multiplied by the dead load.
3. Passive earth pressure on documented fill may be computed as an equivalent fluid having a density of 250 pounds per cubic foot (pcf) with a maximum lateral earth pressure of 2,500 psf.
4. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.
5. All footings should maintain a minimum 7-foot horizontal distance between the base of the footing and any adjacent descending slope, and minimally comply with the guidelines in Section 1808.7 as well as Figure 1808.7.1 of the 2019 CBC.
6. A grade beam reinforced as above and at least 12 inches by 12 inches in cross section should be provided across the garage entrances. If feasible, the base of the grade beam should be at the same elevation as the adjoining footings.
7. Based on the very low expansive soils encountered onsite, the slab thickness, reinforcement, and concrete used should comply with all applicable codes, and in accordance with the foundation designers' recommendations. To improve soil moisture transmission performance (reduce), the slab thickness may be increased as discussed in the "Soil Moisture Transmission" section.
8. Additional recommendations regarding moisture protection from the project structural engineer and project architect should be provided.

The following foundation construction recommendations assume that the soils in the top 7 feet from finish grade will have a very low to low expansion potential (E.I. = 0 to 50). Recommendations by the project's structural engineer or architect, which may exceed the geotechnical consultant's recommendations, should take precedence over the following minimum recommendations.

POST-TENSIONED FOUNDATIONS

The following foundation construction recommendations assume that soils in the upper 7 feet are very low to low expansive in accordance with 2019 CBC. The post-tension foundation designer may exceed these minimal recommendation to increase slab stiffness performance.

The information and recommendations presented in this section are not meant to supercede design by a registered structural engineer or civil engineer qualified to perform post-tensioned design. Post-tensioned foundations should be designed using sound engineering practice and be in accordance with local and 2019 CBC 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 homeowner irrigation, climatic and seasonal changes, and the presence of expansive soils. When the outside soil environment surrounding the slab has a higher moisture content than the area beneath the slab, moisture tends to migrate underneath the slab edges to a distance beyond the slab edges known as a moisture variation distance, and cause the slab edges to lift. Conversely, when the outside soil environment is drier, the moisture 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. Therefore, post-tensioned slabs should have sufficient stiffness and rigidity to resist excessive bending due to non-uniform swell and shrinkage of subgrade soils, particularly within the moisture variation distance, near the slab edges.

To mitigate this possible phenomenon, a combination of soil presaturation and construction of a perimeter “cut-off” wall grade beam should be employed. Perimeter cut-off walls should be a minimum of 12 inches deep for very low to low expansive soils. 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 in width. A vapor retarder should be used and be of sufficient thickness to provide a durable separation of foundation from soils. For additional recommendations see the “Soil Moisture Transmission Considerations” section of this report. The vapor retarder should be sealed to provide a continuous retarder under the entire slab, per the 2019 CBC.

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, 2014), together with its errata’s.

Soil Moisture

Pre-moistening of the slab subgrade soil is recommended owing to soil conditions at the site. 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 or pre-soaking should be evaluated by the geotechnical consultant 72 hours prior to vapor retarder placement. In summary:

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

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.)	15-30

Based on the above, the recommended soil support parameters are tabulated in the following table:

PARAMETERS	VERY LOW TO LOW EXPANSION POTENTIAL (E.I. = 0-50)
e_m center lift	9.0 feet
e_m edge lift	5.2 feet

PARAMETERS	VERY LOW TO LOW EXPANSION POTENTIAL (E.I. = 0-50)
y_m center lift	0.4 inch
y_m edge lift	0.7 inch
Bearing Value ⁽¹⁾	1,000 psf
Lateral Bearing Pressure ⁽³⁾	250 psf
Modulus of Subgrade Reaction (k)	100 pci/inch
Minimum Perimeter Footing Embedment ⁽²⁾	12 to 18 inches
⁽¹⁾ For very low to low expansive soils internal bearing value within the perimeter of the post-tension slab may be increased to 1,500 psf, for a minimum embedment of 12 inches, then by 20 percent for each additional foot of embedment to a maximum of 2,000 psf. No increase in bearing for footing width should be used. ⁽²⁾ As measured below the lowest adjacent compacted subgrade surface without the inclusion of any landscape layer or underslab granular (sand) layers. Very low 12 inches. ⁽³⁾ Maximum lateral bearing should be limited to 2,000 psf for very low to low expansive soils. Note: The use of open-bottom raised planters or extensive landscaping 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 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) parameters may be recommended.

Mat Foundations

In lieu of using a post-tensioned foundation to resist differential settlement or expansive soil effects (if encountered), the Client may consider a mat foundation which uses steel bar reinforcement instead of post-tensioned cables. The structural engineer may supersede the following recommendations based on the planned building loads and use. WRI (Wire Reinforcement Institute) 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), overlying dense formational materials.

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

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

The modulus of subgrade reaction (K_S) and effective plasticity index (P.I.) 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, P.I. = 15 to 24

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 12 inches thick. The bottom of the edge footing should be designed to resist tension, using reinforcement per the structural engineer. The need and arrangement of interior grade beams (stiffening beams) will be in accordance with the structural consultant's recommendations. The recommendations for a mat type of foundation assume that the soils below the slab are compacted fill overlying dense, unweathered formational earth materials. The parameters herein are to mitigate the effects of expansive soils and should be modified to mitigate the effects of the total and differential settlements reported herein.

Confirmation Testing for Final Foundation Design

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

FOUNDATION AND IMPROVEMENT SETTLEMENTS

Designing residential foundations for the existing soil conditions, the estimated settlement and angular distortion values that an individual structure could be subjected to should be evaluated by a qualified structural engineer. In addition, significant site improvements such as retaining walls, sound walls, spas, pools, or other settlement-sensitive improvements

should be evaluated by a structural engineer given the site conditions and geotechnical parameters expressed in this report. The levels of angular distortion were evaluated on a 40-foot length assumed as the minimum dimension of buildings; if, from a structural standpoint, a decreased or increased length over which the differential settlement is assumed to occur is justified, this change should be incorporated into the design. This also applies to the other site improvements previously discussed. The tolerance that pools/spas have for differential settlement is typically less than $\frac{1}{8}$ to $\frac{1}{4}$ inch.

Ground settlement should be anticipated due to primary consolidation and secondary compression of the proposed fills. The total amount of settlement and time over which it occurs is dependent upon various factors, including material type, depth of fill, depth of removals, initial and final moisture content, and in-place density of subsurface materials. Some post-construction settlement is expected and the majority of this settlement is anticipated to occur within 2 to 3 months following grading. Provided that the earthwork and foundation recommendations in this report are adhered, foundations bearing on approved non-detrimentally expansive, engineered fill should be designed to accommodate a differential seismic settlement of up to 1 inch over a 40-foot horizontal span, and a differential static settlement of 1 inch in a 40-foot span (angular distortion = $\frac{1}{480}$).

SOIL MOISTURE TRANSMISSION CONSIDERATIONS

GSI has evaluated the potential for vapor or water transmission through the slabs, in light of typical residential 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, 2022). 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 homeowner) 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 or B criteria, and be installed in accordance with ACI 302.1R-04.
- The 15-mil vapor retarder (ASTM E 1745 - Class A or B) shall be installed per the recommendations of the manufacturer, including all penetrations (i.e., pipe, ducting, rebar, etc.).
- Concrete slabs, including garages, should be underlain by 2 inches of clean sand (S.E. > 30) above a 15 mil vapor retarder (ASTM E 1745 - Class A or Class B, per Engineering Bulletin 119 [Kanare, 2005]). The vapor retarder should in-turn, be underlain by 2 inches of sand (SE>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. The layer of import sand may be eliminated below the vapor retarder, provided the geotechnical consultant has approved the elimination of the additional sand layer, based on the granular nature of exposed pad soils that have been appropriately tested (i.e., sand equivalent [SE $\geq$ 30] and sieve analysis).

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 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 supersede Table 19.3.2.1 of ACI-318 for corrosion or other corrosive requirements. Additional concrete mix design recommendations should be provided by the structural consultant or waterproofing specialist. Concrete finishing and workability should be addressed by the structural consultant and a waterproofing specialist.
- Where slab water/cement ratios are as indicated herein, 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 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 manufacturer's 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.

PRELIMINARY WALL DESIGN PARAMETERS

General

Preliminary recommendations for the design and construction of conventional masonry retaining walls are provided herein. Recommendations for specialty walls (i.e., crib, earthstone, geogrid, etc.) can be provided upon request, and would be based on site specific conditions.

Conventional Retaining Walls

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

Preliminary Retaining Wall Foundation Design

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

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

Minimum Footing Width - 24 inches.

Allowable Bearing Pressure - An allowable bearing pressure of 2,500 pcf may be used in the preliminary design of retaining wall foundations provided that the footing maintains a minimum width of 24 inches and extends at least 18 inches into approved engineered fill overlying dense formational materials. This pressure may be increased by one-third for short-term wind 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 used for a concrete to soil contact when multiplied by the dead load. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

Backfill Soil Density - Soil densities ranging between 120 pcf and 130 pcf may be used in the design of retaining wall foundations. This assumes an average engineered fill compaction of at least 90 percent of the laboratory standard (ASTM D 1557).

Any retaining wall footings near the perimeter of the site will likely need to be deepened into unweathered 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 slope face.

Restrained Walls

Any retaining walls that will be restrained prior to placing and compacting backfill material or that have re-entrant or male corners, should be designed for an at-rest equivalent fluid pressure (EFP) of 55 pcf and 65 pcf for select and very low expansive native backfill, respectively. The design should include any applicable surcharge loading. For areas of male or re-entrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall (2H) laterally from the corner.

Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls up to 10 feet high. Design parameters for walls less than 3 feet in height may be superseded by County/City regional standard design. Active earth pressure may be used for retaining wall design, provided the top of the wall is not restrained from minor deflections. An equivalent

fluid pressure approach may be used to compute the horizontal pressure against the wall. Appropriate fluid unit weights are given below for specific slope gradients of the retained material. These do not include other superimposed loading conditions due to traffic, structures, seismic events or adverse geologic conditions. When wall configurations are finalized, the appropriate loading conditions for superimposed loads can be provided upon request.

For preliminary planning purposes, the structural consultant/wall designer should incorporate the surcharge of traffic on the back of retaining walls where vehicular traffic could occur within horizontal distance “H” from the back of the retaining wall (where “H” equals the wall height). The traffic surcharge may be taken as 100 psf/ft in the upper 5 feet of backfill for light truck and cars traffic. This does not include the surcharge of parked vehicles which should be evaluated at a higher surcharge to account for the effects of seismic loading. Equivalent fluid pressures for the design of cantilevered retaining walls are provided in the following table:

SURFACE SLOPE OF RETAINED MATERIAL (HORIZONTAL:VERTICAL)	EQUIVALENT FLUID WEIGHT P.C.F. (SELECT BACKFILL) ⁽²⁾	EQUIVALENT FLUID WEIGHT P.C.F. (NATIVE BACKFILL) ⁽³⁾
Level ⁽¹⁾	38	45
2 to 1	55	60
⁽¹⁾ Level backfill behind a retaining wall is defined as compacted earth materials, properly drained, without a slope for a distance of 2H behind the wall, where H is the height of the wall. ⁽²⁾ SE $\geq$ 30, P.I. < 15, E.I. < 21, and $\leq$ 10% passing No. 200 sieve. ⁽³⁾ E.I. = 0 to 50, SE $\geq$ 30, P.I. < 15, E.I. < 21, and $\leq$ 15% passing No. 200 sieve.		

Seismic Surcharge

For engineered retaining walls 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 19H 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 19H. For restrained walls, the pressure should be applied as a rectangular distribution. Please note this is for local wall stability only.

The 19H is derived from a Mononobe-Okabe solution for both restrained cantilever walls. This accounts for the increased lateral pressure due to shakedown or movement of the sand fill soil in the zone of influence from the wall or roughly a 45° - $\phi/2$ plane away from the back of the wall. The 19H seismic surcharge is derived from the formula:

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

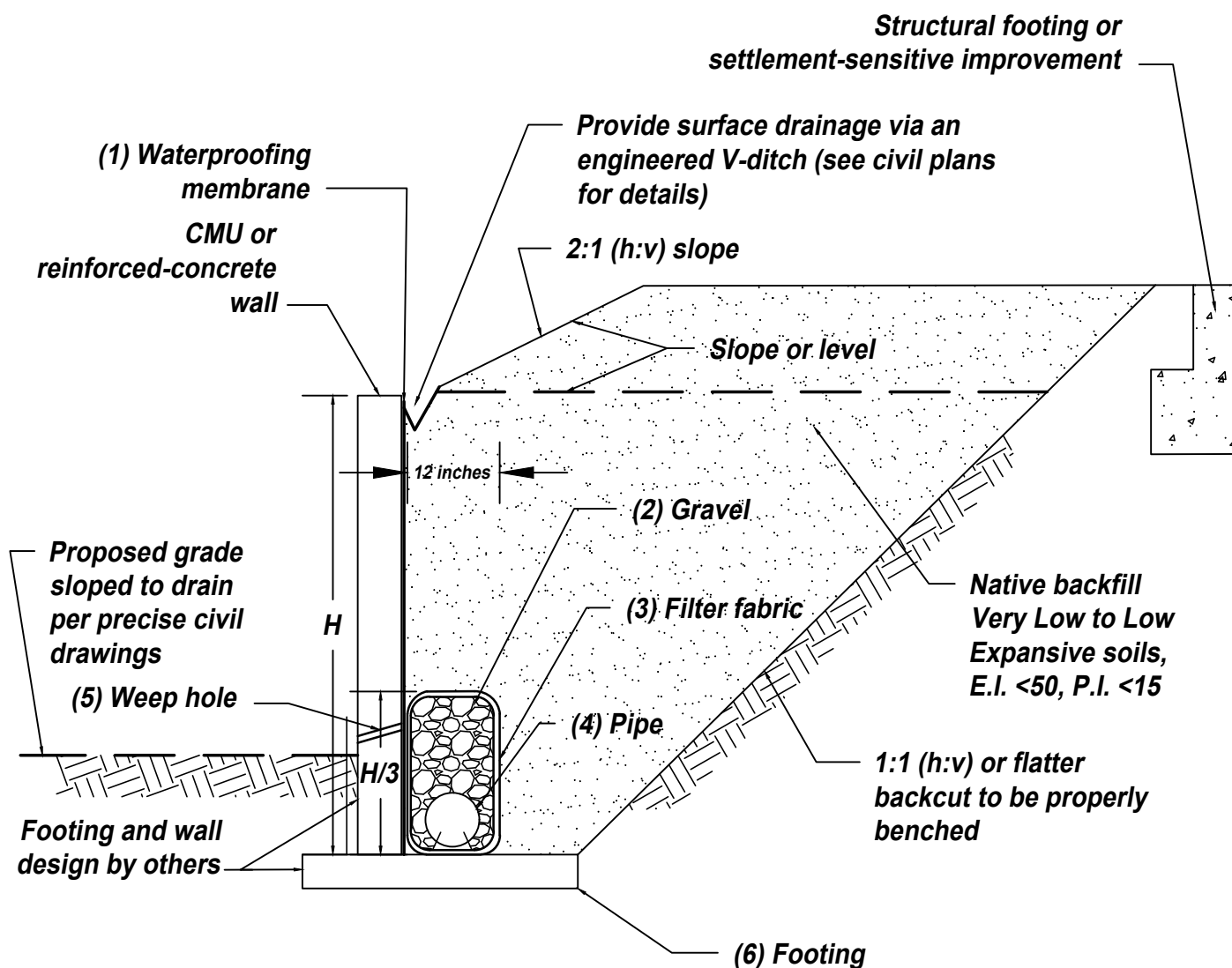
Where:

P_h	=	Seismic increment.
a_h	=	Probabilistic horizontal site acceleration with a percentage of "g."
γ_t	=	total unit weight (115 to 125 pcf for site soils @ 90% relative compaction).
H	=	Height of the wall from the bottom of the footing or point of pile fixity.

Retaining Wall Backfill and Drainage

Positive drainage must be provided behind all retaining walls in the form of gravel wrapped in geofabric and outlets. A backdrain system is considered necessary for retaining walls that are 2 feet or greater in height. Details 1, 2, and 3, present the 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 3/4-inch to 1 1/2-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. = 20, continuous Class 2 permeable drain materials should be used behind the wall. This material should be continuous (i.e., full height) behind the wall, and it should be constructed in accordance with the enclosed Detail 1 (Typical Retaining Wall Backfill and Drainage Detail). For limited access and confined areas, (panel) drainage behind the wall may be constructed in accordance with Detail 2 (Retaining Wall Backfill and Subdrain Detail Geotextile Drain). Materials with an expansion index (E.I.) potential of greater than 20 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), 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. $\leq$ 50). Proper surface drainage should also be provided. For additional mitigation, consideration should be given to applying a water-proof membrane to the back of all retaining structures. The use of a waterstop should be considered for all concrete and masonry joints.



(1) Waterproofing membrane.

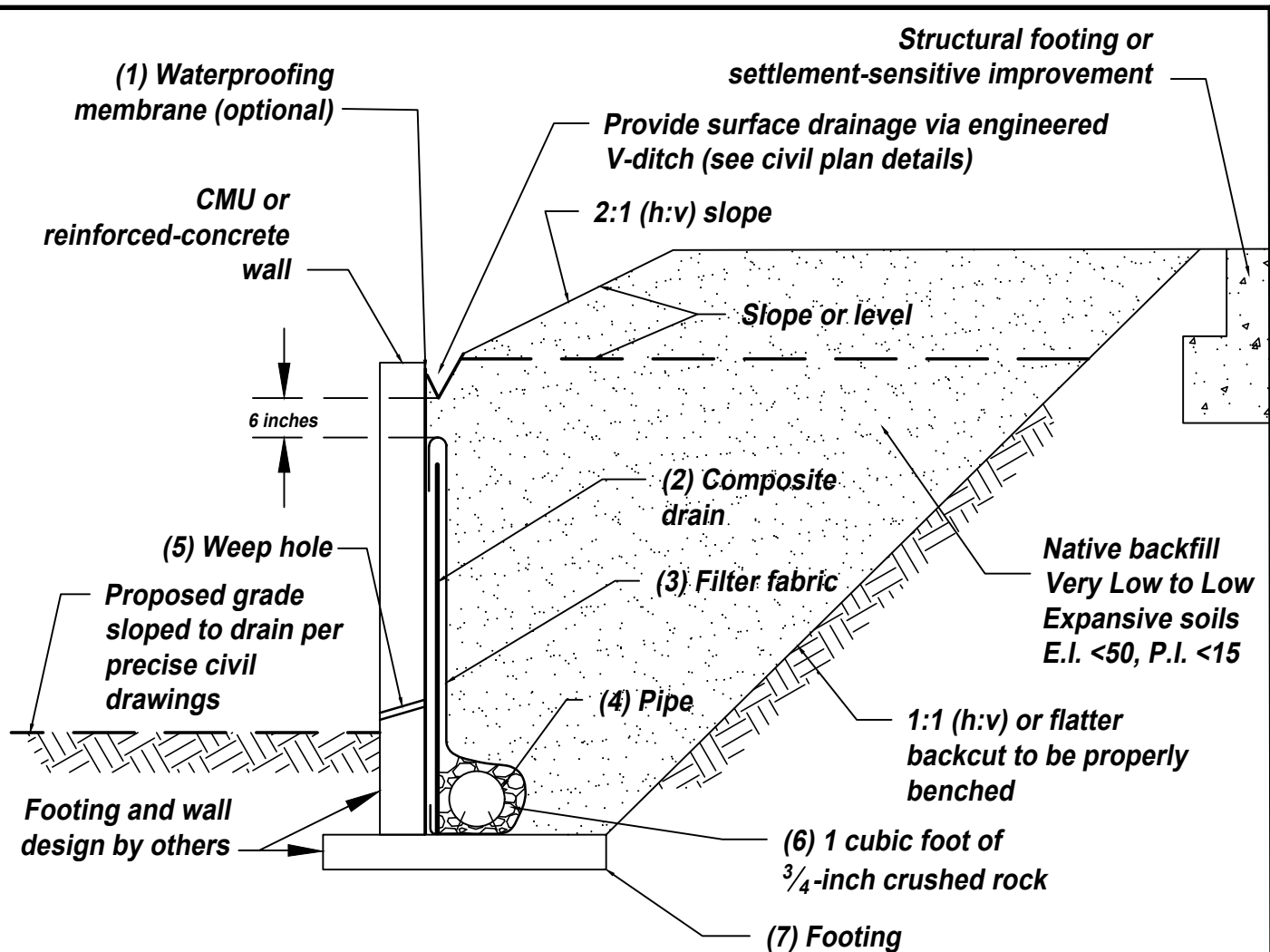
(2) Gravel: Clean, crushed, $\frac{3}{4}$ to $1\frac{1}{2}$ inch.

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

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

(5) Weep holes: For CMU walls, Omit grout every other block, at or slightly above finished surface. For reinforced concrete walls, minimum 2-inch diameter weep holes spaced 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 using level fill or cut natural earth materials, an additional "heel" drain will likely be required by geotechnical consultant.



(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).

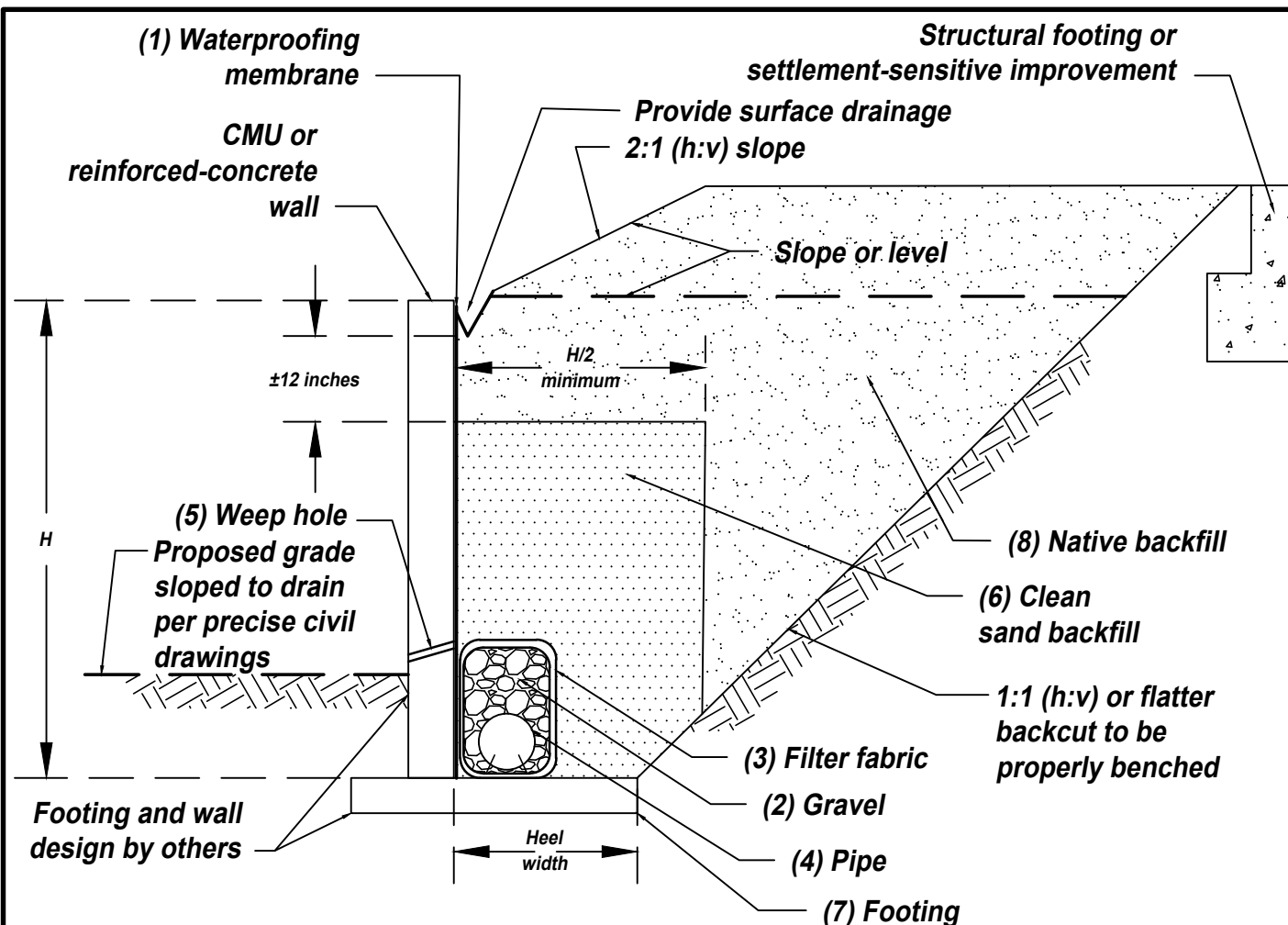
(3) Filter fabric: Mirafi 140N or approved equivalent; place fabric flap behind core.

(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 holes: For CMU walls, Omit grout every other block, at or slightly above finished surface. For reinforced concrete walls, minimum 2-inch diameter weep holes spaced 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, 3/4 to 1 1/2 inch.

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



(1) Waterproofing membrane: Liquid boot or approved mastic equivalent.

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

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

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

(5) Weep hole: For CMU walls, Omit grout every other block, at or slightly above finished surface. For reinforced concrete walls, minimum 2-inch diameter weep hole spaced 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 using 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.

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 and Settlement-Sensitive Improvements

Footings and settlement-sensitive improvements 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. Please note that pool setbacks are specifically addressed in the 2019 CBC (Section 1808.7.3) and these pool setbacks as well as other related pool recommendations in the 2019 CBC should be followed.

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 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. ≤ 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, $\frac{1}{2}$ to $\frac{3}{8}$ 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 grade on the lots 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.

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 R-value of 76, 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 improvements.

The preliminary pavement sections presented in the following table are based on general estimated Traffic Indices (T.I.s), the City of Rialto guidelines (Standard No. S-100) for a local street, a collector street, a secondary arterial roadway, and the guidelines presented in the California Department of Transportation (2020) "Highway Design Manual" seventh edition. It is our understanding that the minimum pavement section required by the City of Rialto for a local street is 4 inches of AC (asphaltic concrete) on 6 inches of Class 2 aggregate base for an estimated T.I. of 5.0 to 7.0. Based on the R-value test results of R=72, the following preliminary pavement designs are presented. The minimum City thicknesses exceed those calculated for the T.I. and R-values shown here. Applicable sections of City/County ordinances should be followed during design of public roads, fire access lanes, etc.

STREET CLASSIFICATION	ESTIMATED TRAFFIC INDEX (T.I.)	STANDARD PAVEMENT DESIGNS		
		R-VALUE	AC INCHES	CLASS 2 BASE ROCK ¹ INCHES
Local Street	5.0	72	4.0*	6.0*
Collector Street	6.0	72	4.0*	6.0*
Secondary Arterial Street	7.0	72	4.0*	6.0*
¹ Assumed R-value for aggregate base rock R=78 - Cal-Trans standard Class 2 base.				
* City of Rialto minimum asphaltic concrete or aggregate base thickness.				

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 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 San Bernardino minimum, as determined by ASTM test designation 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 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 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 used.

ONSITE INFILTRATION-RUNOFF RETENTION SYSTEMS

General

If onsite infiltration-runoff retention systems (OIRRS) are 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 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 or estimated size of the OIRRS, may require additional infiltration testing.

Some of the methods which are used 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.
- If infiltration into the onsite soils is planned, infiltration system design should be based on actual infiltration testing results/data, preferably using double-ring infiltrometer testing (ASTM D 3385) to determine the infiltration rate of the earth materials being contemplated for infiltration. Alternatively, converted percolation rates may be used.
- Wherever possible, infiltration systems should not be installed within 50 feet of the tops of slopes steeper than 15 percent or within $H/3$ from the tops of slopes (where H equals the height of slope).
- Wherever possible, infiltrations systems should not be placed within a distance of $H/2$ from the toes of slopes (where H equals the height of slope).
- Impermeable liners and subdrains should be used along the bottom of bioretention swales/basins located within the influence of slopes. Impermeable liners used in conjunction with 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.]).

- Subdrains should consist of at least 4-inch diameter Schedule 40 or SDR 35 drain pipe with perforations oriented down. The drain pipe should be sleeved with a filter sock.
- The landscape architect should be notified of the location of the proposed OIRRS. If landscaping is proposed within the OIRRS, consideration should be given to the type of vegetation chosen and their potential effect upon subsurface improvements (i.e., some trees/shrubs will have an effect on subsurface improvements with their extensive root systems). Over-watering landscape areas above, or adjacent to, the proposed OIRRS could adversely affect performance of the system.
- Areas adjacent to, or within, the OIRRS that are subject to inundation should be properly protected against scouring, undermining, and erosion, in accordance with the recommendations of the design engineer.
- Seismic shaking may result in the formation of a seiche which could potential overtop the banks of an OIRRS and result in down-gradient flooding and scour.
- 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 used for the infiltration galleries/chambers. Debris filter systems will need to be self cleaning and periodically and regularly maintained on a regular basis. Provisions for the regular and periodic maintenance of any debris filter system is recommended and this condition should be disclosed to all interested/affected parties.
- Infiltrations systems should not be installed within 8 feet of building foundations utility trenches, and walls, or a 1:1 (h:v) slope (down and away) from the bottom elements of these improvements. Alternatively, deepened foundations or pile/pier supported improvements may be used.
- Infiltrations systems should not be installed adjacent to pavement or hardscape improvements. Alternatively, deepened/thickened edges and curbs or impermeable liners may be used in areas adjoining the OIRRS.
- As with any OIRRS, localized ponding and groundwater seepage should be anticipated. The potential for seepage or perched groundwater to occur after site development should be disclosed to all interested/affected parties.
- Installation of infiltrations systems should avoid expansive soils ($E.I. \geq 51$) 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 be 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, or overgrown vegetation (including root systems) should be considered. An appropriate inspection schedule will need to be adopted and provided to all interested/affected parties.
- Any designed system will require regular and periodic maintenance, which may include rehabilitation or complete replacement of the filter media (e.g., sand, gravel, filter fabrics, topsoils, mulch, etc.) or other components used 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.
- 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 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, or seepage along the utility trench line backfill. If utility trenches cross 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 or slurry backfill.
- The use of OIRRS above existing utilities that might degrade/corrode with the introduction of water/seepage should be avoided.

DEVELOPMENT CRITERIA

Slope Maintenance and Planting

Water has been shown to weaken the inherent strength of all earth materials. Slope stability is significantly reduced by overly wet conditions. Positive surface drainage away from slopes should be maintained and only the amount of irrigation necessary to sustain plant life should be provided for planted slopes. Over-watering should be avoided as it adversely affects site improvements, and causes perched groundwater conditions. Graded slopes constructed using 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. Using plants other than those recommended above will increase the potential for perched water, staining, mold, etc., to develop. A rodent control program to prevent burrowing should be implemented. Irrigation of natural (ungraded) slope areas is generally not recommended. These recommendations regarding plant type, irrigation practices, and rodent control should be provided to each homeowner. Over-steepening of slopes should be avoided during building construction activities and landscaping.

