

**DUE DILIGENCE GEOTECHNICAL EVALUATION
AND INFILTRATION FEASIBILITY TESTING
PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
(FARM BUREAU PROJECT), 9.32-ACRES
21150 AND 21160 BOX SPRINGS ROAD
CITY OF MORENO VALLEY
RIVERSIDE COUNTY, CALIFORNIA
(APNS 256-200-002, -003, AND -004)**

GeoSoils, Inc.

FOR

**FOREMOST PACIFIC GROUP, LLC
27271 LAS RAMBLES, SUITE 100
MISSION VIEJO, CALIFORNIA 92691**

W.O. 8676-A-SC

NOVEMBER 9, 2023



Geotechnical • Geologic • Coastal • Environmental

18451 Collier Avenue, Suite A • Lake Elsinore, California 92530 • (951) 471-0700 • FAX (951) 471-0702 • www.geosoilsinc.com

November 9, 2023

W.O. 8676-A-SC

Foremost Pacific Group, LLC
27271 Las Rambles, Suite 100
Mission Viejo, California 92691

Attention: Mr. Bryan Bergeron

Subject: Due Diligence Geotechnical Evaluation and Infiltration Feasibility Testing,
Proposed Multi-Family Residential Development (Farm Bureau Project),
9.32-Acres, 21150 and 21160 Box Springs Road, City of Moreno Valley,
Riverside County, California (APNs 256-200-002, -003, and -004)

Dear Mr. Bergeron:

In accordance with your request and authorization, GeoSoils, Inc. (GSI) is presenting the results of our due diligence geotechnical evaluation and infiltration feasibility testing for the proposed Farm Bureau project located at 21150 and 21160 Box Springs Road in the City of Moreno Valley, Riverside County, California. The primary purpose of this study was to evaluate the onsite soils and geologic conditions and their effects on the proposed multi-family residential development, from a geotechnical point of view. A secondary purpose of this study was to provide infiltration feasibility testing for the proposed stormwater BMP basin, a limited offsite pavement section thickness evaluation of bounding exterior roadways, preparation of general earthwork and grading guidelines, and development criteria in light of proposed multi-family residential development and site geologic conditions.

EXECUTIVE SUMMARY

Based on our review of readily available data, our recent subsurface investigation, associated laboratory testing, and geologic and engineering analyses, the proposed development of the project site appears suitable for its intended multi-family residential use from a geotechnical viewpoint, provided the recommendations presented in the text of this report are properly implemented. The primary developmental considerations are summarized below:

- The proposed 9.32-acre Farm Bureau project is located at 21150 and 21160 Box Springs Road in the City of Moreno Valley, Riverside County, California. Based on our recent subsurface investigation, and published geologic mapping by Morton and Cox (2001), the site is mantled by a thin layer of early-Pleistocene-age very old alluvial fan deposits (Qvof), underlain by unnamed late-Cenozoic


sedimentary bedrock (Qts). During our subsurface investigation, Cretaceous-age granitic bedrock, of a tonalite composition (Kt) was also encountered, and is mapped just north of the project boundary (Morton and Cox, 2001).

- Our review indicates no known active faults are crossing the site, and the site is not located within an Alquist-Priolo Earthquake Fault Zone (California Department of Conservation, California Geological Survey [CGS], 2018). In addition, the site is not located within a County of Riverside fault zone. Based on our review of the Riverside County Information Technology website (RCIT, 2023), the site is located within a zone of “low” to “very low” liquefaction potential, and is characterized as being potentially susceptible to subsidence (RCIT, 2023). Further discussions of the potentials for liquefaction and subsidence are provided within following sections of this report.
- As encountered during our recent field work, the site is mantled by up to 2½ to 20 feet of very old alluvial fan deposits, underlain by sedimentary and granitic bedrock materials at depth. Due to the relatively low density and lack of uniformity, all near-surface weathered deposits (upper 3 to 7 feet) of alluvial and bedrock materials (sedimentary and granitic), are considered unsuitable for the support of settlement-sensitive improvements or additional engineered fill. Based on the available subsurface data obtained, the thickness of these potentially compressible earth materials is anticipated to be approximately 3 to 7 feet, with an average of approximately 5 feet across the site, with the potential for localized deeper removals.
- Seeps, springs, mottled soils, or other indications of a high regional groundwater level were not noted at the subject site during the time of our field investigation. Based on our review of the California Department of Water Resources Water Data Library (CDWR; 2023), nearby well measurements reviewed appear to be regional groundwater measurements at depths of 56.4 and 83.6 feet (Station No. 339387N1172448W001, and No. 339480N1172444W001, respectively) at sites with similar ground surface elevations. As such, the regional groundwater table, at depth, is not anticipated to significantly affect the planned improvements, provided the conclusions and recommendations presented herein are properly incorporated into the design and construction of the project.
- An evaluation of storm water infiltration feasibility within the current BMP locations (selected by the Client) indicates a relatively low infiltration potential at the two specific locations tested. Further discussions of the test procedures used, onsite USDA soil groups, general infiltration siting requirements and limitations, along with the converted infiltration rates obtained are presented herein.
- Adverse geologic features that would preclude project feasibility (e.g., landslides, shallow regional groundwater, subsidence, active faulting, etc.) were not encountered.
- The recommendations presented in this report should be incorporated into preliminary planning, design, and construction considerations of the project.

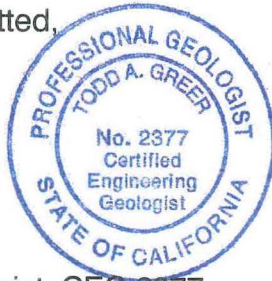
The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to contact our office.

Respectfully submitted,

GeoSoils, Inc.



Todd A. Greer
Engineering Geologist, CEG 2377



Stephen J. Coover
Geotechnical Engineer, GE 2057



MAM/TAG/JPF/SJC/sh

Distribution: (1) Addressee (PDF via email)

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Appendix C - Seismic Data	Rear of Text
Appendix D - Laboratory Test Results	Rear of Text
Appendix E - Field Percolation Data Sheets	Rear of Text
Appendix F - General Earthwork and Grading Guidelines	Rear of Text

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SCOPE OF SERVICES

The scope of our services has included the following:

1. Review of online and in-house geologic maps and literature for the site, review of the conceptual site plan provided and development assessment report by AEI CASC Engineering (ACE, undated), including aerial photographs provided by Google Earth Pro (GEP, 2023) and the United States Department of Agricultural (USDA, 1980, see Appendix A).
2. Geologic site reconnaissance and geologic mapping of significant earth materials units and sedimentary structures.
3. The advancement of five (5) exploratory borings across the site for geotechnical logging and soil sample collection, and to evaluate subsurface conditions onsite. In addition, two (2) relatively shallow borings were advanced within the locations of the stormwater BMPs for infiltration feasibility testing.
4. The advancement of two (2) asphaltic concrete (AC) cores via core-drilled samples on the bounding exterior roadway using a subcontractor. After the advancement of the cores through the AC of the roadways structural section, the crushed aggregate base was removed by hand methods, at each coring location, and the AC and aggregate base sections were measured and logged (Appendix B). The final cap consisted of black-dyed Portland Cement Concrete (PCC).
5. General areal seismicity and liquefaction evaluation (Appendix C).
6. 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, and sulfate/corrosion testing of the materials encountered during our field study. Results of our laboratory testing are provided in Appendix D.
7. Appropriate engineering and geologic analyses of data collected and preparation of this report and accompaniments.

SITE LOCATION

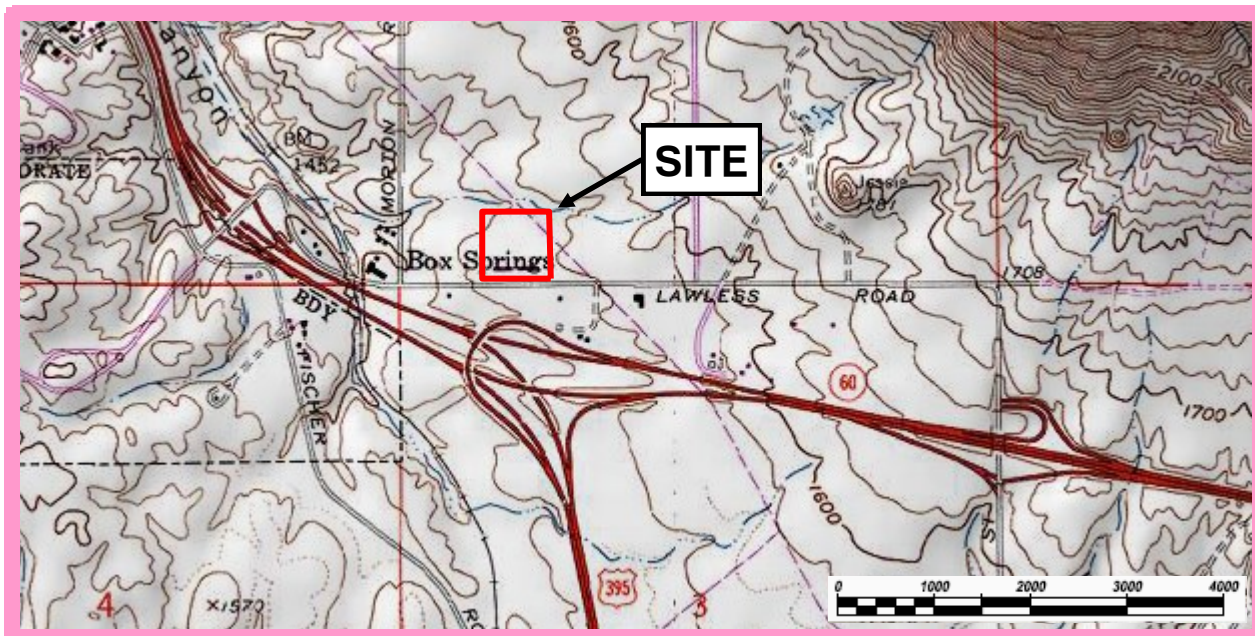
The subject 9.32-acre site is located at 21150 and 21160 Box Springs Road in the City of Moreno Valley, Riverside County, California (see Figure 1, Site Location Map). Based on our review, a majority of the site is currently vacant and undeveloped; however, two (2) commercial buildings occupy the southern side of the site, at the above addresses fronting Box Springs Road. Topographically, the property consists of gently-sloping terrain that varies in elevation from approximately 1,533 feet MSL (Mean Sea Level) near the northwest corner of the site to approximately 1,565 feet MSL near the northeast corner of the property. Therefore, overall relief is on the order of approximately 32 feet. A State aqueduct pipeline easement is located in the northeast corner of the property, however no improvements nor grading are proposed in the easement. Based on our field investigation and review of geologic mapping by Morton and Cox (2001), the site is mantled by a relatively thin layer of early-Pleistocene-age very old alluvial fan deposits (Qvof), underlain by unnamed late-Cenozoic sedimentary bedrock (Qts). During our subsurface investigation Cretaceous-age granitic bedrock, of a tonalite composition (Kt) was also encountered, and is mapped just north of the project boundary (Morton and Cox, 2001). As discussed previously, the site is located within in an area designated as having a “low” to “very low” potential for liquefaction during a seismic event, and the site is characterized as being potentially susceptible to subsidence (RCIT, 2023).

PROPOSED DEVELOPMENT

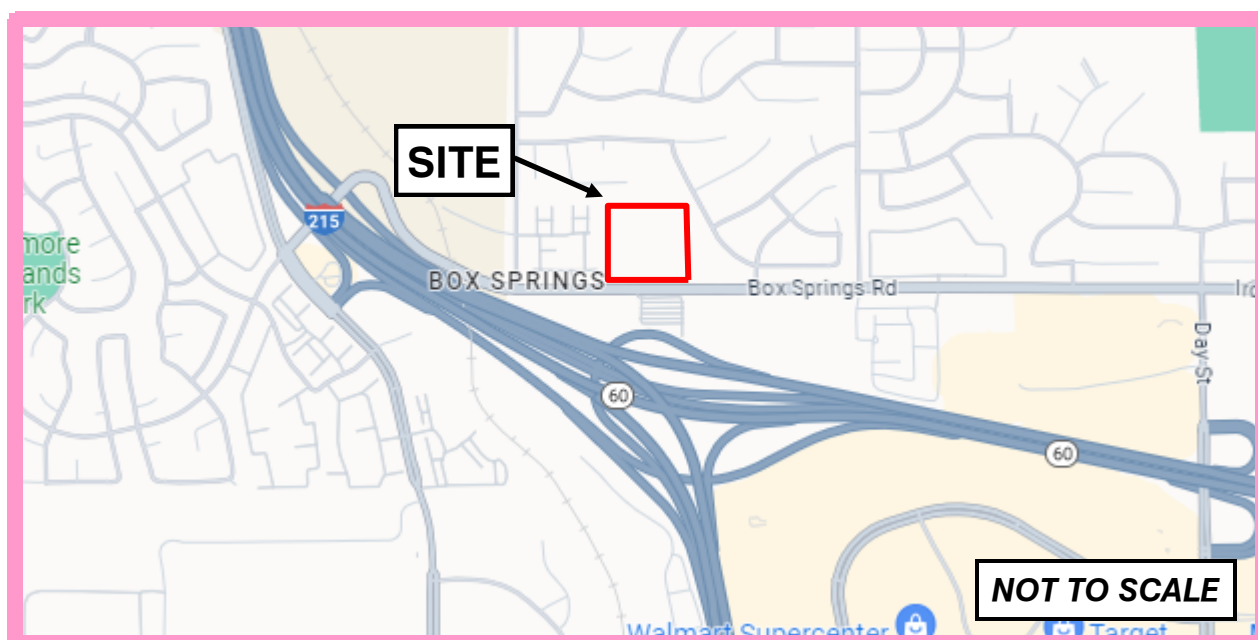
GSI understands that the proposed improvements for the Farm Bureau project would include the development of a residential townhome complex, a recreation facility, water quality stormwater BMPs, associated underground utilities and roadway/parking infrastructure improvements, including perimeter block and retaining wall improvements. Typical cut and fill grading techniques would be necessary to achieve the design grades. We further understand that the multi-family residences are proposed as one or two-story structures, with slab-on-grade/continuous footings using typical wood-frame and stucco type construction. Building loads are assumed to be typical for these types of relatively light structures. Sewage disposal is to be accommodated by tying into the regional system.

FIELD STUDIES

As indicated above, field studies conducted during our evaluation of the property for this investigation consisted of geologic reconnaissance mapping, the advancement of a total of seven (7) exploratory borings across the property for evaluation of near-surface soil and geologic conditions, sample collection, and infiltration feasibility testing. Field exploration was performed on September 18, 2023, with associated infiltration testing conducted on September 19, 2023. The borings were advanced using a hollow-stem drill rig. The



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



Base Map: Google Maps, Copyright 2023, Map Data Copyright 2023 Google

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<h1 style="text-align: center;">SITE LOCATION MAP</h1> <p style="text-align: right;">Figure 1</p>	

borings were observed and logged by a geologist from our firm who also collected representative soil samples for appropriate laboratory testing. The logs of the borings are presented in Appendix B. The approximate locations of the exploratory borings and infiltration test borings advanced for this study are presented on Figure 2 (Geotechnical Map) which uses the undated site plan provided as a base map.

GEOLOGY

Regional Geologic Setting

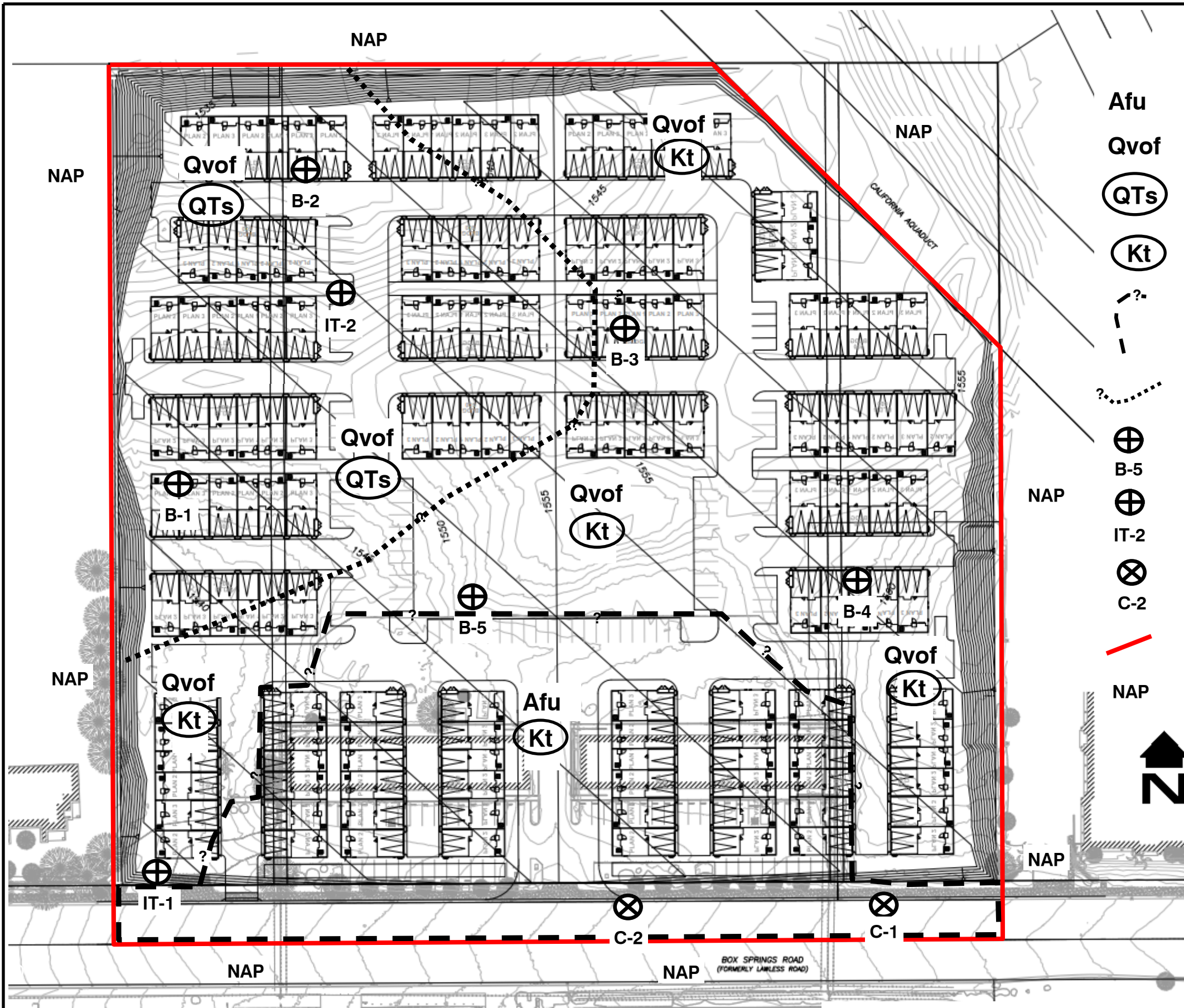
The property lies within the Perris Block, a relatively stable area located between the Elsinore and San Jacinto fault zones, in a prominent natural geomorphic province in southwestern California known as the Peninsular Ranges. The Peninsular Ranges are characterized by steep, elongated ranges and valleys that trend northwesterly. This province is typified by plutonic and metamorphic rocks (bedrock) which comprise the majority of the mountain masses, with relatively thin volcanic and sedimentary deposits discontinuously overlying the bedrock, and with Plio/Pleistocene-age to older Quaternary-age alluvial fan deposits filling in the valleys and younger alluvium filling in the incised drainages. The alluvial deposits are derived from the water borne deposition of the products of weathering and erosion of the bedrock.