Drainage

Adequate lot surface drainage is a very important factor in reducing the likelihood of adverse performance of foundations, hardscape, and slopes. Surface drainage should be sufficient to prevent ponding of water anywhere on a lot, and especially near structures and tops of slopes. Lot surface drainage should be carefully taken into consideration during fine grading, landscaping, and building construction. Therefore, care should be taken that future landscaping or construction activities do not create adverse drainage conditions. Positive site drainage within lots and common areas should be provided and maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond or seep into the ground. In general, the area within 5 feet around a structure should slope away from the structure. We recommend that unpaved lawn and landscape areas have a minimum gradient of 1 percent sloping away from structures, and whenever possible, should be above adjacent paved areas. Consideration should be given to avoiding construction of planters adjacent to structures (buildings, pools, spas, etc.). 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 used 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.

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 used. 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 recompact to 90 percent minimum relative compaction.

Subsurface and Surface Water

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

Site Improvements

If any additional improvements (e.g., pools, spas, etc.) are planned for the site, recommendations concerning the geological or geotechnical aspects of design and construction of said improvements could be provided upon request. Pools or spas should not be constructed without specific design and construction recommendations from GSI, and this construction recommendation should be provided to the homeowners, any homeowners association, 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 after 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 or subcontractors, or homeowners, etc., that may perform such work.

Utility Trench Backfill

1. All interior utility trench backfill should be brought to at least 2 percent above optimum moisture content and then compacted to obtain a minimum relative compaction of 90 percent of the laboratory standard. As an alternative for shallow (12-inch to 18-inch) under-slab trenches, sand having a sand equivalent value of 30 or greater may be used 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 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 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, after the issuance of this report.
- When any developer or homeowner improvements, such as flatwork, spas, pools, walls, etc., are constructed, prior to construction.
- A report of geotechnical observation and testing should be provided at the conclusion of each of the above stages, in order to provide concise and clear documentation of site work, or to comply with code requirements.
- GSI should review project sales documents to homeowners/homeowners associations for geotechnical aspects, including irrigation practices, the conditions outlined above, etc., prior to any sales. At that stage, GSI will provide homeowners maintenance guidelines which should be incorporated into such documents.

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 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 or improvement's designer should confirm to GSI and the governing agency, in writing, that the proposed foundations or improvements can tolerate the amount of differential settlement 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 or further geotechnical studies may be warranted.

LIMITATIONS

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

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

APPENDIX A
REFERENCES

APPENDIX A

REFERENCES

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APPENDIX B
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: W.O. 8377-A

DATE: 06-29-2022

JOB NAME: Beard Realty

CALCULATION NAME: Test Run Analysis

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

SITE COORDINATES:

SITE LATITUDE: 34.1403
SITE LONGITUDE: 117.3668

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. 8377-A-SC
PLATE B-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.
SAN JACINTO-SAN BERNARDINO	2.0(3.2)	6.7	0.765	XI
SAN ANDREAS - San Bernardino M-1	5.1(8.2)	7.5	0.700	XI
SAN ANDREAS - SB-Coach. M-1b-2	5.1(8.2)	7.7	0.727	XI
SAN ANDREAS - SB-Coach. M-2b	5.1(8.2)	7.7	0.727	XI
SAN ANDREAS - Whole M-1a	5.1(8.2)	8.0	0.759	XI
CUCAMONGA	7.0(11.2)	6.9	0.705	XI
CLEGHORN	10.1(16.2)	6.5	0.305	IX
SAN JACINTO-SAN J.VLY-CASA LOMA	11.6(18.7)	6.9	0.337	IX
NORTH FRONTAL FAULT ZONE (West)	12.7(20.5)	7.2	0.521	X
SAN ANDREAS - 1857 Rupture M-2a	12.9(20.7)	7.8	0.501	X
SAN ANDREAS - Cho-Moj M-1b-1	12.9(20.7)	7.8	0.501	X
SAN ANDREAS - Mojave M-1c-3	12.9(20.7)	7.4	0.410	X
SAN JOSE	19.0(30.5)	6.4	0.219	IX
SIERRA MADRE	20.6(33.1)	7.2	0.338	IX
CHINO-CENTRAL AVE. (Elsinore)	22.9(36.8)	6.7	0.219	IX
WHITTIER	24.3(39.1)	6.8	0.156	VIII
ELSINORE (GLEN IVY)	25.2(40.5)	6.8	0.151	VIII
CLAMSHELL-SAWPIT	29.6(47.7)	6.5	0.147	VIII
PUENTE HILLS BLIND THRUST	29.7(47.8)	7.1	0.222	IX
HELENDALE - S. LOCKHARDT	31.9(51.3)	7.3	0.167	VIII
NORTH FRONTAL FAULT ZONE (East)	33.3(53.6)	6.7	0.149	VIII
ELSINORE (TEMECULA)	34.4(55.4)	6.8	0.109	VII
RAYMOND	35.9(57.7)	6.5	0.121	VII
PINTO MOUNTAIN	37.5(60.3)	7.2	0.132	VIII
SAN JACINTO-ANZA	37.8(60.9)	7.2	0.130	VIII
SAN JOAQUIN HILLS	41.6(67.0)	6.6	0.111	VII
VERDUGO	41.8(67.2)	6.9	0.135	VIII
UPPER ELYSIAN PARK BLIND THRUST	42.3(68.0)	6.4	0.096	VII
LENWOOD-LOCKHART-OLD WOMAN SPRGS	43.9(70.7)	7.5	0.138	VIII
JOHNSON VALLEY (Northern)	47.5(76.4)	6.7	0.073	VII
NEWPORT-INGLEWOOD (L.A.Basin)	48.1(77.4)	7.1	0.095	VII
NEWPORT-INGLEWOOD (Offshore)	49.3(79.3)	7.1	0.092	VII
HOLLYWOOD	49.5(79.7)	6.4	0.081	VII
LANDERS	52.8(85.0)	7.3	0.099	VII
SAN ANDREAS - Coachella M-1c-5	53.5(86.1)	7.2	0.091	VII
SAN GABRIEL	53.7(86.4)	7.2	0.091	VII
SIERRA MADRE (San Fernando)	54.2(87.3)	6.7	0.089	VII
EMERSON So. - COPPER MTN.	54.4(87.6)	7.0	0.078	VII
BURNT MTN.	54.6(87.9)	6.5	0.055	VI
EUREKA PEAK	55.6(89.5)	6.4	0.051	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.
ELSINORE (JULIAN)	56.5(91.0)	7.1	0.080	VII
GRAVEL HILLS - HARPER LAKE	56.7(91.2)	7.1	0.080	VII
PALOS VERDES	58.1(93.5)	7.3	0.090	VII
NORTHRIDGE (E. Oak Ridge)	60.4(97.2)	7.0	0.099	VII
CALICO - HIDALGO	60.4(97.2)	7.3	0.086	VII
SANTA MONICA	60.6(97.6)	6.6	0.074	VII

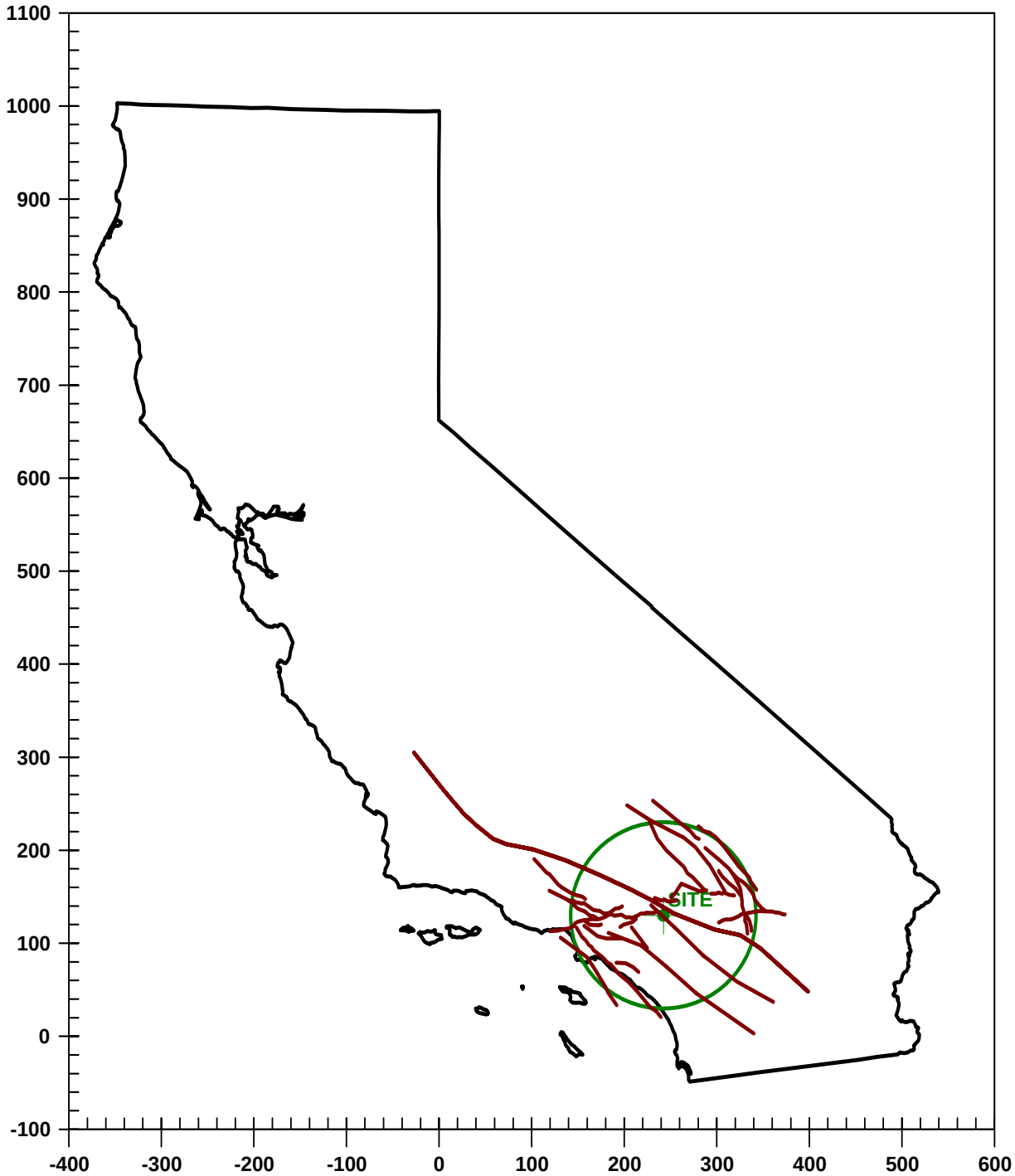
 -END OF SEARCH- 46 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE SAN JACINTO-SAN BERNARDINO FAULT IS CLOSEST TO THE SITE.
 IT IS ABOUT 2.0 MILES (3.2 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.7652 g

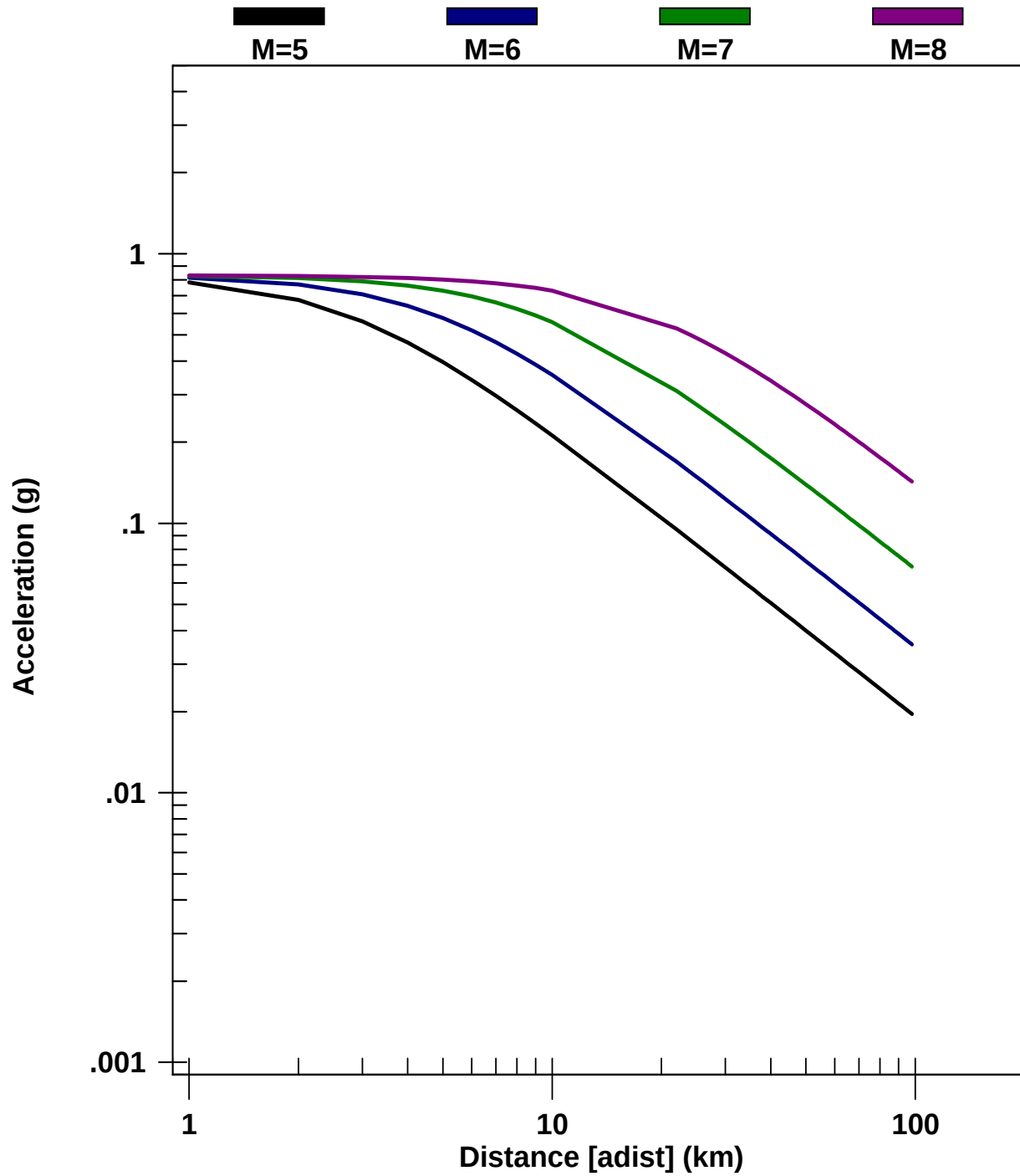
CALIFORNIA FAULT MAP

Beard Realty



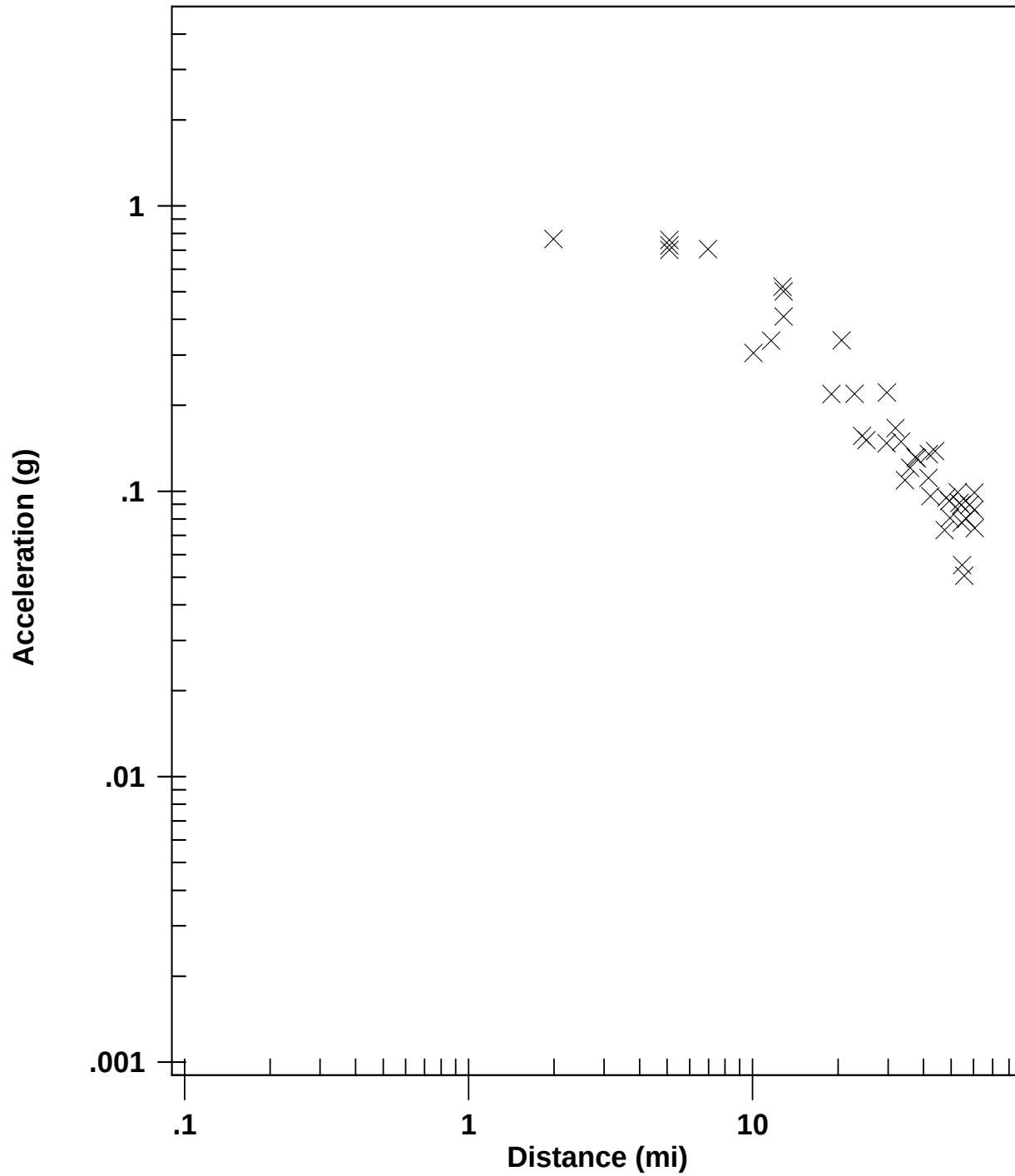
STRIKE-SLIP FAULTS

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



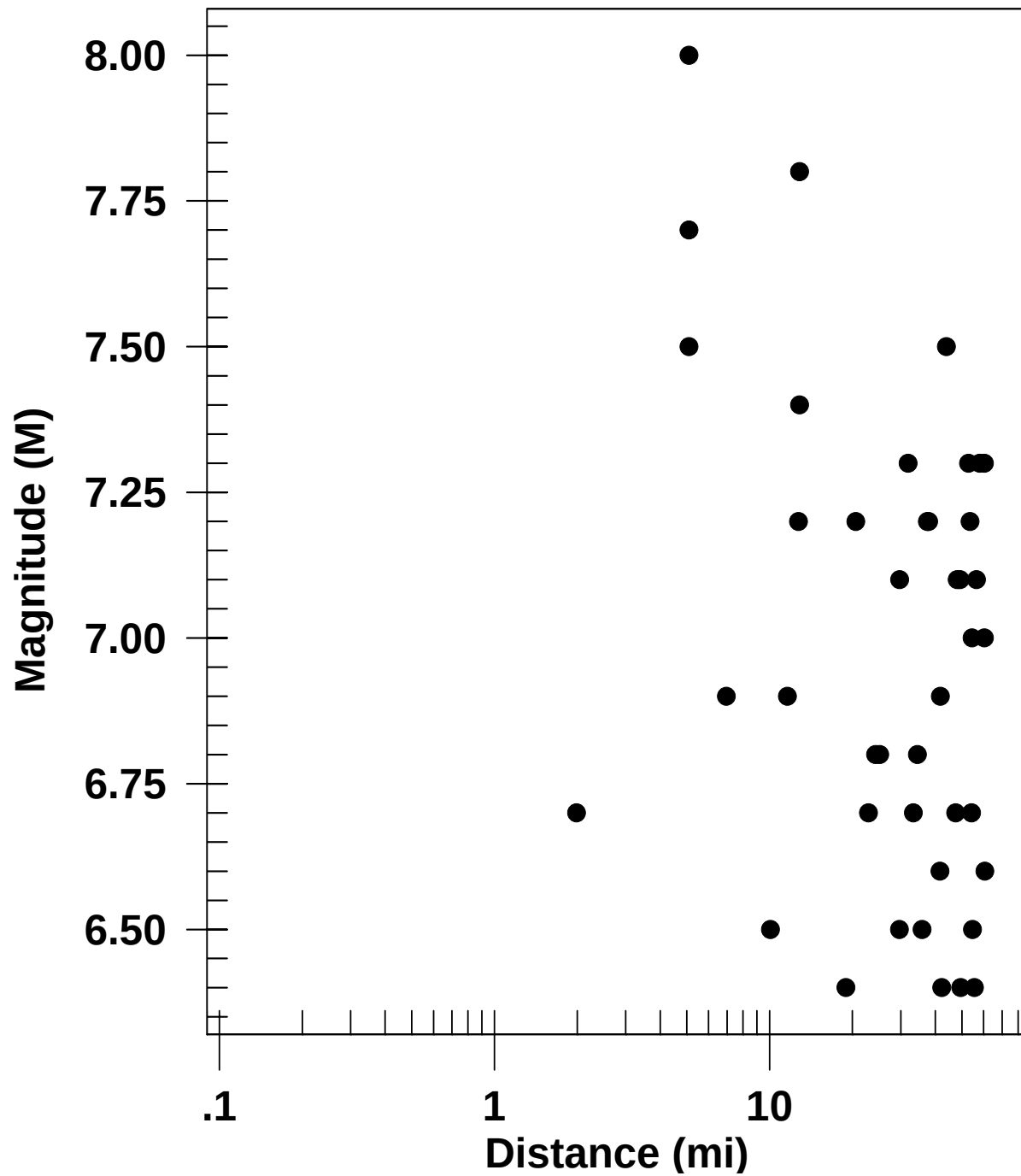
Beard Realty

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EARTHQUAKE MAGNITUDES & DISTANCES

Beard Realty



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

JOB NUMBER: 8377-A

DATE: 06-29-2022

JOB NAME: Beard Realty_Rialto

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

MAGNITUDE RANGE:

MINIMUM MAGNITUDE: 5.00
MAXIMUM MAGNITUDE: 9.00

SITE COORDINATES:

SITE LATITUDE: 34.1403
SITE LONGITUDE: 117.3668

SEARCH DATES:

START DATE: 1800
END DATE: 2021

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

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	34.2000	117.4000	07/22/1899	046 0.0	0.0	5.50	0.293	IX	4.5(7.3)
MGI	34.1000	117.3000	07/15/1905	2041 0.0	0.0	5.30	0.248	IX	4.7(7.6)
DMG	34.0000	117.2500	07/23/1923	73026.0	0.0	6.25	0.225	IX	11.8(18.9)
MGI	34.0000	117.5000	12/16/1858	10 0 0.0	0.0	7.00	0.336	IX	12.3(19.8)
DMG	34.2700	117.5400	09/12/1970	143053.0	8.0	5.40	0.117	VII	13.3(21.5)
DMG	34.3000	117.5000	07/22/1899	2032 0.0	0.0	6.50	0.231	IX	13.4(21.5)
DMG	34.2000	117.1000	09/20/1907	154 0.0	0.0	6.00	0.145	VIII	15.8(25.4)
DMG	34.3000	117.6000	07/30/1894	512 0.0	0.0	6.00	0.132	VIII	17.3(27.8)
GSP	34.1400	117.7000	02/28/1990	234336.6	5.0	5.20	0.073	VII	19.0(30.6)
DMG	33.9000	117.2000	12/19/1880	0 0 0.0	0.0	6.00	0.119	VII	19.1(30.8)
DMG	34.3700	117.6500	12/08/1812	15 0 0.0	0.0	7.00	0.191	VIII	22.6(36.4)
DMG	34.2670	116.9670	08/29/1943	34513.0	0.0	5.50	0.068	VI	24.4(39.3)
DMG	34.1800	116.9200	01/16/1930	034 3.6	0.0	5.10	0.051	VI	25.7(41.3)
DMG	34.1800	116.9200	01/16/1930	02433.9	0.0	5.20	0.054	VI	25.7(41.3)
GSG	33.9530	117.7610	07/29/2008	184215.7	14.0	5.30	0.057	VI	26.0(41.8)
GSP	34.2900	116.9460	02/10/2001	210505.8	9.0	5.10	0.050	VI	26.2(42.1)
MGI	33.8000	117.6000	04/22/1918	2115 0.0	0.0	5.00	0.046	VI	27.0(43.5)
GSP	34.1950	116.8620	08/17/1992	204152.1	11.0	5.30	0.050	VI	29.1(46.8)
GSP	34.1630	116.8550	06/28/1992	144321.0	6.0	5.30	0.050	VI	29.3(47.1)
GSP	34.3400	116.9000	11/27/1992	160057.5	1.0	5.30	0.049	VI	30.0(48.3)
DMG	33.7000	117.4000	05/15/1910	1547 0.0	0.0	6.00	0.074	VII	30.5(49.0)
DMG	33.7000	117.4000	04/11/1910	757 0.0	0.0	5.00	0.040	V	30.5(49.0)
DMG	33.7000	117.4000	05/13/1910	620 0.0	0.0	5.00	0.040	V	30.5(49.0)
DMG	34.2000	117.9000	08/28/1889	215 0.0	0.0	5.50	0.054	VI	30.7(49.5)
GSP	34.2390	116.8370	07/09/1992	014357.6	0.0	5.30	0.047	VI	31.0(49.9)
GSP	34.3690	116.8970	12/04/1992	020857.5	3.0	5.30	0.047	VI	31.1(50.1)
GSN	34.2030	116.8270	06/28/1992	150530.7	5.0	6.70	0.113	VII	31.1(50.1)
DMG	33.8000	117.0000	12/25/1899	1225 0.0	0.0	6.40	0.092	VII	31.5(50.7)
DMG	33.6990	117.5110	05/31/1938	83455.4	10.0	5.50	0.052	VI	31.6(50.8)
GSG	34.3100	116.8480	02/22/2003	121910.6	1.0	5.20	0.043	VI	31.8(51.2)
DMG	33.9500	116.8500	09/28/1946	719 9.0	0.0	5.00	0.038	V	32.3(52.1)
DMG	34.1000	116.8000	10/24/1935	1448 7.6	0.0	5.10	0.040	V	32.5(52.3)
DMG	33.7500	117.0000	06/06/1918	2232 0.0	0.0	5.00	0.036	V	34.2(55.0)
DMG	33.7500	117.0000	04/21/1918	223225.0	0.0	6.80	0.110	VII	34.2(55.0)
GSP	33.9325	117.9158	03/29/2014	040942.2	5.1	5.10	0.038	V	34.5(55.6)
GSP	34.2620	118.0020	06/28/1991	144354.5	11.0	5.40	0.041	V	37.2(59.9)
MGI	34.0000	118.0000	12/25/1903	1745 0.0	0.0	5.00	0.033	V	37.5(60.3)
DMG	34.1000	116.7000	02/07/1889	520 0.0	0.0	5.30	0.038	V	38.2(61.5)
DMG	33.9760	116.7210	06/12/1944	104534.7	10.0	5.10	0.033	V	38.6(62.2)
DMG	33.9940	116.7120	06/12/1944	111636.0	10.0	5.30	0.037	V	38.8(62.4)
DMG	33.7100	116.9250	09/23/1963	144152.6	16.5	5.00	0.031	V	39.0(62.8)
PAS	34.0610	118.0790	10/01/1987	144220.0	9.5	5.90	0.051	VI	41.1(66.1)
MGI	34.1000	118.1000	07/11/1855	415 0.0	0.0	6.30	0.064	VI	42.0(67.6)
PAS	34.0730	118.0980	10/04/1987	105938.2	8.2	5.30	0.034	V	42.1(67.7)
PAS	33.9980	116.6060	07/08/1986	92044.5	11.7	5.60	0.039	V	44.6(71.8)
DMG	33.7500	118.0830	03/11/1933	2 9 0.0	0.0	5.00	0.025	V	49.1(79.0)
DMG	33.7500	118.0830	03/11/1933	323 0.0	0.0	5.00	0.025	V	49.1(79.0)
DMG	33.7500	118.0830	03/13/1933	131828.0	0.0	5.30	0.029	V	49.1(79.0)
DMG	33.7500	118.0830	03/11/1933	230 0.0	0.0	5.10	0.026	V	49.1(79.0)
DMG	33.7500	118.0830	03/11/1933	910 0.0	0.0	5.10	0.026	V	49.1(79.0)
GSP	34.3410	116.5290	06/28/1992	124053.5	6.0	5.20	0.027	V	49.8(80.1)
DMG	33.6170	117.9670	03/11/1933	154 7.8	0.0	6.30	0.053	VI	49.9(80.3)
DMG	33.6830	118.0500	03/11/1933	658 3.0	0.0	5.50	0.032	V	50.3(80.9)

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	34.0170	116.5000	07/25/1947	61949.0	0.0	5.20	0.027	V	50.3(80.9)
DMG	34.0170	116.5000	07/24/1947	221046.0	0.0	5.50	0.032	V	50.3(80.9)