Site Geology

Based on our recent subsurface investigation, and published geologic mapping by Morton and Cox (2001), the site is mantled by a relatively thin layer of early-Pleistocene-age very old alluvial fan deposits (Qvof), underlain by unnamed late-Cenozoic sedimentary bedrock (Qts). During our subsurface investigation Cretaceous-age granitic bedrock, of a tonalite composition (Kt) was also encountered, and is mapped just north of the project boundary (Morton and Cox, 2001). Based on our subsurface investigation and geologic reconnaissance mapping, the near-surface very old alluvial fan deposits have been recently tilled (disturbed) for weed abatement purposes.

Site Earth Materials

As discussed above, the earth materials encountered during our subsurface investigation included early-Pleistocene-age very old alluvial fan deposits, unnamed late-Cenozoic sedimentary rocks, and undifferentiated Cretaceous tonalite (Morton and Cox, 2001). Mappable geologic units are shown on Figure 2 (Geotechnical Map), and these units are described as follows, from youngest to oldest:



GSI LEGEND

- Afu** Artificial Fill - Undocumented
- Qvof** Quaternary - Very Old Alluvial Fan Deposits.
- (QTs)** Late-Cenozoic - Sedimentary Rocks (Circled Where Buried).
- (Kt)** Cretaceous Tonalite, Undifferentiated (Circled Where Buried).
- - - ? -** Approximate Location of Geologic Contact, Queried where Uncertain.
- . - . -** Approximate Location of Concealed Geologic Contact, Queried where Uncertain.
- ⊕** Approximate Location of Exploratory Boring.
- B-5** Approximate Location of Infiltration Test Boring.
- IT-2** Approximate Location of Infiltration Test Boring.
- ⊗** Approximate Location of Asphalt Core.
- C-2** Approximate Location of Asphalt Core.
- - -** Approximate Boundary of Property Under the Purview of this Report.
- NAP** Not A Part of this Study.

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.



Figure 2

GEOTECHNICAL MAP

W.O. 8676-A-SC	DATE: 11/23	SCALE: 1'=100'
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Base Map Provided by Foremost Pacific, 2023

Quaternary - Very Old Alluvial Fan Deposits (Map Symbol - Qvof)

As encountered during our recent field work, the site is mantled by up to 2½ to 20 feet of very old alluvial fan deposits, underlain by sedimentary and granitic bedrock materials at depth. These materials were observed to consist primarily of brown to reddish brown silty sands and sandy silts. The very old alluvial fan deposits were generally dry to moist, with a loose consistency near the surface, likely due to previous tilling for weed abatement purposes. At depth, these alluvial materials were generally moist to saturated, with a medium stiff to hard consistency, where silty. Expansion index (E.I.) testing performed on a representative sample of the fan deposits indicates a very low expansive soil condition (E.I. 0-20), on a preliminary basis. However, at the conclusion of site grading, low expansive soils (E.I. 21-50) may not be precluded from occurring. The near-surface weathered old alluvial fan deposits (upper 3 to 7 feet) are considered unsuitable for the support of settlement-sensitive improvements (i.e., building foundations, concrete slab-on-grade floors, site walls, pavements, exterior hardscape, etc.) and will require surficial removal and recompaction during rough grading. The very old alluvial fan deposits may be reused for compacted fills, provided that they have been cleansed of deleterious materials (i.e., weeds, grasses, and concentrations of organic matter), prior to placement onsite as engineered fill.

Unnamed Late-Cenozoic Sedimentary Rocks (Map Symbol - QTs)

As encountered onsite, late-Cenozoic sedimentary rocks were logged underlying the alluvial fan deposits on the northwestern portion of the site. Upon drilling, the sedimentary bedrock deposits generally consisted of grayish brown to dark gray silty sands. The sedimentary rock deposits were generally damp, and were dense to very dense with depth; however, they were penetrated with the hollow stem auger drill rig, with moderate production, to depths ranging from 16½ to 26½ feet below the existing ground surface. If encountered during anticipated remedial removals or planned cuts, any weathered late-Cenozoic sedimentary rock deposits (upper 1 to 2 feet) may also need to be removed and recompacted during rough grading. These sedimentary bedrock deposits may be reused for compacted fills, provided that they have been cleansed of deleterious materials, prior to placement onsite as engineered fill.

Cretaceous-age - Tonalite - Undifferentiated (Map Symbol - Kt)

As encountered onsite, Cretaceous-age granitic rocks, of a tonalite composition (Morton and Cox, 2001) were logged underlying the very old alluvial fan deposits onsite. The Cretaceous-age granitic bedrock was encountered at depths ranging from 2½ feet (hillside areas) to as much as 20 feet below existing grades (low-lying areas). The granitic bedrock materials observed were typically composed of biotite-hornblende tonalite that were grayish brown, dark gray, to yellow in color, and predominately damp in moisture content. The near-surface granitic bedrock appears to be somewhat weathered to moderately weathered, becoming very dense with depth. The granitic bedrock materials

were penetrated with the hollow stem auger drill rig, with slow production, to a depth of 5½ to 10½ feet, where explored. While no bedrock outcrops exist onsite, the possibility of localized areas of non-rippable and non-trenchable bedrock materials may exist onsite in the form of slightly weathered to unweathered bedrock materials at depth. The soils derived from these granitic rocks commonly have a very low expansion potential. However, weathered, near-surface residual soils may have a low expansion potential. These granitic bedrock materials may be reused for compacted fills, provided that they have been cleansed of deleterious materials, and any oversized rock (greater than 12 inches) prior to placement onsite as engineered fill.

GROUNDWATER

Seeps, springs, mottled soils, perched groundwater, or other indications of a high regional groundwater level were not noted at the subject site during the time of our field investigation. Based on our review of the California Department of Water Resources Water Data Library (CDWR; 2023), nearby well measurements reviewed appear to be regional groundwater measurements at depths of 56.4 and 83.6 feet (Station No. 339387N1172448W001, and No. 339480N1172444W001, respectively) at sites with similar ground surface elevations. As such, the regional groundwater table, at depth, is not anticipated to significantly affect the planned improvements, provided the conclusions and recommendations presented herein are properly incorporated into the design and construction of the project. In general, perched groundwater and seepage may occur along fill-alluvial and fill-bedrock contacts, or along discontinuities (sand lenses, bedding planes, joints, and fractures) due to migration from adjacent drainage areas and development during or after periods of above normal or heavy precipitation or irrigation.

FAULTING AND REGIONAL SEISMICITY

Local and Regional Faults

Our review indicates that there are no known active faults crossing this site, and the site is not within an Alquist-Priolo Earthquake Fault Zone (California Geological Survey [CGS], 2018). However, the site is situated in a region subject to strong earthquakes occurring along active faults. These faults include, but are not limited to, the Claremont fault, the Rialto-Colton fault, and others within the San Jacinto fault zone; and the San Andreas Fault system.

According to Blake (2000a), the closest known active faults to the site are the San Bernardino and San Jacinto Valley-Casa Loma segments of the San Jacinto fault zone, and are both located approximately 6.1 miles (9.8 km) north-northeast of the site. Both segments of the San Jacinto fault zone have demonstrated movement in the Holocene Epoch (i.e., last 11,700 years), and therefore, are considered active and are located within

Alquist-Priolo Earthquake Fault Zones (CGS, 2018). Cao, et al. (2003) indicates that the San Bernardino and San Jacinto Valley/Casa Loma segments of the San Jacinto fault zone are both “A” faults and are capable of producing a maximum magnitude (M_w) 6.7 and 6.9 earthquake, respectively. The possibility of ground acceleration, or shaking at the site, may be considered as approximately similar to the Southern California region as a whole.

Seismicity

The acceleration-attenuation relations of Bozorgnia, Campbell, and Niazi (1999), have been incorporated into EQFAULT (Blake, 2000a). For this study, peak horizontal ground accelerations anticipated at the site were determined based on the mean plus 1 - sigma attenuation curves developed by those authors. The EQFAULT computer program 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) is computed by user-selected acceleration-attenuation relations that are contained in EQFAULT. Based on the EQFAULT program, peak horizontal ground accelerations (deterministic acceleration values) from an upper bound event at the site may be on the order of 0.5013g.

Historical Site Acceleration

Historical site seismicity was evaluated with the acceleration-attenuation relations of Bozorgnia, Campbell, and Niazi (1999) and the computer program EQSEARCH (Blake, 2000b). This program was used to perform a search of historical earthquake records for magnitude 5.0 to 9.0 seismic events within a 100 km radius, between the years 1800 to May 8, 2021. Based on the selected acceleration-attenuation relation, a peak horizontal ground acceleration has been estimated, which may have affected the site during the specific seismic events in the past. Based on the available data and attenuation relationship used, the estimated maximum (peak) site acceleration during the period 1800 to May 8, 2021, was 0.447g. In addition, a seismic recurrence curve is also estimated/generated from the historical data (see Appendix C).

Seismic Design Parameters

Based on the site conditions, the following table summarizes the site-specific design criteria obtained from the 2022 CBC (CBSC, 2022), 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, 2023) was used for design (<https://seismicmaps.org/>). The short spectral response uses a period of 0.2 seconds.

2022 CBC SEISMIC DESIGN PARAMETERS		
PARAMETER	VALUE	2022 CBC OR REFERENCE
Risk Category	I, II, or III	Table 1604.5
Site Class	C	Section 1613.2.2/Chap. 20 ASCE 7-16 (p. 203-204)
Spectral Response - (0.2 sec), S_s	1.5g	Section 1613.2.1 Figure 1613.2.1(1)
Spectral Response - (1 sec), S_1	0.6g	Section 1613.2.1 Figure 1613.2.1(2)
Site Coefficient, F_a	1.2	Table 1613.2.3(1)
Site Coefficient, F_v	1.4	Table 1613.2.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec), S_{MS}	1.8g	Section 1613.2.3 (Eqn 16-36)
Maximum Considered Earthquake Spectral Response Acceleration (1 sec), S_{M1}	0.84g	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	1.2g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.56g	Section 1613.2.4 (Eqn 16-39)
PGA_M - Probabilistic Vertical Ground Acceleration may be assumed as about 50% of these values.	0.747g	ASCE 7-16 (Eqn 11.8.1)
Seismic Design Category	D	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⁾	6.1 mi (9.8 km) ⁽²⁾
Upper Bound Earthquake San Jacinto (San Jacinto Valley/Casa Loma) Segment	$M_w = 6.9$ ⁽¹⁾
⁽¹⁾ - Cao, et al. (2003)	
⁽²⁾ - USGS (2021)	

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 2022 CBC (CBSC, 2022) 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.

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, recommended remedial site grading, and typical site development procedures:

- Liquefaction
- Lateral Spreading
- Subsidence
- Ground Lurching or Shallow Ground Rupture
- Tsunami
- Seiche

A review of the Riverside County Information Technology (RCIT, 2023), or “Map My County” website indicates that the site is not located within a County of Riverside fault zone. However, based on our review, the site is located within a zone of “low” to “very low” liquefaction potential, and is characterized as being potentially susceptible to subsidence (RCIT, 2023). Our general liquefaction screening evaluation (pursuant to Special Publication 117 [CGS, 2008 SP117]) indicates that the potential for liquefaction and associated adverse effects within the site is considered low, based on the dense/medium stiff to very dense/hard very old alluvial fan deposits which underlie the site at shallow depths, the very dense/hard bedrock materials (sedimentary and granitic) at depth, and the anticipated removal and recompaction of near-surface potentially compressible soils during site grading activities.

The effects of areal subsidence generally occur at the transition or boundaries between low-lying areas and adjacent hillside terrain, where materials of substantially different engineering properties (i.e., thick alluvium vs. bedrock) are present, or in areas of overdraft owing to groundwater withdrawal, usually where bounded by Neogene faults. Our review of available data, as well as stereoscopic aerial photographs (USDA, 1980), showed no features generally associated with areal subsidence (i.e., radially-directed drainages flowing into a depression(s), linearity of depressions associated with mountain fronts, etc.), directly on the project site. In view of the nature of the underlying very old alluvial fan deposits, the shallow bedrock materials, and lack of onsite faulting, the potential for this phenomena to affect the site is considered very low.

In addition, ground fissures are generally associated with excessive groundwater withdrawal and associated subsidence, or active faulting. Our review did not reveal any information that active faulting or excessive groundwater withdrawal, or ground fissures, or hydroconsolidation in the specific site location, is occurring at this time. Therefore, the potential for ground fissures is also considered very low.

MASS WASTING/LANDSLIDE SUSCEPTIBILITY

Mass wasting refers to the various processes by which earth materials are moved down slope in response to the force of gravity. Examples of these processes include slope creep, surficial failures, and deep-seated landslides. Creep is the slowest form of mass wasting and generally involves the outer 5 to 10 feet of a slope surface. During heavy rains, such as those in El Niño years, creep-affected materials may become saturated, resulting in a more rapid form of downslope movement (i.e., landslides or surficial failures). For this relatively low relief (gently-sloping) site, geomorphic expressions indicative of past mass wasting events (i.e., scarps and hummocky terrain) were not observed on the property during our field studies, nor in our review of regional geologic mapping. Further, no adverse geologic structures were encountered during our subsurface exploration. Regional geologic maps do not indicate the presence of landslides on the property. However, based on the granular nature of the onsite earth materials, the onsite soils are considered erosive. Therefore, slopes composed of these materials may be subject to rilling, gullyng, sloughing, depending on rainfall severity and surface drainage practices. Such risks can be minimized through properly designed, and regularly and periodically maintained surface drainage.

LABORATORY TESTING

Classification

Soils were classified visually according to the Unified Soils Classification System. The soil classifications are shown on the Boring Logs presented in Appendix B. The Laboratory Test Results are presented in Appendix D.

Moisture-Density

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

Laboratory Standard

The maximum dry density and optimum moisture content was determined for the major soil type encountered within the exploratory borings. The laboratory standard used was ASTM D 1557. The moisture-density relationships obtained are shown below:

SOIL TYPE	BORING AND DEPTH (ft.)	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
Silty Sand, Brown (Rock Corrected)	B-3 @ 0-5	137.5	6.2
Silty Sand, Brown (Uncorrected)	B-3 @ 0-5	134.6	6.9

Expansion Potential

Expansion index (E.I.) testing was performed on a representative sample of site earth materials. E.I. test results are presented in the following Table. Additional E.I. testing should be conducted at the conclusion of site grading to further evaluate the preliminary test results obtained.

SOIL TYPE	LOCATION & DEPTH (FT)	EXPANSION INDEX (E.I.)	EXPANSION POTENTIAL
Brown, Silty SAND	B-3 @ 0-5	<5	Very Low

Soluble Sulfates/Corrosion

A representative sample of site soil was analyzed for soluble sulfates, chloride, pH, and resistivity. The soluble sulfate and corrosion potential results are presented in the following Table. Additional sulfate/corrosion testing should be conducted at the conclusion of site grading to further evaluate the preliminary test results obtained.

LOCATION AND DEPTH (FT.)	SOLUBLE SULFATES (PERCENTAGE BY WEIGHT)	CHLORIDE (PPM)	pH	RESISTIVITY (OHMS-CM)
B-3 @ 0-5	<0.003	43	7.5	8,800

For preliminary planning purposes, and based upon the soluble sulfate test results and the latest edition of the 2022 CBC (CBSC, 2022), the soluble sulfate content is considered Class "S0" per the ACI 318-14 (0.00 to 0.10 soil percentage by weight is considered Class "S0"). As such, sulfate-resistant concrete is currently not anticipated. Based on the results of the resistivity and pH testing, the onsite soils are generally considered slightly alkaline (a pH of 7.4 to 7.8 is considered slightly alkaline), and are considered moderately corrosive to ferrous metals in a saturated state (2,000 to 10,000 ohm-cm is considered moderately corrosive). Chlorides are generally low.

Although the site soils are categorized as being moderately corrosive toward ferrous metals, 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 laboratory test results obtained, a consulting corrosion engineer may be retained to provide specific recommendations for foundations, utility piping, etc, as warranted.

Direct Shear Testing

Direct shear testing was performed on a remolded sample of site earth materials collected from the borings in general accordance with ASTM D 3080. The shear testing results are provided in the following table, and in Appendix D.