DMG	34.0170	116.5000	07/26/1947	24941.0	0.0	5.10	0.025	V	50.3(80.9)
DMG	34.0170	116.5000	07/25/1947	04631.0	0.0	5.00	0.024	V	50.3(80.9)
DMG	33.7000	118.0670	03/11/1933	85457.0	0.0	5.10	0.025	V	50.3(81.0)
DMG	33.7000	118.0670	03/11/1933	51022.0	0.0	5.10	0.025	V	50.3(81.0)
DMG	33.7830	118.1330	10/02/1933	91017.6	0.0	5.40	0.030	V	50.3(81.0)
MGI	34.0800	118.2600	07/16/1920	18 8 0.0	0.0	5.00	0.023	IV	51.2(82.4)
T-A	34.0000	118.2500	01/10/1856	0 0 0.0	0.0	5.00	0.023	IV	51.4(82.8)
T-A	34.0000	118.2500	03/26/1860	0 0 0.0	0.0	5.00	0.023	IV	51.4(82.8)
T-A	34.0000	118.2500	09/23/1827	0 0 0.0	0.0	5.00	0.023	IV	51.4(82.8)
DMG	33.6170	118.0170	03/14/1933	19 150.0	0.0	5.10	0.024	V	51.9(83.5)
DMG	33.5750	117.9830	03/11/1933	518 4.0	0.0	5.20	0.026	V	52.6(84.7)
GSP	34.3320	116.4620	07/01/1992	074029.9	9.0	5.40	0.028	V	53.3(85.8)
GSN	34.2010	116.4360	06/28/1992	115734.1	1.0	7.60	0.122	VII	53.3(85.8)
GSP	34.1390	116.4310	06/28/1992	123640.6	10.0	5.10	0.024	IV	53.5(86.1)
DMG	34.5190	118.1980	08/23/1952	10 9 7.1	13.1	5.00	0.022	IV	54.1(87.1)
PAS	34.3270	116.4450	03/15/1979	21 716.5	2.5	5.20	0.025	V	54.2(87.2)
MGI	34.0000	118.3000	09/03/1905	540 0.0	0.0	5.30	0.026	V	54.2(87.3)
GSP	34.1080	116.4040	06/29/1992	141338.8	9.0	5.40	0.027	V	55.1(88.6)
DMG	33.8500	118.2670	03/11/1933	1425 0.0	0.0	5.00	0.022	IV	55.3(89.0)
GSP	34.2680	116.4020	06/16/1994	162427.5	3.0	5.00	0.021	IV	55.8(89.8)
PAS	34.5160	116.4950	06/01/1975	13849.2	4.5	5.20	0.024	IV	56.1(90.2)
DMG	33.7830	118.2500	11/14/1941	84136.3	0.0	5.40	0.027	V	56.3(90.6)
GSP	34.0640	116.3610	09/15/1992	084711.3	9.0	5.20	0.023	IV	57.7(92.9)
DMG	33.9330	116.3830	12/04/1948	234317.0	0.0	6.50	0.052	VI	58.1(93.5)
DMG	34.0670	116.3330	05/18/1940	55120.2	0.0	5.20	0.023	IV	59.3(95.5)
DMG	34.0670	116.3330	05/18/1940	72132.7	0.0	5.00	0.020	IV	59.3(95.5)
GSP	34.0290	116.3210	08/21/1993	014638.4	9.0	5.00	0.020	IV	60.3(97.0)
DMG	34.0830	116.3000	05/18/1940	5 358.5	0.0	5.40	0.025	V	61.1(98.3)
GSP	33.9610	116.3180	04/23/1992	045023.0	12.0	6.10	0.038	V	61.3(98.6)
DMG	34.4110	118.4010	02/09/1971	14 1 8.0	8.0	5.80	0.031	V	61.9(99.6)
DMG	34.4110	118.4010	02/09/1971	14 041.8	8.4	6.40	0.045	VI	61.9(99.6)
DMG	34.4110	118.4010	02/09/1971	141028.0	8.0	5.30	0.023	IV	61.9(99.6)
DMG	34.4110	118.4010	02/09/1971	14 244.0	8.0	5.80	0.031	V	61.9(99.6)
GSP	33.5290	116.5720	06/12/2005	154146.5	14.0	5.20	0.021	IV	62.1(100.0)

-END OF SEARCH- 89 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 2021

LENGTH OF SEARCH TIME: 222 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 4.5 MILES (7.3 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.6

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.336 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:

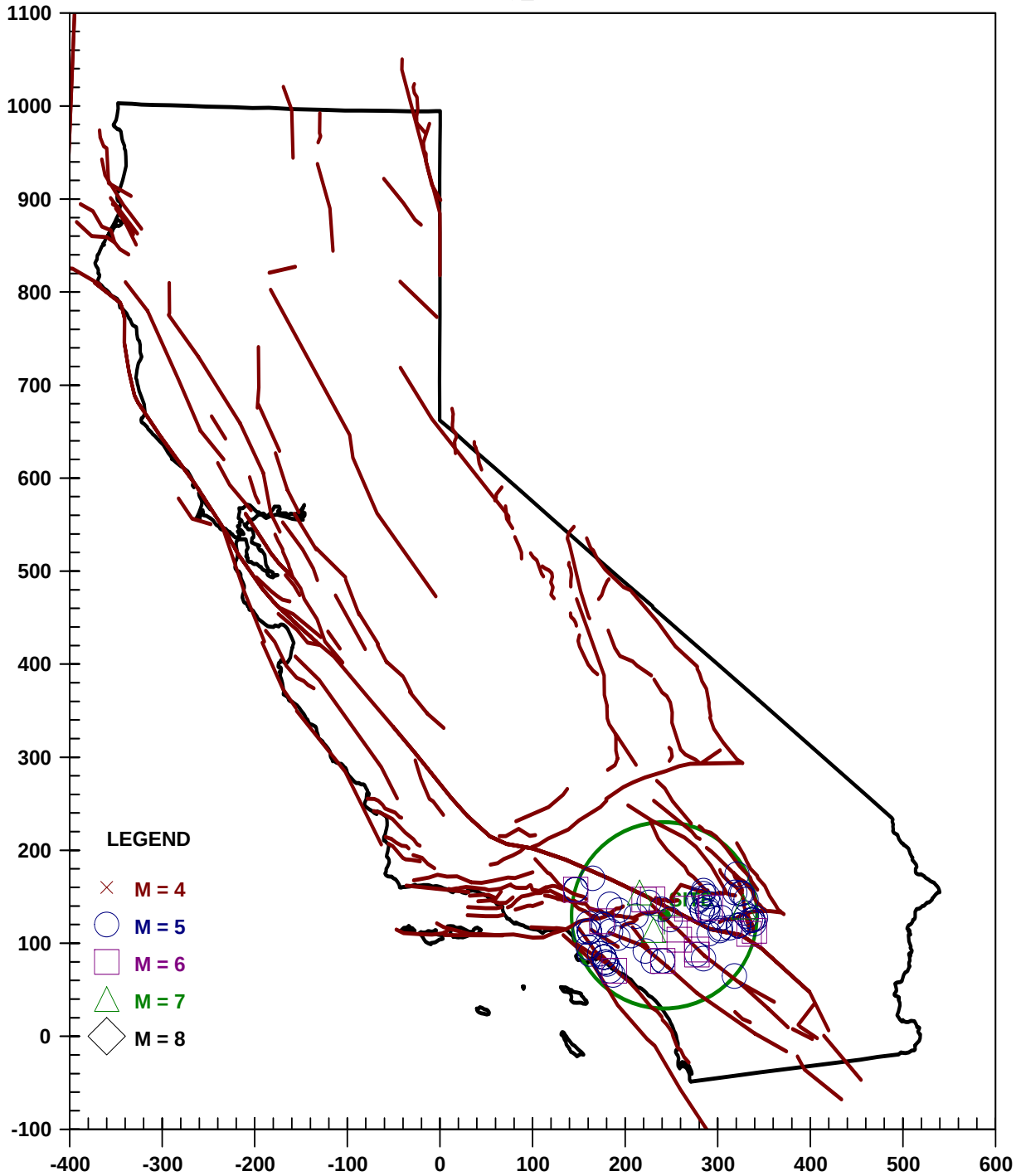
a-value= 1.312
b-value= 0.391
beta-value= 0.901

TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	89	0.40090
4.5	89	0.40090
5.0	89	0.40090
5.5	27	0.12162
6.0	17	0.07658
6.5	7	0.03153
7.0	3	0.01351
7.5	1	0.00450

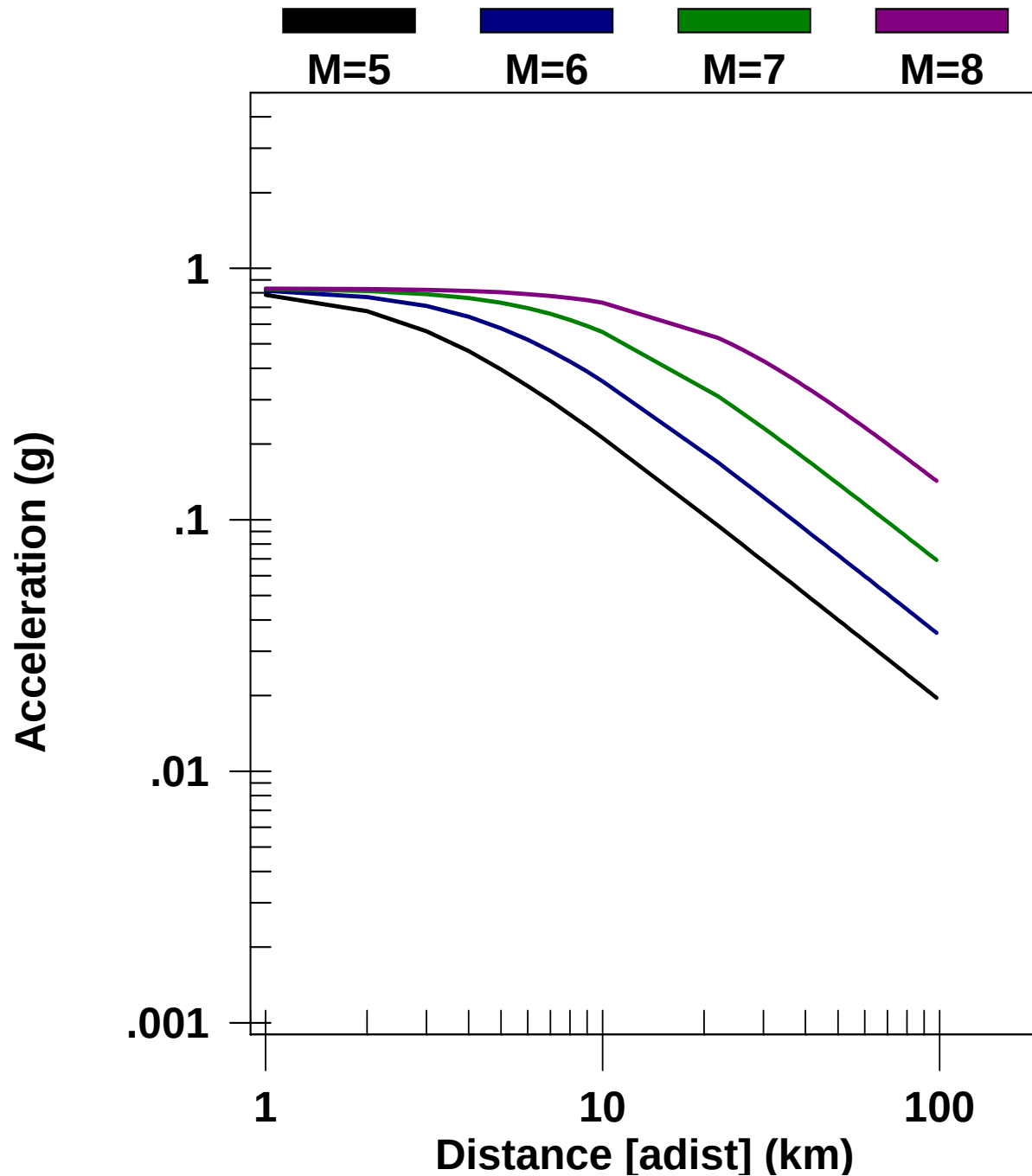
EARTHQUAKE EPICENTER MAP

Beard Realty_Rialto



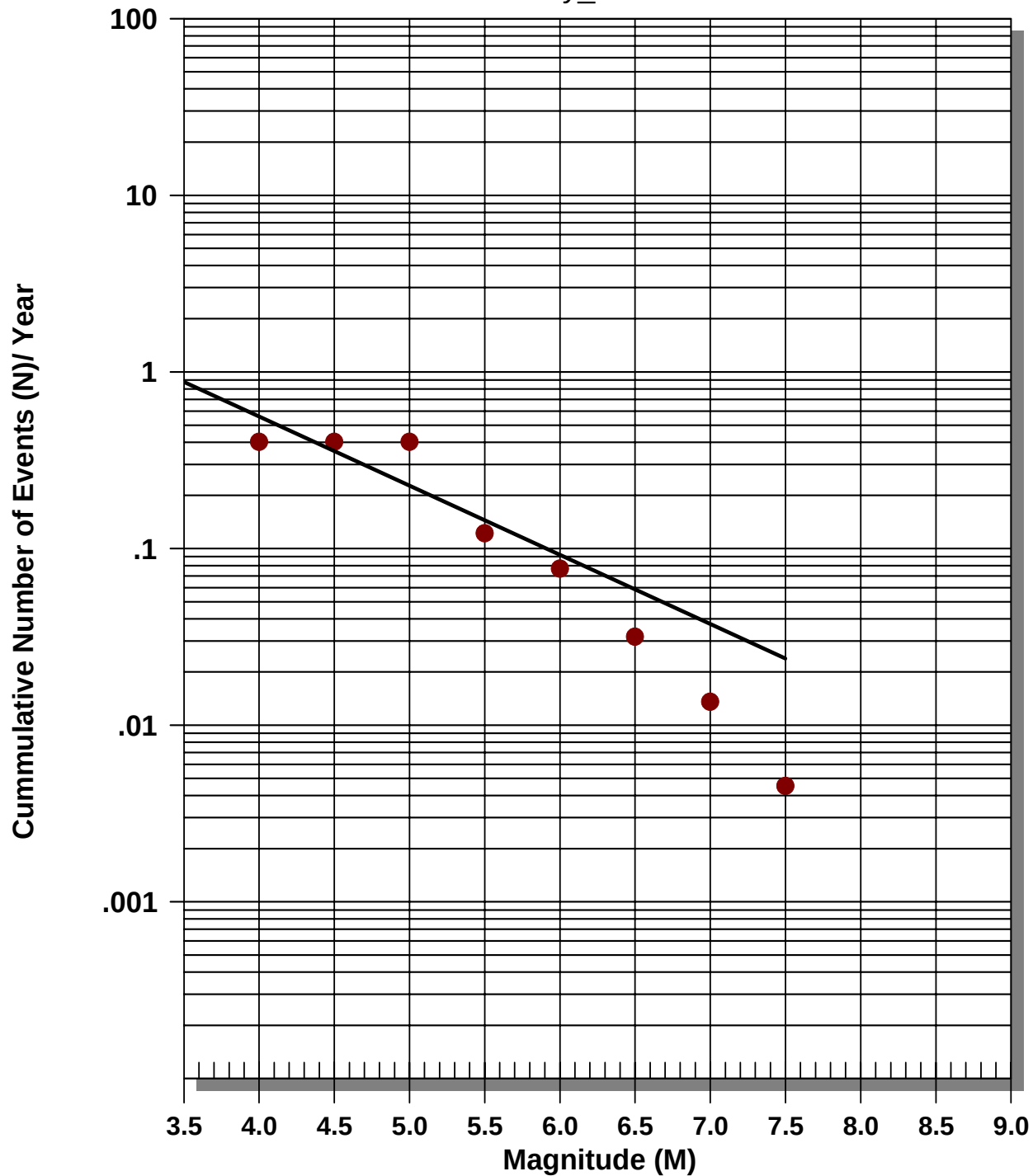
STRIKE-SLIP FAULTS

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



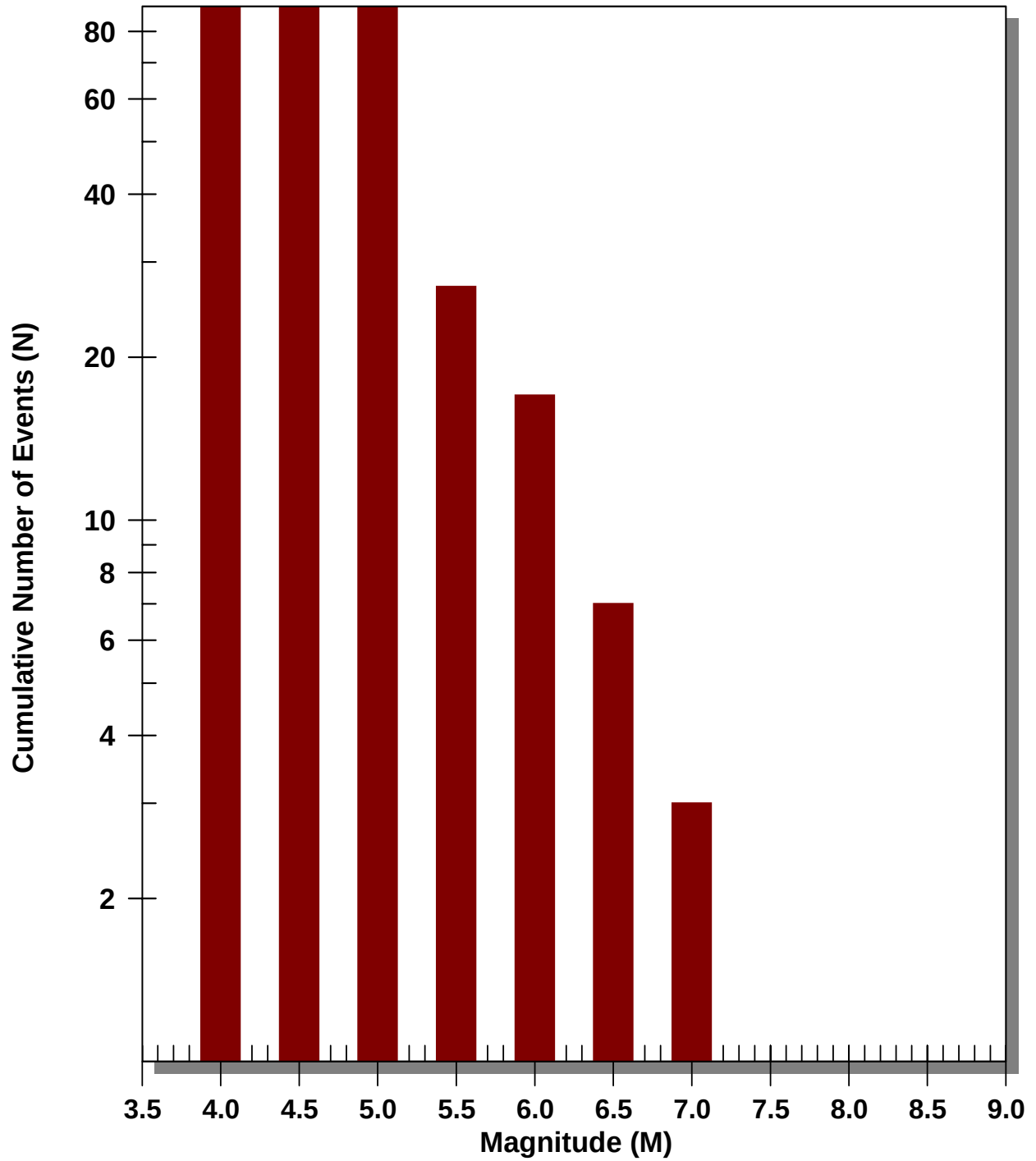
EARTHQUAKE RECURRENCE CURVE

Beard Realty_Rialto



Number of Earthquakes (N) Above Magnitude (M)

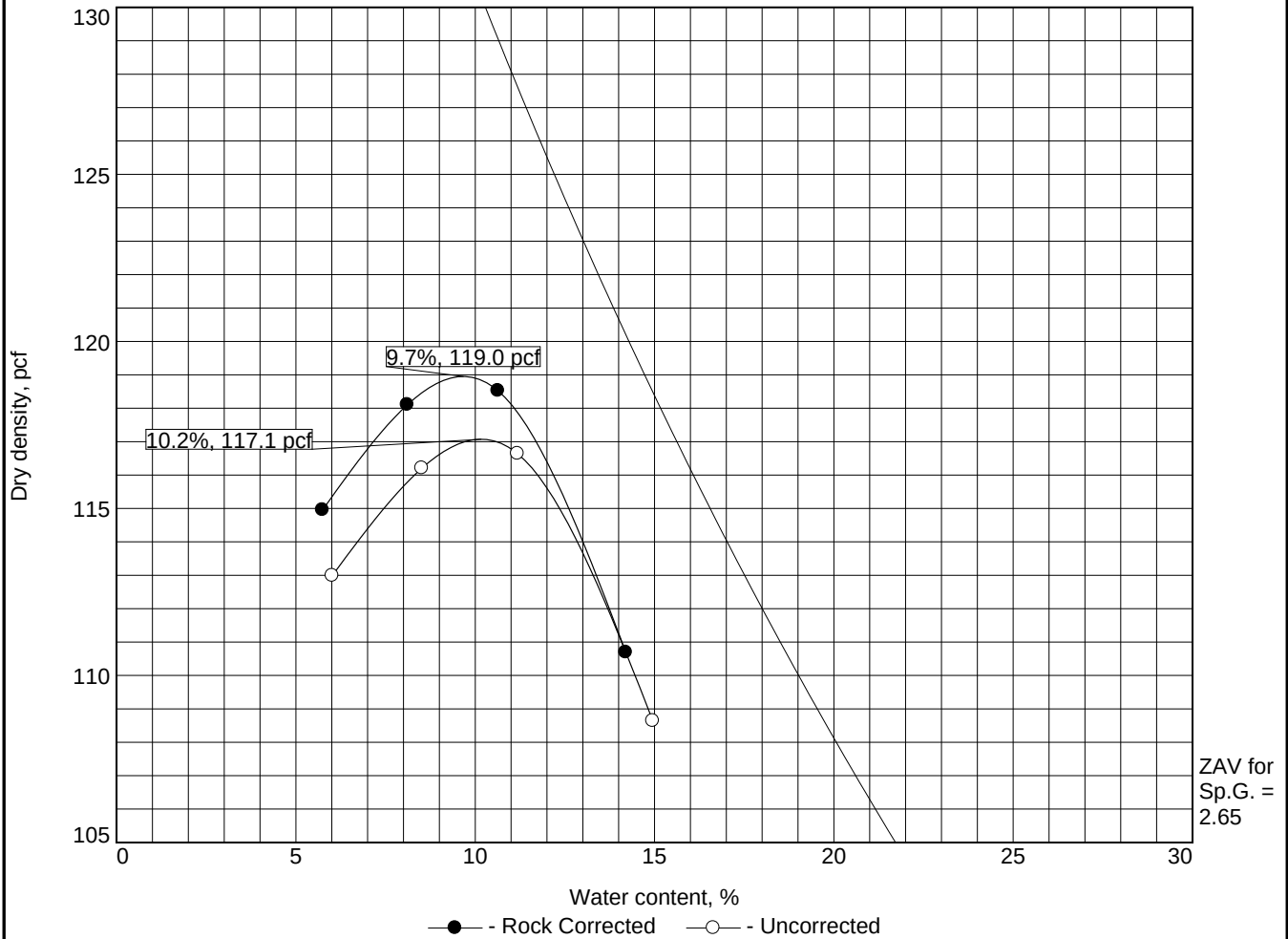
Beard Realty_Rialto



APPENDIX C

LABORATORY TEST RESULTS

COMPACTION TEST REPORT For Curve No. TP-1, 0-4ft



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

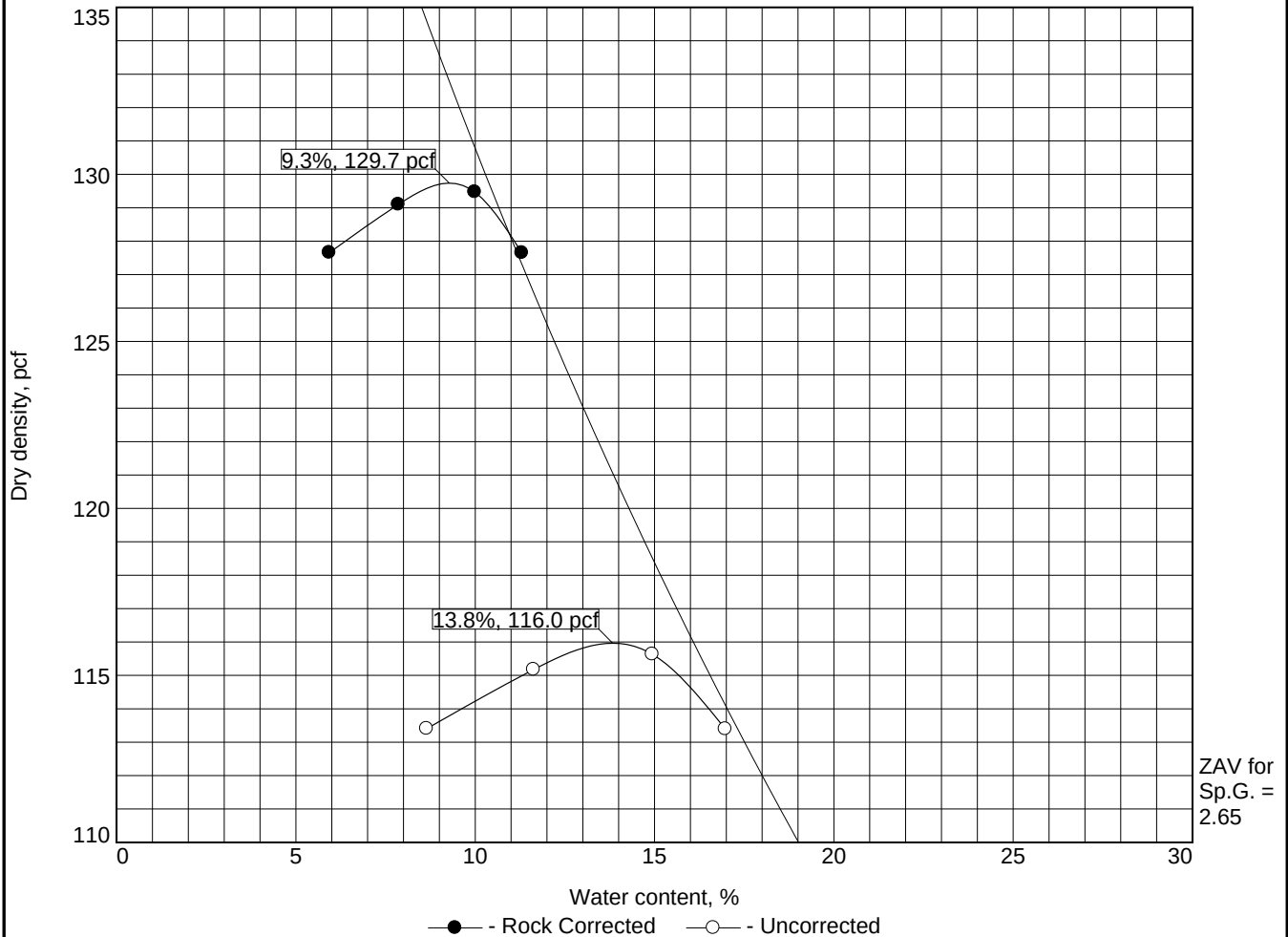
Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
0-4	SM			2.65			5.4	

ROCK CORRECTED TEST RESULTS		UNCORRECTED	MATERIAL DESCRIPTION
Maximum dry density = 119.0 pcf		117.1 pcf	Dark Olive Gray Silty Sand
Optimum moisture = 9.7 %		10.2 %	
Project No. 8377-A-SC Client: Beard Realty Project: Rialto 4-acres Date: 7-29-22 Source of Sample: TP-1 Sample Number: TP-1			Remarks:
GeoSoils, Inc.			
			Plate

Plate

Tested By: TR Checked By: TR

COMPACTION TEST REPORT For Curve No. TP-1, 7-15ft



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

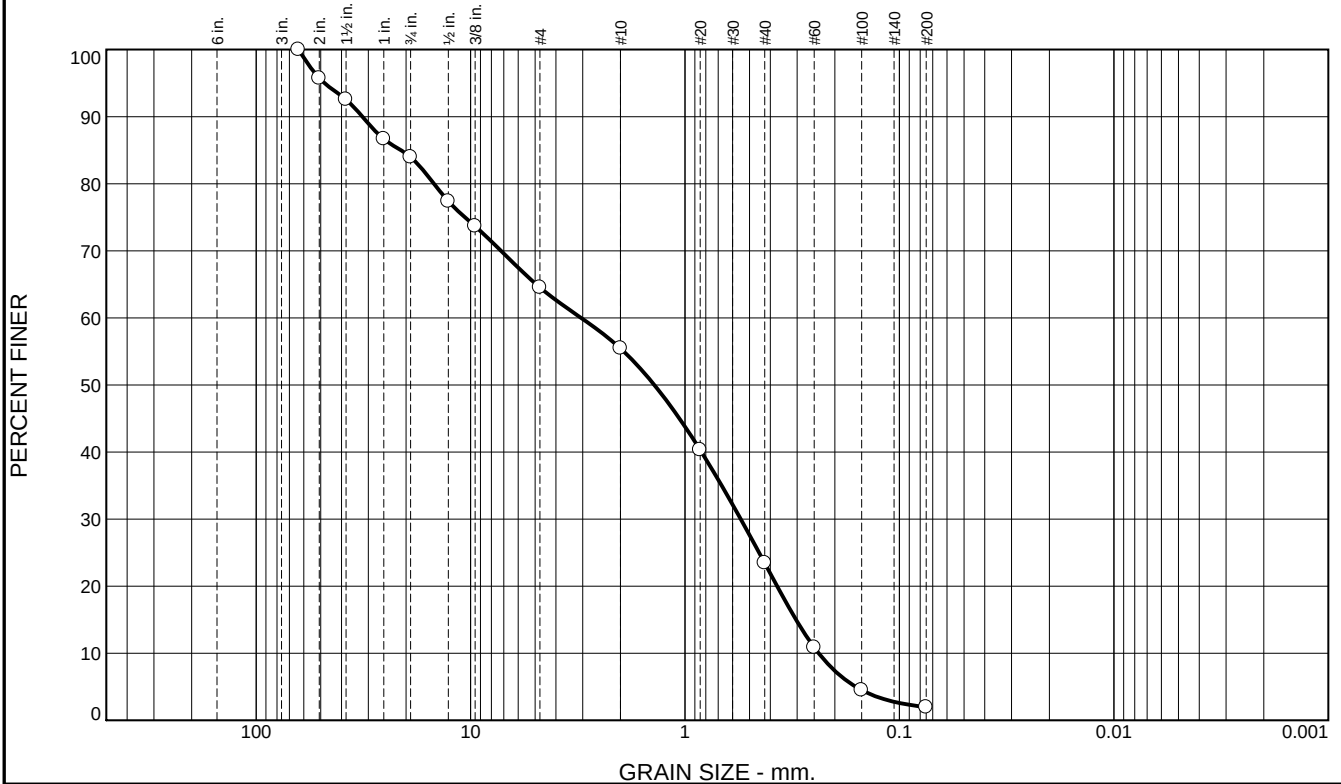
Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
7-15	SW			2.65			35.5	1.9

ROCK CORRECTED TEST RESULTS		UNCORRECTED	MATERIAL DESCRIPTION
Maximum dry density = 129.7 pcf		116.0 pcf	Gray Well Graded Sand w/Gravel
Optimum moisture = 9.3 %		13.8 %	
Project No. 8377-A-SC Client: Beard Realty Project: Rialto 4-acres Date: 8-1-22 Source of Sample: TP-1 Sample Number: TP-1			Remarks:
GeoSoils, Inc. Plate			

Plate

Tested By: TR Checked By: TR

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	16.0	19.5	9.1	32.0	21.5	1.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2.5	100.0		
2.0	95.7		
1.5	92.6		
1.0	86.7		
0.75	84.0		
0.5	77.4		
0.375	73.7		
#4	64.5		
#10	55.4		
#20	40.3		
#40	23.4		
#60	10.9		
#100	4.5		
#200	1.9		

* (no specification provided)

Material Description
Gray Well Graded Sand w/Gravel

Atterberg Limits
 PL= LL= PI=

Coefficients
 D₉₀= 31.9471 D₈₅= 21.1198 D₆₀= 3.0514
 D₅₀= 1.3902 D₃₀= 0.5497 D₁₅= 0.3038
 D₁₀= 0.2383 C_u= 12.80 C_c= 0.42