SAMPLE LOCATION AND DEPTH (FT)	PRIMARY		RESIDUAL	
	COHESION (PSF)	FRICTION ANGLE (DEGREES)	COHESION (PSF)	FRICTION ANGLE (DEGREES)
B-3 @ 0-5	210	38	110	37

PERCOLATION/INFILTRATION TESTING

In general accordance with guidelines of the Riverside County Flood Control (RCFC, 2011) Design Handbook for Low Impact Development Best Management Practices, and following errata (RCFC, 2016), two (2) percolation tests were conducted at the locations provided by the client (see Geotechnical Map, Figure 2). The percolation testing was conducted within the proposed areas of the BMP systems, at a depth of 5 feet. The percolation/infiltration testing was performed to further evaluate site conditions with respect to the proposed water quality BMP systems that will retain and filter onsite storm water. The percolation testing was performed in general conformance with the RCFC (2011 and 2016) and CASQA (2003) design handbooks for such testing. The field percolation testing and geologic logging were performed by a staff geologist from our firm. Logs of the borings advanced for this study are included in Appendix B. The field percolation data sheets are presented in Appendix E. Procedures for testing are outlined briefly below:

Percolation Test Procedures

- Test Borings:**
1. Diameter - 8 inches.
 2. After the removal of loose materials, 2 inches of gravel was placed on the bottom of each test cup.
 3. A perforated pipe was then installed within each test location to facilitate accurate field measurements and prevent caving during the pre-soak period and next day testing.

- Pre-Soaking:** After the installation of the perforated pipes, the boring was filled with clear water several times. The pre-soak period for percolation tests continued overnight, as all the water did not seep away while the tester was present.
- Sandy Soil Test:** During the sandy soil test period, two (2) consecutive measurements were conducted at each test location at intervals of approximately 25 minutes. Less than 6 inches of water seeped away during each of the two (2) test measurements within the percolation test locations, therefore a 30 minute test period was selected for standard (percolation) testing.
- Testing:** After required pre-soak period and sandy soil test periods, percolation testing measurements were made. A column of clear water was re-established within each of the test locations. The drop in water level was measured from a fixed reference point, refilling after each test measurement. For each test, a series of test measurements were taken for a minimum of six (6) hours, at time intervals of 30 minutes.
- Accuracy:** All test measurements were read to the nearest 1/4-inch.
- Test Results:** Calculations from our field testing indicate percolation rates of 10 minutes/inch in both locations. Per the RCFC (2011) guidelines, the percolation rates obtained were then converted to infiltration rates utilizing the “Porchet Method,” to be used by the design engineer for appropriate sizing of the BMP systems (RCFC, 2011). The converted infiltration rates obtained varied between 0.56 and 0.42 inches/hour, with an average of 0.49 inches/hour. Typically, the lowest infiltration rate obtained is applied to the design; therefore, an unfactored (i.e., no factor of safety) design infiltration rate (DIR) of 0.42 inches/hour may be used by the design engineer. The converted infiltration rates, along with the formulas used are presented in Figure 3 (Percolation Rate to Infiltration Rate Conversion).

USDA Site Soil Groups, Soil Units, Ksat Values

Our review of the United States Department of Agriculture (USDA, 2023) Web Soil Survey (<https://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm>), indicates four (4) major soil units underlie the site. These include the Fallbrook fine sandy loam, the Hanford coarse sandy loam, the Monserate sandy loam, and the Monserate sandy loam, shallow. The Fallbrook fine sandy loam (FkD2) and the Monserate sandy loam (MmC2) are the two soil units that generally underlie the areas of the proposed stormwater basins, with FkD2 comprising the majority of the northwestern section of the site and FkD2 and MmC2 both appearing in the southwest. Based on our review, the soils onsite belong to Hydrologic

Percolation Rate to Infiltration Rate Conversion

$$* \text{ Infiltration Rate (I}_t\text{)} = \frac{\Delta H \pi r^2 60}{\Delta t (\pi r^2 + 2\pi r H_{\text{avg}})} = \frac{\Delta H 60 r}{\Delta t (r + 2H_{\text{avg}})}$$

Where:

- I_t = tested infiltration rate, inches/hour
- ΔH = change in head over the time interval, inches
- Δt = time interval, minutes
- r = effective radius of test hole
- H_{avg} = average head over the time interval, inches

		Δt	Init Level	Fnl Level	ΔH	H_{avg}	I_t	Low = 0.42 Average = 0.49 **DIR= 0.42
Infiltration Test Numbers	IT-1 @ 5 ft.	30	21	18	3	19 1/2	0.56	
	IT-2 @ 5 ft.	30	28	25	3	26 1/2	0.42	

* Conversion per the "Porchet Method" (RCFWCD, 2011)

** DIR = Design Infiltration Rate

Soil Groups “A,” “C,” and “D,” with the capacity of the most limiting soil layer to transmit water classified as “very low” to “high”. The “ K_{sat} ” value (i.e., hydraulic conductivity or infiltration rate) for the soil type onsite was evaluated to be within the range of 0.00 (very low) to 5.95 (high) inches per hour. The results of site specific infiltration testing (this study) generally appear consistent with the findings presented in the USDA data set.

Infiltration Basin Siting Requirements

Our review of the general infiltration basin siting requirements and limitations (CASQA, 2003), indicates sites characterized as belonging to Hydrologic Soil Group “A,” “B,” and “C” may be suitable for infiltration, requiring a minimum soil infiltration rate of 0.5 inches/hour (CASQA, 2003). Based on our review of historic regional groundwater levels and recent onsite subsurface investigation, and provided a relatively shallow BMP systems are constructed, a minimum 10-foot vertical separation from the bottom of the BMP systems to the top of historic high groundwater levels will be maintained.

The design engineer will need to review basin siting requirements by CASQA (2003) and the converted rates obtained during this study with respect to the proposed water quality BMP systems. An appropriate factor of safety (FOS), per the RCFC (2011) BMP design handbook, should also be applied by the design engineer.

Onsite Storm Water Quality Best Management Practice (BMP) Systems

It is our understanding that onsite infiltration-runoff retention systems (OIRRS) are planned for Best Management Practices (BMPs) or Low Impact Development (LID) principles for the project, certain guidelines must be followed in the planning, design, and construction of such systems. Such systems, 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. Locally, relatively impermeable formations include: clayey surficial soils, igneous and metamorphic bedrock, as well as future fine grained fill soils.

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.
- Wherever possible, infiltration systems should not be installed within 50 feet of the tops of slopes steeper than 15 percent or within $H/3$ from the tops of slopes (where “H” equals the height of slope).
- Impermeable liners used in conjunction with basins should consist of a 30-mil polyvinyl chloride (PVC) membrane that is covered by a minimum of 12 inches of clean soil, free from rocks and debris, at a maximum inclination of 4:1 (h:v), and meets the following minimum specifications:

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

- Wherever possible, infiltrations systems should not be placed within a distance of $H/2$ from the toes of slopes (where H equals the height of slope).
- The landscape architect should be notified of the location of the proposed OIRRS. If landscaping is proposed 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.

- Infiltrations systems should not be installed within 8 feet of building foundations utility trenches, and walls, or a 1:1 (horizontal to vertical [h:v]) slope (down and away) from the bottom elements of these improvements. Alternatively, deepened foundations 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 (Expansion Index [E.I.] ≥ 51) or soils with a relatively high plasticity index (P.I. > 20).
- Infiltration systems should not be installed where the vertical separation of the groundwater level is less than 10 feet from the base of the system.
- Where permeable pavements are planned as part of the system, the site Traffic Index (T.I.) Should be less than 25,000 Average Daily Traffic (ADT), as recommended in Allen, et al. (2011).
- Infiltration systems should be designed using a suitable factor of safety (FOS) to account for uncertainties in the known infiltration rates (as generally required by the controlling authorities), and reduction in performance over time.
- As with any OIRRS, proper care will need to 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.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

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

- Earth materials characteristics and depth to competent bearing material.
- On-going expansion and corrosion potential of site soils.
- Erosiveness of site earth materials.
- Potential for perched water during and following site development.
- Regional seismic activity.

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

EARTHWORK CONSTRUCTION RECOMMENDATIONS

General

All earthwork should conform to the guidelines presented in the 2022 CBC (CBSC, 2022), the requirements of the City of Moreno Valley and County of Riverside, and the General

Earthwork and Grading Guidelines presented in Appendix F, except where specifically superceded in the text of this report. Prior to earthwork, a GSI representative should be present at the preconstruction meeting to provide additional earthwork guidelines, if needed, and review the rough grading and earthwork schedule.

During earthwork construction, all site preparation and the general grading procedures of the contractor should be observed and the fill selectively tested by a representative(s) of GSI. If unusual or unexpected conditions are exposed in the field, they should be reviewed by this office and, if warranted, modified or additional recommendations will be offered. All applicable requirements of local and national construction and general industry safety orders, the Occupational Safety and Health Act (OSHA, 2011), and the Construction Safety Act should be met. It is the onsite general contractor's and individual subcontractors' responsibility to provide a safe working environment for our field staff who are onsite. GSI does not consult in the area of safety engineering.

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 document or further evaluate geologic conditions. Although unlikely, if adverse geologic structures are encountered, supplemental recommendations and earthwork may be warranted.
3. In general, and based upon the available data to date, regional groundwater will not be a factor in site development or underground utility installation. However, seepage may be encountered along fill contacts or throughout the site along with seasonal perched water. Seepage and a transient perched water table can also develop along, or near, the contact between near surface fills and the underlying native soil, most likely after heavy rains, or due to irrigation practices or other factors not evident at the time of our review. This may occur after development. As such, the need for localized subdrainage systems for the control of seepage and perched water should be anticipated.
4. Based upon the proposed development plan and our field exploration, the very old alluvial fan deposits, and bedrock materials (sedimentary and granitic) throughout the site should be readily rippable with conventional earthwork equipment, in good working order.
5. Due to the non-cohesive nature of some of the onsite materials, some caving and sloughing should be anticipated in all 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 F. Specific recommendations are provided below.

Demolition/Grubbing

1. Any existing surficial/subsurface structures (i.e., foundations, septic systems, etc.), trees and major vegetation, bushes, 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 or drain lines, septic systems, or other subsurface structures that are uncovered during the recommended removals, so that appropriate remedial recommendations can be provided.
3. Cavities or loose soils remaining after demolition and site clearance should be cleaned out, observed by the soils engineer, processed, and replaced with fill that has been moisture conditioned to at least optimum moisture content and compacted to at least 90 percent of the laboratory standard (ASTM D 1557), if not removed by proposed cuts.

Treatment of Existing Ground

1. All near-surface weathered very old alluvial fan deposits and bedrock materials (sedimentary and granitic) should be removed to either competent very old alluvial fan deposits or bedrock materials (i.e., greater than or equal to 85 percent compaction, or greater than or equal to 105 pcf for in-place native materials). For preliminary planning purposes, removal depths are estimated to be approximately 3 to 7 feet, with an average of approximately 5 feet across the site, with the potential for localized deeper removals. However, a minimum of 2 feet of compacted fill should underlie proposed building foundations. 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 recompact to a minimum relative compaction of 90 percent of the laboratory standard.
3. The existing site soils and earth materials may be reused as compacted fill provided that any significant concentrations of vegetation and miscellaneous trash/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 significant vegetation and debris prior to fill placement.
2. 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. While appreciable amounts of oversized rock are currently not anticipated during site grading, several oversized boulders and oversized rocks are located in the northwest corner of the property, and appear associated with grading of the adjoining property to the west. Any oversized rock materials greater than 12 inches in diameter should not be placed within 10 feet of finish grade. General guidelines for rock placement are provided in Appendix F.
4. Any import materials should be observed and determined suitable by the soils engineer prior to placement on the site. Foundation designs may be altered if import materials have a greater sulfate or expansion values than the onsite materials encountered during our preliminary investigation.

Transition and Overexcavation Areas

In order to reduce the potential for differential settlements between cut and fill materials or materials of differing engineering properties, 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, if any, should be overexcavated and capped with at least 3 feet of fill or 2 feet below the bottom of proposed footings, whichever is greater.

Fill Quality

Surficial onsite soils (very old alluvial fan and bedrock materials) generally consist of admixtures of silty sand, sand, clayey sand, and localized silt. Rippable zones of weathered bedrock (sedimentary and granitic) will likely generate good quality fill consisting of silty sand to sand and brittle rock fragments within the near surface, while oversized materials (floaters 12-inch plus) may not be precluded at depth, or locally near the surface. As discussed previously, several oversized boulders and oversized rocks are located in the northwest corner of the property. Per Code, the placement of oversized materials (i.e., 12-inch and larger) should be no closer than 10 feet from finish grade. Hold down depths of oversized material should also consider other site improvements, such as swimming pools and spas, as well as disclosure issues. General guidelines for rock placement are provided in Appendix F.

Preliminary Earthwork Factors

Preliminary earthwork factors (shrinkage and bulking) for the subject property have been estimated based upon our field and laboratory testing, visual site observations, and experience in the site area. It is apparent that shrinkage would vary with depth and with areal extent over the site based on previous site use. Variables include vegetation, weed control, discing, and previous filling or exploring. However, all these factors are difficult to define in a three-dimensional fashion.

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

Very Old Alluvial Fan Deposits	5% to 10% shrinkage
Cenozoic Sedimentary Rock Deposits (weathered - upper 5')	3% to 5% shrinkage
Cenozoic Sedimentary Rock Deposits (unweathered)	2% to 3% bulking
Cretaceous Granitic Bedrock Deposits (weathered- upper 3')	1% to 2% shrinkage
Cretaceous Granitic Bedrock Deposits (unweathered)	3% to 7% bulking

An additional shrinkage factor item would include the removal of root systems of individual large plants or trees. These plants and trees vary in size, but when pulled, may result in a loss of ½ to 1 cubic yard, to locally greater than 1 cubic yard of volume, respectively. The above facts indicate that earthwork balance for the site may 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.

Subdrains

The possibility that local seepage may be encountered at the subject site is considered low. However, the need for subdrainage systems for the control of localized groundwater seepage cannot be precluded. If required, subdrainage for slopes and embankments should adhere to the specifications in Appendix F, which should be incorporated into the project plans and construction documents.

Slope Considerations and Slope Design

A detailed evaluation of slope stability is beyond the scope of this current preliminary report, and generally not required for slopes less than 30 feet in vertical height, at gradients of 2:1 (h:v), or flatter. However, a general discussion of slope stability is included herein. Based on our experience on adjacent projects, proposed fill and cut slopes constructed onsite, to the heights proposed, should be grossly and surficially stable provided the recommendations contained herein are implemented during site development.

All slopes should be designed and constructed in accordance with the minimum requirements of the City of Moreno Valley and County of Riverside, the recommendations

in the General Earthwork and Grading Guidelines section of this report (Appendix F), 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 30 feet in height. Fill slopes should be properly built and compacted to a minimum relative compaction of 90 percent throughout, including the slope surfaces.
2. Cut slopes should be designed at gradients of 2:1 (h:v) or flatter and should not exceed about 30 feet in height. While stabilization of such slopes is not anticipated, locally adverse geologic conditions (i.e., daylighted joints/fractures, severely weathered bedrock deposits may be encountered which may require remedial grading, stabilization, or laying back of the slope to an angle flatter than the adverse geologic condition. All slope configurations should minimally conform to City/County guidelines. General guidelines for slope construction are presented in Appendix F.
3. Local areas of highly to severely weathered bedrock or non-cohesive materials may be present. Should these materials be exposed in cut slopes, the potential for long term maintenance or possible slope failure exists. Evaluation of cut slopes during grading would be necessary in order to identify any areas of severely weathered rock or non-cohesive materials. Should any of 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. Where loose materials are exposed on the cut slopes, the project's engineering geologist would require that the slope be cleaned prior to making their final inspection. Final approval of the cut slope can only be made after the slope being fully cut and cleaned.
4. Loose rock debris and fines remaining on the face of the cut slopes should be removed during grading. This can be accomplished by the use of a slope-board, as warranted. Cut slopes should be mapped by the project engineering geologist during grading to allow amendments to the recommendations should exposed conditions warrant alternation of the design or stabilization.
5. 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.

PRELIMINARY RECOMMENDATIONS - FOUNDATIONS

General

Preliminary recommendations for foundation design and construction are provided in the following sections. These preliminary recommendations have been developed from our

understanding of the currently planned site development, site observations, subsurface exploration, laboratory testing, and engineering analyses. Foundation design should be re-evaluated at the conclusion of site grading/remedial earthwork for the as-graded soil conditions. Although not anticipated, revisions to these recommendations may be necessary. In the event that the information concerning the proposed development plan is not correct, or any changes in the design, location or loading conditions of the proposed additions are made, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report are modified or approved in writing by this office.

The information and recommendations presented in this section are not meant to supercede design by the project structural engineer or civil engineer specializing in structural design. Upon request, GSI could provide additional input/consultation regarding soil parameters, as related to foundation design.

Expansive/Corrosive Soils

The laboratory testing conducted for this study indicates that the majority of onsite soils do not meet the criteria of detrimentally expansive soils as defined in Section 1803.5.3 of the 2022 CBC. However, some site soils are considered expansive. With recommended site grading, the overall expansive character of site soils is anticipated to be non-detrimentally expansive. Should site grading result in areas underlain with expansive soils, foundation systems constructed within the influence of detrimentally expansive soils (i.e., E.I. > 20 and P.I. \geq 15) will require specific design to resist expansive soil effects per Sections 1808.6.1 or 1808.6.2 of the 2022 CBC, and should be reviewed by the project structural engineer.

Preliminary testing indicates that site soils present a negligible sulfate exposure (exposure Class "S0" (per Table 19.3.2.1 of ACI 318R-14) to concrete. However, reinforced concrete mix design for foundations, slab-on-grade floors, and pavements should also conform to Exposure Classes "S0", "W0", and "C1" in Table 19.3.1.1 of ACI 318R-14, as concrete would likely be exposed to moisture.

Preliminary Foundation Design

The following preliminary foundation design recommendations are presented as a minimum criteria from a soils engineering viewpoint.