Classification
 USCS= SW AASHTO=

Remarks

Source of Sample: TP-1
Sample Number: TP-1

Depth: 7-15

Date: 7-28-22



Client: Beard Realty
Project: Rialto 4-acres

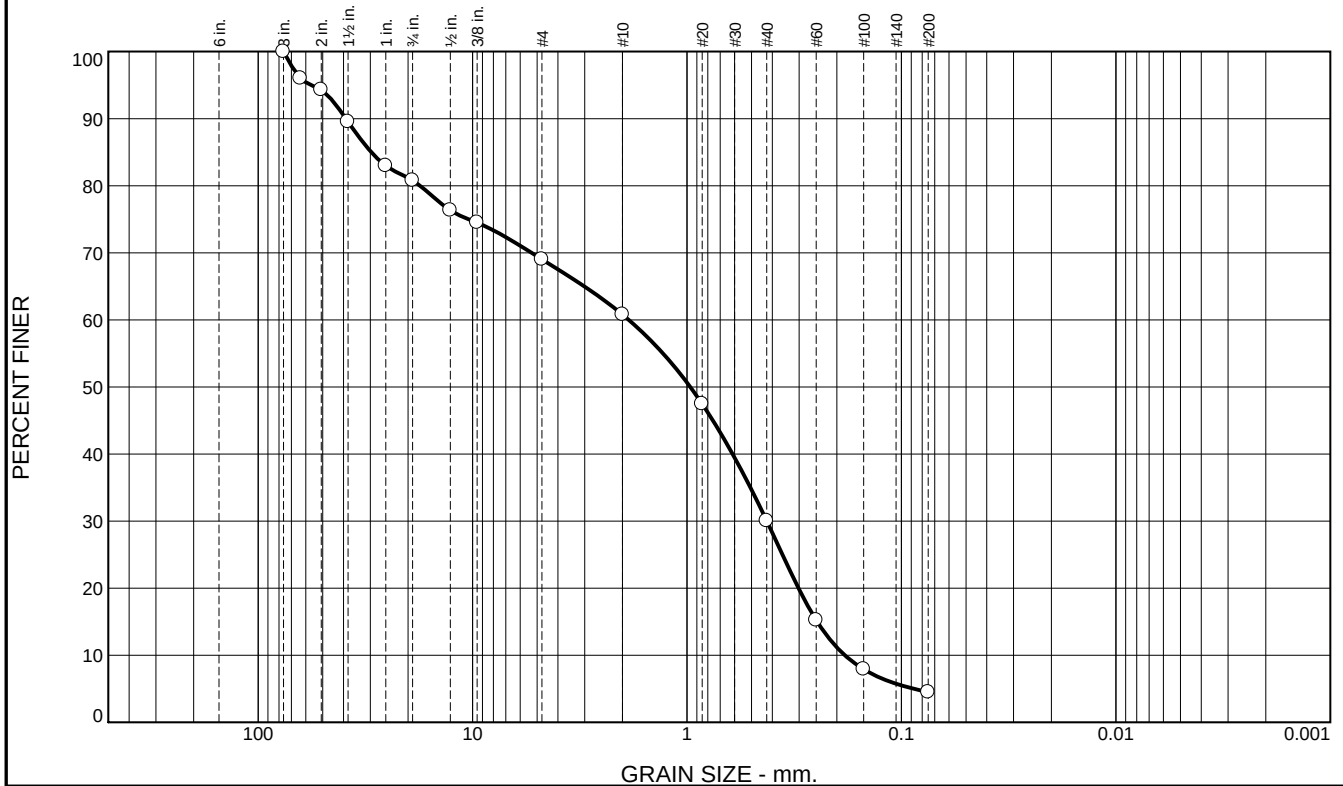
Project No: 8377-A-SC

Plate

Tested By: TR

Checked By: TR

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	19.2	11.8	8.2	30.8	25.5	4.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3	100.0		
2.5	96.0		
2.0	94.3		
1.5	89.5		
1.0	83.0		
0.75	80.8		
0.5	76.3		
0.375	74.5		
#4	69.0		
#10	60.8		
#20	47.5		
#40	30.0		
#60	15.2		
#100	7.9		
#200	4.5		

* (no specification provided)

Material Description

Gray Well-Graded Sand w/Gravel

Atterberg Limits

PL=

LL=

PI=

Coefficients

D₉₀= 39.0064

D₅₀= 0.9660

D₁₀= 0.1837

D₈₅= 29.6898

D₃₀= 0.4245

C_u= 10.17

D₆₀= 1.8685

D₁₅= 0.2474

C_c= 0.53

Classification

USCS= SW

AASHTO=

Remarks

Source of Sample: TP-2
Sample Number: TP-2

Depth: 5-7

Date: 7-28-22



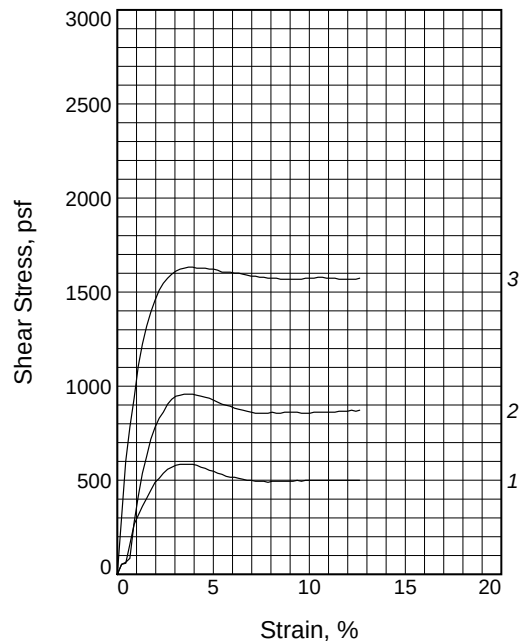
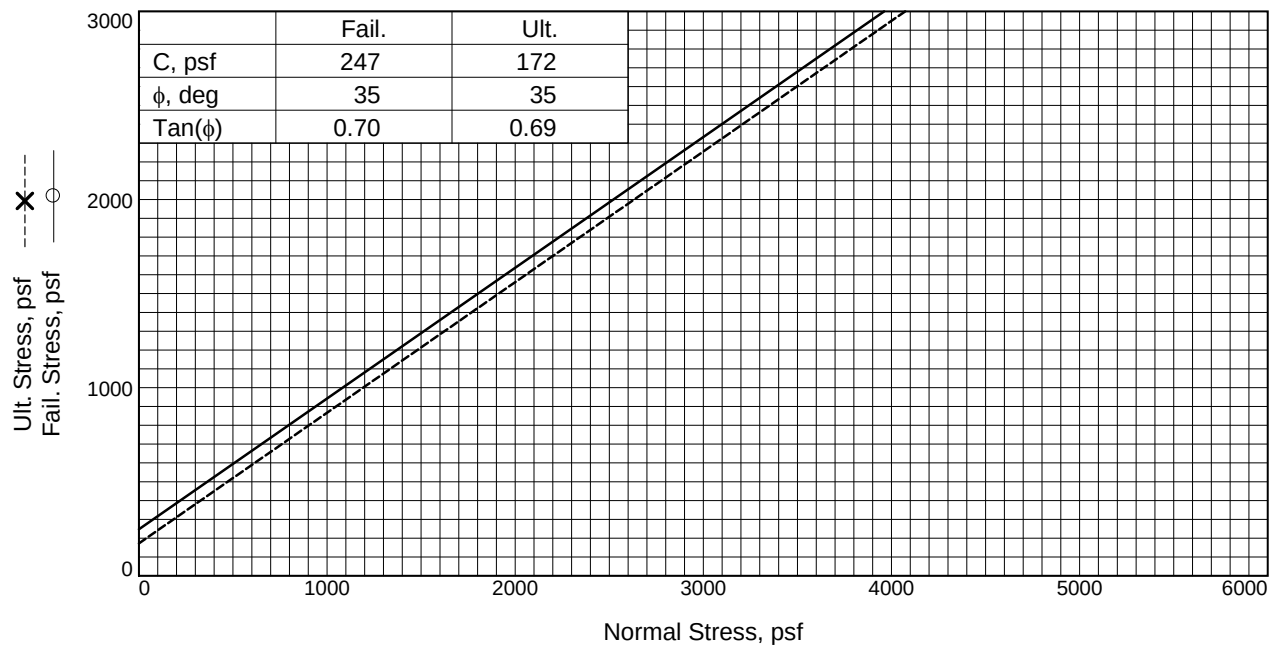
Client: Beard Realty
Project: Rialto 4-acres

Project No: 8377-A-SC

Plate

Tested By: TR

Checked By: TR



Sample No.		1	2	3
Initial	Water Content, %	10.2	10.2	10.2
	Dry Density, pcf	105.5	105.5	105.4
	Saturation, %	47.6	47.6	47.5
	Void Ratio	0.5681	0.5681	0.5692
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	18.7	18.7	18.7
	Dry Density, pcf	105.8	106.2	106.7
	Saturation, %	87.9	88.9	90.0
	Void Ratio	0.5634	0.5571	0.5504
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	0.99	0.99
Normal Stress, psf		500	1000	2000
Fail. Stress, psf		585	957	1632
Strain, %		3.5	3.7	3.7
Ult. Stress, psf		495	856	1569
Strain, %		7.2	7.2	8.7
Strain rate, in./min.		0.005	0.005	0.005

Sample Type: Remolded
Description: Dark Olive Gray Silty Sand

Specific Gravity= 2.65

Remarks:

Plate _____

Client: Beard Realty

Project: Rialto 4-acres

Source of Sample: TP-1

Depth: 0-4

Sample Number: TP-1

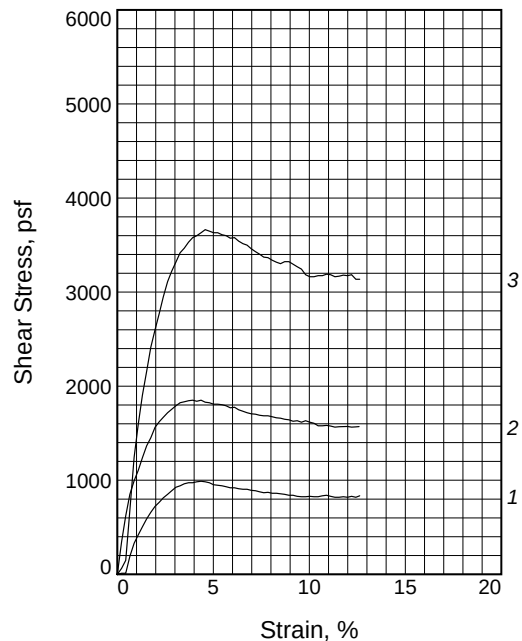
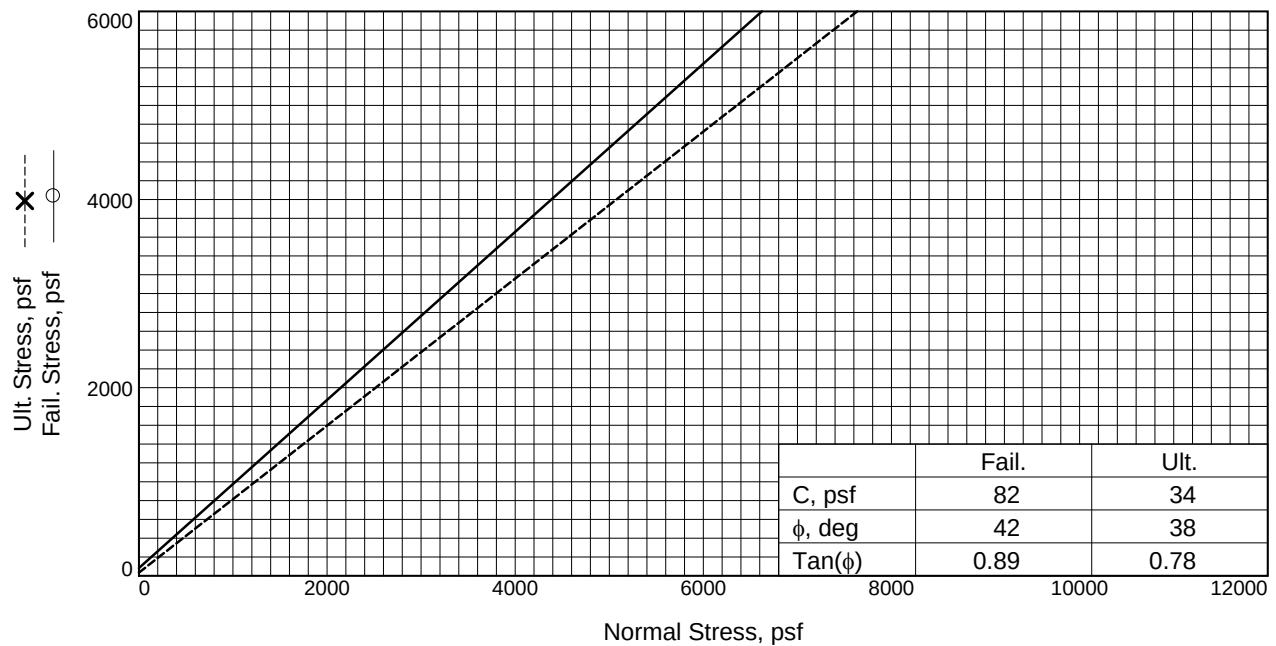
Proj. No.: 8377-A-SC

Date Sampled:

GeoSoils, Inc.

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

W.O. 8377-A-SC
 PLATE C-5



Sample No.		1	2	3
Initial	Water Content, %	13.8	13.8	13.8
	Dry Density, pcf	104.7	104.7	104.6
	Saturation, %	63.1	63.0	62.8
	Void Ratio	0.5796	0.5807	0.5818
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	18.0	17.8	17.7
	Dry Density, pcf	105.2	105.5	105.9
	Saturation, %	83.2	83.0	83.4
	Void Ratio	0.5732	0.5681	0.5629
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	0.99	0.99
Normal Stress, psf		1000	2000	4000
Fail. Stress, psf		989	1850	3664
Strain, %		4.4	3.9	4.6
Ult. Stress, psf		824	1582	3164
Strain, %		9.6	10.9	11.3
Strain rate, in./min.		0.005	0.004	0.004

Sample Type: Remolded

Description: Gray Well Graded Sand w/Gravel

Specific Gravity: 2.65

Remarks:

Plate _____

Client: Beard Realty

Project: Rialto 4-acres

Source of Sample: TP-1

Depth: 7-15

Sample Number: TP-1

Proj. No.: 8377-A-SC

Date Sampled: 7-28-22

GeoSoils, Inc.

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

W.O. 8377-A-SC
PLATE C-6

TEST SPECIMEN		A	B	C	D
Compactor air pressure	PSI	350	350	350	
Water added	%	2.3	2.7	3.4	
Moisture at compaction	%	10.3	10.7	11.4	
Height of sample	IN	2.7	2.68	2.67	
Dry density	PCF	117.6	116.1	115.5	
R-Value by exudation		80	72	71	
R-Value by exudation, corrected		82	76	74	
Exudation pressure	PSI	780	370	220	
Stability thickness	FT	0.26	0.36	0.37	
Expansion pressure thickness	FT	0.73	0.50	0.27	

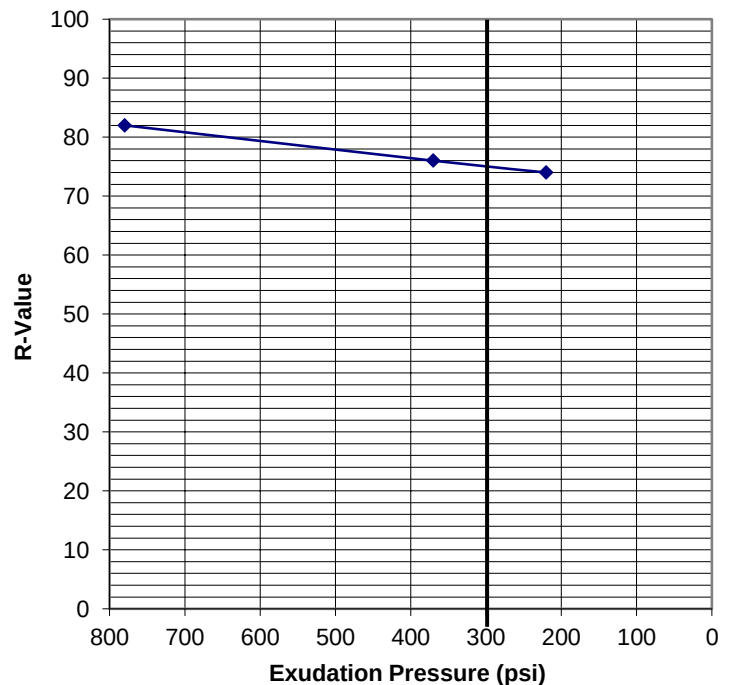
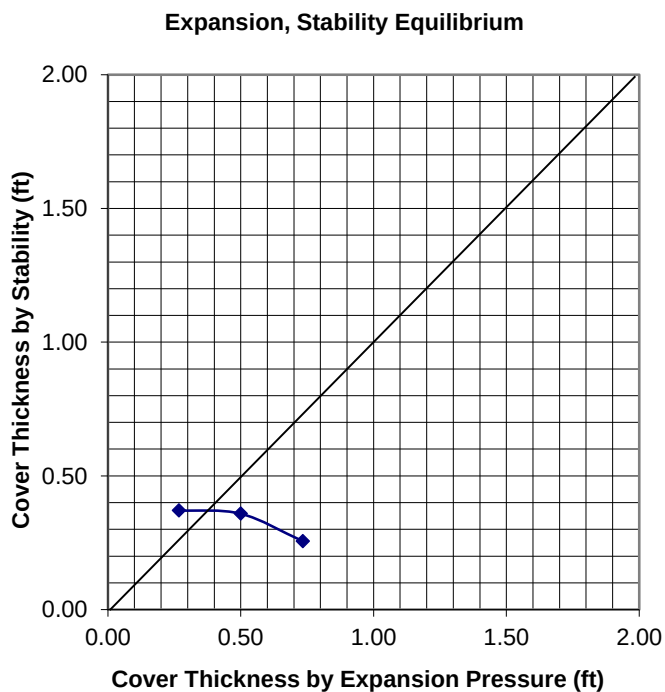
DESIGN CALCULATION DATA

Traffic index, assumed	5.0
Gravel equivalent factor, assumed	1.25
Expansion, stability equilibrium	0.36
R-Value by expansion	72
R-Value by exudation	75
R-Value at equilibrium	72

SAMPLE INFORMATION

Sample Location:	TP-1, 0-4ft
Sample Description:	Brown Silty Sand
Notes:	-
	1% Retained on 3/4 inch sieve
Test Method:	Cal-Trans Test 301

R-Value By Exudation



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

9/2/2010

R - VALUE TEST RESULTS

Project:	Beard Realty	W.O. 8377-A-SC
Number:	8377-A-SC	PLATE C-7
Date:	July 2022	Plate: 1



5741 Palmer Way, Carlsbad CA 92010

Phone (760) 438-3155

CORROSION REPORT SUMMARY

Project No: 8377-A-SC

Project Name: Beard Realty

Report Date: June 24, 2022

SAMPLE ID	pH (H+)	Minimum Resistivity (ohm/cm)	Sulfate Content (wt%)	Chloride Content (mg/kg)
TP-1, 0-4ft	6.0	11000	0.006	53

Samples testing in accordance with:

pH - CTM 643, Resistivity - CTM 643

Sulfate - CTM 417, Chloride - CTM 422

Remarks: _____

APPENDIX D

SLOPE STABILITY ANALYSIS

APPENDIX D

2-DIMENSIONAL SLOPE STABILITY ANALYSIS INTRODUCTION OF GSTABL7 v.2 COMPUTER PROGRAM

Introduction

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

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

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

General Information

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

1. The Stability of Slopes, by E.N. Bromhead, Surrey University Press, Chapman and Hall, N.Y., 411 pages, ISBN 412 01061 5, 1992.
2. Rock Slope Engineering, by E. Hoek and J.W. Bray, Inst. of Mining and Metallurgy, London, England, Third Edition, 358 pages, ISBN 0 900488 573, 1981.
3. Landslides: Analysis and Control, by R.L. Schuster and R.J. Krizek (editors), Special Report 176, Transportation Research Board, National Academy of Sciences, 234 pages, ISBN 0 309 02804 3, 1978.

GSTABL7 v.2 Features

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

1. Allows user to calculate factors of safety for static stability and dynamic stability situations.
2. Allows user to analyze stability situations with different failure modes.
3. Allows user to edit input for slope geometry and calculate corresponding factor of safety.
4. Allows user to readily review on-screen the input slope geometry.
5. Allows user to automatically generate and analyze unlimited number of circular, non-circular and block-shaped failure surfaces (i.e., bedding plane, slide plane, etc.).

Input Data

Input data includes the following items:

1. Unit weight, residual cohesion, residual friction angle, peak cohesion, and peak friction angle of fill material, bedding plane, and bedrock, respectively. Residual cohesion and friction angle is used for static stability analysis, where as peak cohesion and friction angle is for dynamic stability analysis.
2. Slope geometry and surcharge boundary loads.
3. Apparent dip of bedding plane can be specified in angular range (i.e., from 0 to 90 degrees).
4. Pseudo-static earthquake loading (an earthquake loading of 0.15 g was used in the analysis).

Seismic Discussion

Seismic stability analyses were approximated using a pseudo-static approach. The major difficulty in the pseudo-static approach arises from the appropriate selection of the seismic coefficient used in the analysis. The use of a static inertia force equal to this acceleration during an earthquake (rigid-body response) would be conservative for several reasons including: (1) only low height, stiff/dense embankments or embankments in confined areas may respond essentially as rigid structures; (2) an earthquake's inertia force is enacted on a mass for a short time period. Therefore, replacing a transient force by a pseudo-static force representing the maximum acceleration is considered unrealistic; (3) assuming that

total pseudo-static loading is applied evenly throughout the embankment for an extended period of time is an incorrect assumption, as the length of the failure surface analyzed is usually much greater than the wave length of seismic waves generated by earthquakes; and (4) the seismic waves would place portions of the mass in compression and some in tension, resulting in only a limited portion of the failure surface analyzed moving in a downslope direction, at any one instant of time.

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

The method developed by Krinitzsky, Gould, and Edinger (1993) which was in turn based on Taniguchi and Sasaki (T&S, 1986), was referenced. This method is based on empirical data and the performance of existing earth embankments during seismic loading. Our review of "Guidelines for Evaluating and Mitigating Seismic Hazards in California (Davis, 1997) indicates the State of California recommends using pseudo-static coefficient of 0.2 / for design earthquakes of M 8.25 or greater and using 0.1 / for earthquake parameter M 6.5. Therefore, for conservatism a seismic coefficient of 0.15 / was used in our analysis.

Output Information

Output information includes:

1. All input data.
2. Factors of safety for the ten most critical surfaces for static and pseudo-static stability situation.
3. High quality plots can be generated. The plots include the slope geometry, the critical surfaces and the factor of safety.
4. Note, that in the analysis, a minimum of 100 trial surfaces were analyzed for each section for either static or pseudo-static analyses.

Results of Slope Stability Analyses

Table D-1 shows parameters utilized in slope stability calculations. Summaries of the slope stability analysis are presented in Table D-2. Detailed output information is presented on Plates D-1 through D-4. A representative geologic cross-section is presented on Plate D-5.

TABLE D-1

SOIL PARAMETERS USED

EARTH MATERIALS	UNIT WEIGHT (MOIST)	UNIT WEIGHT (SATURATED)	C (psf)	Φ (degrees)
Quaternary Young Alluvial Fan Deposits (Qyf ₃)	125	130	172	35
Quaternary Young Alluvial Fan Deposits (Qyf ₄)	127	131	34	38

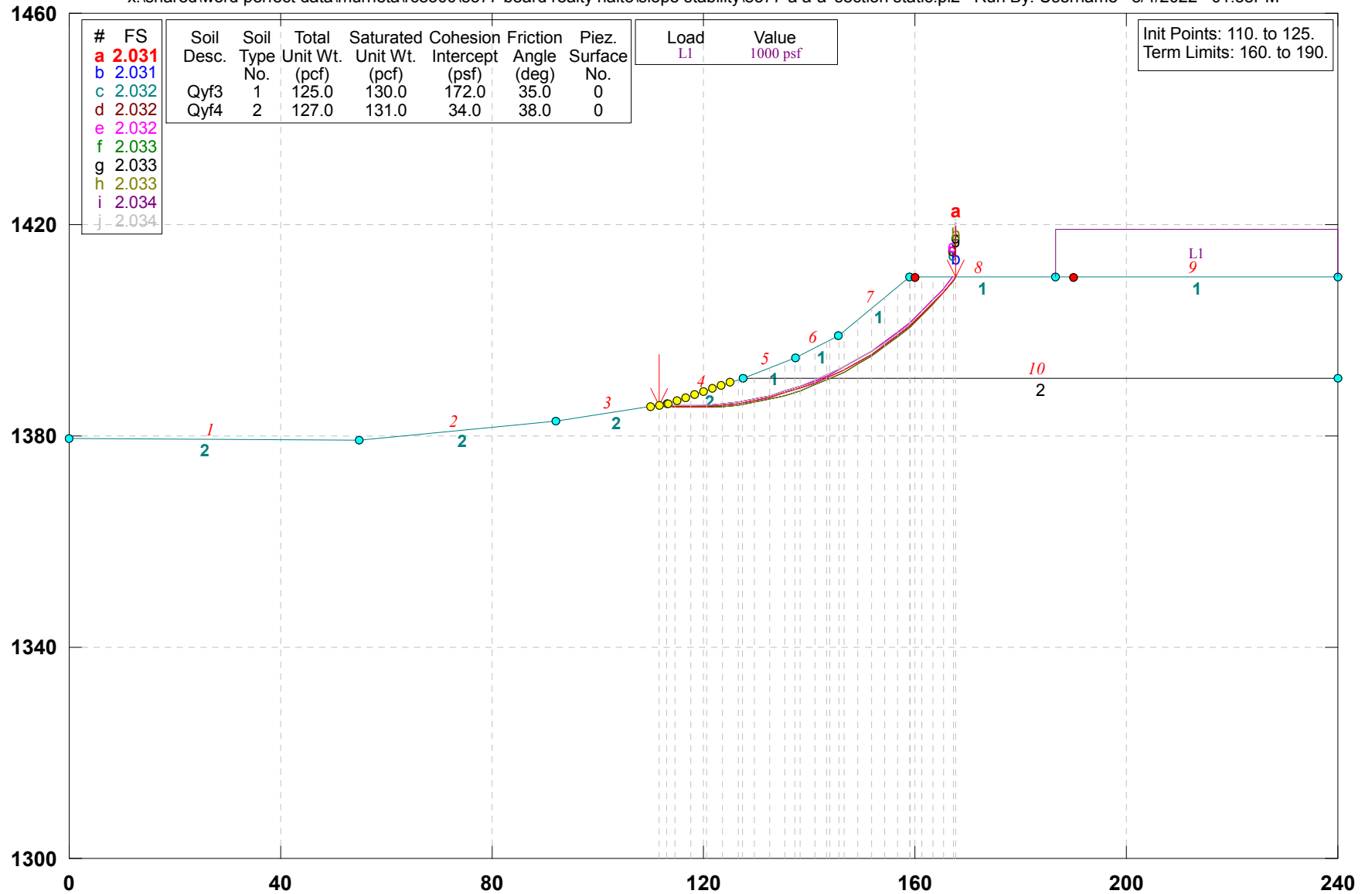
TABLE D-2

SUMMARY OF STABILITY ANALYSIS

STABILITY	SLOPE CONFIGURATION	FACTORS OF SAFETY		REMARKS
		STATIC	SEISMIC	
Cross-Section A - A'	Natural Slope 30-Foot 2:1 (h:v)	2.03	1.48	Bishop Method
Qyf ₃ (Unit 3)	Natural Slope 2:1 (h:v)	1.56	---	Surficial
Qyf ₄ (Unit 4)	Natural Slope 2:1 (h:v)	0.96	---	Surficial

8377-A BEARD REALTY SECTION A-A' STATIC

x:\shared\word perfect data\murrieta\rc8300\8377 beard realty rialto\slope stability\8377-a a-a' section static.pl2 Run By: Username 8/4/2022 01:38PM

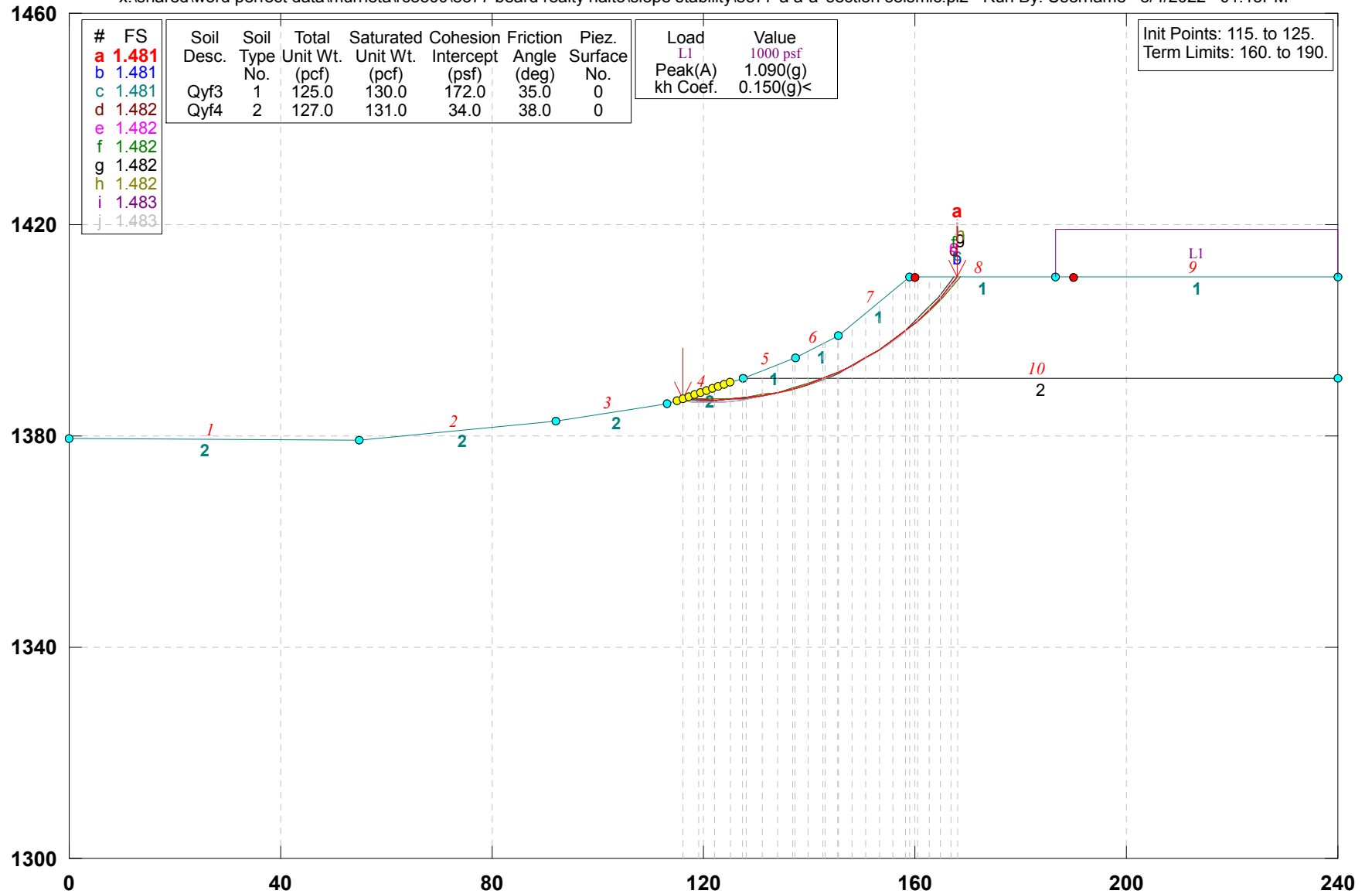


GSTABL7 v.2 FSmin=2.031
Safety Factors Are Calculated By The Modified Bishop Method