1. The foundation systems should be designed and constructed in accordance with guidelines presented in the 2022 CBC.
2. An allowable bearing value of 2,000 pounds per square foot (psf) may be used for the design of footings that maintain a minimum width of 12 inches and a minimum depth of 12 inches (below the lowest adjacent grade) and are founded entirely into

properly compacted, engineered fill or suitable formation. This value may be increased by 20 percent for each additional 12 inches in footing depth to a maximum value of 2,500 psf. These values may be increased by one-third when considering short duration seismic or wind loads. Foundation embedment depth excludes concrete slabs-on-grade, or slab underlayment. Foundations should not simultaneously bear on formation and engineered fill.

3. For foundations deriving passive resistance from engineered fill, a passive earth pressure may be computed as an equivalent fluid having a density of 250 pcf, with a maximum earth pressure of 2,500 psf.
4. The upper 6 inches of passive pressure should be neglected if not confined by slabs or pavement.
5. For lateral sliding resistance, a 0.35 coefficient of friction may be used for a concrete to soil contact when multiplied by the dead load.
6. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.
7. All footing setbacks from slopes should comply with Figure 1808.7.1 of the 2022 CBC. GSI recommends a minimum horizontal setback distance of 7 feet as measured from the bottom, outboard edge of the footing to the slope face.
8. Footings for structures adjacent to retaining walls should be deepened so as to extend below a 1:1 projection from the heel of the wall. Alternatively, walls may be designed to accommodate structural loads from buildings or appurtenances as described in the “Retaining Wall” section of this report.
9. Provided that the earthwork and foundation recommendations in this reported are adhered, foundations bearing on engineered fill should be minimally designed to accommodate a differential settlement of 1 inch over a 40-foot horizontal span (angular distortion = $1/480$).

Preliminary Foundation Construction Recommendations

Current laboratory testing indicates that the majority of onsite soils do not meet the criteria of detrimentally expansive soils as defined in Section 1803.5.3 of the 2022 CBC. The following foundation construction recommendations are presented as a minimum criteria from a soils engineering viewpoint. These recommendations are intended to support planned improvements underlain by at least 7 feet of non-detrimentally expansive soils (i.e., E.I. < 21 and P.I. < 15). Expansive soils may not be precluded from occurring locally. Should foundations be underlain by expansive soils at depths of less than 7 feet, they will require specific design modifications to mitigate expansive soil effects as required in Sections 1808.6.1 or 1808.6.2 of the 2022 CBC (CBSC, 2022).

1. Exterior and interior footings should be founded into engineered fill, or suitable formation, at minimum depths of 12 inches, 18 inches, or 24 inches below the lowest adjacent grade, for planned one-, two-, or three-story floor loads, respectively, with footing width per Code. Isolated, exterior column and panel pads, or wall footings, should be at least 24 inches square, and founded at a minimum depth of 18 inches into properly engineered fill or suitable formation. All footings should be reinforced with four No. 4 reinforcing bars, two placed near the top and two placed near the bottom of the footing. Reinforcement of pad footings should be provided by the projects structural engineer.
2. All exterior column footings, and perimeter wall footings, should be tied together via grade beams in at least one direction for very low expansive soils, and two directions for low expansive soils, if encountered. The grade beam should be at least 12 inches square in cross section, and should be provided with a minimum of one No. 4 reinforcing bar at the top, and one No. 4 reinforcing bar at the bottom of the grade beam. The base of the reinforced grade beam should be at the same elevation as the adjoining footings.
3. A grade beam, reinforced as previously recommended, and at least 12 inches square, should be provided across large (garage) entrances. The base of the reinforced grade beam should be at the same elevation as the adjoining footings.
4. Based on the very low to potentially low expansive soils that underlie the subject lots, the slab thickness, reinforcement, and concrete used should comply with all applicable codes, and in accordance with the foundation designers' recommendations. At minimum, concrete slabs should be reinforced with No. 3 reinforcement bars placed at 18-inch on center, in two horizontally perpendicular directions (i.e., long axis and short axis). For slab underlayment, please refer to the "Soil Moisture Transmission" section of this report.
5. All slab reinforcement should be supported to ensure proper mid-slab height positioning during placement of the concrete. "Hooking" of reinforcement is not an acceptable method of positioning.
6. The slab subgrade should be pre-soaked to at least the soils "optimum moisture content," to a depth of at least 12 inches, prior to the placement of underlayment sand and vapor retarder. Slab subgrade pre-soaking should be evaluated by the geotechnical consultant within 72 hours of the placement of the underlayment sand and vapor retarder.
7. Reinforced concrete mix design should conform to Exposure Classes "S0", "W0", and "C1" in Table 19.3.2.1 of ACI 318R-14.

Post-Tension Foundation Systems

Post-tension foundation systems may be used to mitigate the damaging effects of seismic activity or shrink/swell effects of expansive soil conditions, if encountered, on the proposed building foundations and slab-on-grade floors. The post-tension foundation designer may elect to exceed the minimal recommendations, provided herein, to increase slab stiffness performance. Post-tension (PT) design may be either ribbed or mat-type. The latter is also referred to as uniform thickness foundation (UTF). The use of a UTF is an alternative to the traditional ribbed-type. The UTF offers a reduction in grade beams. That is to say a UTF typically uses a single perimeter grade beam and “shovel” footings, but has a thicker slab than the ribbed-type. UTF perimeter footings may use an allowable vertical bearing value of 1,500 psf, if founded into approved engineered fill overlying dense unweathered sedimentary bedrock.

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-tension foundations should be designed using sound engineering practice and be in accordance with local and 2022 CBC requirements and Post Tensioning Institute (PTI) methodologies (PTI; 2004, 2008, 2012, 2013, and 2014). Upon request, GSI can provide additional data/consultation regarding soil parameters as related to post-tension foundation design.

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

Slab Subgrade Pre-Soaking

Pre-moistening of the slab subgrade soil is recommended. The moisture content of the subgrade soils should be equal to or greater than optimum moisture to a depth equivalent to the perimeter grade beam or cut-off wall depth in the slab areas (typically 12, or 18, inches for very low to low expansive soil conditions, respectively).

Pre-moistening or pre-soaking should be evaluated by the soils engineer 72 hours prior to vapor retarder placement. In summary:

EXPANSION POTENTIAL	PAD SOIL MOISTURE	CONSTRUCTION METHOD	SOIL MOISTURE RETENTION
Very Low (E.I. = 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.
Low (E.I. = 21-50)	Upper 12 inches of pad soil moisture 2 percent over optimum (or 1.2 times)	Wetting or reprocessing	Periodically wet or cover with plastic after trenching. Evaluation 72 hours prior to placement of concrete.

Perimeter Cut-Off Walls

Perimeter cut-off walls should be at least 12, or 18 inches deep for very low to low expansive soil conditions, respectively. The cut-off walls may be integrated into the slab design or independent of the slab. The cut-off walls should be a minimum of 6 inches thick (wide). The bottom of the perimeter cut-off wall should be designed to resist tension, using cable or reinforcement per the structural engineer.

Post-Tension Foundation Design

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

Post-Tension Foundation 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. Additional testing is recommended either during or following grading, and prior to foundation construction to further evaluate the soil

conditions within the upper 7 to 15 feet of pad grade. The following table presents suggested minimum coefficients to be used in the Post-Tensioning Institute design method:

Thornthwaite Moisture Index	-20 inches/year
Correction Factor for Irrigation	20 inches/year
Depth to Constant Soil Suction	7 feet or overexcavation depth, whichever is greater
Constant soil Suction (pf)	3.6
Moisture Velocity	0.7 inches/month
Plasticity Index (P.I.)*	15-45
* - The effective plasticity index should be evaluated for the upper 7 to 15 feet of earth materials.	

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

TABLE 1 - POST-TENSION FOUNDATION DESIGN	
DESIGN PARAMETER ⁽⁴⁾	VERY LOW TO LOW EXPANSION ⁽⁵⁾ (E.I. = 0-50)
e _m center lift	9.0 feet
e _m edge lift	5.2 feet
y _m center lift	0.4 inches
y _m edge lift	0.7 inch
Bearing Value ⁽¹⁾	1,500 psf ⁽¹⁾
Lateral Pressure	250 psf
Subgrade Modulus (k)	100 pci/inch
Minimum Perimeter Footing Embedment ⁽²⁾	12 inches
⁽¹⁾ Internal bearing values within the perimeter of the post-tension slab for very low to low expansive soil conditions may be increased to 2,000 psf for a minimum embedment of 12 inches, then by 20 percent for each additional foot of embedment to a maximum of 2,500 psf. ⁽²⁾ For medium to very high expansive soil conditions, internal bearing values within the perimeter of the post-tension slab may be increased to 1,500 psf for a minimum embedment of 12 inches, then by 20 percent for each additional foot of embedment to a maximum of 2,000 psf. ⁽³⁾ As measured below the lowest adjacent compacted subgrade surface (not including slab underlayment layer thickness). ⁽⁴⁾ Post-tension slab design should also be evaluated with respect to the potential differential settlements provided in this report. ⁽⁵⁾ Category Criteria: Category I Expansion Index < 50 (very low to low), or Max fill less than 25 feet thick, or fill differential less than 10 feet.	

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] third edition) parameters may be recommended. All exterior columns not supported by the post-tensioned foundation should be supported by 24 square inch isolated footings extending at least 18 inches into approved engineered fill. Exterior column footings should be tied to the post-tensioned foundation with 12 square inch, reinforced grade beams in at least one direction for very low expansive soils and two directions for low expansive soils.

Mat Foundations

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

Mat Foundation Design

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

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

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

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

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

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

Specific pre-moistening/pre-soaking and moisture testing of the slab subgrade are recommended, as previously provided in this report. Slab subgrade moisture conditioning/pre-soaking should conform to the recommendations previously provided for post-tension foundation systems.

Confirmation Testing for Final Foundation Design

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

PRELIMINARY FOUNDATION AND IMPROVEMENT SETTLEMENTS

In addition to designing slab systems (post-tension or other) for the soil conditions described herein, the estimated settlement and angular distortion values that an individual structure could be subjected to should be evaluated by a structural engineer. The levels of angular distortion were evaluated on a 40-foot length assumed as minimum dimension of buildings; if, from a structural standpoint, a decreased or increased length over which the tilt is assumed to occur is justified, this change should be incorporated into the design. The structures should be evaluated and designed for the combination of the soil parameters presented above, and the estimated total settlement, differential settlement and angular distortions provided herein.

The footings or slabs should be designed to accommodate a total static settlement of up to 2 inches and a static differential settlement of 1 inch, i.e., at least 1 inch in a 40-foot span. The structural engineer should consider these settlements and the performance of the foundation as well as the overlying structure. In addition to the above, the structural engineer should also consider estimated settlements due to short duration seismic loading and applicable load combinations, as required by the controlling authorities or the 2022 CBC. Post-construction settlement of the fill should generally be mitigated by

appropriate foundation design currently recommended, provided the design parameters presented herein are properly used in design of foundation systems.

These settlement estimates indicated above have been based on City of Moreno Valley and Riverside County overexcavation requirements, and do not preclude top of slope deformation (within Code setback zones) and settlement due to fills that have been saturated from utility leaks, pool leaks, prevailing climatic conditions, or excessive landscape irrigation.

SOIL MOISTURE TRANSMISSION CONSIDERATIONS

GSI has evaluated the potential for vapor or water transmission through the concrete floor slab, in light of typical floor coverings and improvements. Please note that slab moisture emission rates range from about 2 to 27 lbs/24 hours/1,000 square feet from a typical slab (Kanare, 2005), while floor covering manufacturers generally recommend about 3 lbs/24 hours as an upper limit. The recommendations in this section are not intended to preclude the transmission of water or vapor through the foundation or slabs. Foundation systems and slabs shall not allow water or water vapor to enter into the structure so as to cause damage to another building component or to limit the installation of the type of flooring materials typically used for the particular application (State of California, 2023). 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 versus benefit analysis (owner expectations and repairs/replacement), along with disclosure to all interested/affected parties.

Vapor transmission will occur in new slab-on-grade floors as a result of chemical reactions taking place within the curing concrete. Vapor transmission through concrete floor slabs as a result of concrete curing has the potential to adversely affect sensitive floor coverings depending on the thickness of the concrete floor slab and the duration of time between the placement of concrete and the floor covering. It is possible that a slab moisture sealant may be needed prior to the placement of sensitive floor coverings if a thick slab-on-grade floor is used and the time frame between concrete and floor covering placement is relatively short.

Considering the E.I. test results presented herein, and known soil conditions in the region, the anticipated typical water vapor transmission rates, floor coverings, and improvements (to be chosen by the Client or project architect) that can tolerate vapor transmission rates without significant distress, the following alternatives are provided:

- Concrete slab-on-grade floors (including garage slabs) should be thicker.
- Concrete slab underlayment should consist of a 15-mil vapor retarder, or equivalent, with all laps sealed per the 2019 CBC and the manufacturer’s recommendation.

The vapor retarder should comply with the ASTM E 1745 - Class A criteria, and be installed in accordance with ACI 302.1R-04 and ASTM E 1643.

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

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

- 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 concrete admixtures are 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. The vapor retarder contractor should have representatives onsite during the initial installation.

PRELIMINARY WALL DESIGN PARAMETERS

General

Recommendations for the design and construction of conventional masonry retaining walls are provided below. Recommendations for specialty walls (i.e., crib, earthstone, mechanically stabilized earth [MSE], gravity, 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 50 are used to backfill any retaining wall. The type of backfill (i.e., select or native), should be specified by the wall designer, and clearly shown on the plans. Building walls, below grade, should be water-proofed. Waterproofing should also be provided for site retaining walls in order to reduce the potential for efflorescence staining.

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 125 pcf and 135 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 native deposits for adequate vertical and lateral bearing support. All retaining wall footing setbacks from slopes should comply with Figure 1808.7.1 of the 2022 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 City of Moreno Valley standard design. Active earth pressure may be used for retaining wall design, provided the top of the wall is not restrained from minor deflections. An equivalent fluid pressure approach may be used to compute the horizontal pressure against the wall. Appropriate fluid unit weights are given below for specific slope gradients of the retained material. These do not include other superimposed loading conditions due to traffic, structures, seismic events or adverse geologic conditions. When wall configurations are finalized, the appropriate loading conditions for superimposed loads can be provided upon request.

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

SURFACE SLOPE OF RETAINED MATERIAL (HORIZONTAL:VERTICAL)	EQUIVALENT FLUID WEIGHT P.C.F. (SELECT BACKFILL)⁽²⁾	EQUIVALENT FLUID WEIGHT P.C.F. (NATIVE BACKFILL)⁽³⁾
Level ⁽¹⁾	38	50
2 to 1	55	65
⁽¹⁾ Level backfill behind a retaining wall is defined as compacted earth materials, properly drained, without a slope for a distance of 2H behind the wall, where H is the height of the wall. ⁽²⁾ SE > 30, P.I. < 15, E.I. < 21, and < 10% passing No. 200 sieve. ⁽³⁾ E.I. = 0 to 50, SE > 30, P.I. < 15, E.I. < 21, and < 15% passing No. 200 sieve.		

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 2022 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 18H 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 18H. For restrained walls, the pressure should be applied as a rectangular distribution. Please note this is for local wall stability only.

The 18H 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 18H seismic surcharge is derived from the formula:

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

Where:

P_h	=	Seismic increment.
a_h	=	Probabilistic horizontal site acceleration with a percentage of "g."
γ_t	=	total unit weight (125 to 135 pcf for site soils @ 90 percent relative compaction).
H	=	Height of the wall from the bottom of the footing or point of pile fixity.

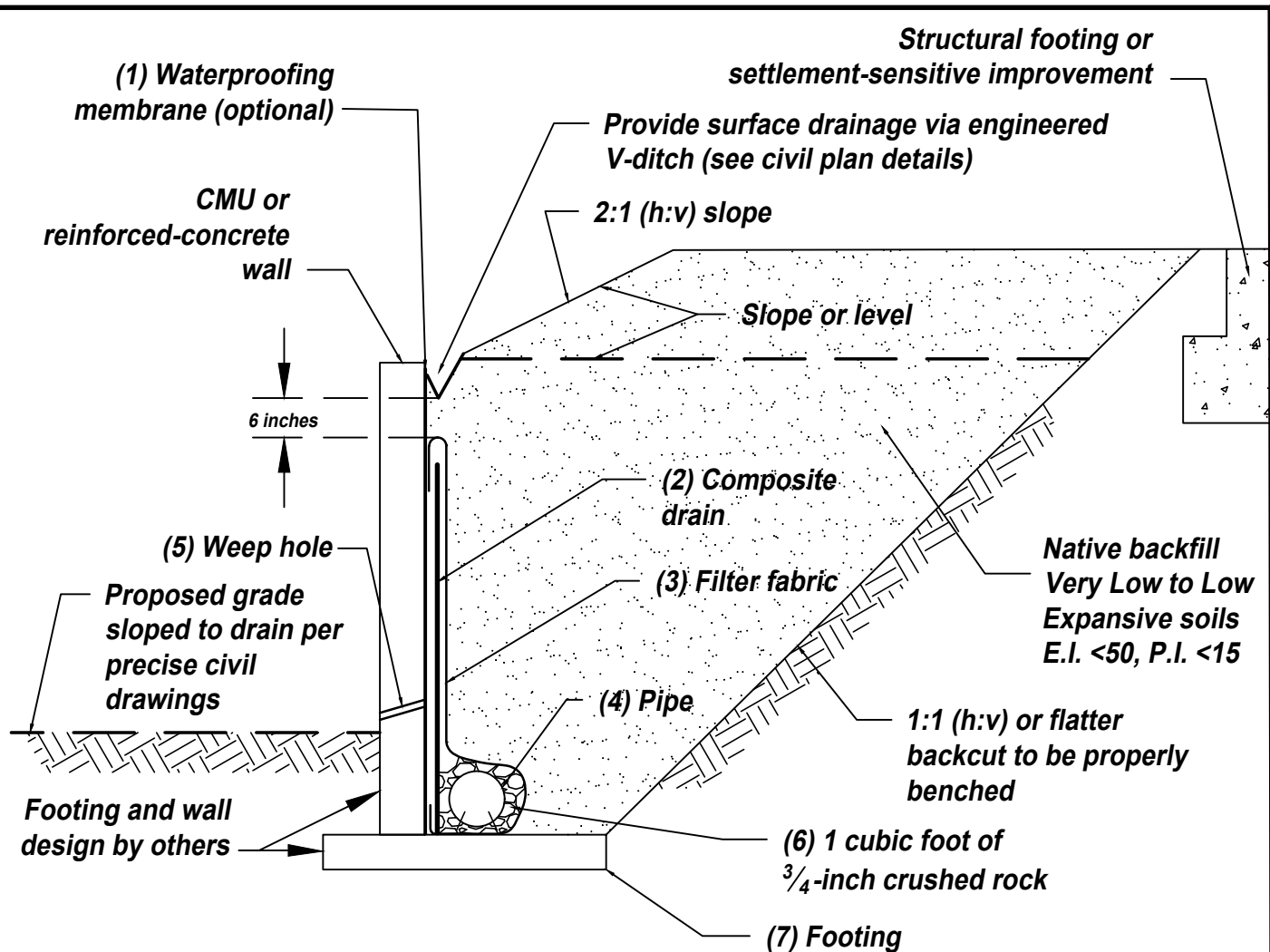
Retaining Wall Backfill and Drainage

Positive drainage must be provided behind all retaining walls in the form of gravel wrapped in geofabric and outlets. A backdrain system is considered necessary for retaining walls that are 2 feet or greater in height. Details 1, 2, and 3, present the backdrainage options discussed below. Backdrains should consist of a 4-inch diameter perforated PVC or ABS pipe encased in either Class 2 permeable filter material or $\frac{3}{4}$ -inch to 1½-inch gravel wrapped in approved filter fabric (Mirafi 140 or equivalent). For select backfill, the filter material should extend a minimum of 1 horizontal foot behind the base of the walls and upward at least 1 foot. For native backfill that has up to E.I. = 20, continuous Class 2 permeable drain materials should be used behind the wall. This material should be continuous (i.e., full length) 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. ≤ 50). Proper surface drainage should also be provided. For additional mitigation, consideration should be given to applying a water-proof membrane to the back of all retaining structures. The use of a waterstop should be considered for all concrete and masonry joints.