W.O. 8377-A-SC
Plate D-1

8377-A BEARD REALTY SECTION A-A' SEISMIC

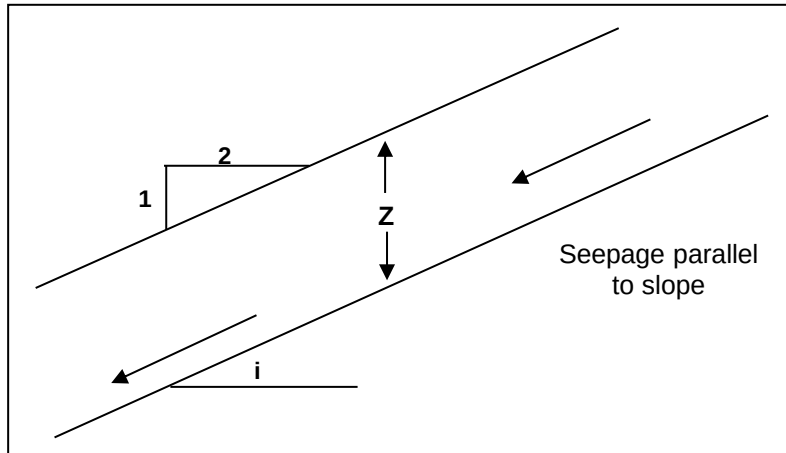
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GSTABL7 v.2 FSmin=1.481
Safety Factors Are Calculated By The Modified Bishop Method

W.O. 8377-A-SC
Plate D-2

SURFICIAL SLOPE STABILITY ANALYSIS



Tract/Project:	Beard Realty, Rialto
Material Type:	Native 2:1 Slope Configuration
	Qyf3 (Unit 3)

Depth of Saturation (z)	4	feet
Slope Angle (i) (for 2:1 slopes)	26.6	degrees
Unit Weight of Water (γ_w)	62.4	lb/ft ³
Saturated Unit Weight of Soil (γ_{sat})	127	lb/ft ³
Apparent Angle of Internal Friction (ϕ)	35	degrees
Apparent Cohesion (C)	172	lb/ft ²

$$F_s = \text{Static Safety Factor} = \frac{z (\gamma_{sat} - \gamma_w) \cos^2(i) \tan(\phi) + C}{z (\gamma_{sat}) \sin(i) \cos(i)}$$

DEPTH OF SATURATION	SLOPE	FACTOR OF SAFETY
4 FEET	2:1	1.56



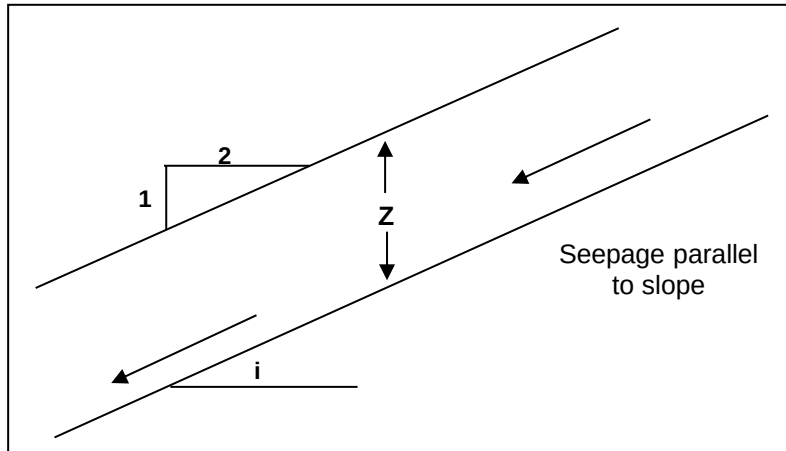
W.O. 8377-A-SC

SURFICIAL SLOPE STABILITY

2: 1 SLOPE

Plate D-3

SURFICIAL SLOPE STABILITY ANALYSIS



Tract/Project:	Beard Realty, Rialto
Material Type:	Native 2:1 Slope Configuration
	Qyf (Unit 4)

Depth of Saturation (z)	4	feet
Slope Angle (i) (for 2:1 slopes)	26.6	degrees
Unit Weight of Water (γ_w)	62.4	lb/ft ³
Saturated Unit Weight of Soil (γ_{sat})	127	lb/ft ³
Apparent Angle of Internal Friction (ϕ)	38	degrees
Apparent Cohesion (C)	34	lb/ft ²

$$F_s = \text{Static Safety Factor} = \frac{z (\gamma_{sat} - \gamma_w) \cos^2(i) \tan(\phi) + C}{z (\gamma_{sat}) \sin(i) \cos(i)}$$

DEPTH OF SATURATION	SLOPE	FACTOR OF SAFETY
4 FEET	2:1	0.96



W.O. 8377-A-SC

SURFICIAL SLOPE STABILITY

2: 1 SLOPE

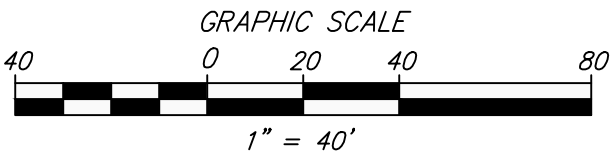
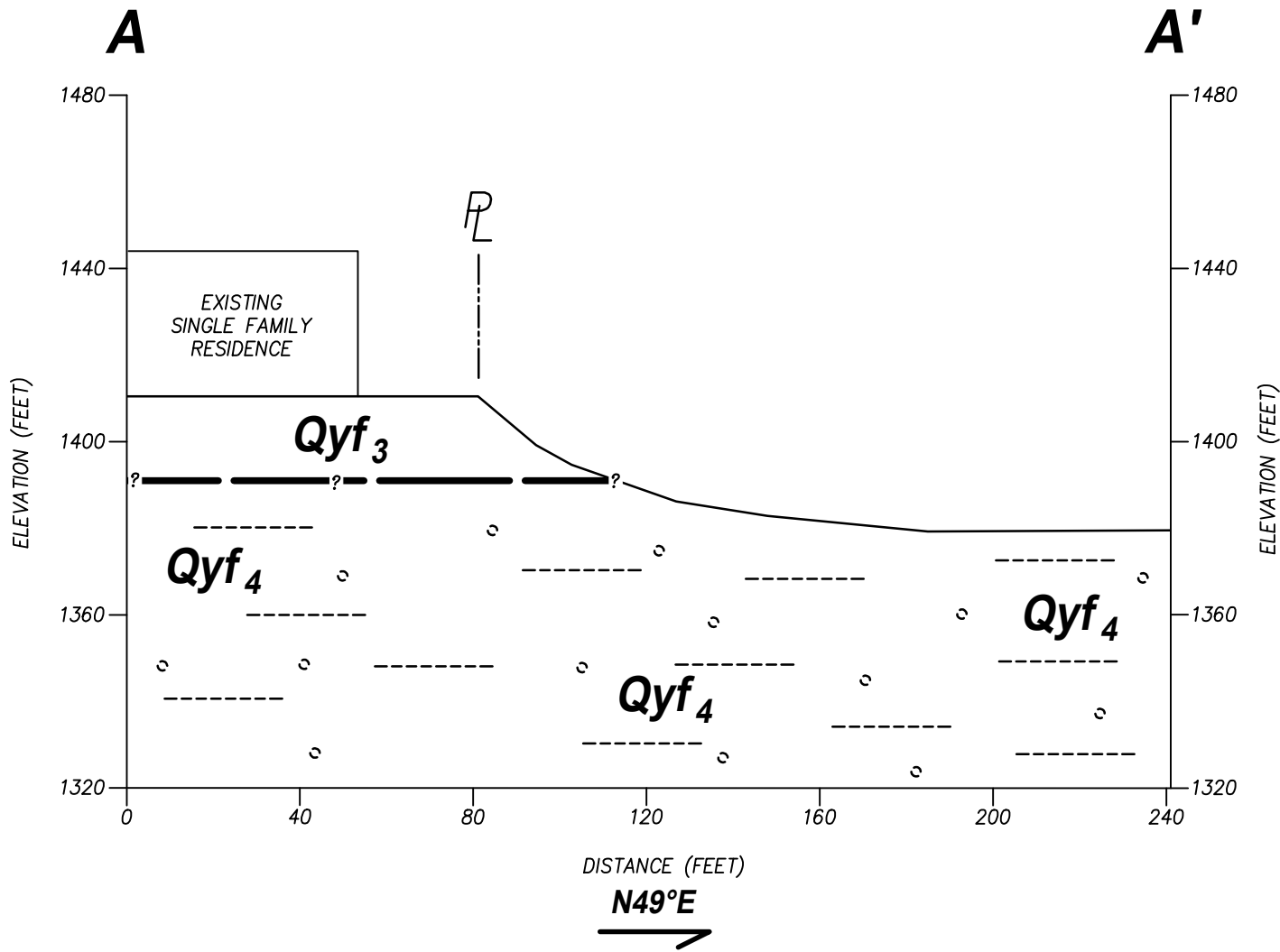
Plate D-4

GSI LEGEND

Qyf₃ — QUATERNARY YOUNG ALLUVIAL FAN DEPOSITS – UNIT 3

Qyf₄ — QUATERNARY YOUNG ALLUVIAL FAN DEPOSITS – UNIT 4

—?— — APPROXIMATE LOCATION OF GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN



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.



**GEOLOGIC
CROSS SECTION A-A'**

Plate D-5

W.O. 8377-A-SC | DATE: 08/22 | SCALE: 1" = 40'

APPENDIX E

GENERAL EARTHWORK, GRADING GUIDELINES AND PRELIMINARY CRITERIA

GENERAL EARTHWORK, GRADING GUIDELINES, AND PRELIMINARY CRITERIA

General

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

The contractor is responsible for the satisfactory completion of all earthwork in accordance with provisions of the project plans and specifications and latest adopted Code. In the case of conflict, the most onerous provisions shall prevail. The project geotechnical engineer and engineering geologist (geotechnical consultant), 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 $\frac{1}{2}$ 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 used 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 used as fill material for the subject project may contain some rock. Appropriately, the need for rock disposal may be necessary during grading operations on the site. From a geotechnical standpoint, the depth of any rocks, rock fills, or rock blankets, should be a sufficient distance from finish grade. This depth is generally the same as any overexcavation due to cut-fill transitions in hard rock areas, and generally facilitates the excavation of structural footings and substructures. Should deeper excavations be proposed (i.e., deepened footings, utility trenching, swimming pools, spas, etc.), the developer may consider increasing the hold-down depth of any rocky fills to be placed, as appropriate. In addition, some agencies/jurisdictions mandate a specific hold-down depth for oversize materials placed in fills. The hold-down depth, and potential to encounter oversize rock, both within fills, and occurring in cut or natural areas, would need to be disclosed to all interested/affected parties. Once approved by the governing agency, the hold-down depth for oversized rock (i.e., greater than 12 inches) in fills on this project is provided as 10 feet, unless specified differently in the text of this report. The governing agency may require that these materials need to be deeper, crushed, or reduced to less than 12 inches in maximum dimension, at their discretion.

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

If import material is required for grading, representative samples of the materials to be used as compacted fill should be analyzed in the laboratory by the geotechnical consultant to evaluate its 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 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 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 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:

2. An extra piece of equipment consisting of a heavy, short-shanked sheepfoot should be used to roll (horizontal) parallel to the slopes continuously as fill is placed. The sheepfoot 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, during compaction operations.
4. After completion of the slope, the slope face should be shaped with a small tractor and then grid-rolled with a sheepsfoot 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 or moisture content, the contractor will be responsible to rip, water, mix, and recompact the slope material as necessary to achieve compaction and adequate moisture conditions. 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, 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, 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 or be planted in accordance with the project specifications or as recommended by a landscape architect. Such protection or planning should be undertaken as soon as practical after completion of grading.

PRELIMINARY OUTDOOR POOL/SPA DESIGN RECOMMENDATIONS

The following preliminary recommendations are provided for consideration in pool/spa design and planning. Actual recommendations should be provided by a qualified geotechnical consultant, based on site specific geotechnical conditions, including a subsurface investigation, differential settlement potential, expansive and corrosive soil potential, proximity of the proposed pool/spa to any slopes with regard to slope creep and lateral fill extension, as well as slope setbacks per Code, and geometry of the proposed improvements. Recommendations for pools/spas or deck flatwork underlain by expansive soils, or for areas with differential settlement greater than 1/4-inch over 40 feet horizontally, will be more onerous than the preliminary recommendations presented below.

The 1:1 (h:v) influence zone of any nearby retaining wall site structures should be delineated on the project civil drawings with the pool/spa. This 1:1 (h:v) zone is defined as a plane up from the lower-most heel of the retaining structure, to the daylight grade of

the nearby building pad or slope. If pools/spas or associated pool/spa improvements are constructed within this zone, they should be re-positioned (horizontally or vertically) so that they are supported by earth materials that are outside or below this 1:1 plane. If this is not possible given the area of the building pad, the owner should consider eliminating these improvements or allow for increased potential for lateral/vertical deformations and associated distress that may render these improvements unusable in the future, unless they are periodically repaired and maintained. The conditions and recommendations presented herein should be disclosed to all homeowners and any interested/affected parties.

General

1. The equivalent fluid pressure to be used for the pool/spa design should be 60 pounds per cubic foot (pcf) for pool/spa walls with level backfill, and 75 pcf for a 2:1 sloped backfill condition. In addition, backdrains should be provided behind pool/spa walls subjacent to slopes.
2. Passive earth pressure may be computed as an equivalent fluid having a density of 150 pcf, to a maximum lateral earth pressure of 1,000 pounds per square foot (psf).
3. An allowable coefficient of friction between soil and concrete of 0.30 may be used with the dead load forces.
4. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.
5. Where pools/spas are planned near structures, appropriate surcharge loads need to be incorporated into design and construction by the pool/spa designer. This includes, but is not limited to landscape berms, decorative walls, footings, built-in barbeques, utility poles, etc.
6. All pool/spa walls should be designed as “free standing” and be capable of supporting the water in the pool/spa without soil support. The shape of pool/spa in cross section and plan view may affect the performance of the pool, from a geotechnical standpoint. Pools and spas should also be designed in accordance with the latest adopted Code. Minimally, the bottoms of the pools/spas, should maintain a distance $H/3$, where H is the height of the slope (in feet), from the slope face. This distance should not be less than 7 feet, nor need not be greater than 40 feet.
7. The soil beneath the pool/spa bottom should be uniformly moist with the same stiffness throughout. If a fill/cut transition occurs beneath the pool/spa bottom, the cut portion should be overexcavated to a minimum depth of 48 inches, and replaced with compacted fill, such that there is a uniform blanket that is a minimum of 48 inches below the pool/spa shell. If very low expansive soil is used for fill, the

fill should be placed at a minimum of 95 percent relative compaction, at optimum moisture conditions. This requirement should be 90 percent relative compaction at over optimum moisture if the pool/spa is constructed within or near expansive soils. The potential for grading or re-grading of the pool/spa bottom, and attendant potential for shoring or slot excavation, needs to be considered during all aspects of pool/spa planning, design, and construction.

8. If the pool/spa is founded entirely in compacted fill placed during rough grading, the deepest portion of the pool/spa should correspond with the thickest fill on the lot.
9. Hydrostatic pressure relief valves should be incorporated into the pool and spa designs. A pool/spa under-drain system is also recommended, with an appropriate outlet for discharge.
10. All fittings and pipe joints, particularly fittings in the side of the pool or spa, should be properly sealed to prevent water from leaking into the adjacent soils materials, and be fitted with slip or expandible joints between connections transecting varying soil conditions.
11. An elastic expansion joint (flexible waterproof sealant) should be installed to prevent water from seeping into the soil at all deck joints.
12. A reinforced grade beam should be placed around skimmer inlets to provide support and mitigate cracking around the skimmer face.
13. In order to reduce unsightly cracking, deck slabs should minimally be 4 inches thick, and reinforced with No. 3 reinforcing bars at 18 inches on-center. All slab reinforcement should be supported to ensure proper mid-slab positioning during the placement of concrete. Wire mesh reinforcing is specifically not recommended. Deck slabs should not be tied to the pool/spa structure. Pre-moistening or pre-soaking of the slab subgrade is recommended, to a depth of 12 inches (optimum moisture content), or 18 inches (120 percent of the soil's optimum moisture content, or 3 percent over optimum moisture content, whichever is greater), for very low to low, and medium expansive soils, respectively. This moisture content should be maintained in the subgrade soils during concrete placement to promote uniform curing of the concrete and minimize the development of unsightly shrinkage cracks. Slab underlayment should consist of a 1- to 2-inch leveling course of sand (S.E.>30) and a minimum of 4 to 6 inches of Class 2 base compacted to 90 percent. Deck slabs within the H/3 zone, where H is the height of the slope (in feet), will have an increased potential for distress relative to other areas outside of the H/3 zone. If distress is undesirable, improvements, deck slabs or flatwork should not be constructed closer than H/3 or 7 feet (whichever is greater) from the slope face, in order to reduce, but not eliminate, this potential.

14. Pool/spa bottom or deck slabs should be founded entirely on competent bedrock, or properly compacted fill. Fill should be compacted to achieve a minimum 90 percent relative compaction, as discussed above. Prior to pouring concrete, subgrade soils below the pool/spa decking should be thoroughly watered to achieve a moisture content that is at least 2 percent above optimum moisture content, to a depth of at least 18 inches below the bottom of slabs. This moisture content should be maintained in the subgrade soils during concrete placement to promote uniform curing of the concrete and minimize the development of unsightly shrinkage cracks.
15. In order to reduce unsightly cracking, the outer edges of pool/spa decking to be bordered by landscaping, and the edges immediately adjacent to the pool/spa, should be underlain by an 8-inch wide concrete cutoff shoulder (thickened edge) extending to a depth of at least 12 inches below the bottoms of the slabs to mitigate excessive infiltration of water under the pool/spa deck. These thickened edges should be reinforced with two No. 4 bars, one at the top and one at the bottom. Deck slabs may be minimally reinforced with No. 3 reinforcing bars placed at 18 inches on-center, in both directions. All slab reinforcement should be supported on chairs to ensure proper mid-slab positioning during the placement of concrete.
16. Surface and shrinkage cracking of the finish slab may be reduced if a low slump and water-cement ratio are maintained during concrete placement. Concrete used should have a minimum compressive strength of 4,000 psi. Excessive water added to concrete prior to placement is likely to cause shrinkage cracking, and should be avoided. Some concrete shrinkage cracking, however, is unavoidable.
17. Joint and sawcut locations for the pool/spa deck should be determined by the design engineer or contractor. However, spacings should not exceed 6 feet on center.
18. Considering the nature of the onsite earth materials, it should be anticipated that caving or sloughing could be a factor in subsurface excavations and trenching. Shoring or excavating the trench walls/backcuts at the angle of repose (typically 25 to 45 degrees), should be anticipated. All excavations should be observed by a representative of the geotechnical consultant, including the project geologist or geotechnical engineer, prior to workers entering the excavation or trench, and minimally conform to Cal/OSHA ("Type C" soils may be assumed), state, and local safety codes. Should adverse conditions exist, appropriate recommendations should be offered at that time by the geotechnical consultant. GSI does not consult in the area of safety engineering and the safety of the construction crew is the responsibility of the pool/spa builder.
19. It is imperative that adequate provisions for surface drainage are incorporated by the homeowners into their overall improvement scheme. Ponding water, ground saturation and flow over slope faces, are all situations which must be avoided to enhance long term performance of the pool/spa and associated improvements, and reduce the likelihood of distress.

20. Regardless of the methods employed, once the pool/spa is filled with water, should it be emptied, there exists some potential that if emptied, significant distress may occur. Accordingly, once filled, the pool/spa should not be emptied unless evaluated by the geotechnical consultant and the pool/spa builder.
21. For pools/spas built within (all or part) of the Code setback or geotechnical setback, as indicated in the site geotechnical documents, special foundations are recommended to mitigate the affects of creep, lateral fill extension, expansive soils and settlement on the proposed pool/spa. Most municipalities or County reviewers do not consider these effects in pool/spa plan approvals. As such, where pools/spas are proposed on 20 feet or more of fill, medium or highly expansive soils, or rock fill with limited “cap soils” and built within Code setbacks, or within the influence of the creep zone, or lateral fill extension, the following should be considered during design and construction:
- OPTION A:** Shallow foundations with or without overexcavation of the pool/spa “shell,” such that the pool/spa is surrounded by 5 feet of very low to low expansive soils (without irreducible particles greater than 6 inches), and the pool/spa walls closer to the slope(s) are designed to be free standing. GSI recommends a pool/spa under-drain or blanket system (see attached Typical Pool/Spa Detail). The pool/spa builders and owner in this optional construction technique should be generally satisfied with pool/spa performance under this scenario; however, some settlement, tilting, cracking, and leakage of the pool/spa is likely over the life of the project.
- OPTION B:** Pier supported pool/spa foundations with or without overexcavation of the pool/spa shell such that the pool/spa is surrounded by 5 feet of very low to low expansive soils (without irreducible particles greater than 6 inches), and the pool/spa walls closer to the slope(s) are designed to be free standing. The need for a pool/spa under-drain system may be installed for leak detection purposes. Piers that support the pool/spa should be a minimum of 12 inches in diameter and at a spacing to provide vertical and lateral support of the pool/spa, in accordance with the pool/spa designers recommendations current applicable Codes. The pool/spa builder and owner in this second scenario construction technique should be more satisfied with pool/spa performance. This construction will reduce settlement and creep effects on the pool/spa; however, it will not eliminate these potentials, nor make the pool/spa “leak-free.”
22. The temperature of the water lines for spas and pools may affect the corrosion properties of site soils, thus, a corrosion specialist should be retained to review all spa and pool plans, and provide mitigative recommendations, as warranted. Concrete mix design should be reviewed by a qualified corrosion consultant and materials engineer.
23. All pool/spa utility trenches should be compacted to 90 percent of the laboratory standard, under the full-time observation and testing of a qualified geotechnical

consultant. Utility trench bottoms should be sloped away from the primary structure on the property (typically the residence).

24. Pool and spa utility lines should not cross the primary structure's utility lines (i.e., not stacked, or sharing of trenches, etc.).
25. The pool/spa or associated utilities should not intercept, interrupt, or otherwise adversely impact any area drain, roof drain, or other drainage conveyances. If it is necessary to modify, move, or disrupt existing area drains, subdrains, or tightlines, then the design civil engineer should be consulted, and mitigative measures provided. Such measures should be further reviewed and approved by the geotechnical consultant, prior to proceeding with any further construction.
26. The geotechnical consultant should review and approve all aspects of pool/spa and flatwork design prior to construction. A design civil engineer should review all aspects of such design, including drainage and setback conditions. Prior to acceptance of the pool/spa construction, the project builder, geotechnical consultant and civil designer should evaluate the performance of the area drains and other site drainage pipes, following pool/spa construction.
27. All aspects of construction should be reviewed and approved by the geotechnical consultant, including during excavation, prior to the placement of any additional fill, prior to the placement of any reinforcement or pouring of any concrete.
28. Any changes in design or location of the pool/spa should be reviewed and approved by the geotechnical and design civil engineer prior to construction. Field adjustments should not be allowed until written approval of the proposed field changes are obtained from the geotechnical and design civil engineer.
29. Disclosure should be made to homeowners and builders, contractors, and any interested/affected parties, that pools/spas built within about 15 feet of the top of a slope, or $H/3$, where H is the height of the slope (in feet), will experience some movement or tilting. While the pool/spa shell or coping may not necessarily crack, the levelness of the pool/spa will likely tilt toward the slope, and may not be esthetically pleasing. The same is true with decking, flatwork and other improvements in this zone.
30. Failure to adhere to the above recommendations will significantly increase the potential for distress to the pool/spa, flatwork, etc.
31. Local seismicity or the design earthquake will cause some distress to the pool/spa and decking or flatwork, possibly including total functional and economic loss.
32. The information and recommendations discussed above should be provided to any contractors or subcontractors, or homeowners, interested/affected parties, etc., that may perform or may be affected by such work.

JOB SAFETY

General

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

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

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

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

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

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

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

Test Pits Location, Orientation, and Clearance

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

Test pits should be excavated so that the spoil pile is placed away from oncoming traffic, whenever possible. The technician's vehicle is to be placed next to the test pit, opposite

the spoil pile. This necessitates the fill be maintained in a driveable condition. Alternatively, the contractor may wish to park a piece of equipment in front of the test holes, particularly in small fill areas or those with limited access.

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

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

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

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

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

Trench and Vertical Excavation

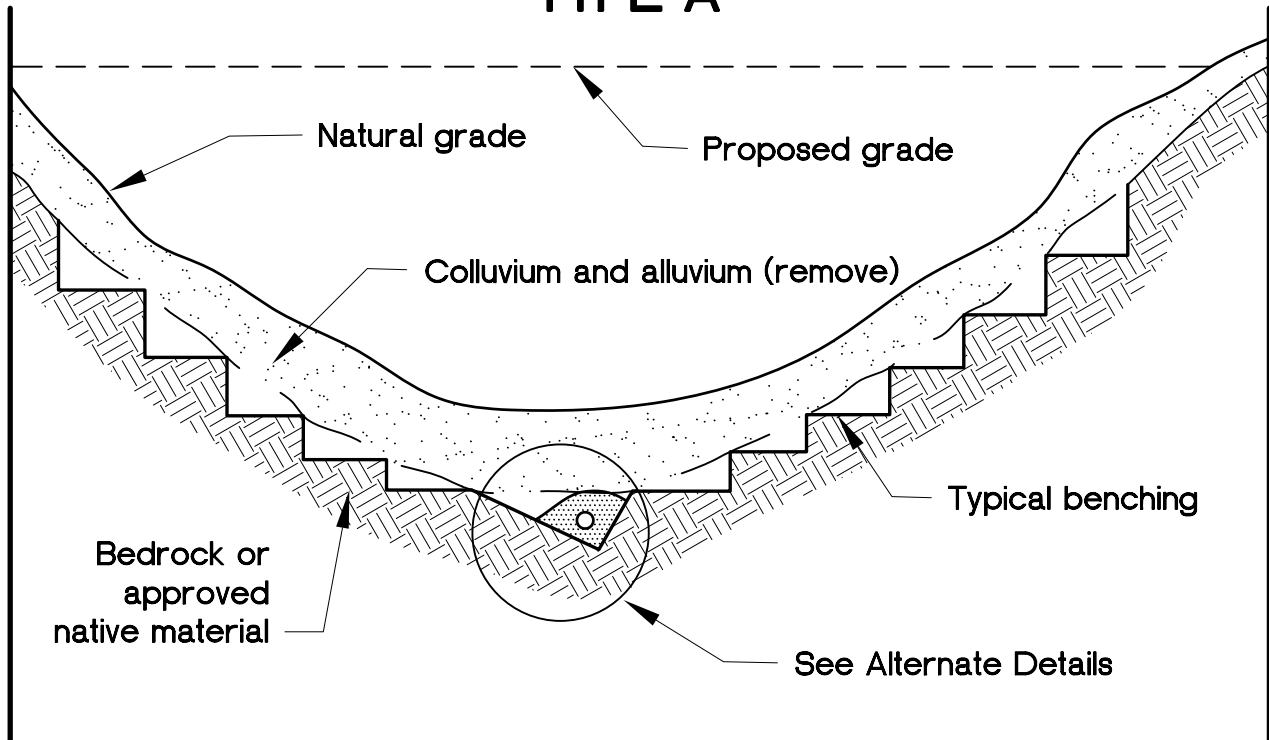
It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Our personnel are directed not to enter any excavation or vertical cut which: 1) is 5 feet or deeper unless shored or laid back; 2) displays any evidence of instability, has any loose rock or other debris which could fall into the trench; or 3) displays any other evidence of any unsafe conditions regardless of depth.

All trench excavations or vertical cuts in excess of 5 feet deep, which any person enters, should be shored or laid back. Trench access should be provided in accordance with Cal/OSHA or state and local standards. Our personnel are directed not to enter any trench by being lowered or "riding down" on the equipment.