Wall/Retaining Wall Footing Transitions

Site walls are anticipated to be founded on footings designed in accordance with the recommendations in this report. Should wall footings transition from cut to fill, the structural consultant/wall designer may specify either:



(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).

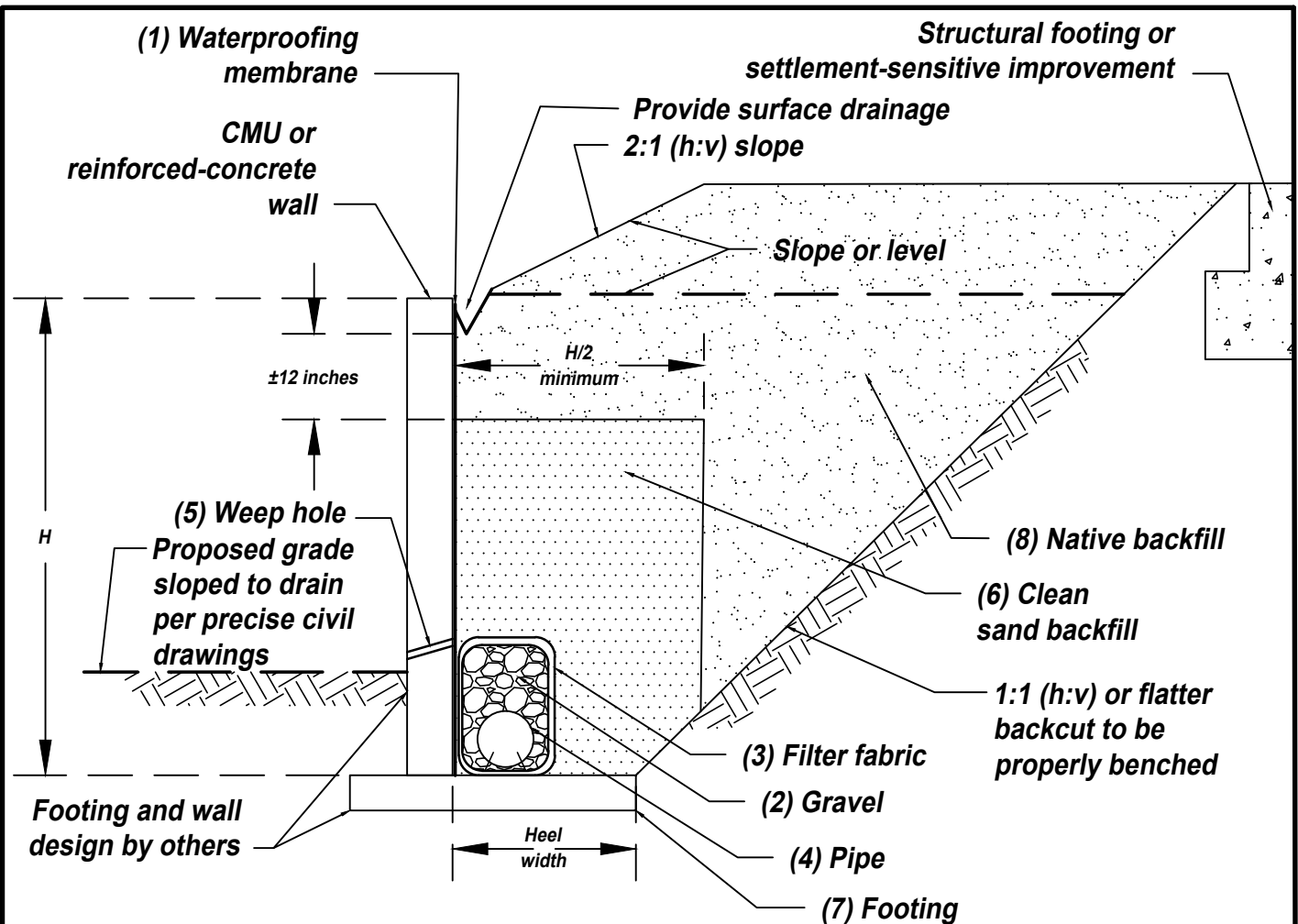
(3) Filter fabric: Mirafi 140N or approved equivalent; place fabric flap behind core.

(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.



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(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.

- a) A minimum of a 2-foot overexcavation and recompaction of cut materials for a distance of 2H, from the point of transition.
- b) Increase of the amount of reinforcing steel and wall detailing (i.e., expansion joints or crack control joints) such that a angular distortion of 1/360 for a distance of 2H on either side of the transition may be accommodated. Expansion joints should be placed no greater than 20 feet on-center, in accordance with the structural engineer's/wall designer's recommendations, regardless of whether or not transition conditions exist. Expansion joints should be sealed with a flexible, non-shrink grout.
- c) Embed the footings entirely into native formational material (i.e., deepened footings).

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

PRELIMINARY PAVEMENT DESIGN AND CONSTRUCTION

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 a general review of pavement designs for other nearby projects, and for estimation and bidding purposes, the pavement sections provided herein should be considered for preliminary design. Typically, actual pavement sections will likely vary, therefore final pavement sections should be based on actual R-value testing performed during, or shortly after, roadway grading for any proposed street and driveway/parking area improvements.

Asphaltic Concrete (AC) Pavements

The preliminary design for Asphaltic Concrete (AC), and Portland Cement Concrete Pavement (PCCP) was evaluated based on an assumed R-value of 30, and the use of concrete shoulders (curb and gutter) at the edge of PCC pavement. GSI does not recommend the use of an ADTT value of less than 25 for any pavement section, unless the ADTT significantly less than 25 is certified by a civil engineer specializing in traffic engineering.

It is our understanding that the "minimum" pavement section required by the City of Moreno Valley for a local street, modified local street, or collector is 3.6 inches of AC on 6 inches of Crushed Aggregate Base (CAB) for a traffic index of 6.0 to 7.0 (Standard Plan No. MVSI-100A-2). Per the standard plan the City of Moreno Valley does not allow the use of Crushed Miscellaneous Base (CMB). Typically actual pavement sections will likely vary,

therefore final pavement sections should be based on actual Resistance Value (R-value) testing performed during, or shortly after, roadway grading for any proposed street and roadway area improvements. Applicable sections of City of Moreno Valley ordinances should be followed during design of public roads, fire access lanes, etc.

Preliminary Pavement Design

The preliminary pavement sections presented in the following table are based on the assumed R-value of 30, the general Traffic Indices (T.I.'s) used by the City of Moreno Valley (MVSI-100A-2) for a rural (street), local (street), collector (road), and an arterial (road), and the guidelines presented in the latest revision to the California Department of Transportation (Caltrans, 2020) "Highway Design Manual" seventh edition. It is our understanding that the minimum pavement section required by the City of Moreno Valley for a rural (road) and general local (road) is 3.6 inches of AC (asphaltic concrete) on 6.0 inches of Class 2 aggregate base for a T.I. of 5.0 to 6.0. Based on the assumed R-value of 50 obtained (i.e., R=50), the following preliminary pavement designs are presented. Applicable sections of City ordinances should be followed during design of public roads, fire access lanes, etc.

Asphaltic Concrete Pavement (ACP)

Preliminary asphaltic concrete (AC) pavement sections are presented in the following table:

STREET CLASSIFICATION	TRAFFIC INDEX (T.I.) ¹	STANDARD PAVEMENT DESIGNS		
		R-VALUE	AC INCHES	CLASS 2 BASE ROCK ² INCHES
Local Street or Modified Local Street	6.0	30	3.6*	7.0
Collector (Road)	7.0	30	3.6*	10.0
Industrial Collector (Road)	10.0	30	6.0*	14.5
¹ T.I.s are based on City street design criteria (City of Moreno Valley, Standard Plan No. MVSI-100A-2). ² Denotes standard Caltrans Class 2 aggregate base rock @ ≥ 78 , SE ≥ 25 . * Denotes City of Moreno Valley minimum asphaltic concrete or aggregate base.				

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.

Limited Pavement Thickness Evaluation

As discussed previously, a secondary purpose of our study was to provide a limited offsite pavement section thickness evaluation of the exterior roadway. Our limited study included the advancement of two (2) exploratory AC cores via core-drilled samples on Box Springs Road using a subcontractor. After the advancement of the cores through the AC of the roadways structural section, the crushed aggregate base was removed by hand methods at each coring location, and the AC and aggregate base sections were measured and logged. The approximate locations of the AC cores advanced during our investigation are depicted on Figure 2 (Geotechnical Map).

Based on our site reconnaissance and field review localized areas of the offsite roadway, Box Springs Road, exhibits minimal distress (structural, surficial, and aesthetic). According to a map from the City of Moreno Valley Public Works Department, rehabilitation of the subject portion of Box Spring Road was proposed in November of 2022, and occurred some time after April 2023 (GEP, 2023) but before the date of our site investigation (September, 2023). While reports on the extent of these improvements are currently unavailable at this time, the following items were noted during our limited offsite pavement section thickness evaluation.

- It is our understanding that the recent rehabilitation of the subject portion of Box Spring Road has included a slurry-seal, and re-striping of the offsite roadway.
- The existing AC section in the subject portion of Box Springs Road was observed to be in good condition, with minor cracking within the middle lanes along construction lay-down seams.
- The offsite roadways AC section was generally design deficient at both coring locations, while the aggregate base was design compliant. A comparison of the existing roadway structural section measurements vs. the City minimum roadway structural sections for an “Arterial Roadway” (Standard Plan No. MVSI-100A-2) are presented on Table 1. A negative number within Table 1 indicates an AC or aggregate base section deficiency, while a positive number indicates an excess AC or aggregate base section.

TABLE 1										
EXISTING VS. RECOMMENDED MINIMUM ROADWAY SECTIONS*										
CORE	LOCATION	EXISTING AC (IN)	MINIMUM AC (IN)	Δ AC (IN)	% OF DESIGN**	EXISTING BASE (IN)	MINIMUM BASE (IN)	Δ BASE (IN)	% OF DESIGN**	COMMENT
C-1	Box Springs Rd. East	5.00	6.0	-1.00	-16.67	12.00	12.0	0.00	0.00	AC - Design Deficient. Base - Design Compliant
C-2	Box Springs Rd. West	4.75	6.0	-1.25	-20.83	12.00	12.0	0.00	0.00	AC - Design Deficient. Base - Design Compliant
* City of Murrieta Minimum Roadway Design Sections (Standard No.120)										
** Positive numbers indicate exceeds design recommendations; negative numbers indicates design deficient										

The roadways AC thickness was generally design deficient; while roadways base thickness was design compliant, per current City standards.

Portland Concrete Cement (PCC) Pavement

The preliminary design for Portland Cement Concrete Pavement (PCCP) was evaluated using an assumed subgrade R-value of 30, a modulus of rupture (MR) of 420 and 500 psi. GSI does not recommend the use of an ADTT value of less than 25 for any pavement section, unless the ADTT significantly less than 25 is certified by a civil engineer specializing in traffic engineering. The preliminary PCCP sections are provided in the following table:

PORTLAND CONCRETE CEMENT PAVEMENTS (PCCP)*					
TRAFFIC AREAS	CONCRETE TYPE	PCCP THICKNESS (INCHES)	TRAFFIC AREAS	CONCRETE TYPE	PCCP THICKNESS (INCHES)
Light Vehicles	520-C-2500	6.0	Dumpster Aprons (Trash Service)	520-C-2500	8.0
	560-C-3250	5.0		560-C-3250	7.0
NOTE: All PCCP is designed as un-reinforced and bearing directly on compacted subgrade. However, a 4-inch thick leveling course of compacted aggregate base, or crushed rock may be considered to improve performance. All PCCP should be properly detailed (jointing, etc.) per the industry standard. Pavements may be additionally reinforced with #4 reinforcing bars, placed 18 inches on center, each way, for improved performance.					
* To be re-evaluated based on exposed field conditions and R-values obtained following rough grading.					

The transition of the pavement from parking to traffic lanes should be made over a distance of 24 inches with crack control joints (weaken plane) or contact joints at the end of the transition. A minimum 4-inch layer of base rock in traffic and bus stop areas should be considered to improve traffic lane performance. Base rock may consist of either ¾-inch minus crushed rock or Caltrans Class 2 aggregate base. Crushed rock may be compacted by vibratory methods. Aggregate base should be compacted to a minimum relative compaction of 95 percent.

Weakened Plane Joints

Transverse and longitudinal weakened plane joints may be constructed per Greenbook Standard Specifications (2021), Section 302-6.5, or the structural/civil engineer.

Transverse weakened plane joints should be spaced no farther than 15 feet apart and no closer than 5 feet. Longitudinal weakened plane joints should be spaced no farther than 20 feet apart, but not less than 5 feet.

Expansion Joints and Contact Joints

Transverse expansion joints should be constructed at 120-foot spacings, or in accordance with City standards. Transverse and longitudinal contact joints should be constructed in accordance with the recommendations of the design engineer. Within large slab areas, joint spacings should be no greater than 20 feet.

Slab Reinforcement

The preliminary PCC Pavements for this project are designed as unreinforced and should perform adequately, assuming proper construction. If additional control of internal slab stresses (i.e., curing shrinkage, thermal expansion and contraction), and the effects of expansive soil subgrades is desired, then the use of No. 4 reinforcing bars, 18 inches on center each way, should be considered.

Subgrade should be compacted to a minimum relative compaction of 95 percent. Aggregate base compaction should be 95 percent of the maximum dry density (ASTM D 1557). If adverse conditions (i.e., saturated ground, etc.) are encountered during preparation of subgrade, special construction methods may need to be employed. These recommendations should be considered preliminary. R-value testing and pavement design analysis should be performed upon completion of grading for the project.

Concrete/Pervious Pavers

Concrete pavers should be underlain by a minimum of 8 inches of aggregate base, overlain by a leveling-course of sand, or per the manufacturers guidelines. Manufacturer's guidelines should be reviewed for concordance with the intent of the geotechnical report and the underlying soil conditions. Prior to aggregate base placement the subgrade soils should be compacted to a minimum relative compaction of 95 percent. Aggregate base compaction should also be 95 percent of the maximum dry density (ASTM D-1557), and follow the pavement grading recommendations provided below, as warranted.

PAVEMENT GRADING RECOMMENDATIONS

General

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

Subgrade

Within access drives and parking areas, all surficial deposits of loose soil material should be removed and recompact 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 City minimum, as evaluated by ASTM Test Method 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 ensure a uniform firm and unyielding surface. All grading and fill placement should be observed by the project geotechnical consultant or his representative.

Aggregate Base Rock

Compaction tests are required for the recommended base section. Minimum relative compaction required will be 95 percent of the laboratory maximum density as evaluated by ASTM Test Designation D 1557. Base aggregate should be in accordance to the Caltrans Class 2 base rock (minimum R-value=78). The City of Moreno Valley does not accept crushed miscellaneous base (CMB) as acceptable materials for roadway base.

Drainage

Positive drainage should be provided for all surface water to drain towards the area swale, curb and gutter, catch basin, 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. 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.

Additional Considerations

To mitigate perched groundwater, consideration should be given to installation of subgrade separators (cut-offs) between pavement subgrade and landscape areas (such as planting strips in parkways), 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 aggregate base rock, if used.