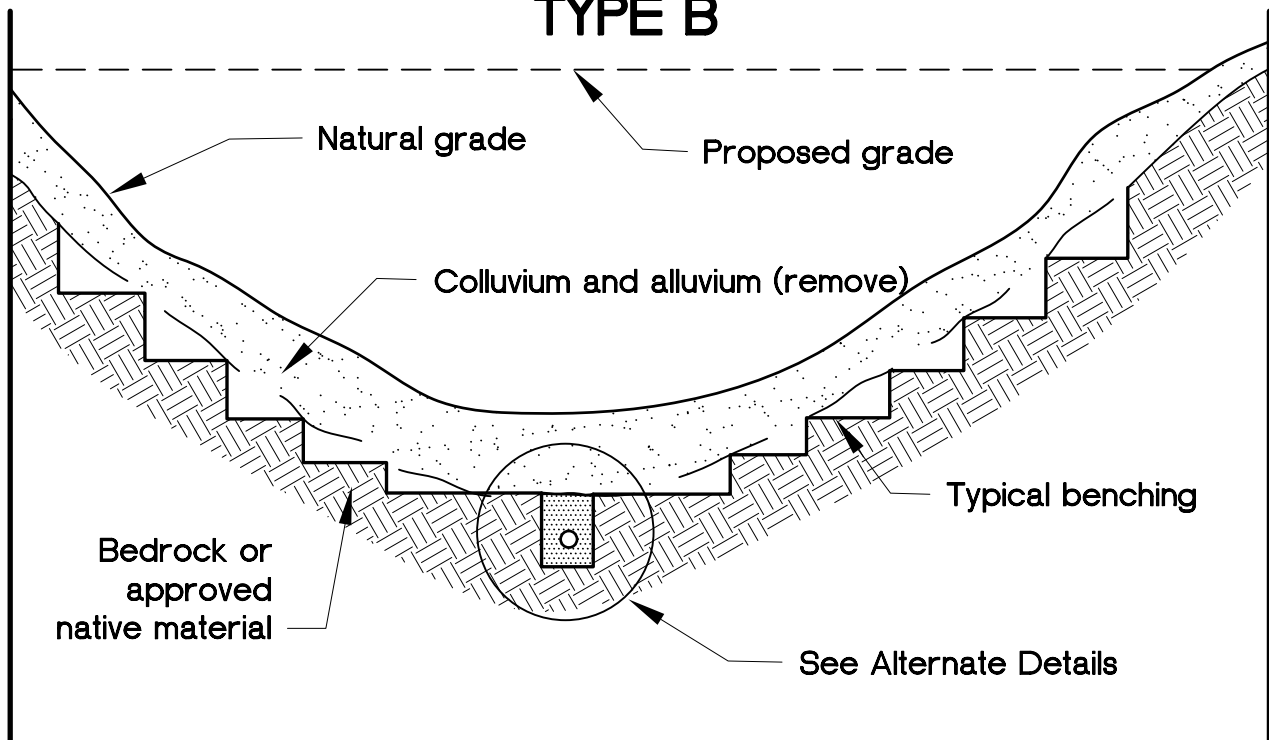
If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraw and notify his/her supervisor. The contractor's representative will be contacted in an effort to affect a solution. All backfill not tested due to safety concerns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

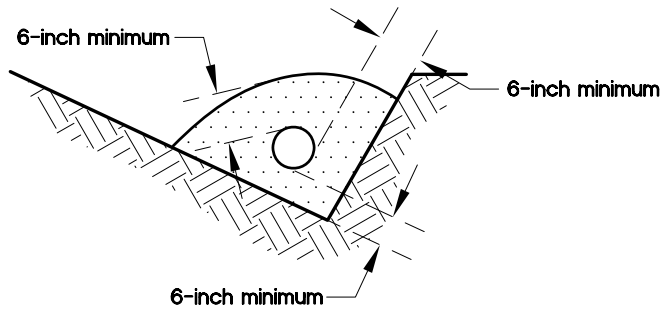
TYPE A



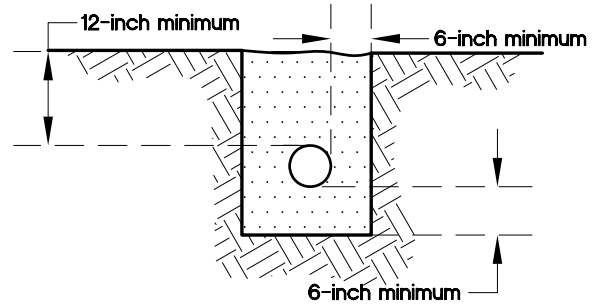
TYPE B



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



A-1



B-1

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

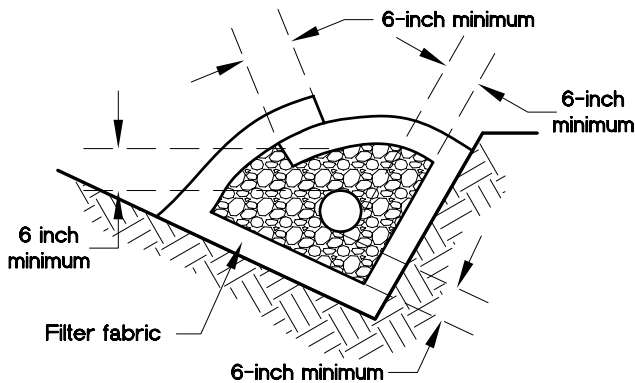
Perforated pipe: 6-inch-diameter ABS or PVC pipe or approved substitute with minimum 8 perforations ($\frac{1}{4}$ -inch diameter) per lineal foot in bottom half of pipe (ASTM D-2751, SDR-35, or ASTM D-1527, Schd. 40).

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

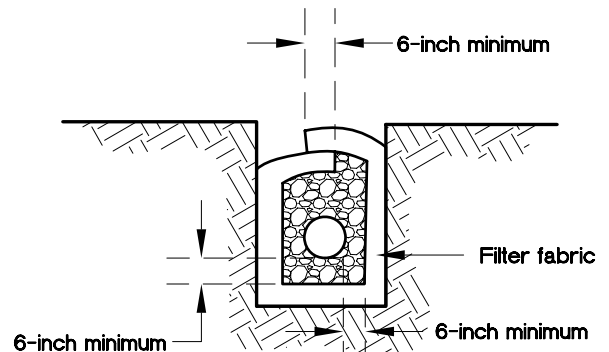
FILTER MATERIAL

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

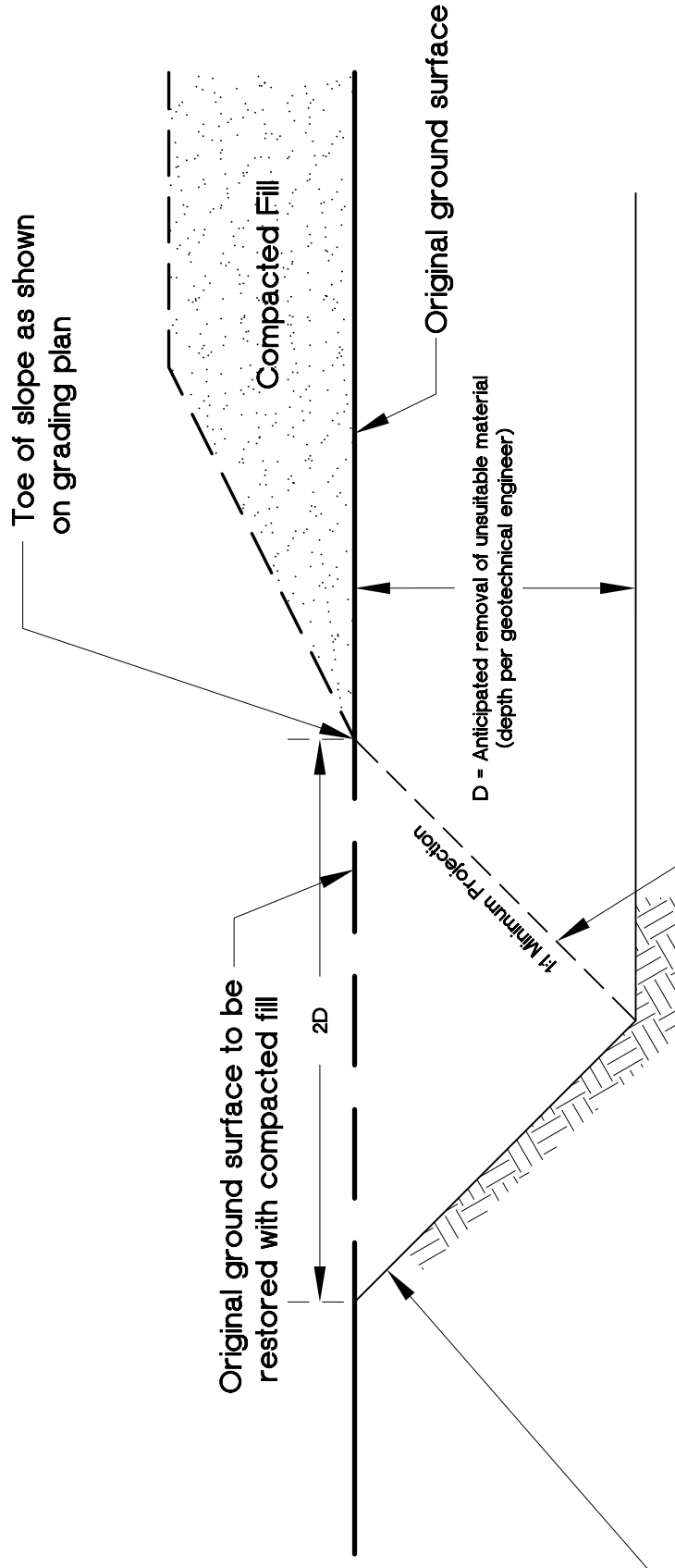
Gravel Material: 9 cubic feet per lineal foot.

Perforated Pipe: See Alternate 1

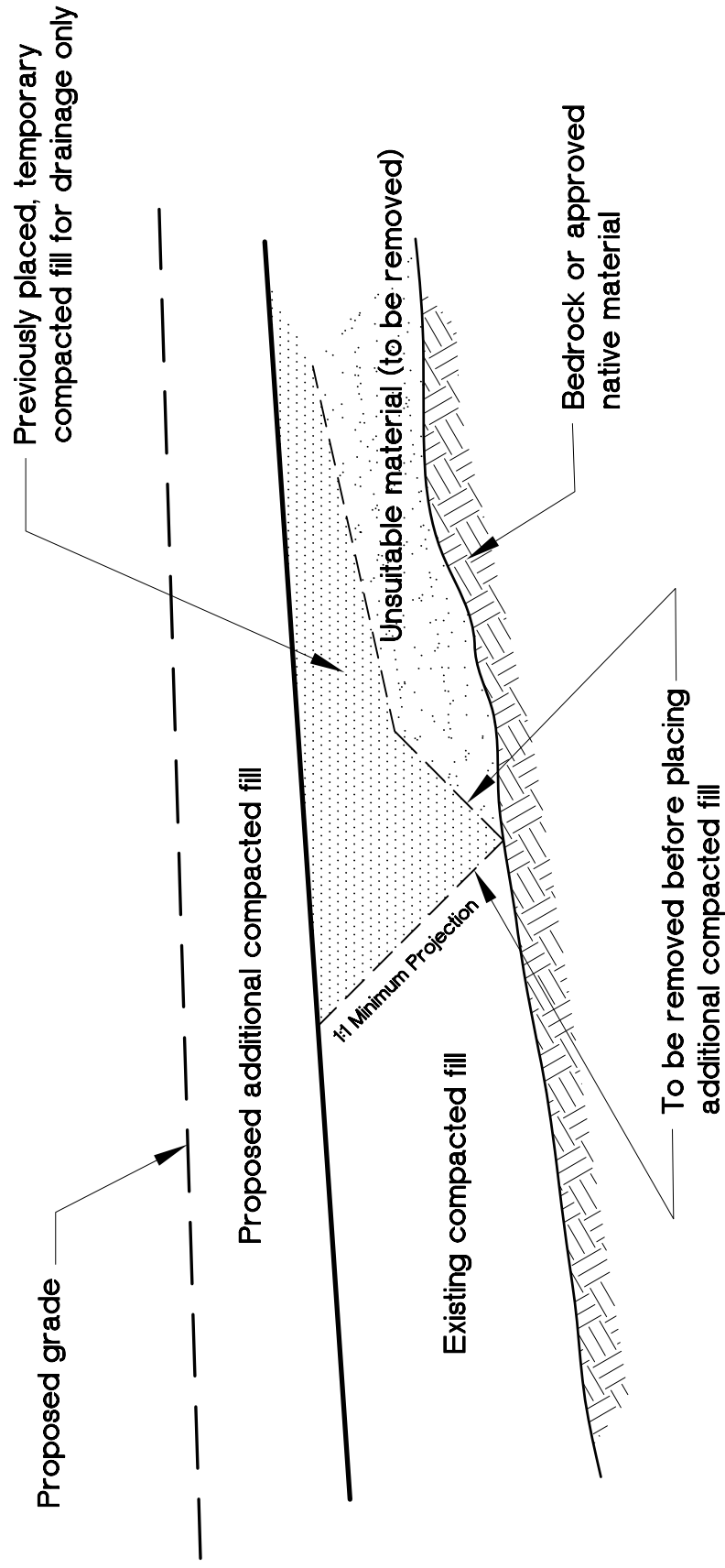
Gravel: Clean $\frac{3}{4}$ -inch rock or approved substitute.

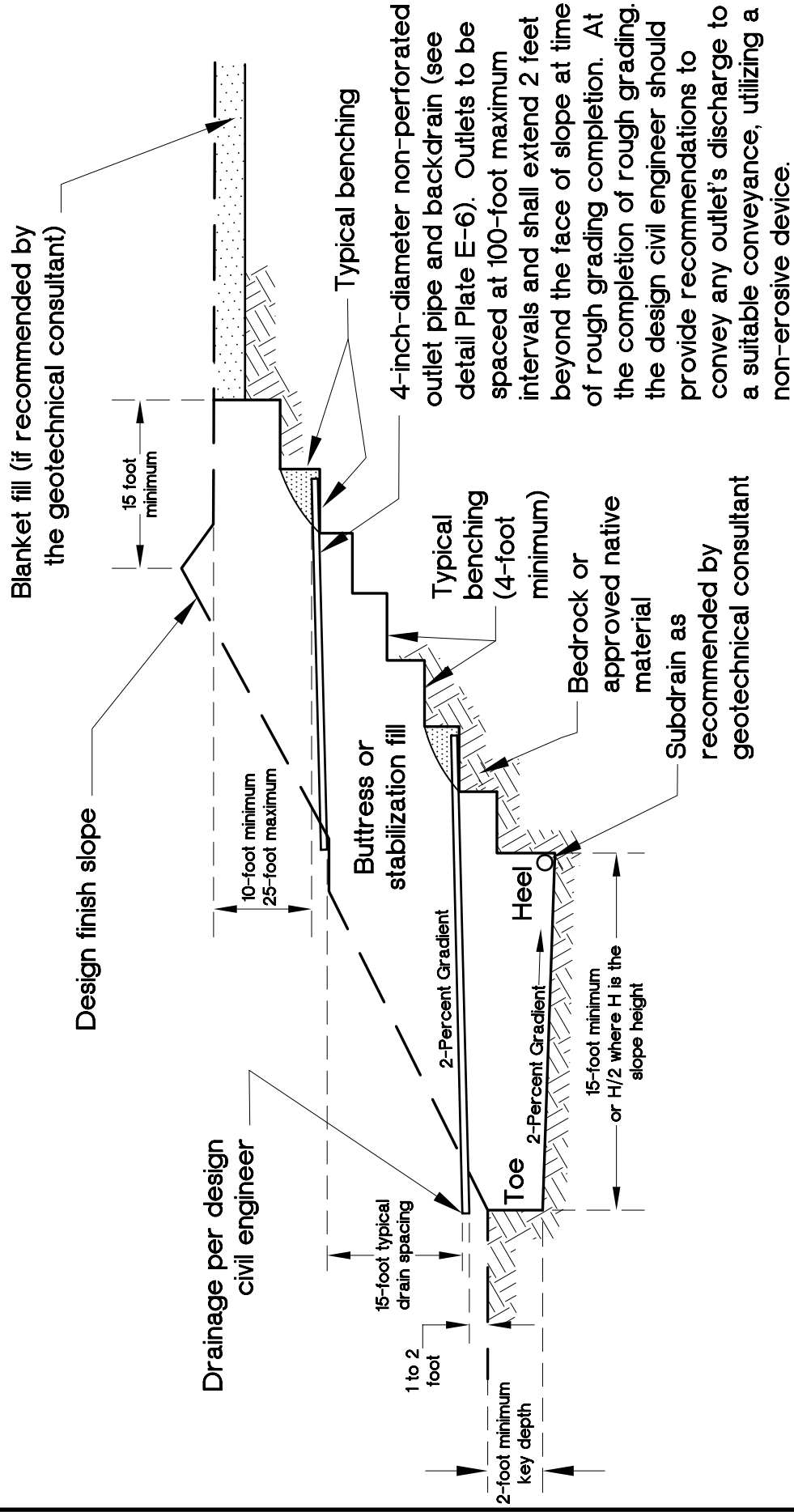
Filter Fabric: Mirafi 140 or approved substitute.

ALTERNATE 2: PERFORATED PIPE, GRAVEL, AND FILTER FABRIC

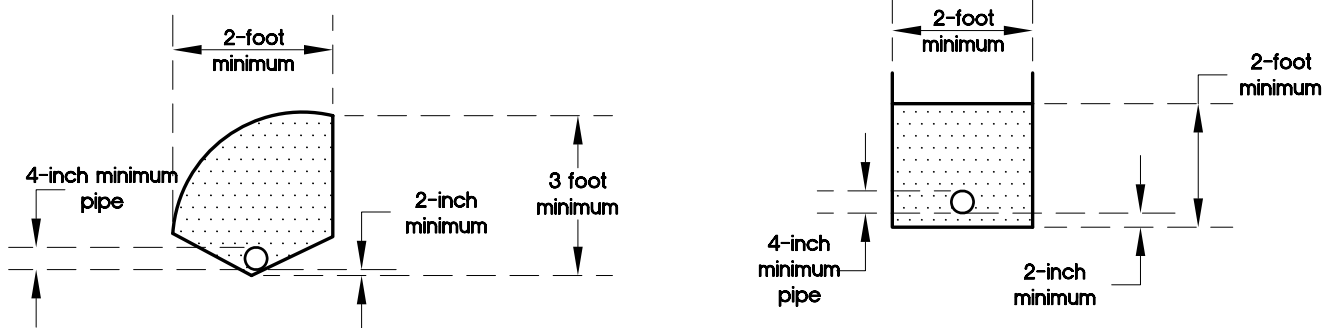


Provide a 1:1 (H:V) minimum projection from toe of slope as shown on grading plan to the recommended removal depth. Slope height, site conditions, and/or local conditions could dictate flatter projections.





TYPICAL STABILIZATION / BUTTRESS FILL DETAIL



Filter Material: Minimum of 5 cubic feet per lineal foot of pipe or 4 cubic feet per lineal foot 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.

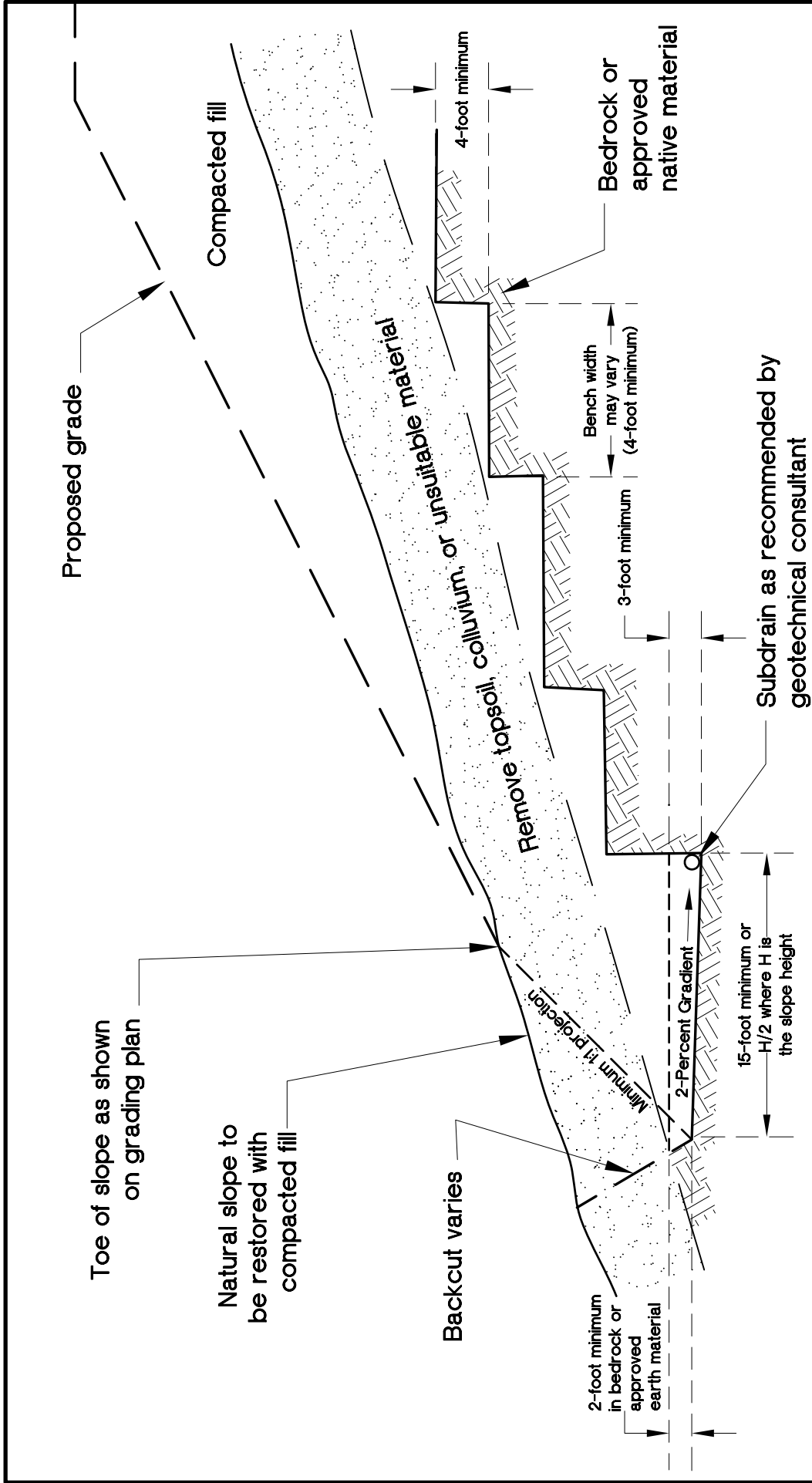
- Notes:**
1. Trench for outlet pipes to be backfilled and compacted with onsite soil.
 2. Backdrains and lateral drains shall be located at elevation of every bench drain. First drain located at elevation just above lower lot grade. Additional drains may be required at the discretion of the geotechnical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

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

Sieve Size	Percent Passing
1 inch	100
¾ inch	90-100
⅜ inch	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

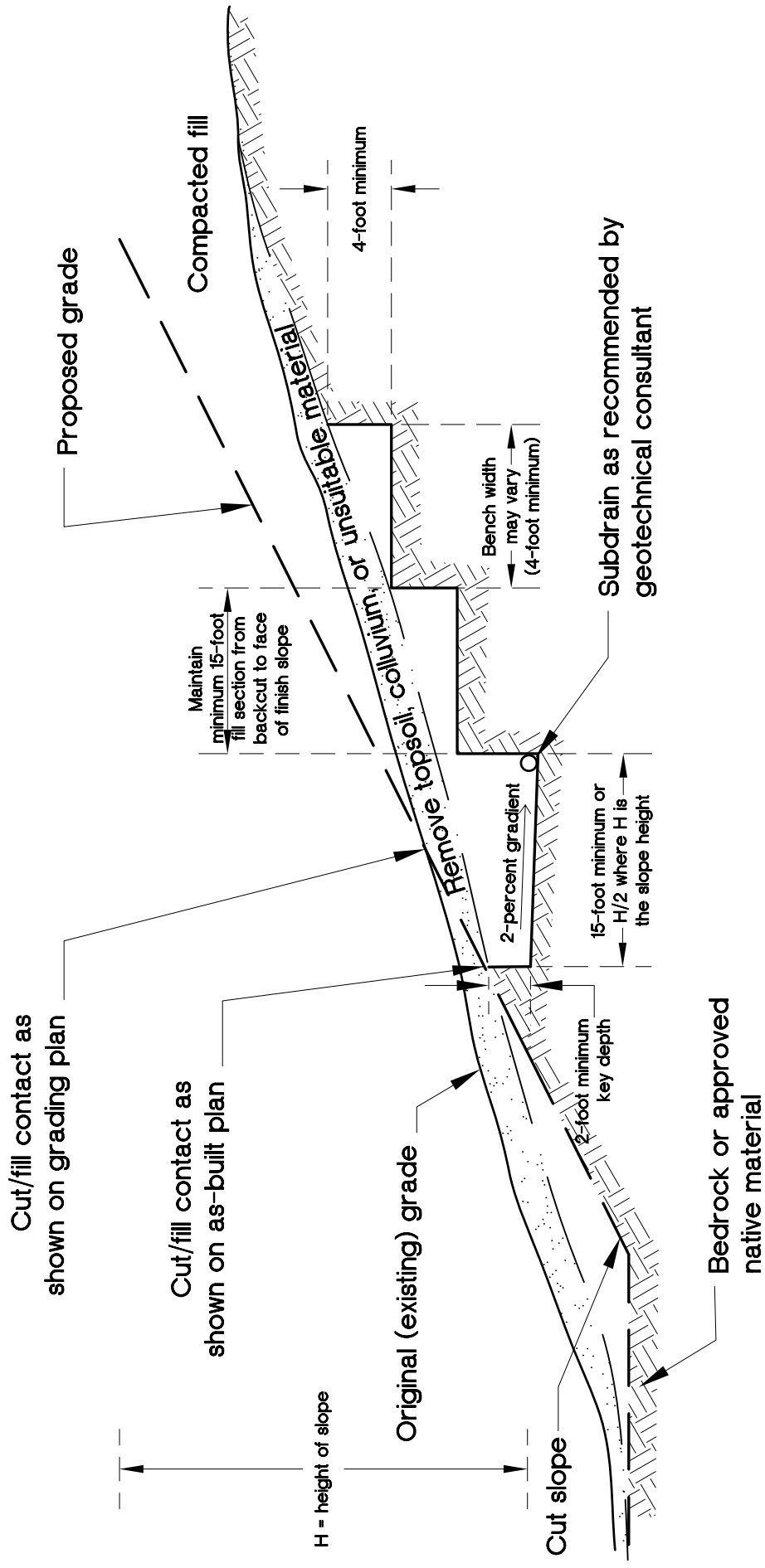
Gravel shall be of the following specification or an approved equivalent.

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

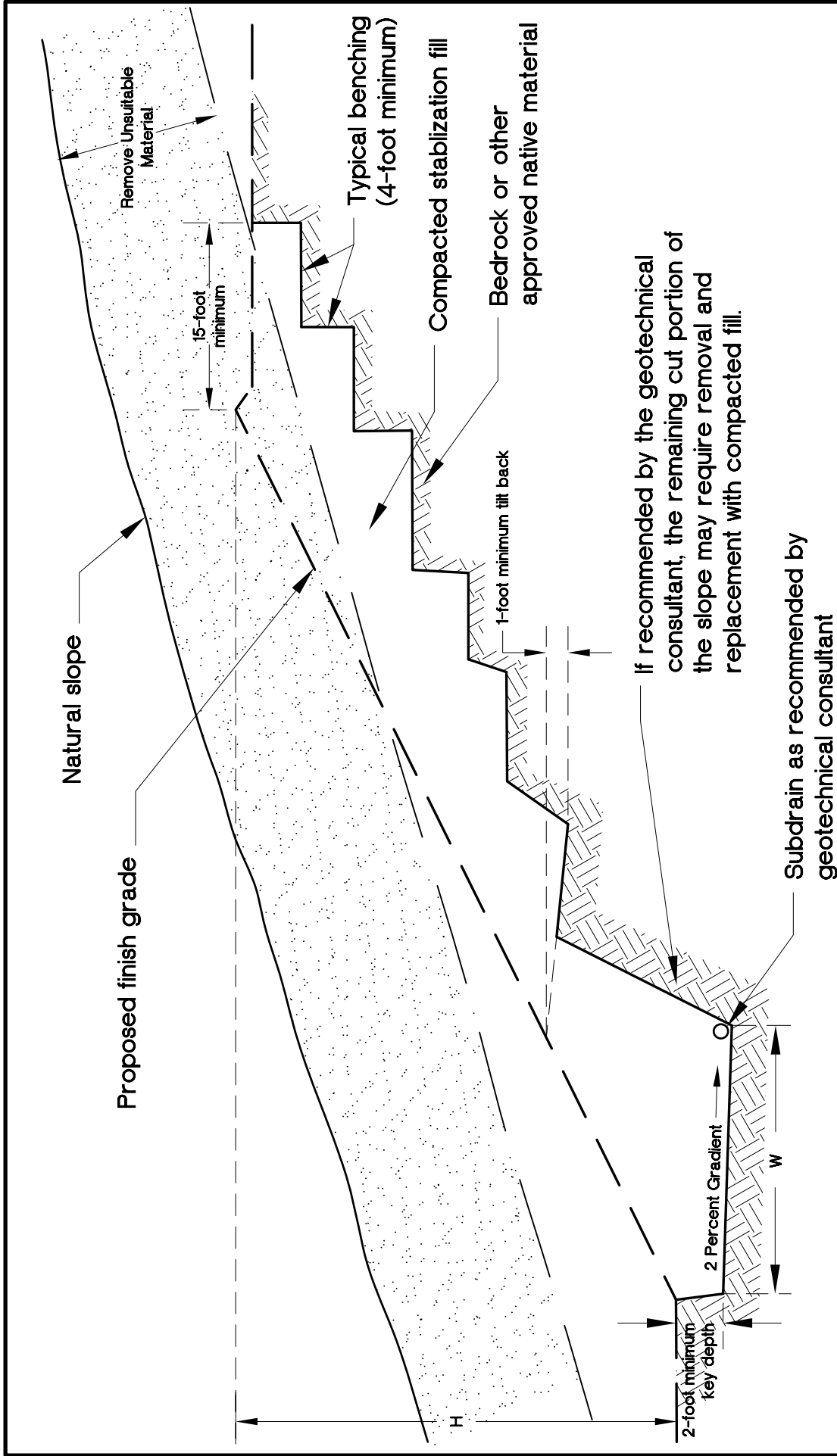


NOTES:

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.

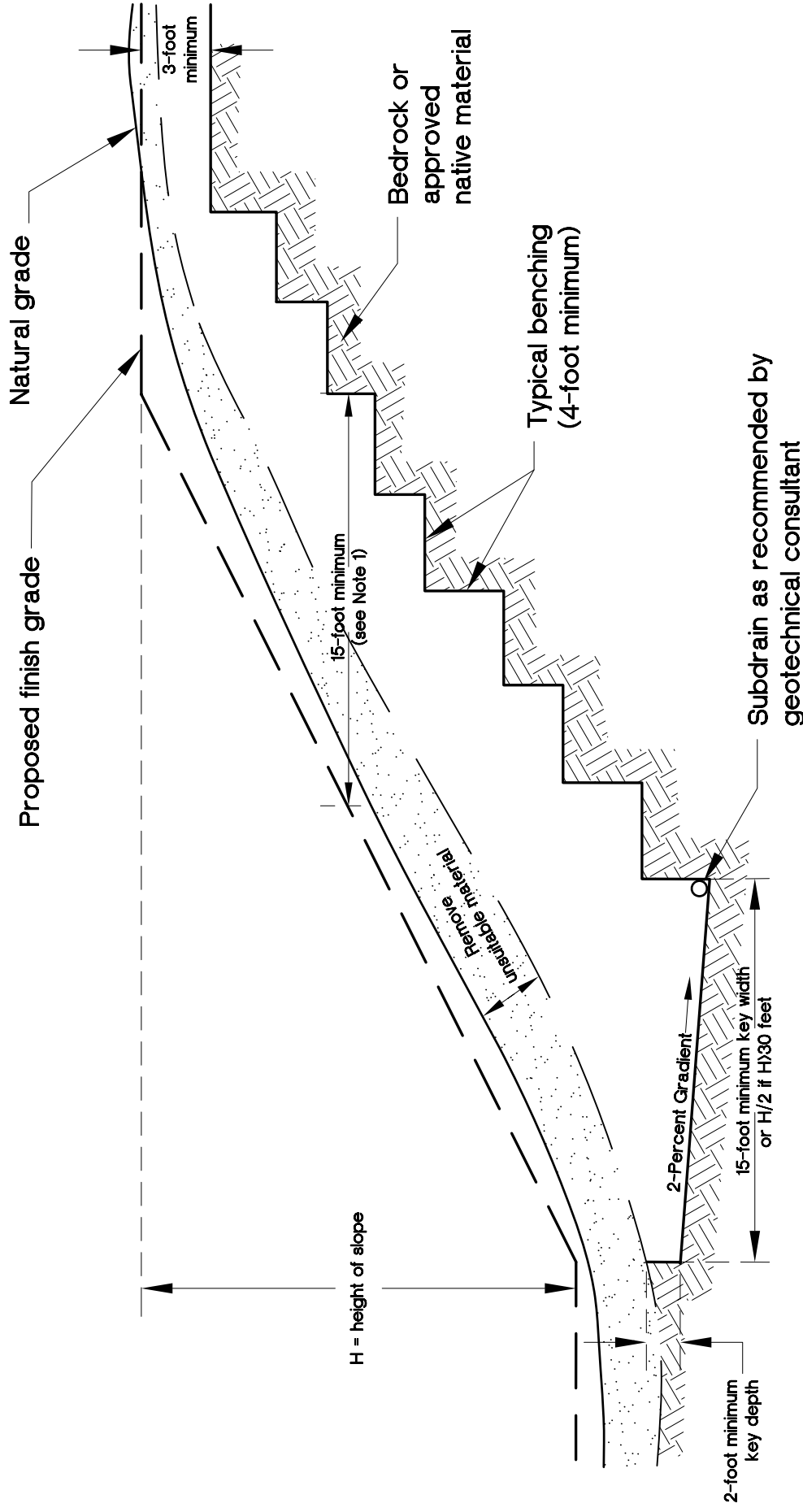


NOTE: The cut portion of the slope should be excavated and evaluated by the geotechnical consultant prior to construction of the fill portion.