DRIVEWAYS, CONCRETE APRONS, FLATWORK, AND OTHER IMPROVEMENTS

The 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 that end, it is recommended that the developer should notify any owners or interested/affected parties of this long-term potential for distress. To reduce the likelihood of distress, the following recommendations are presented for all exterior flatwork:

1. The subgrade area for sidewalk slabs should be compacted to achieve a minimum 90 percent relative compaction, the subgrade area for access drive slabs and concrete aprons should be compacted to achieve a minimum 95 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 concrete placement.
2. Exterior concrete slabs should be cast over a non-yielding surface, consisting of a 4-inch layer of Class 2 base, crushed rock, gravel, or clean sand (or City minimum, whichever is greater), that should be compacted and level prior to placement of concrete. If very low expansive soils are present, the base, rock, gravel, or sand may be deleted. The layer or subgrade should be wet-down completely prior to placement of concrete, to minimize loss of concrete moisture to the surrounding earth materials.
3. Exterior sidewalk slabs should be a minimum of 4 inches thick. Access drive slabs should be a minimum of 5 inches thick. 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. Trash disposal (dumpster) area aprons should be a minimum of 6 inches thick and meet minimum City standards, as necessary.
4. Curbs next to slopes should have a thickened edge similar to drives and approaches.
5. The use of transverse and longitudinal control joints are recommended to help control slab cracking due to concrete shrinkage or expansion. Two ways to mitigate such cracking are: a) add a sufficient amount of reinforcing steel, increasing tensile strength of the slab; and, b) provide an adequate amount of control 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. Presoaking, as indicated earlier, is recommended for slab subsoils.

6. No traffic should be allowed upon the newly placed concrete slabs until they have been properly cured to within 75 percent of design strength. Concrete compression strength should be a minimum of 2,500 psi.
7. Access drives, sidewalks, and patio/exterior slabs adjacent to the structure should be separated from the structure with thick expansion joint filler material. In areas directly adjacent to a continuous source of moisture (i.e., irrigation, planters, etc.), all joints should be additionally sealed with flexible mastic.
8. Planters and walls (sound walls or retaining walls) should not be tied to the structure.
9. 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.
10. Any masonry landscape or sound 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.
11. If settlement concerns or expansive soils are present, utilities may be enclosed within a closed utilidor (vault) or designed with flexible connections to accommodate differential settlement and expansive soil conditions.
12. Positive site drainage should be maintained at all times. Finish grade on the building pad 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 drainage gradients are not periodically maintained by the owner or interested/affected parties.
13. 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.

14. 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 (PCA) 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.

DEVELOPMENT CRITERIA

Slope Deformation

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

It is generally not practical to attempt to eliminate the effects of either slope creep or LFE. Suitable mitigative measures to reduce the potential of lateral deformation typically include: setback of improvements from the slope faces (per 2019 CBC), positive structural separations (i.e., joints) between improvements, and stiffening and deepening of foundations. Expansion joints in walls should be placed no greater than 20 feet on-center, and in accordance with the structural engineer's recommendations. All of these measures are recommended for design of structures and improvements. The ramifications of the above conditions, and recommendations for mitigation, should be provided to each homeowner or any homeowners association.

Slope Maintenance and Planting

Water has been shown to weaken the inherent strength of all earth materials. Slope stability is significantly reduced by overly wet conditions. Positive surface drainage away from slopes should be maintained and only the amount of irrigation necessary to sustain plant life should be provided for planted slopes. Over-watering should be avoided as it adversely affects site improvements, and causes perched groundwater conditions. Graded

slopes constructed utilizing onsite materials would be erosive. Eroded debris may be minimized and surficial slope stability enhanced by establishing and maintaining a suitable vegetation cover soon after construction. Compaction to the face of fill slopes would tend to minimize short-term erosion until vegetation is established. Plants selected for landscaping should be light weight, deep rooted types that require little water and are capable of surviving the prevailing climate. Jute-type matting or other fibrous covers may aid in allowing the establishment of a sparse plant cover. Utilizing plants other than those recommended above will increase the potential for perched water, staining, mold, etc., to develop. A rodent control program to prevent burrowing should be implemented. Irrigation of natural (ungraded) slope areas is generally not recommended. These recommendations regarding plant type, irrigation practices, and rodent control should be provided to 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, retaining walls, etc.). Pad drainage should be directed toward the street or other approved area(s). 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., trash enclosures, walls, etc.) are planned for the site, recommendations concerning the geological or geotechnical aspects of design and construction of said improvements could be provided upon request. This office should be notified in advance of any fill placement, grading of the site, or trench backfilling after rough grading has been completed. This includes any grading, utility trench and retaining wall backfills, flatwork, etc.

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 superseded 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. Given the potentially erosive nature of the low expansive (low cohesive) soils, poor drainage or heavy rain events could destabilize trenches. 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., Stego Wrap, Husky Guard, 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, 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.

The findings of this study are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or inappropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this study may be invalidated wholly or partially by changes outside our control. Therefore, this study and the recommendations contained herein are subject to review and should not be relied upon after a period of three years.

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

APPENDIX A
REFERENCES

APPENDIX A

REFERENCES

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APPENDIX B
BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM					CONSISTENCY OR RELATIVE DENSITY																													
Major Divisions			Group Symbols	Typical Names	CRITERIA																													
Coarse-Grained Soils More than 50% retained on No. 200 sieve	Gravels 50% or more of coarse fraction retained on No. 4 sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines	<div><u>Standard Penetration Test</u></div> <div><div>Penetration Resistance N (blows/ft)</div><div>Relative Density</div></div> <table><tr><td>0 - 4</td><td>Very loose</td></tr><tr><td>4 - 10</td><td>Loose</td></tr><tr><td>10 - 30</td><td>Medium</td></tr><tr><td>30 - 50</td><td>Dense</td></tr><tr><td>> 50</td><td>Very dense</td></tr></table>			0 - 4	Very loose	4 - 10	Loose	10 - 30	Medium	30 - 50	Dense	> 50	Very dense																	
			0 - 4	Very loose																														
		4 - 10	Loose																															
		10 - 30	Medium																															
	30 - 50	Dense																																
	> 50	Very dense																																
	GP	Poorly graded gravels and gravel-sand mixtures, little or no fines																																
	Gravel with	GM	Silty gravels gravel-sand-silt mixtures																															
		GC	Clayey gravels, gravel-sand-clay mixtures																															
	Sands more than 50% of coarse fraction passes No. 4 sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines																														
SP			Poorly graded sands and gravelly sands, little or no fines																															
Sands with Fines		SM	Silty sands, sand-silt mixtures																															
		SC	Clayey sands, sand-clay mixtures																															
		<div><u>Standard Penetration Test</u></div> <div><div>Penetration Resistance N (blows/ft)</div><div>Consistency</div><div>Unconfined Compressive Strength (tons/ft²)</div></div> <table><tr><td><2</td><td>Very Soft</td><td><0.25</td></tr><tr><td>2 - 4</td><td>Soft</td><td>0.25 - .050</td></tr><tr><td>4 - 8</td><td>Medium</td><td>0.50 - 1.00</td></tr><tr><td>8 - 15</td><td>Stiff</td><td>1.00 - 2.00</td></tr><tr><td>15 - 30</td><td>Very Stiff</td><td>2.00 - 4.00</td></tr><tr><td>>30</td><td>Hard</td><td>>4.00</td></tr></table>			<2	Very Soft	<0.25	2 - 4	Soft	0.25 - .050	4 - 8	Medium	0.50 - 1.00	8 - 15	Stiff	1.00 - 2.00	15 - 30	Very Stiff	2.00 - 4.00	>30	Hard	>4.00												
					<2	Very Soft	<0.25																											
2 - 4	Soft				0.25 - .050																													
4 - 8	Medium				0.50 - 1.00																													
8 - 15	Stiff				1.00 - 2.00																													
15 - 30	Very Stiff				2.00 - 4.00																													
>30	Hard				>4.00																													
Fine-Grained Soils 50% or more passes No. 200 sieve	Silts and Clays Liquid limit 50% or less				ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands																												
					CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays																												
					OL	Organic silts and organic silty clays of low plasticity																												
	Silts and Clays Liquid limit greater than 50%	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts																															
		CH	Inorganic clays of high plasticity, fat clays																															
		OH	Organic clays of medium to high plasticity																															
Highly Organic Soils		PT	Peat, mucic, and other highly organic soils																															
<div>3"3/4"#4#10#40#200 U.S. Standard Sieve</div> <table><tr><th rowspan="2">Unified Soil Classification</th><th rowspan="2">Cobbles</th><th colspan="2">Gravel</th><th colspan="3">Sand</th><th rowspan="2">Silt or Clay</th></tr><tr><th>coarse</th><th>fine</th><th>coarse</th><th>medium</th><th>fine</th></tr></table>					Unified Soil Classification	Cobbles	Gravel		Sand			Silt or Clay	coarse	fine	coarse	medium	fine																	
Unified Soil Classification	Cobbles	Gravel		Sand			Silt or Clay																											
		coarse	fine	coarse	medium	fine																												
<div><div><u>MOISTURE CONDITIONS</u></div><div><table><tr><td>Dry</td><td>Absence of moisture; dusty, dry to the touch</td><td>trace</td><td>0 - 5 %</td><td>C</td><td>Core Sample</td></tr><tr><td>Slightly Moist</td><td>Below optimum moisture content for compaction</td><td>few</td><td>5 - 10 %</td><td>S</td><td>SPT Sample</td></tr><tr><td>Moist</td><td>Near optimum moisture content</td><td>little</td><td>10 - 25 %</td><td>B</td><td>Bulk Sample</td></tr><tr><td>Very Moist</td><td>Above optimum moisture content</td><td>some</td><td>25 - 45 %</td><td>—</td><td>Groundwater</td></tr><tr><td>Wet</td><td>Visible free water; below water table</td><td></td><td></td><td>Qp</td><td>Pocket Penetrometer</td></tr></table></div><div><u>MATERIAL QUANTITY</u></div><div><u>OTHER SYMBOLS</u></div></div>					Dry	Absence of moisture; dusty, dry to the touch	trace	0 - 5 %	C	Core Sample	Slightly Moist	Below optimum moisture content for compaction	few	5 - 10 %	S	SPT Sample	Moist	Near optimum moisture content	little	10 - 25 %	B	Bulk Sample	Very Moist	Above optimum moisture content	some	25 - 45 %	—	Groundwater	Wet	Visible free water; below water table			Qp	Pocket Penetrometer
Dry	Absence of moisture; dusty, dry to the touch	trace	0 - 5 %	C	Core Sample																													
Slightly Moist	Below optimum moisture content for compaction	few	5 - 10 %	S	SPT Sample																													
Moist	Near optimum moisture content	little	10 - 25 %	B	Bulk Sample																													
Very Moist	Above optimum moisture content	some	25 - 45 %	—	Groundwater																													
Wet	Visible free water; below water table			Qp	Pocket Penetrometer																													
BASIC LOG FORMAT: Group name, Group symbol, (grain size), color, moisture, consistency or relative density. Additional comments: odor, presence of roots, mica, gypsum, coarse grained particles, etc.																																		
EXAMPLE: Sand (SP), fine to medium grained, brown, moist, loose, trace silt, little fine gravel, few cobbles up to 4" in size, some hair roots and rootlets.																																		

GeoSoils, Inc.

BORING LOG

PROJECT: FPG FARM BUREAU, BOX SPRINGS,
MORENO VALLEY

W.O. 8676-A-SC BORING B-1 SHEET 1 OF 1

DATE EXCAVATED 9-18-23 LOGGED BY: MAM APPROX. ELEV.: 1,544'MSL

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

Depth (ft.)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
	Bulk	Undisturbed	Blows/Ft.					
0			36	SM	130.8	7.5	75.3	QUATERNARY VERY OLD ALLUVIAL FAN DEPOSITS (Qvof): @ 0', SILTY SAND, brown, damp to moist, loose to medium dense; fine to coarse sand, organic debris at surface.
5			29/ 50-3"	SM	128.7	3.9	36.4	BEDROCK LATE-CENOZOIC SEDIMENTARY ROCKS (QTs): @ 5', Upon drilling breaks down to SILTY SAND, strong brown to dark gray to reddish yellow, damp, very dense; fine to coarse grained bedrock.
10			50-5"	SM		2.2		@ 10', As per 5', grayish brown.
15			50-2"	SM		1.7		@ 15', As per 5', dark gray.
20			50-6"	SM		2.6		@ 20', As per 15'.
25			50-2"	SM		4.5		@ 25', As per 15'.
30								Total Depth = 26.5'. Practical refusal. No caving or groundwater encountered. Backfilled 9-18-23.

☒ Standard Penetration Test
☐ Undisturbed, Ring Sample

☐ Groundwater
☐ Seepage

GeoSoils, Inc.

PLATE B-2

GeoSoils, Inc.




BORING LOG

PROJECT: FPG FARM BUREAU, BOX SPRINGS,
MORENO VALLEY



W.O. 8676-A-SC BORING B-2 SHEET 1 OF 1

DATE EXCAVATED 9-18-23 LOGGED BY: MAM APPROX. ELEV.: 1,535'MSL

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

Depth (ft.)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
	Bulk	Undisturbed	Blows/Ft.					
0				SM				QUATERNARY VERY OLD ALLUVIAL FAN DEPOSITS (Qvof): @ 0', SILTY SAND, brown, damp to moist, medium dense; fine to coarse sand, organic debris at surface.
5			18	ML		10.1		@ 5', SANDY SILT, reddish brown, moist to wet, stiff; fine to coarse sand.
10			50-4"	SM	116.1	7.7	47.9	BEDROCK LATE CENOZOIC SEDIMENTARY ROCKS (QTs): @ 7', Drill rig chatter; upon drilling breaks down to silty sand, grayish brown, damp, dense. @ 10', SILTY SAND, gray to grayish brown, damp, very dense; fine to coarse grained bedrock, slightly weathered.
15			50-4"	SM		3.7		@ 15', SILTY SAND, gray, dry, very dense; fine to coarse grained bedrock.
20								Total Depth = 16.5'. Practical refusal. No caving or groundwater encountered. Backfilled 9-18-23.
25								
30								

☒ Standard Penetration Test
☐ Undisturbed, Ring Sample

 Groundwater
 Seepage

GeoSoils, Inc.

PLATE B-3

GeoSoils, Inc.

BORING LOG

PROJECT: FPG FARM BUREAU, BOX SPRINGS,
MORENO VALLEY

W.O. 8676-A-SC BORING B-3 SHEET 1 OF 1

DATE EXCAVATED 9-18-23 LOGGED BY: MAM APPROX. ELEV.: 1,547'MSL

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

Depth (ft.)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
	Bulk	Undisturbed	Blows/Ft.					
0				SM				QUATERNARY VERY OLD ALLUVIAL FAN DEPOSITS (Qvof): @ 0', SILTY SAND, brown, dry to moist, loose; fine to coarse sand; organic debris at surface. @ 2', SANDY SILT, brown to reddish brown, moist to wet, soft; fine to coarse sand.
5			6	ML	114.2	12.2	72.2	
10			54	ML		10.6		@ 5', As per 2', red brown, moist, medium stiff. @ 10', As per 2', hard.
15			81	ML	120.2	16.0	100	@ 15', As per 10', saturated; coarse grained. @ 18', Operator added water.
20			50-6"	SM		5.3		BEDROCK CRETACEOUS TONALITE, UNDIFFERENTIATED (Kt): @ 20', Upon drilling breaks down to SILTY SAND, gray to yellow, damp, very dense; fine to coarse grained bedrock, slightly weathered. @ 25', As per 20'.
25			50-2"	SM		7.2		
30								Total Depth = 26.5'. Practical refusal. No caving or groundwater encountered. Backfilled 9-18-23.

☒ Standard Penetration Test
☐ Undisturbed, Ring Sample

☐ Groundwater
☐ Seepage

GeoSoils, Inc.

PLATE B-4

GeoSoils, Inc.


BORING LOG

PROJECT: FPG FARM BUREAU, BOX SPRINGS,
MORENO VALLEY



W.O. 8676-A-SC BORING B-4 SHEET 1 OF 1

DATE EXCAVATED 9-18-23 LOGGED BY: MAM APPROX. ELEV.: 1,563'MSL

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

Depth (ft.)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
	Bulk	Undisturbed	Blows/Ft.					
0				SM				QUATERNARY VERY OLD ALLUVIAL FAN DEPOSITS (Qvof): @ 0', SILTY SAND, pale brown to brown, dry to damp, loose to medium dense; fine to coarse sand, organic debris at surface.
5			35/ 50-4"	SM		3.3		BEDROCK CRETACEOUS TONALITE, UNDIFFERENTIATED (Kt): @ 5', Upon drilling breaks down to SILTY SAND, gray to yellow, damp, very dense; fine to coarse sand, slightly weathered.
10			50-4"	SM		2.6		@ 10', As per 5'.
15								Total Depth = 11.5'. Practical refusal. No caving or groundwater encountered. Backfilled 9-18-23.
20								
25								
30								

☒ Standard Penetration Test
☐ Undisturbed, Ring Sample

 Groundwater
 Seepage

GeoSoils, Inc.

PLATE B-5

GeoSoils, Inc.

BORING LOG

PROJECT: FPG FARM BUREAU, BOX SPRINGS,
MORENO VALLEY

W.O. 8676-A-SC BORING B-5 SHEET 1 OF 1

DATE EXCAVATED 9-18-23 LOGGED BY: MAM APPROX. ELEV.: 1,549'MSL

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

Depth (ft.)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
	Bulk	Undisturbed	Blows/Ft.					
0				SM				QUATERNARY VERY OLD ALLUVIAL FAN DEPOSITS (Qvof): @ 0', SILTY SAND, brown to pale brown, dry to damp, loose to dense; fine to coarse sand, organic debris at surface. BEDROCK CRETACEOUS TONALITE, UNDIFFERENTIATED (Kt): @ 2.5', Upon drilling breaks down to SILTY SAND, light gray to yellow, damp, very dense; fine to coarse grained, weathered. @ 5', As per 2.5'; slightly weathered.
5			50-5"	SM	118.4	3.3	22.1	
			50-5"	SM		3.4		
10								Total Depth = 6.5'. Practical refusal. No caving or groundwater encountered. Backfilled 9-18-23.
15								
20								
25								
30								

☒ Standard Penetration Test
☐ Undisturbed, Ring Sample

☐ Groundwater
☐ Seepage

GeoSoils, Inc.

PLATE B-6

GeoSoils, Inc.