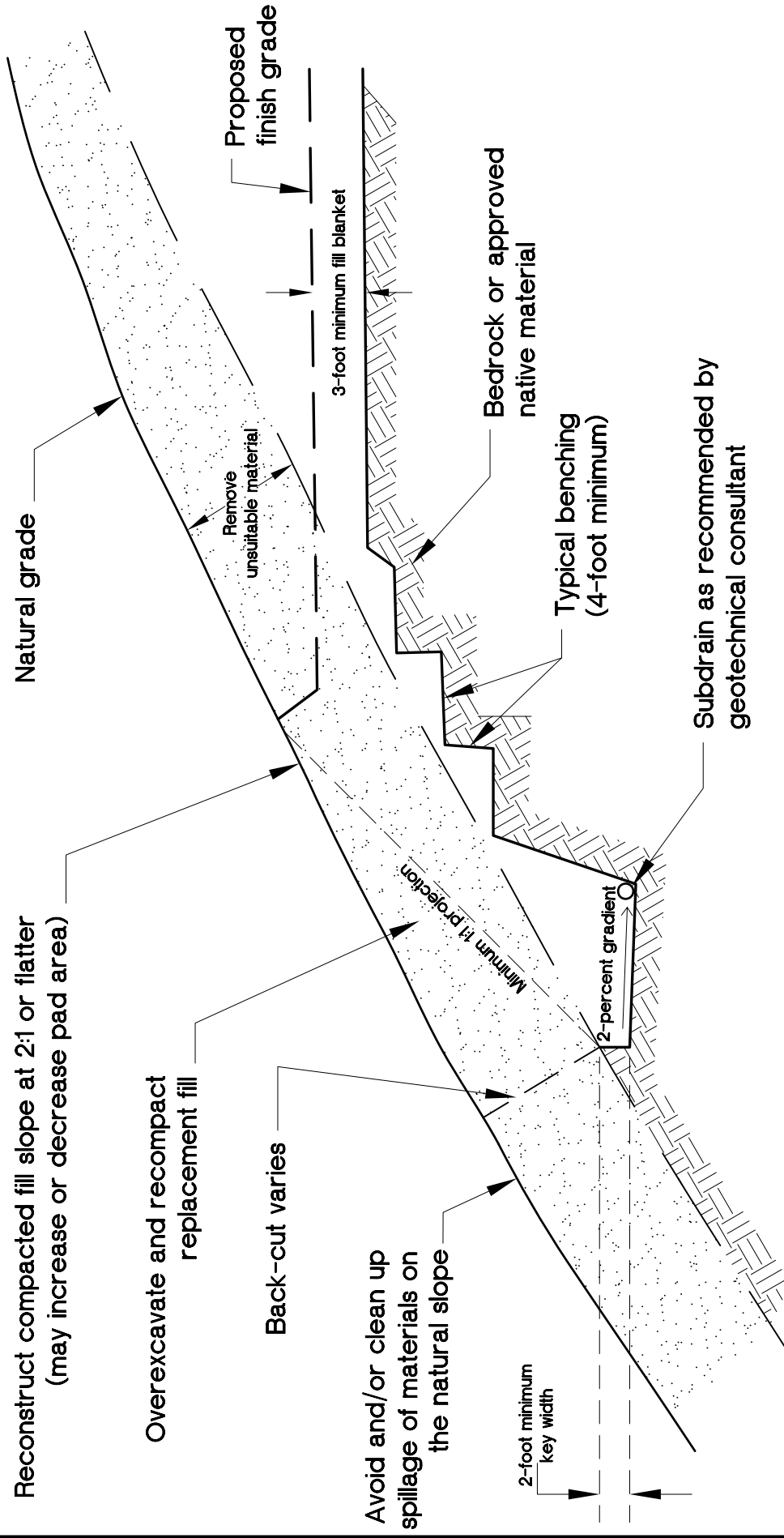


NOTES: 1. Subdrains may be required as specified by the geotechnical consultant.

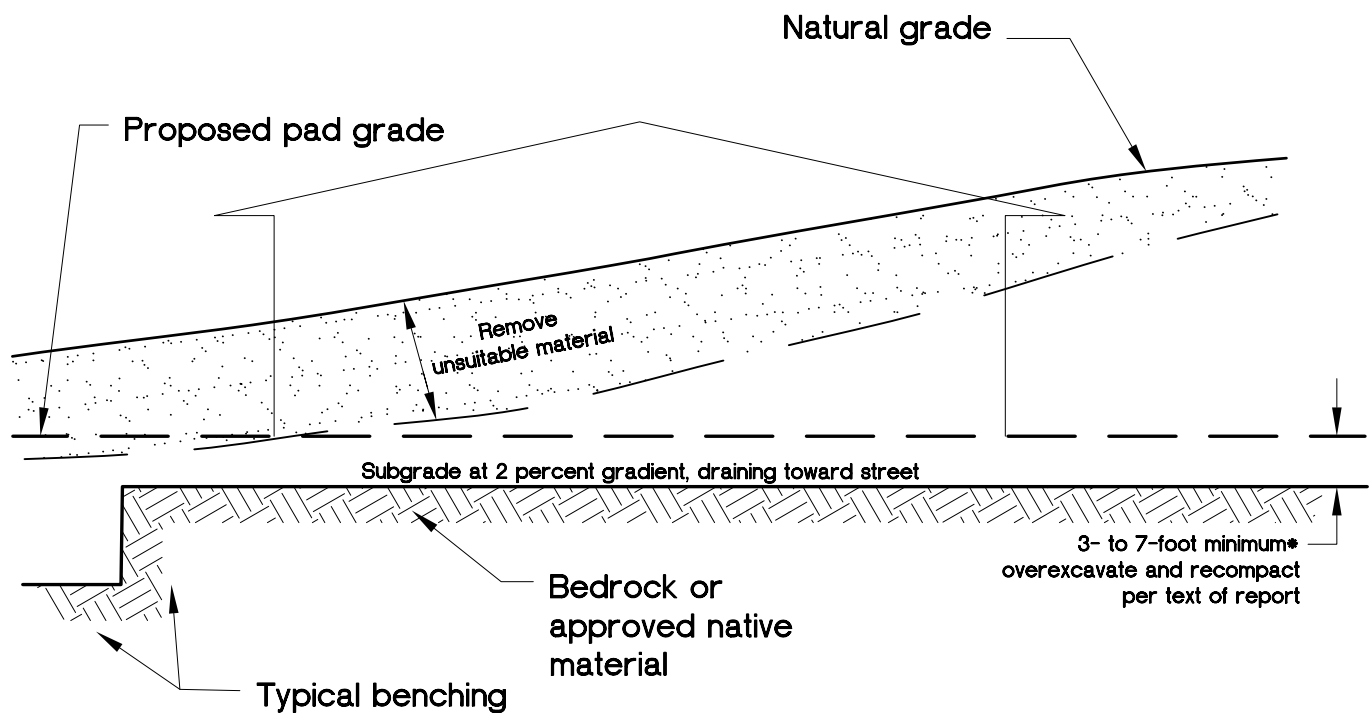
2. W shall be equipment width (15 feet) for slope heights less than 25 feet. For slopes greater than 25 feet, W shall be evaluated by the geotechnical consultant. At no time, shall W be less than $H/2$, where H is the height of the slope.



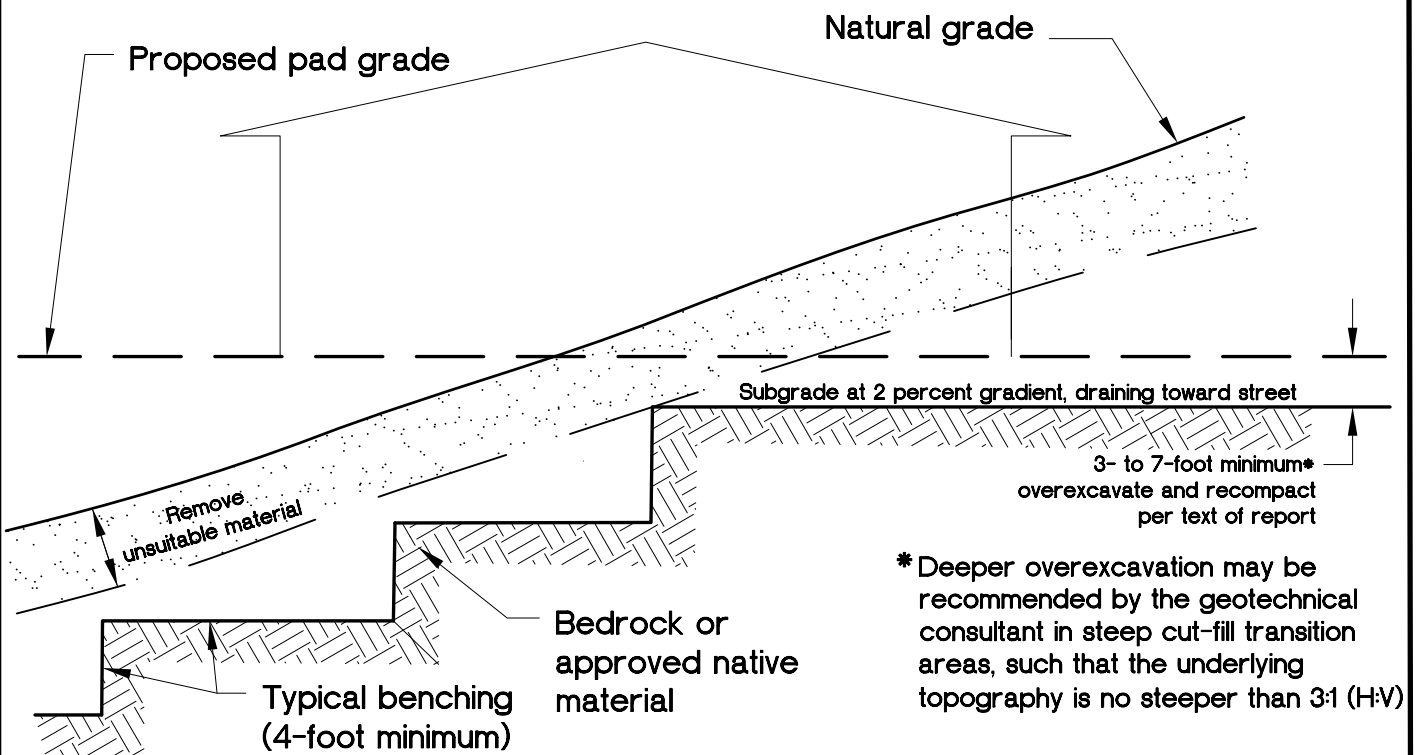
- NOTES:
1. 15-foot minimum to be maintained from proposed finish slope face to backcut.
 2. The need and disposition of drains will be evaluated by the geotechnical consultant based on field conditions.
 3. Pad overexcavation and recompaction should be performed if evaluated to be necessary by the geotechnical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



- 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.

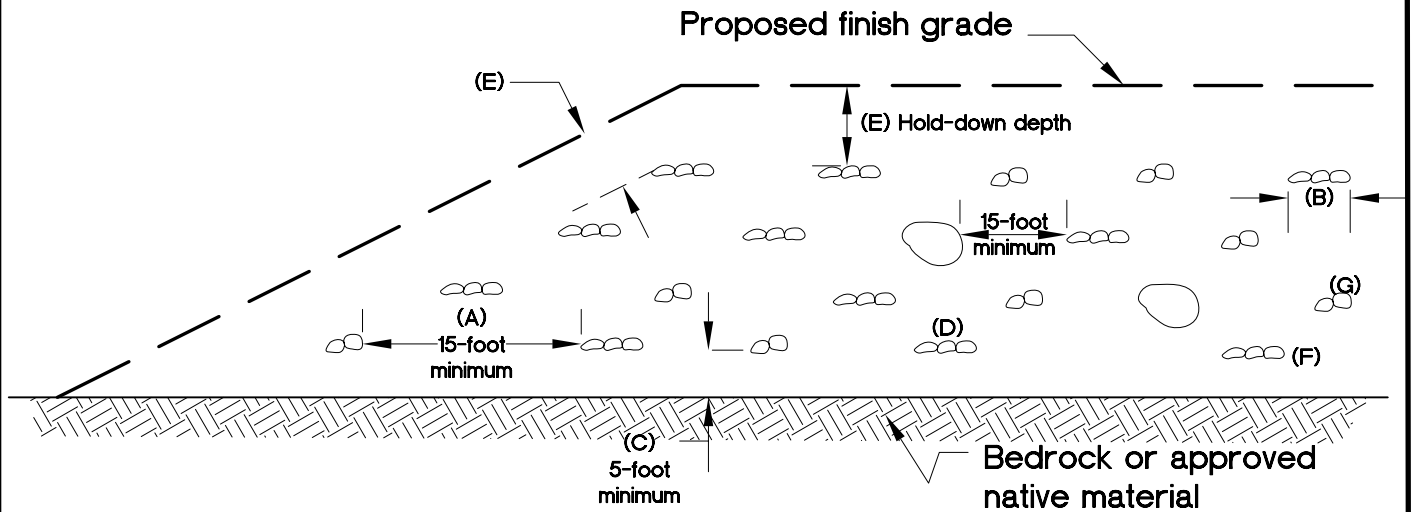


CUT LOT OR MATERIAL-TYPE TRANSITION

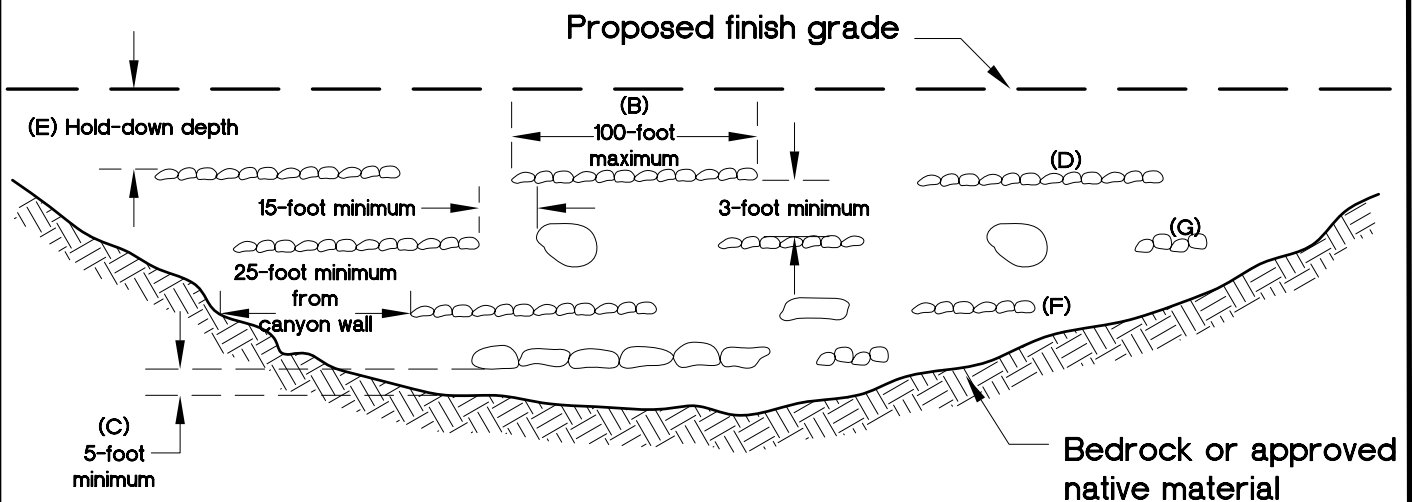


CUT-FILL LOT (DAYLIGHT TRANSITION)

VIEW NORMAL TO SLOPE FACE



VIEW PARALLEL TO SLOPE FACE



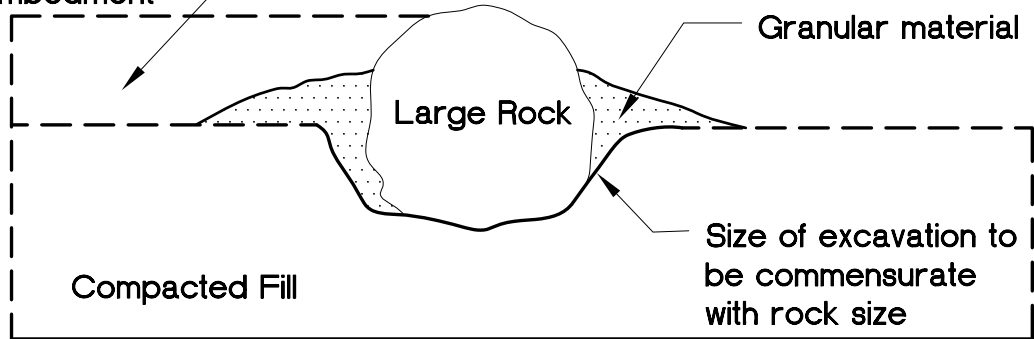
NOTES:

- One equipment width or a minimum of 15 feet between rows (or windrows).
- Height and width may vary depending on rock size and type of equipment. Length of windrow shall be no greater than 100 feet.
- If approved by the geotechnical consultant, windrows may be placed directly on competent material or bedrock, provided adequate space is available for compaction.
- 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.
- Clear area for utility trenches, foundations, and swimming pools; Hold-down depth as specified in text of report, subject to governing agency approval.
- All fill over and around rock windrow shall be compacted to at least 90 percent relative compaction or as recommended.
- 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

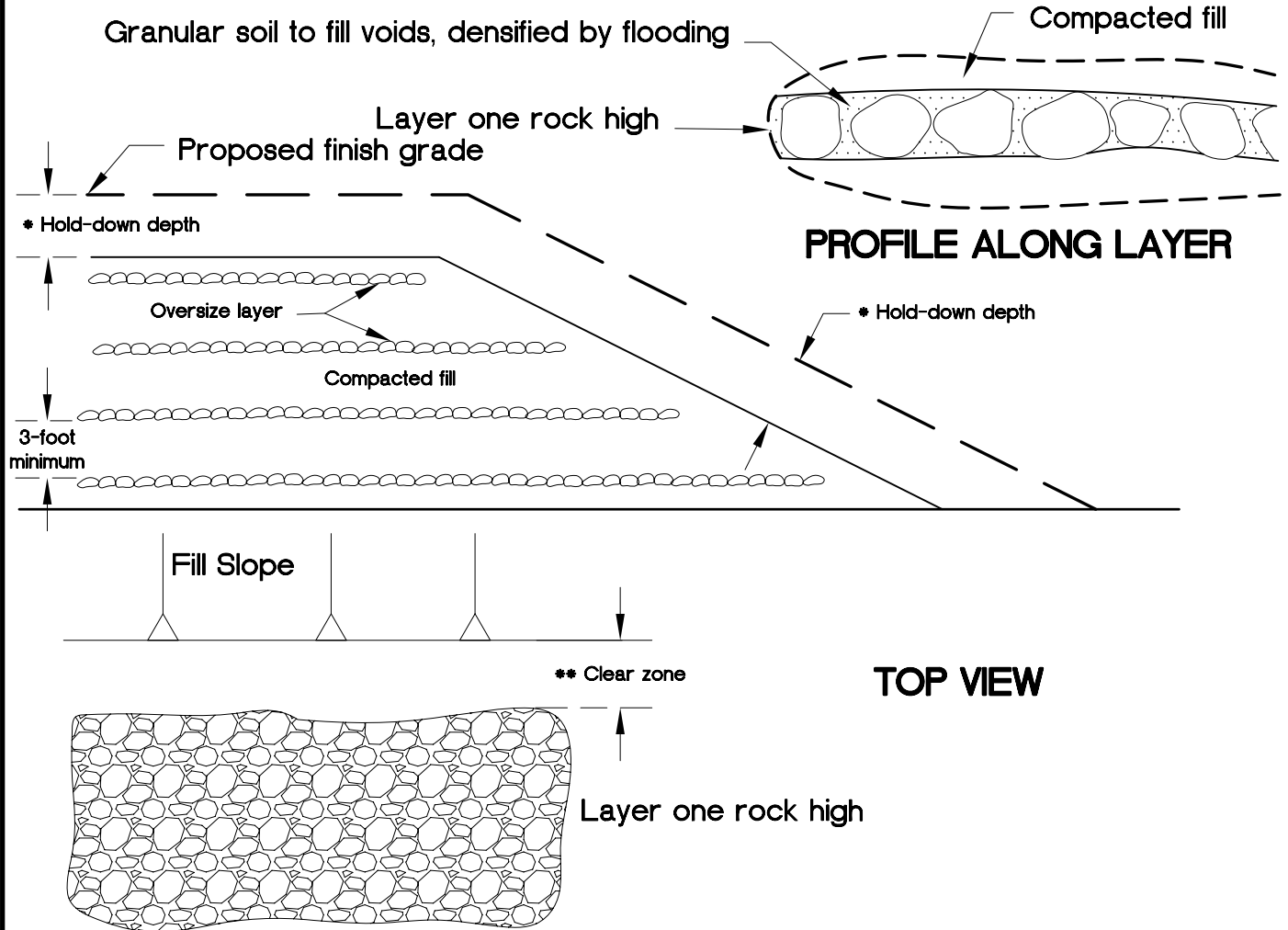
ROCK DISPOSAL PITS

Fill lifts compacted over rock after embedment



ROCK DISPOSAL LAYERS

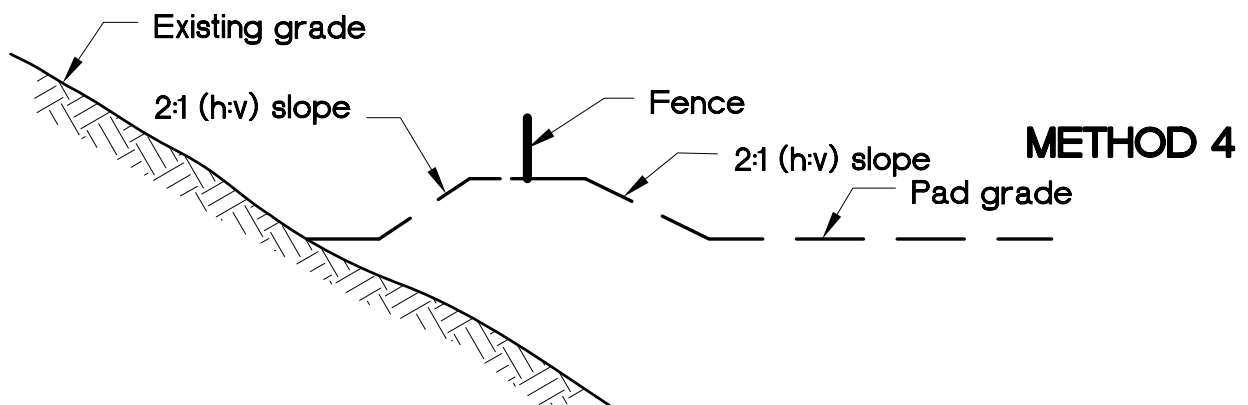
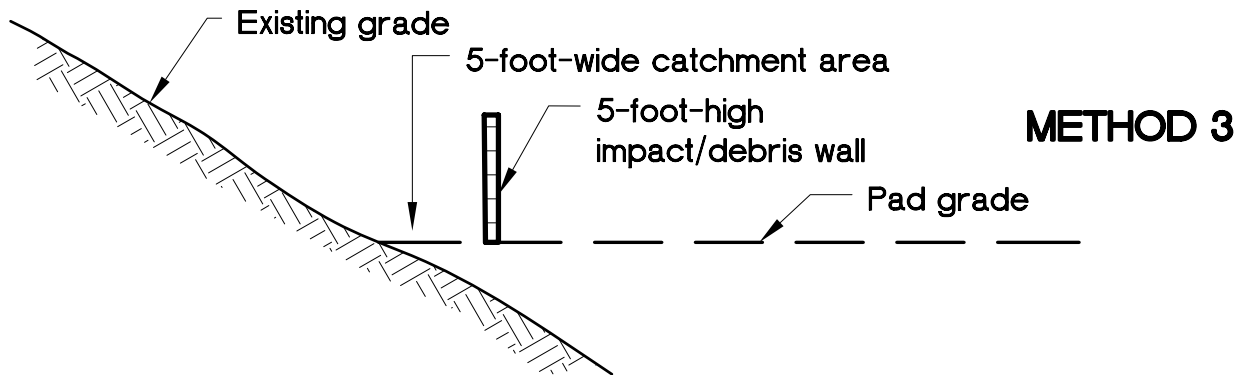
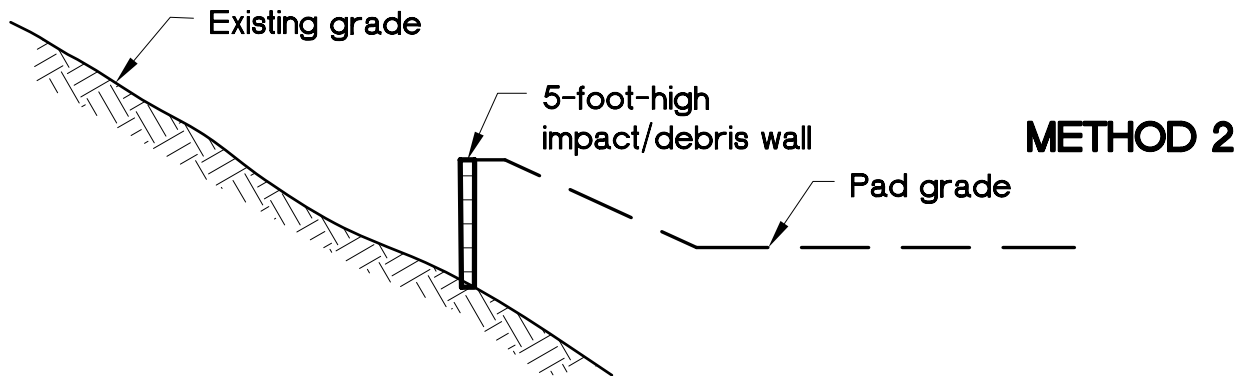
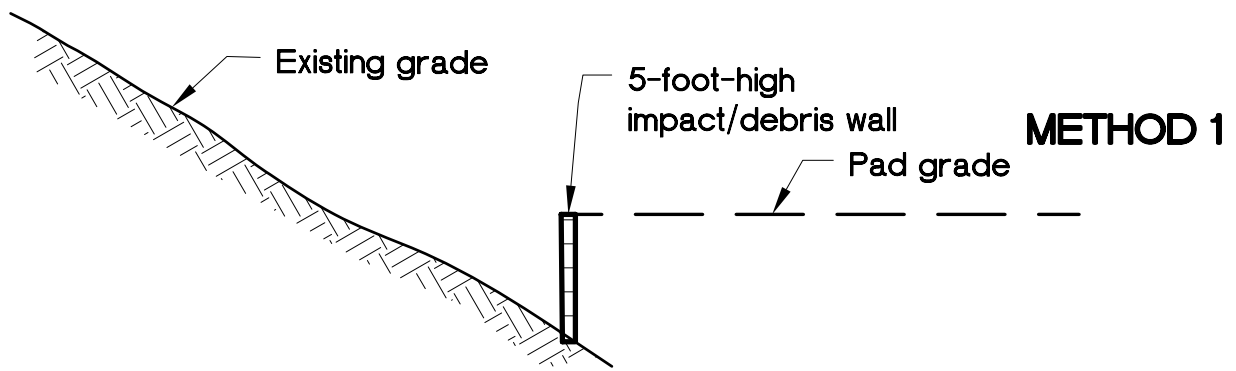
Granular soil to fill voids, densified by flooding



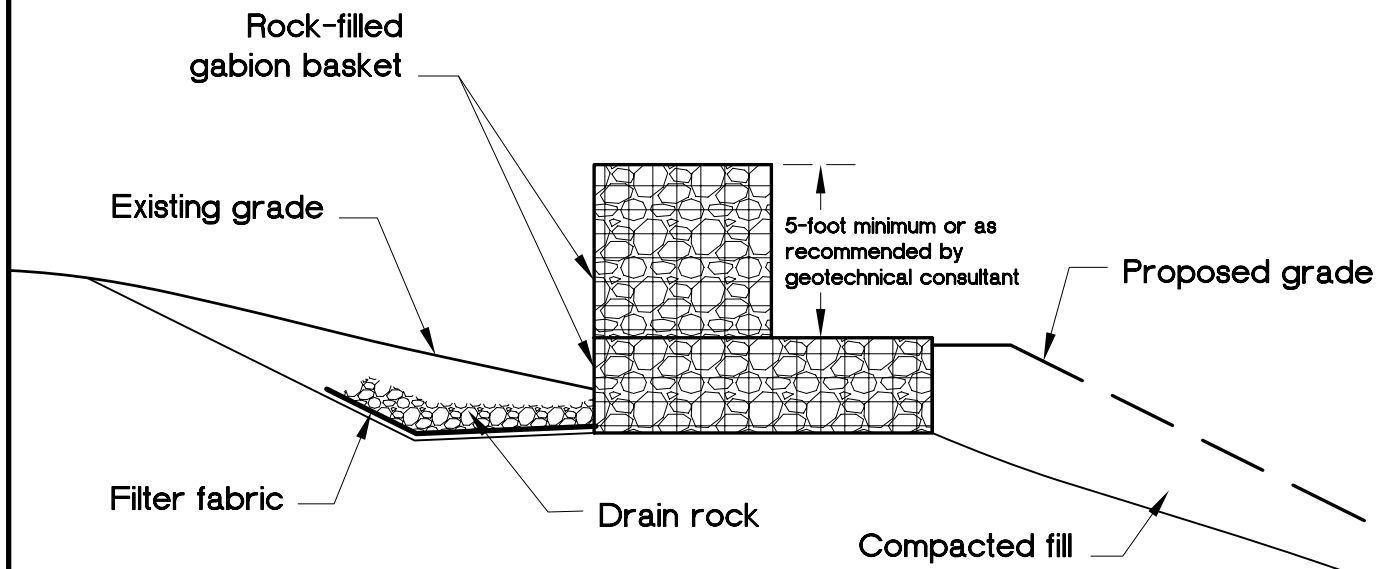
* Hold-down depth or below lowest utility as specified in text of report, subject to governing agency approval.