BORING LOG

PROJECT: FPG FARM BUREAU, BOX SPRINGS,
MORENO VALLEY

W.O. 8676-A-SC BORING IT-1 SHEET 1 OF 1

DATE EXCAVATED 9-18-23 LOGGED BY: MAM APPROX. ELEV.: 1,536'SL

SAMPLE METHOD: 8" HSA (No Sampling)

Depth (ft.)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
	Bulk	Undisturbed	Blows/Ft.					
0				SM				QUATERNARY VERY OLD ALLUVIAL FAN DEPOSITS (Qvof): @ 0', SILTY SAND, light brown, dry to damp, medium dense; fine to coarse sand.
5								Total Depth = 5'. Placed gravel and presoaked @ 10:03 am (no movement on water). Caved around testing pipe. No groundwater encountered. Backfilled 9-19-23.
10								
15								
20								
25								
30								

☒ Standard Penetration Test
☐ Undisturbed, Ring Sample

☐ Groundwater
☐ Seepage

GeoSoils, Inc.

PLATE B-7

GeoSoils, Inc.

BORING LOG

PROJECT: FPG FARM BUREAU, BOX SPRINGS,
MORENO VALLEY

W.O. 8676-A-SC BORING IT-2 SHEET 1 OF 1

DATE EXCAVATED 9-18-23 LOGGED BY: MAM APPROX. ELEV.: 1,538'MSL

SAMPLE METHOD: 8" HSA (No Sampling)

Depth (ft.)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
	Bulk	Undisturbed	Blows/Ft.					
0				SM				QUATERNARY VERY OLD ALLUVIAL FAN DEPOSITS (Qvof): @ 0', SILTY SAND, strong brown, dry to damp, loose to dense; fine to coarse sand, organics at surface.
5								Total Depth = 5', Added gravel and presoaked @ 12:23 pm (no movement on water). No caving or groundwater encountered. Backfilled 9-19-23.
10								
15								
20								
25								
30								

☒ Standard Penetration Test
☐ Undisturbed, Ring Sample

☐ Groundwater
☐ Seepage

GeoSoils, Inc.

PLATE B-8

APPENDIX C

SEISMIC DATA

*
* E Q F A U L T *
*
* Versi on 3.00 *
*

DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 8676

DATE: 10-16-2023

JOB NAME: FPG - Farm Bureau

CALCULATION NAME: Test Run Analysis

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

SITE COORDINATES:

SITE LATITUDE: 33.9476
SITE LONGITUDE: 117.2932

SEARCH RADIUS: 62.14 mi

ATTENUATION RELATION: 12) Bozorgnia Campbell Ni azi (1999) Hor. -Soft Rock-Cor.

UNCERTAINTY (M=Median, S=Sigma): S Number of Sigmas: 1.0

DISTANCE MEASURE: cdist

SCOND: 0

Basement Depth: 5.00 km Campbell SSR: 1 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

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	6.1 (9.8)	6.7	0.459	X
SAN JACINTO-SAN J. VLY-CASA LOMA	6.1 (9.8)	6.9	0.501	X
SAN ANDREAS - San Bernardi no M-1	14.5 (23.3)	7.5	0.354	IX
SAN ANDREAS - SB-Coach. M-2b	14.5 (23.3)	7.7	0.397	X
SAN ANDREAS - SB-Coach. M-1b-2	14.5 (23.3)	7.7	0.397	X
SAN ANDREAS - Whole M-1a	14.5 (23.3)	8.0	0.468	X
ELSINORE (GLEN IVY)	18.0 (28.9)	6.8	0.186	VIII
CHINO-CENTRAL AVE. (El sinore)	18.8 (30.2)	6.7	0.235	IX
CUCAMONGA	20.1 (32.4)	6.9	0.250	IX
WHITTIER	20.4 (32.9)	6.8	0.163	VIII
ELSINORE (TEMECULA)	21.4 (34.4)	6.8	0.156	VIII
NORTH FRONTAL FAULT ZONE (West)	21.8 (35.1)	7.2	0.281	IX
CLEGHORN	22.7 (36.6)	6.5	0.120	VII
SAN JACINTO-ANZA	26.0 (41.8)	7.2	0.168	VIII
SAN JOSE	26.0 (41.9)	6.4	0.138	VIII
SAN ANDREAS - 1857 Rupture M-2a	26.5 (42.6)	7.8	0.248	IX
SAN ANDREAS - Cho-Moj M-1b-1	26.5 (42.6)	7.8	0.248	IX
SAN ANDREAS - Mojave M-1c-3	26.5 (42.6)	7.4	0.189	VIII
SIERRA MADRE	28.6 (46.1)	7.2	0.214	VIII
PUENTE HILLS BLIND THRUST	33.1 (53.2)	7.1	0.174	VIII
PINTO MOUNTAIN	33.7 (54.3)	7.2	0.128	VIII
SAN JOAQUIN HILLS	34.1 (54.9)	6.6	0.120	VII
NORTH FRONTAL FAULT ZONE (East)	34.4 (55.4)	6.7	0.126	VIII
HELENDALE - S. LOCKHARDT	38.0 (61.2)	7.3	0.122	VII
CLAMSHELL-SAWPIT	39.3 (63.2)	6.5	0.096	VII
NEWPORT-INGLEWOOD (Offshore)	42.0 (67.6)	7.1	0.095	VII
ELSINORE (JULIAN)	42.6 (68.6)	7.1	0.094	VII
RAYMOND	42.9 (69.1)	6.5	0.087	VII
NEWPORT-INGLEWOOD (L. A. Basi n)	43.2 (69.6)	7.1	0.093	VII
LENWOOD-LOCKHART-OLD WOMAN SPRGS	47.0 (75.7)	7.5	0.113	VII
UPPER ELYSIAN PARK BLIND THRUST	47.0 (75.7)	6.4	0.075	VII
SAN ANDREAS - Coachella M-1c-5	47.3 (76.1)	7.2	0.090	VII
VERDUGO	49.5 (79.7)	6.9	0.098	VII
BURNT MTN.	51.1 (82.2)	6.5	0.052	VI
LANDERS	51.9 (83.6)	7.3	0.088	VII
JOHNSON VALLEY (Northern)	52.6 (84.6)	6.7	0.057	VI
EUREKA PEAK	52.9 (85.2)	6.4	0.046	VI
PALOS VERDES	55.0 (88.5)	7.3	0.083	VII
HOLLYWOOD	55.2 (88.9)	6.4	0.063	VI
SAN JACINTO-COYOTE CREEK	56.4 (90.7)	6.6	0.050	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.
ROSE CANYON	57.5 (92.6)	7.2	0.074	VII
EMERSON So. - COPPER MTN.	58.8 (94.6)	7.0	0.063	VI
CORONADO BANK	59.4 (95.6)	7.6	0.095	VII
SAN GABRIEL	62.1 (100.0)	7.2	0.068	VI

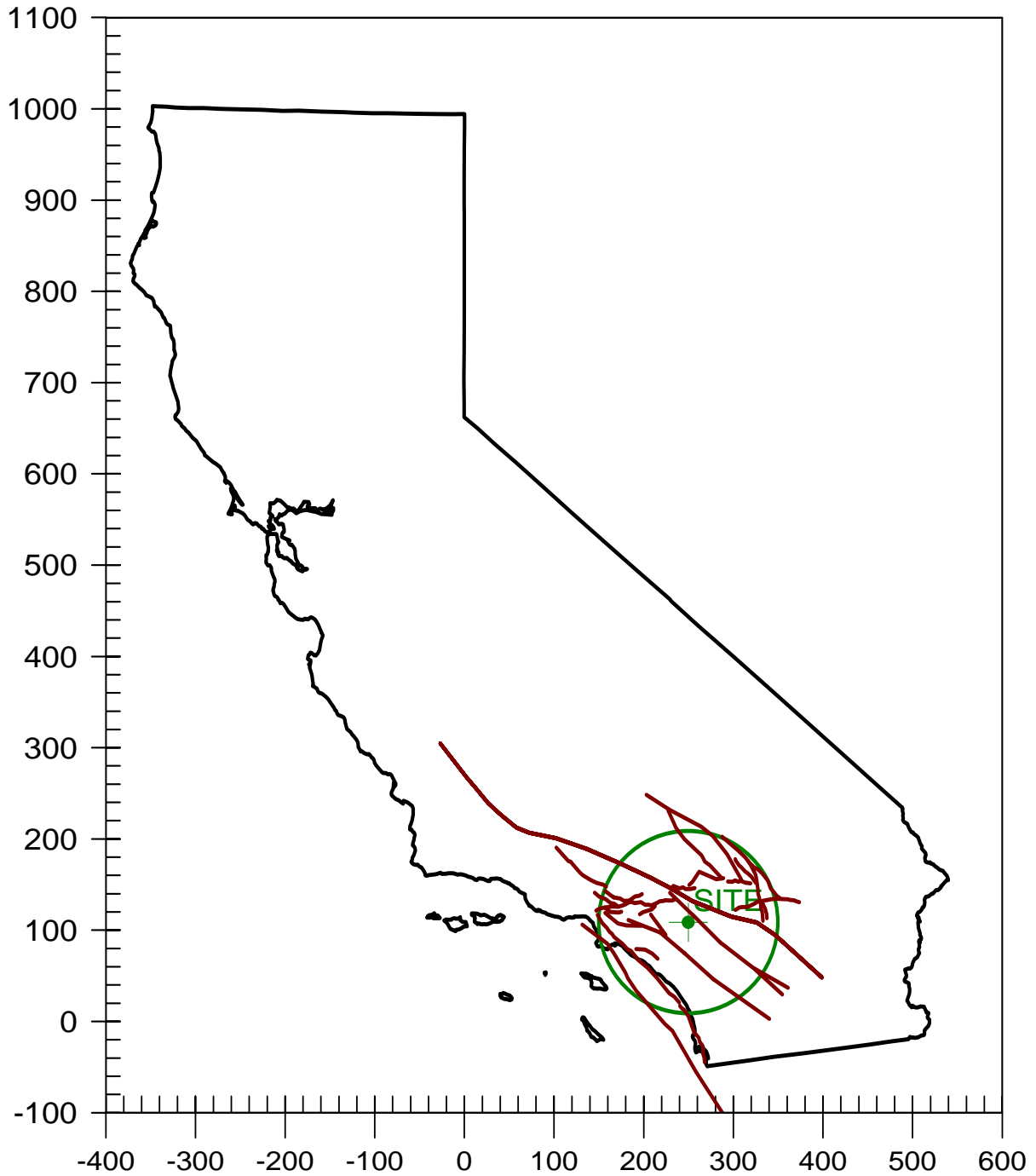
-END OF SEARCH- 44 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE SAN JACINTO-SAN J. VLY-CASA LOMA FAULT IS CLOSEST TO THE SITE.
 IT IS ABOUT 6.1 MILES (9.8 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.5013 g

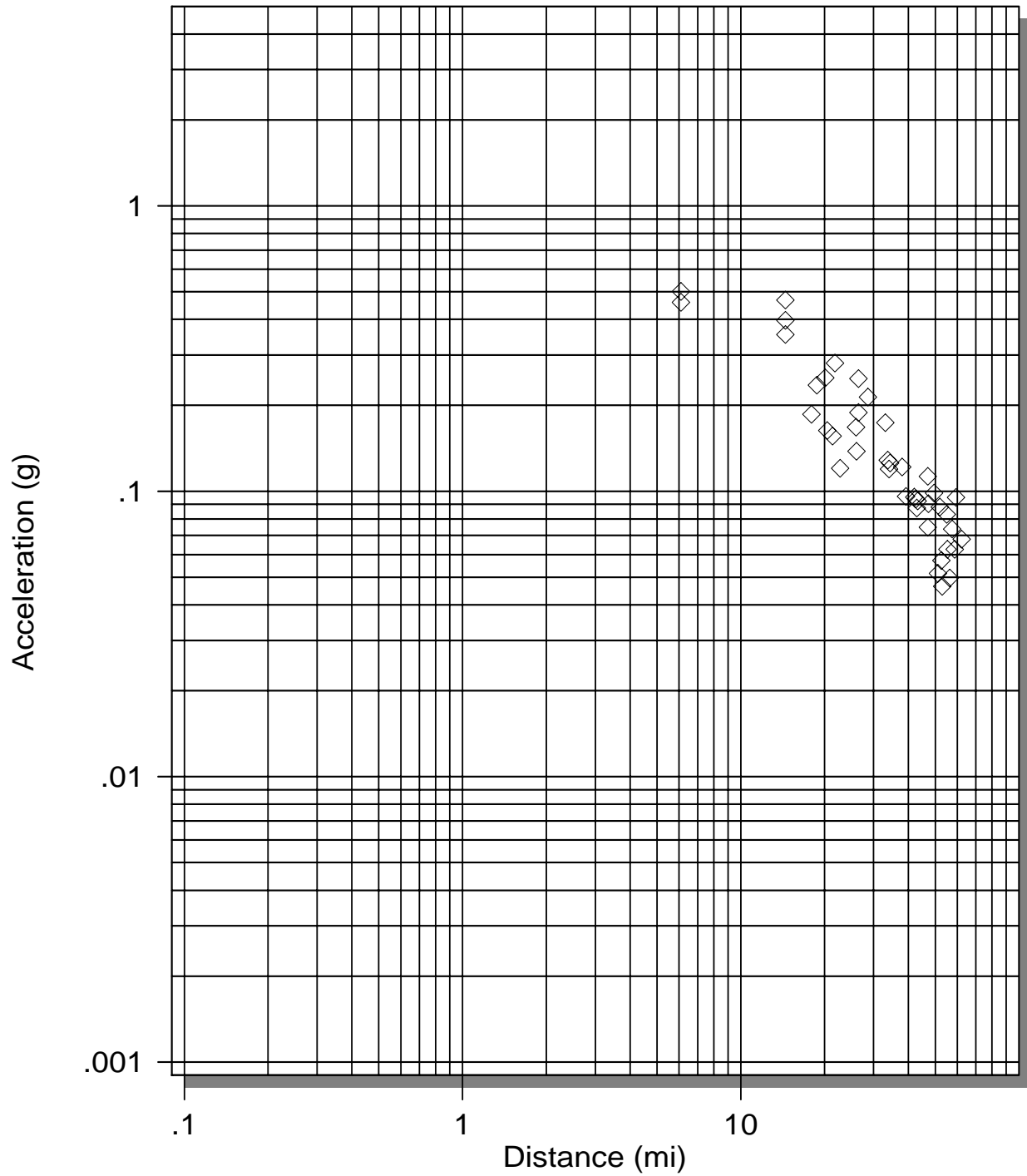
CALIFORNIA FAULT MAP

FPG Farm Bureau



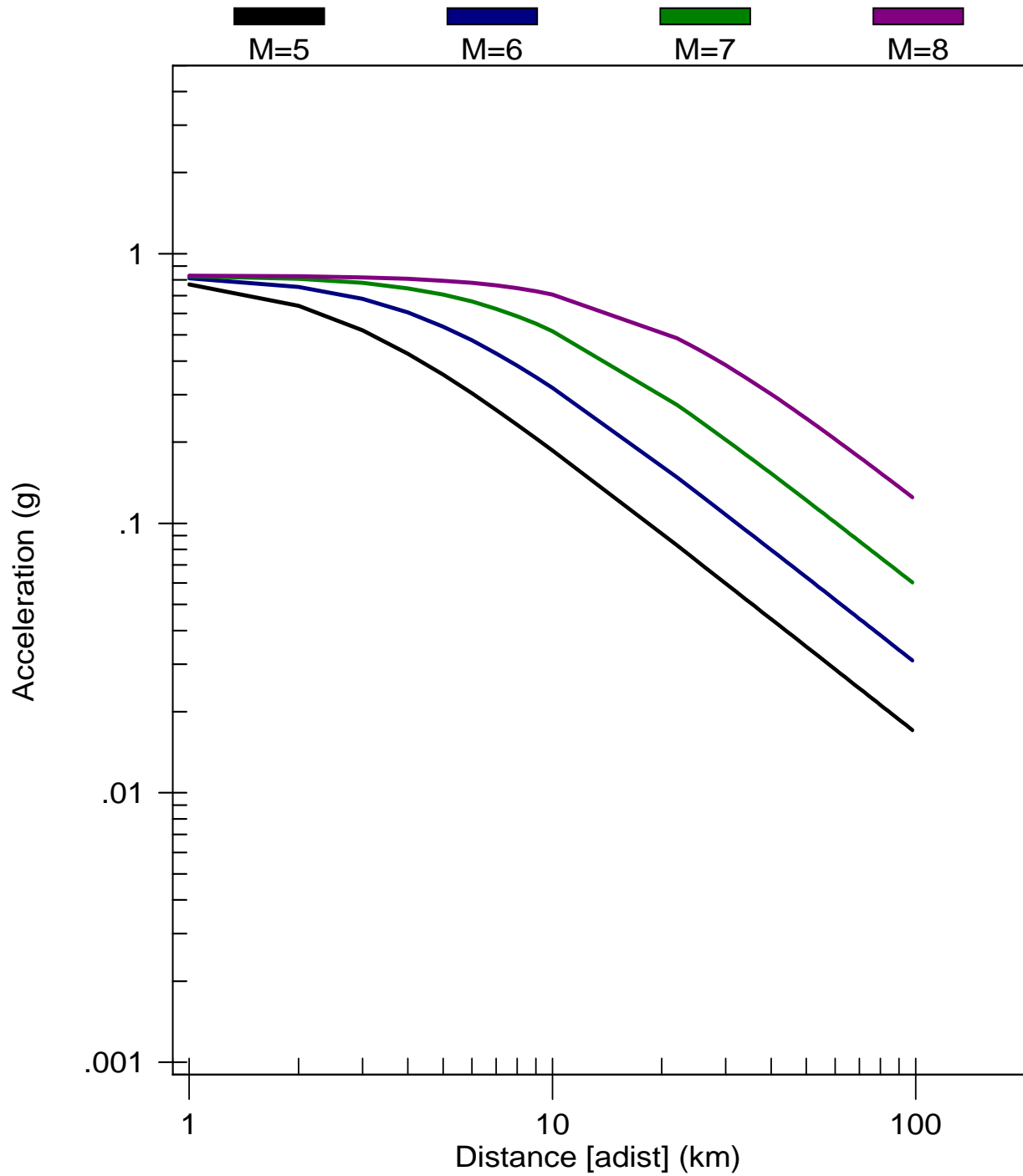
MAXIMUM EARTHQUAKES

FPG Farm Bureau



STRIKE-SLIP FAULTS

12) Bozorgnia Campbell Niazi (1999) Hor.-Soft Rock-Cor.