** Clear zone for utility trenches, foundations, and swimming pools, as specified in text of report.

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



NOT TO SCALE

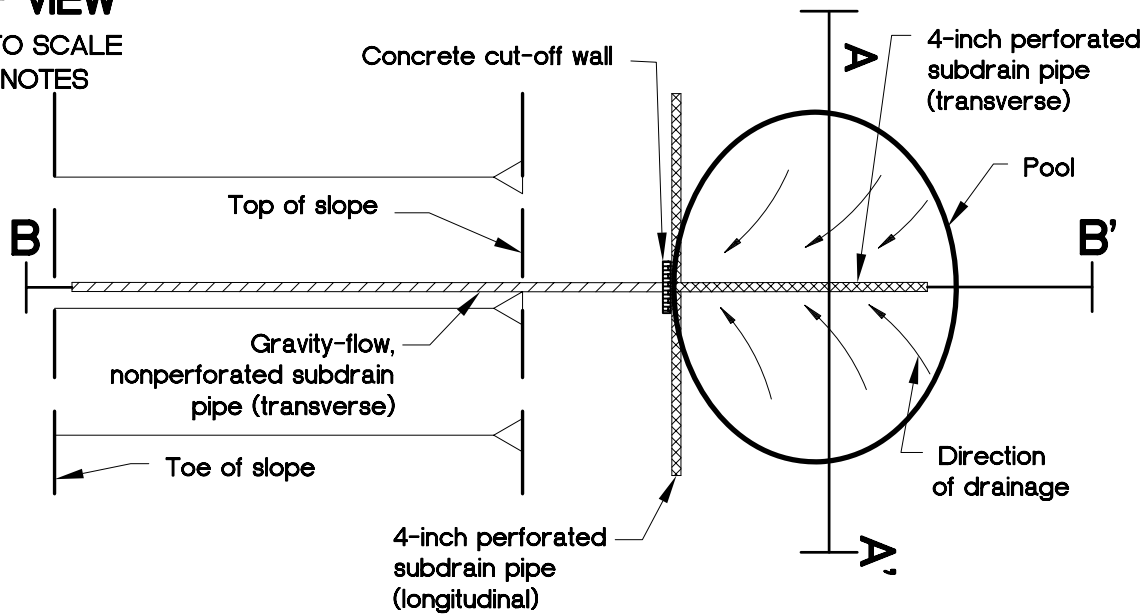


Gabion impact or diversion wall should be constructed at the base of the ascending slope subject to rock fall. Walls need to be constructed with high segments that sustain impact and mitigate potential for overtopping, and low segment that provides channelization of sediments and debris to desired depositional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

From GSA, 1987

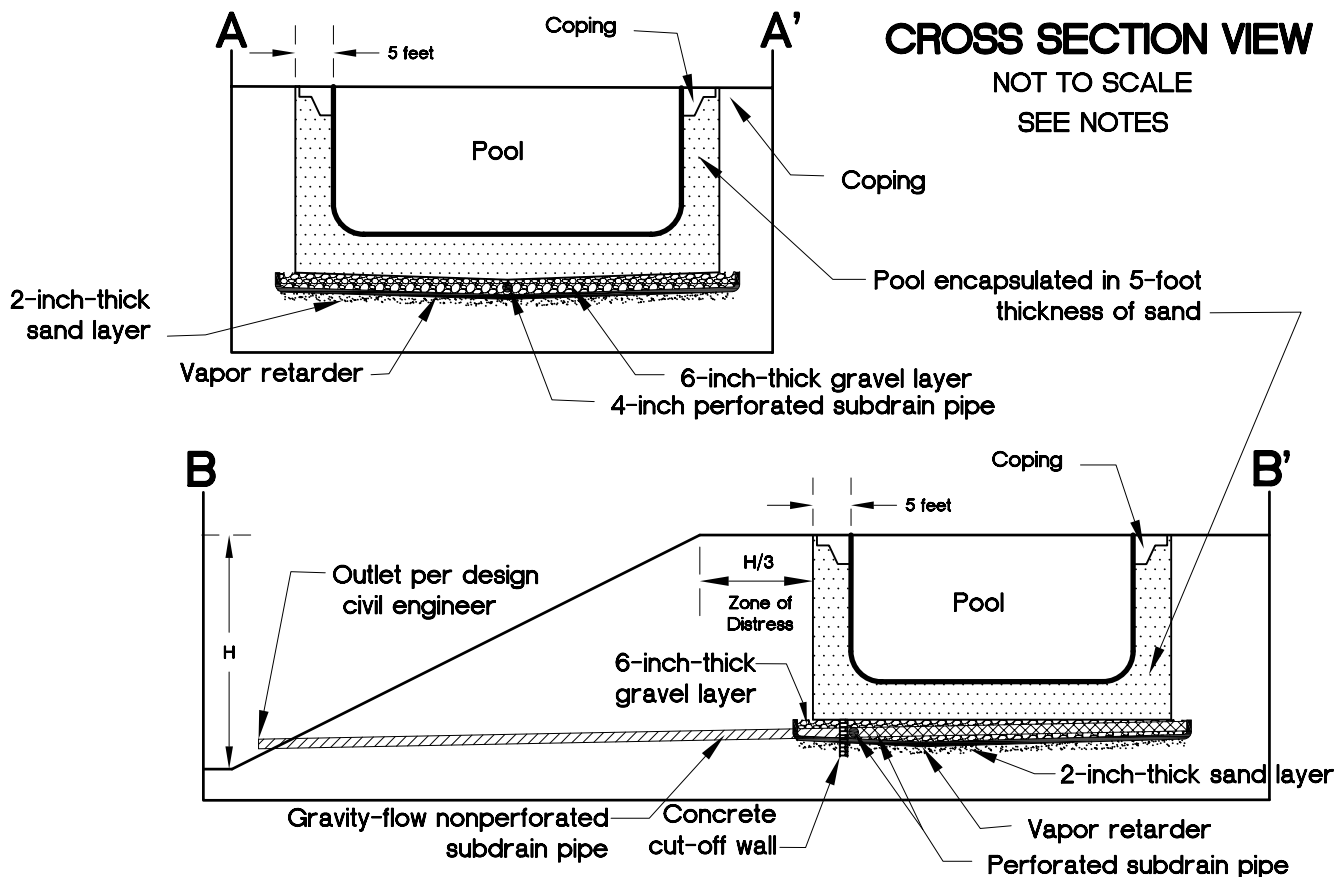
MAP VIEW

NOT TO SCALE
SEE NOTES



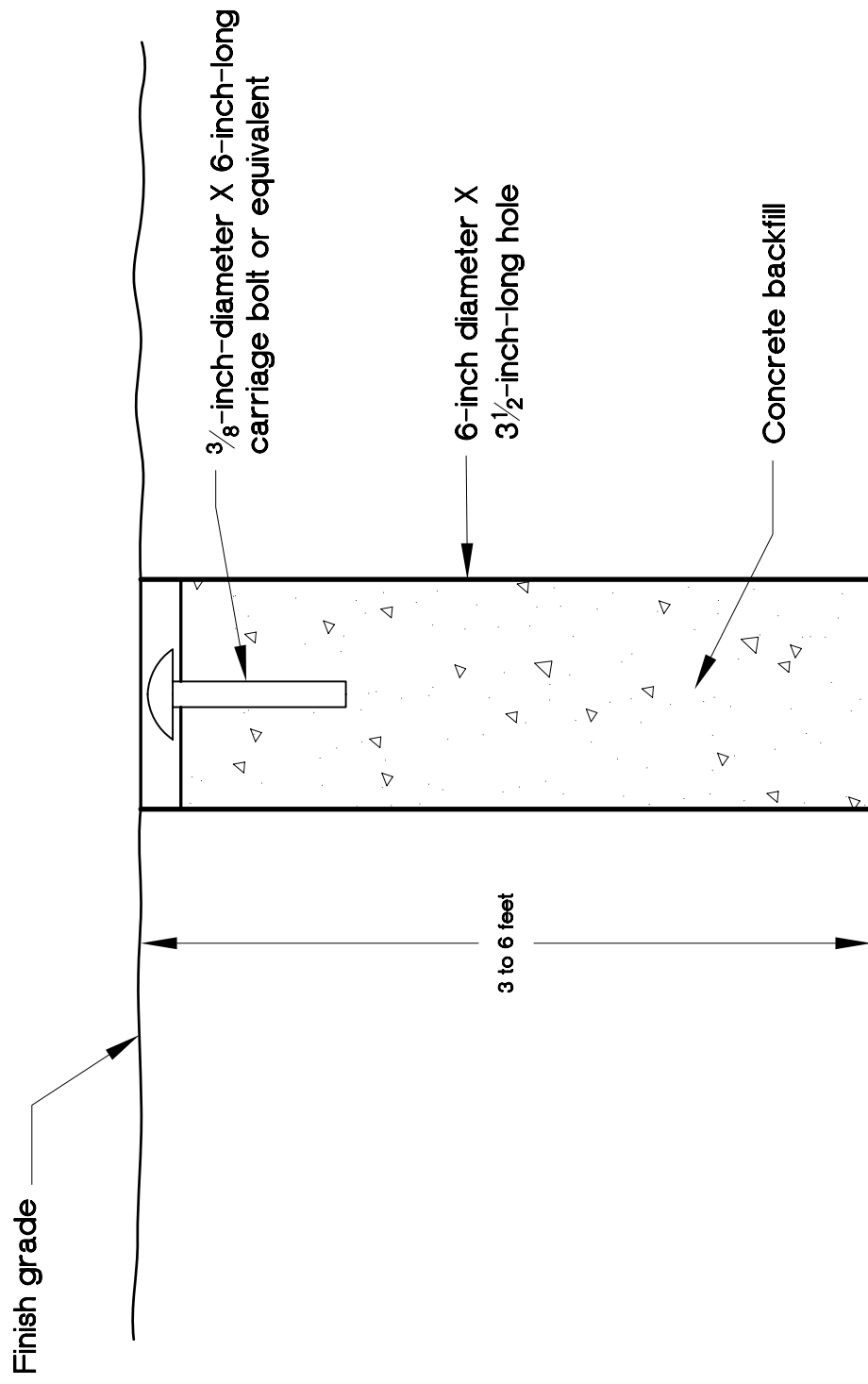
CROSS SECTION VIEW

NOT TO SCALE
SEE NOTES

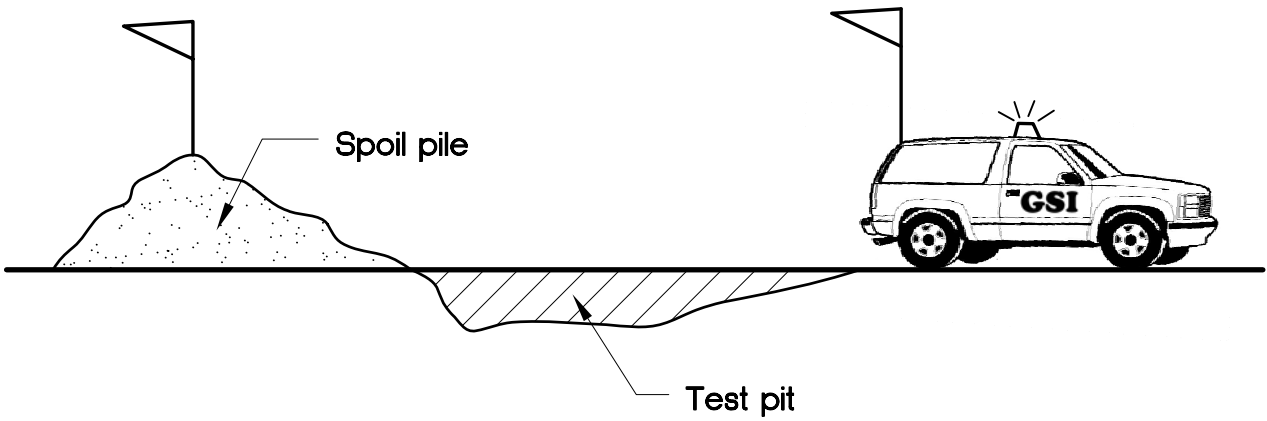


NOTES:

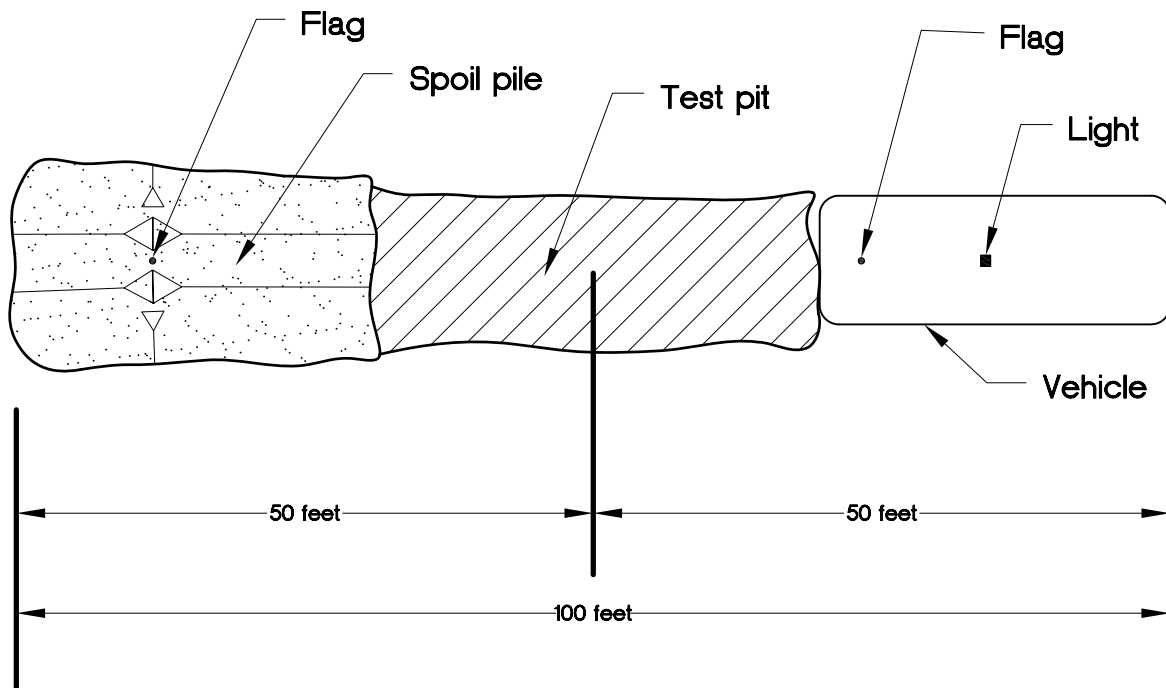
1. 6-inch-thick, clean gravel ($\frac{3}{4}$ to $1\frac{1}{2}$ inch) sub-base encapsulated in Mirafi 140N or equivalent, underlain by a 15-mil vapor retarder, with 4-inch-diameter perforated pipe longitudinal connected to 4-inch-diameter perforated pipe transverse. Connect transverse pipe to 4-inch-diameter 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.

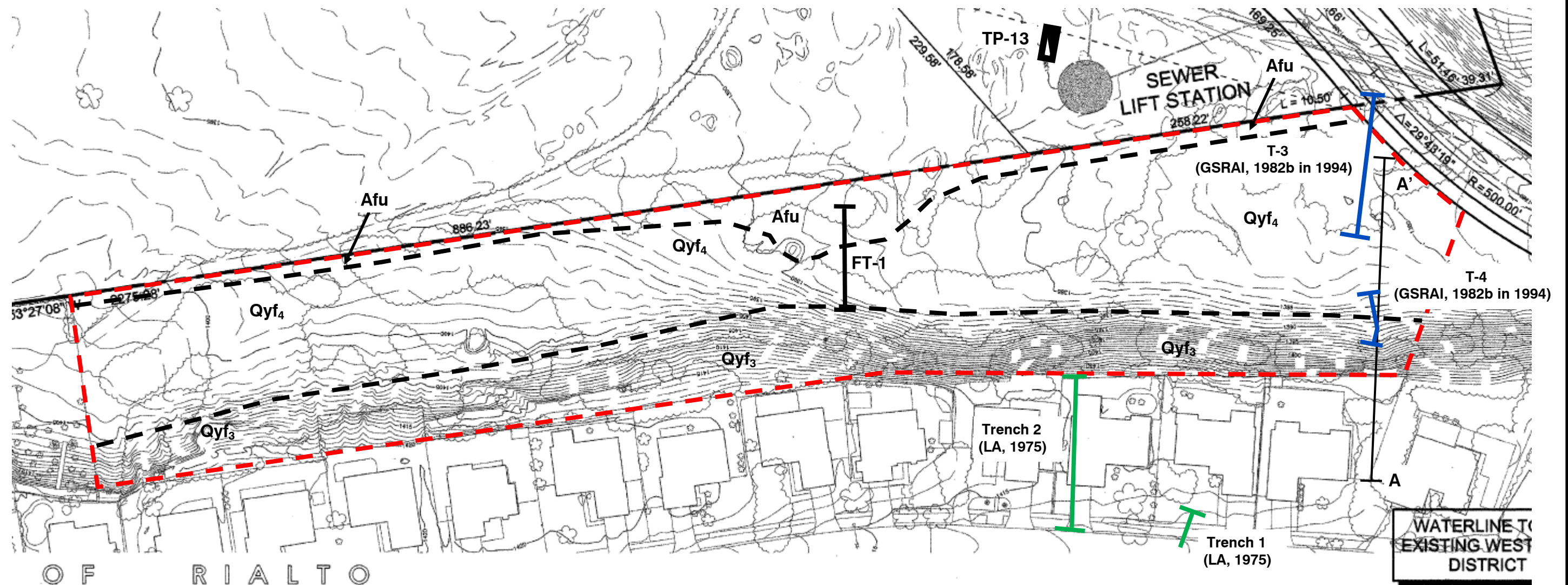


SIDE VIEW



TOP VIEW





GSI LEGEND

FT-1 Approximate Location of Exploratory Fault Finding Trench (This Study).

Afu Artificial Fill - Undocumented

Qyf₄ Quaternary Young Alluvial-Fan Deposits (Unit 4)

Qyf₃ Quaternary Young Alluvial-Fan Deposits (Unit 3)

A-A' Approximate Location of Geologic Cross-Section.

T-4 Approximate Location of Exploratory Fault Finding Trenches by Gary S. Rasmussen and Associates, Inc. (1994).

Trench 2 Approximate Location of Exploratory Fault Finding Trench by Leighton and Associates (1975).

TP-13 Approximate Location of Exploratory Test Pit (GSI, 2018).

- - - Approximate Location of Geologic Contact.

- - - - - Approximate Boundary of Property Under the Purview of this Report.

ALL LOCATIONS ARE APPROXIMATE

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

GeoSoils, Inc.

Plate 1

GEOTECHNICAL MAP

W.O. 8377-A-SC

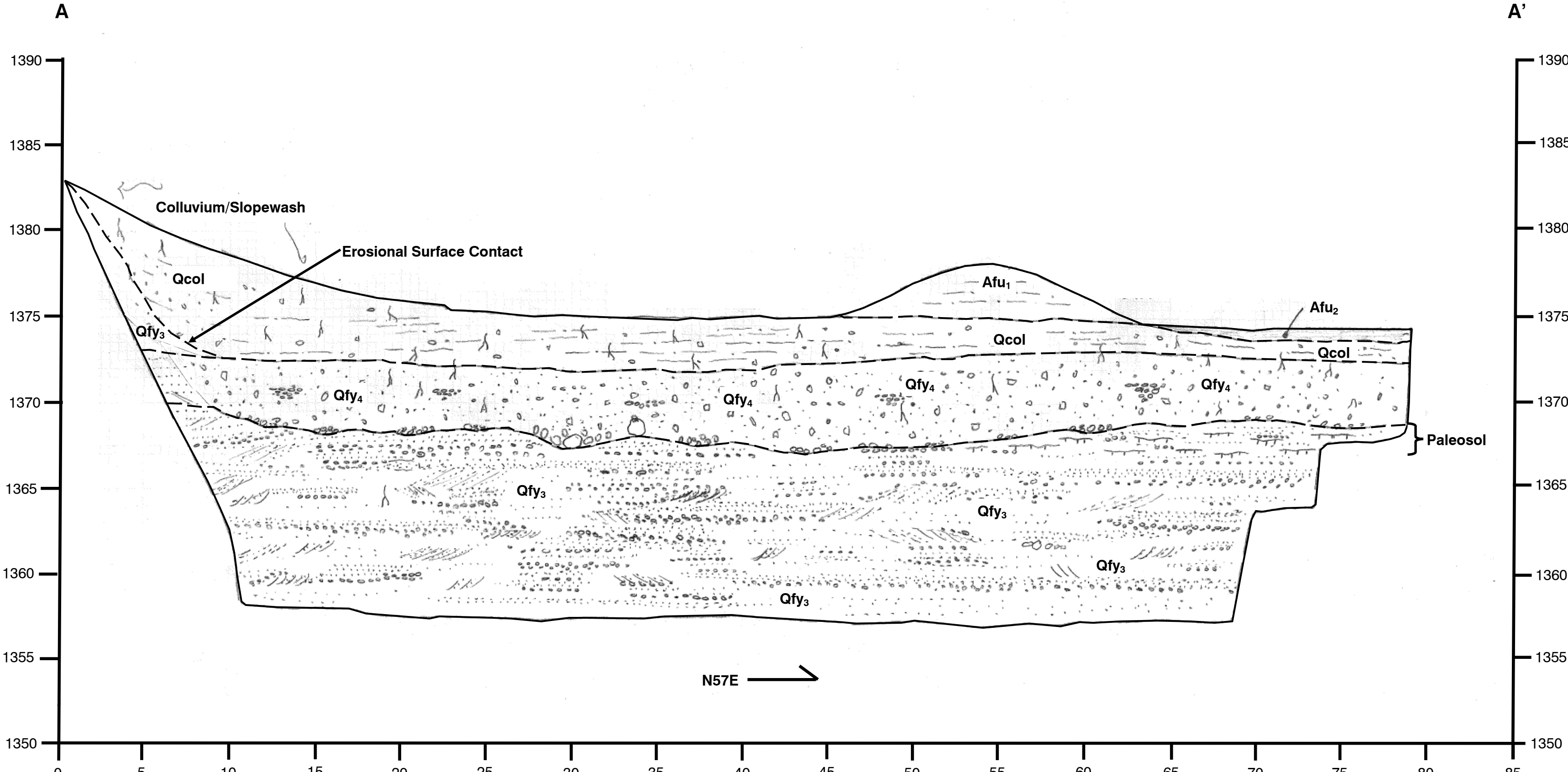
DATE: 08/22

SCALE: 1"=80'

Base Map: Tentative Tract Map 20092, by Otte-Berkeley Groupe, Inc. (2018)

SCALE: 1" = 80'

GSI - FAULT TRENCH LOG (FT-1)



GSI LEGEND

Artificial Fill - Undocumented (Afu₁) - Silty SAND (SM), grayish brown (10YR 5/2), dry, loose; porous, few roots, minor trash and debris, possibly related to construction on adjacent property.

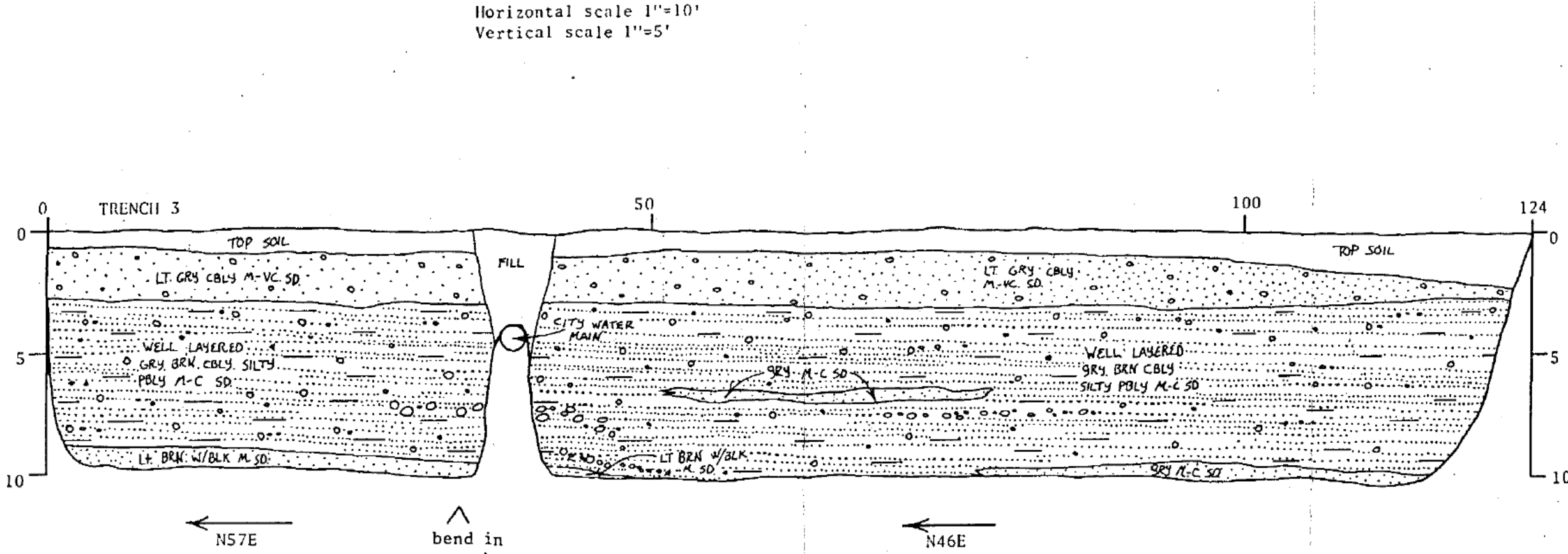
Artificial Fill - Undocumented (Afu₂) - Silty SAND and Sandy SILT (SM/ML), brown (10YR 5/3), dry, loose/soft; porous, few roots, bioturbated.

Colluvium - Slopewash (Qcol) - Sandy SILT/Silty fine SAND (ML/SM), grayish brown (10YR 5/2-5/3) to dark grayish brown to brown (10YR 4/2-4/3), damp, soft/loose; porous, bioturbated, few roots.

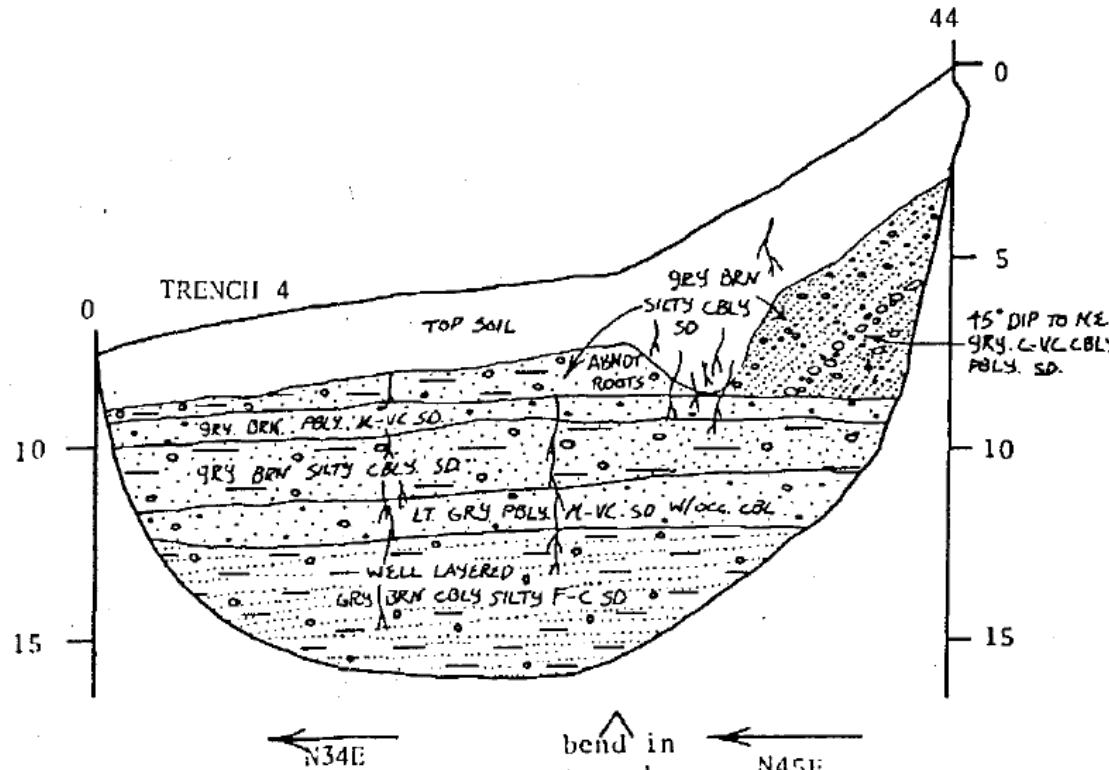
Quaternary Young Alluvial-Fan Deposits - Unit 4 (Qfy₄) - SAND (SP) and Sandy GRAVEL (GW) w/ cobble and boulder sized clasts, light gray to very pale brown (10YR 7/2-7/3), dry to slightly damp, medium dense; fine to coarse grained sands, few roots, gravels and cobbles are generally well to sub-rounded, boulders are rounded to sub-rounded.

Quaternary Young Alluvial-Fan Deposits - Unit 3 (Qf₃) - Gravely SAND
w/ Silt (GW) to Sandy GRAVEL (GW w/ cobble and few boulder sized clasts, light brownish gray (10YR 6/2) to pale brown (10YR 6/3) to light gray (10YR 7/2), dry to slightly damp, loose to medium dense with depth; some well-rounded gravel sized clasts, weak sub-horizontal bedding, laminar to cross-bedded sand lenses (point bar?), typical fining-upwards sequence throughout thinly-bedded 1 to 2-foot depositional sequences, steep erosional/scoured unconformable contact with colluvial - slopewash deposits. Contains localized discontinuous Paleosol (Sta. 53'^{+0.5} to 80'^{+0.5}). Silty SAND, brown (7.5Y 4/4).

GSRA (1982b in 1994) - FAULT TRENCH LOG (FT-3)



GSRA (1982b in 1994) - FAULT TRENCH LOG (FT-4)



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