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ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 8676

DATE: 10-16-2023

JOB NAME: FPG - Farm Bureau

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

MAGNITUDE RANGE:

MINIMUM MAGNITUDE: 5.00
MAXIMUM MAGNITUDE: 9.00

SITE COORDINATES:

SITE LATITUDE: 33.9476
SITE LONGITUDE: 117.2932

SEARCH DATES:

START DATE: 1800
END DATE: 2021

SEARCH RADIUS:

62.1 mi
100.0 km

ATTENUATION RELATION: 12) Bozorgnia Campbell Ni azi (1999) Hor. -Soft Rock-Cor.

UNCERTAINTY (M=Median, S=Sigma): S Number of Sigmas: 1.0

ASSUMED SOURCE TYPE: SS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]

SCOND: 1 Depth Source: A

Basement Depth: 5.00 km Campbell SSR: 1 Campbell SHR: 0

COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km): 3.0

EARTHQUAKE SEARCH RESULTS

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	34.0000	117.2500	07/23/1923	73026.0	0.0	6.25	0.447	X	4.4 (7.0)
DMG	33.9000	117.2000	12/19/1880	0 0 0.0	0.0	6.00	0.300	IX	6.3 (10.1)
MGI	34.1000	117.3000	07/15/1905	2041 0.0	0.0	5.30	0.120	VII	10.5 (16.9)
MGI	34.0000	117.5000	12/16/1858	10 0 0.0	0.0	7.00	0.298	IX	12.4 (19.9)
DMG	33.7000	117.4000	05/13/1910	620 0.0	0.0	5.00	0.060	VI	18.2 (29.2)
DMG	33.7000	117.4000	05/15/1910	1547 0.0	0.0	6.00	0.110	VII	18.2 (29.2)
DMG	33.7000	117.4000	04/11/1910	757 0.0	0.0	5.00	0.060	VI	18.2 (29.2)
DMG	34.2000	117.4000	07/22/1899	046 0.0	0.0	5.50	0.079	VII	18.5 (29.7)
DMG	33.8000	117.0000	12/25/1899	1225 0.0	0.0	6.40	0.131	VIII	19.7 (31.6)
MGI	33.8000	117.6000	04/22/1918	2115 0.0	0.0	5.00	0.053	VI	20.3 (32.7)
DMG	34.2000	117.1000	09/20/1907	154 0.0	0.0	6.00	0.096	VII	20.6 (33.2)
DMG	33.6990	117.5110	05/31/1938	83455.4	10.0	5.50	0.068	VI	21.2 (34.2)
DMG	33.7500	117.0000	06/06/1918	2232 0.0	0.0	5.00	0.050	VI	21.6 (34.8)
DMG	33.7500	117.0000	04/21/1918	223225.0	0.0	6.80	0.154	VIII	21.6 (34.8)
DMG	33.9500	116.8500	09/28/1946	719 9.0	0.0	5.00	0.042	VI	25.4 (40.8)
DMG	34.2700	117.5400	09/12/1970	143053.0	8.0	5.40	0.052	VI	26.4 (42.4)
DMG	34.1800	116.9200	01/16/1930	034 3.6	0.0	5.10	0.043	VI	26.7 (43.0)
DMG	34.1800	116.9200	01/16/1930	02433.9	0.0	5.20	0.045	VI	26.7 (43.0)
DMG	33.7100	116.9250	09/23/1963	144152.6	16.5	5.00	0.040	V	26.7 (43.0)
GSG	33.9530	117.7610	07/29/2008	184215.7	14.0	5.30	0.048	VI	26.8 (43.1)
GSP	34.1400	117.7000	02/28/1990	234336.6	5.0	5.20	0.045	VI	26.8 (43.1)
DMG	34.3000	117.5000	07/22/1899	2032 0.0	0.0	6.50	0.100	VII	27.0 (43.5)
DMG	34.2670	116.9670	08/29/1943	34513.0	0.0	5.50	0.050	VI	28.9 (46.5)
GSP	34.1630	116.8550	06/28/1992	144321.0	6.0	5.30	0.044	VI	29.1 (46.9)
DMG	34.3000	117.6000	07/30/1894	512 0.0	0.0	6.00	0.065	VI	30.0 (48.3)
GSP	34.1950	116.8620	08/17/1992	204152.1	11.0	5.30	0.042	VI	30.0 (48.3)
DMG	34.1000	116.8000	10/24/1935	1448 7.6	0.0	5.10	0.038	V	30.1 (48.5)
GSP	34.2900	116.9460	02/10/2001	210505.8	9.0	5.10	0.037	V	30.9 (49.7)
GSN	34.2030	116.8270	06/28/1992	150530.7	5.0	6.70	0.096	VII	32.0 (51.4)
DMG	33.9760	116.7210	06/12/1944	104534.7	10.0	5.10	0.034	V	32.8 (52.8)
GSP	34.2390	116.8370	07/09/1992	014357.6	0.0	5.30	0.039	V	32.9 (53.0)
DMG	33.9940	116.7120	06/12/1944	111636.0	10.0	5.30	0.038	V	33.4 (53.8)
GSP	34.3400	116.9000	11/27/1992	160057.5	1.0	5.30	0.036	V	35.2 (56.6)
DMG	34.1000	116.7000	02/07/1889	520 0.0	0.0	5.30	0.036	V	35.5 (57.2)
DMG	34.3700	117.6500	12/08/1812	15 0 0.0	0.0	7.00	0.106	VII	35.6 (57.3)
GSP	33.9325	117.9158	03/29/2014	040942.2	5.1	5.10	0.032	V	35.7 (57.4)
GSG	34.3100	116.8480	02/22/2003	121910.6	1.0	5.20	0.033	V	35.7 (57.4)
GSP	34.3690	116.8970	12/04/1992	020857.5	3.0	5.30	0.034	V	36.9 (59.3)
DMG	34.2000	117.9000	08/28/1889	215 0.0	0.0	5.50	0.037	V	38.8 (62.5)
PAS	33.9980	116.6060	07/08/1986	92044.5	11.7	5.60	0.038	V	39.5 (63.6)
MGI	34.0000	118.0000	12/25/1903	1745 0.0	0.0	5.00	0.026	V	40.6 (65.4)
DMG	33.6170	117.9670	03/11/1933	154 7.8	0.0	6.30	0.052	VI	44.9 (72.3)
PAS	34.0610	118.0790	10/01/1987	144220.0	9.5	5.90	0.039	V	45.6 (73.5)
DMG	34.0170	116.5000	07/24/1947	221046.0	0.0	5.50	0.031	V	45.7 (73.5)
DMG	34.0170	116.5000	07/25/1947	04631.0	0.0	5.00	0.023	IV	45.7 (73.5)
DMG	34.0170	116.5000	07/25/1947	61949.0	0.0	5.20	0.026	V	45.7 (73.5)
DMG	34.0170	116.5000	07/26/1947	24941.0	0.0	5.10	0.024	V	45.7 (73.5)
GSP	34.2620	118.0020	06/28/1991	144354.5	11.0	5.40	0.029	V	46.0 (74.0)
PAS	34.0730	118.0980	10/04/1987	105938.2	8.2	5.30	0.027	V	46.9 (75.4)
DMG	33.6830	118.0500	03/11/1933	658 3.0	0.0	5.50	0.030	V	47.1 (75.8)
DMG	33.5750	117.9830	03/11/1933	518 4.0	0.0	5.20	0.025	V	47.2 (76.0)
DMG	33.7500	118.0830	03/11/1933	323 0.0	0.0	5.00	0.022	IV	47.3 (76.1)
DMG	33.7500	118.0830	03/13/1933	131828.0	0.0	5.30	0.026	V	47.3 (76.1)

EARTHQUAKE SEARCH RESULTS

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	33.7500	118.0830	03/11/1933	2 9 0.0	0.0	5.00	0.022	IV	47.3 (76.1)
DMG	33.7500	118.0830	03/11/1933	910 0.0	0.0	5.10	0.024	IV	47.3 (76.1)

DMG	33. 7500	118. 0830	03/11/1933	230 0. 0	0. 0	5. 10	0. 024	IV	47. 3(76. 1)
MGI	34. 1000	118. 1000	07/11/1855	415 0. 0	0. 0	6. 30	0. 049	VI	47. 3(76. 2)
DMG	33. 6170	118. 0170	03/14/1933	19 150. 0	0. 0	5. 10	0. 023	IV	47. 4(76. 3)
DMG	33. 7000	118. 0670	03/11/1933	51022. 0	0. 0	5. 10	0. 023	IV	47. 6(76. 5)
DMG	33. 7000	118. 0670	03/11/1933	85457. 0	0. 0	5. 10	0. 023	IV	47. 6(76. 5)
DMG	33. 7830	118. 1330	10/02/1933	91017. 6	0. 0	5. 40	0. 027	V	49. 5(79. 6)
GSP	33. 5290	116. 5720	06/12/2005	154146. 5	14. 0	5. 20	0. 023	IV	50. 5(81. 3)
GSP	34. 1390	116. 4310	06/28/1992	123640. 6	10. 0	5. 10	0. 022	IV	51. 1(82. 2)
GSP	34. 3410	116. 5290	06/28/1992	124053. 5	6. 0	5. 20	0. 023	IV	51. 4(82. 8)
GSN	34. 2010	116. 4360	06/28/1992	115734. 1	1. 0	7. 60	0. 109	VI I	52. 0(83. 8)
GSP	34. 1080	116. 4040	06/29/1992	141338. 8	9. 0	5. 40	0. 025	V	52. 1(83. 8)
DMG	33. 9330	116. 3830	12/04/1948	234317. 0	0. 0	6. 50	0. 050	VI	52. 1(83. 9)
GSP	34. 0640	116. 3610	09/15/1992	084711. 3	9. 0	5. 20	0. 022	IV	54. 0(86. 8)
GSP	33. 5080	116. 5140	10/31/2001	075616. 6	15. 0	5. 10	0. 020	IV	54. 1(87. 0)
PAS	33. 5010	116. 5130	02/25/1980	104738. 5	13. 6	5. 50	0. 026	V	54. 4(87. 5)
GSP	34. 3320	116. 4620	07/01/1992	074029. 9	9. 0	5. 40	0. 024	V	54. 4(87. 6)
T-A	34. 0000	118. 2500	09/23/1827	0 0 0. 0	0. 0	5. 00	0. 019	IV	54. 9(88. 3)
T-A	34. 0000	118. 2500	01/10/1856	0 0 0. 0	0. 0	5. 00	0. 019	IV	54. 9(88. 3)
T-A	34. 0000	118. 2500	03/26/1860	0 0 0. 0	0. 0	5. 00	0. 019	IV	54. 9(88. 3)
DMG	33. 5000	116. 5000	09/30/1916	211 0. 0	0. 0	5. 00	0. 019	IV	55. 0(88. 6)
PAS	34. 3270	116. 4450	03/15/1979	21 716. 5	2. 5	5. 20	0. 021	IV	55. 1(88. 7)
GSP	34. 2680	116. 4020	06/16/1994	162427. 5	3. 0	5. 00	0. 019	IV	55. 5(89. 4)
DMG	34. 0670	116. 3330	05/18/1940	55120. 2	0. 0	5. 20	0. 021	IV	55. 6(89. 4)
DMG	34. 0670	116. 3330	05/18/1940	72132. 7	0. 0	5. 00	0. 019	IV	55. 6(89. 4)
GSP	33. 9610	116. 3180	04/23/1992	045023. 0	12. 0	6. 10	0. 036	V	55. 9(89. 9)
GSP	34. 0290	116. 3210	08/21/1993	014638. 4	9. 0	5. 00	0. 019	IV	55. 9(90. 0)
DMG	33. 7830	118. 2500	11/14/1941	84136. 3	0. 0	5. 40	0. 023	IV	56. 0(90. 1)
MGI	34. 0800	118. 2600	07/16/1920	18 8 0. 0	0. 0	5. 00	0. 019	IV	56. 1(90. 2)
DMG	33. 8500	118. 2670	03/11/1933	1425 0. 0	0. 0	5. 00	0. 019	IV	56. 2(90. 5)
DMG	34. 0830	116. 3000	05/18/1940	5 358. 5	0. 0	5. 40	0. 023	IV	57. 6(92. 7)
MGI	34. 0000	118. 3000	09/03/1905	540 0. 0	0. 0	5. 30	0. 021	IV	57. 8(92. 9)
GSP	33. 9020	116. 2840	07/24/1992	181436. 2	9. 0	5. 00	0. 018	IV	57. 9(93. 2)
GSG	33. 4200	116. 4890	07/07/2010	235333. 5	14. 0	5. 50	0. 024	IV	58. 8(94. 7)
GSP	33. 8760	116. 2670	06/29/1992	160142. 8	1. 0	5. 20	0. 020	IV	59. 0(95. 0)
PAS	34. 5160	116. 4950	06/01/1975	13849. 2	4. 5	5. 20	0. 019	IV	60. 1(96. 8)
GSP	33. 4315	116. 4427	06/10/2016	080438. 7	12. 3	5. 19	0. 019	IV	60. 5(97. 3)
DMG	33. 2000	116. 7000	01/01/1920	235 0. 0	0. 0	5. 00	0. 017	IV	61. 9(99. 6)

-END OF SEARCH- 92 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 4.4 MILES (7.0 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.6

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.447 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:

a-value= 1.424

b-value= 0.414

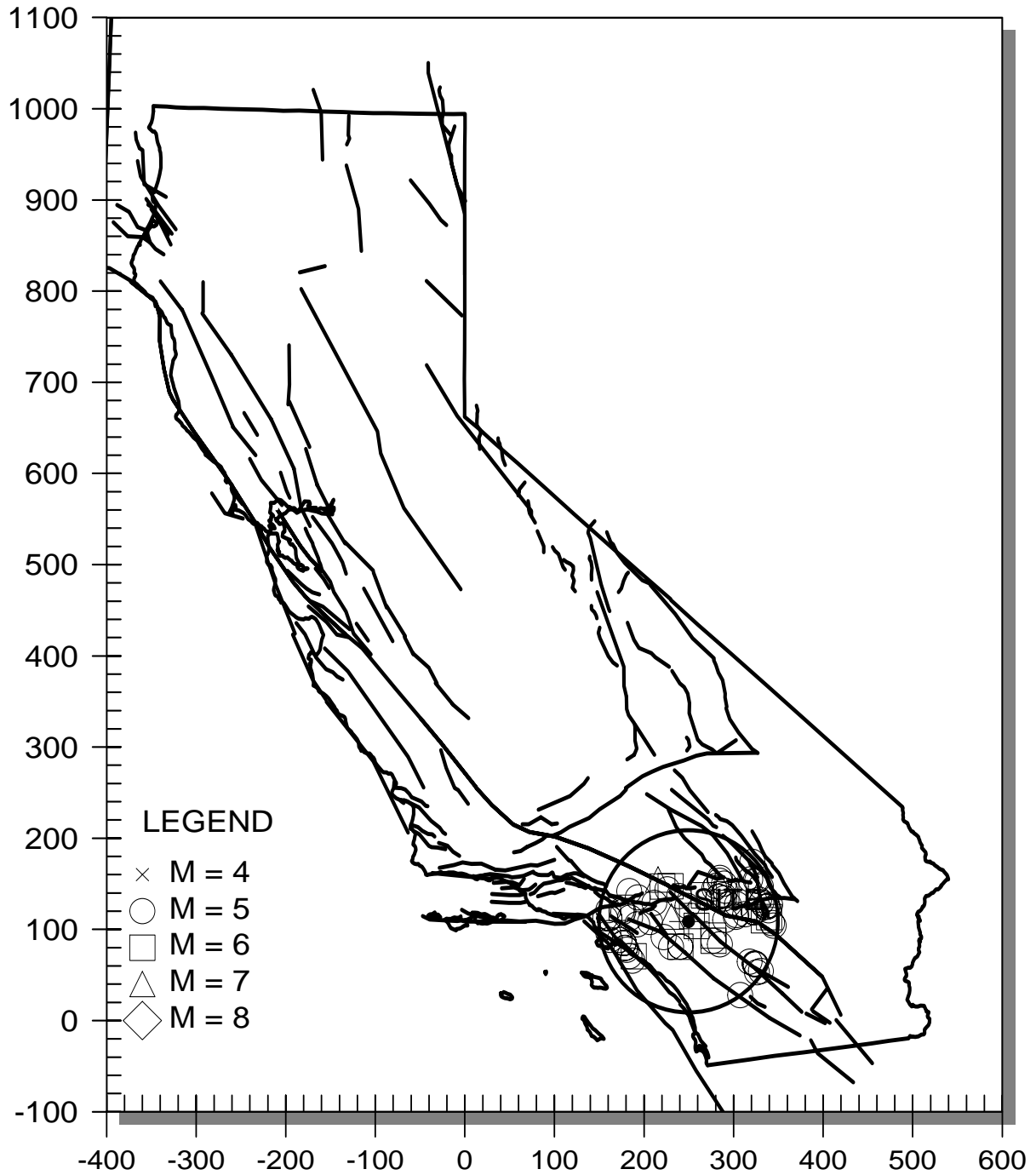
beta-value= 0.952

TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake Magni tude	Number of Times Exceeded	Cumul ative No. / Year
4. 0	92	0. 41441
4. 5	92	0. 41441
5. 0	92	0. 41441
5. 5	26	0. 11712
6. 0	16	0. 07207
6. 5	7	0. 03153
7. 0	3	0. 01351
7. 5	1	0. 00450

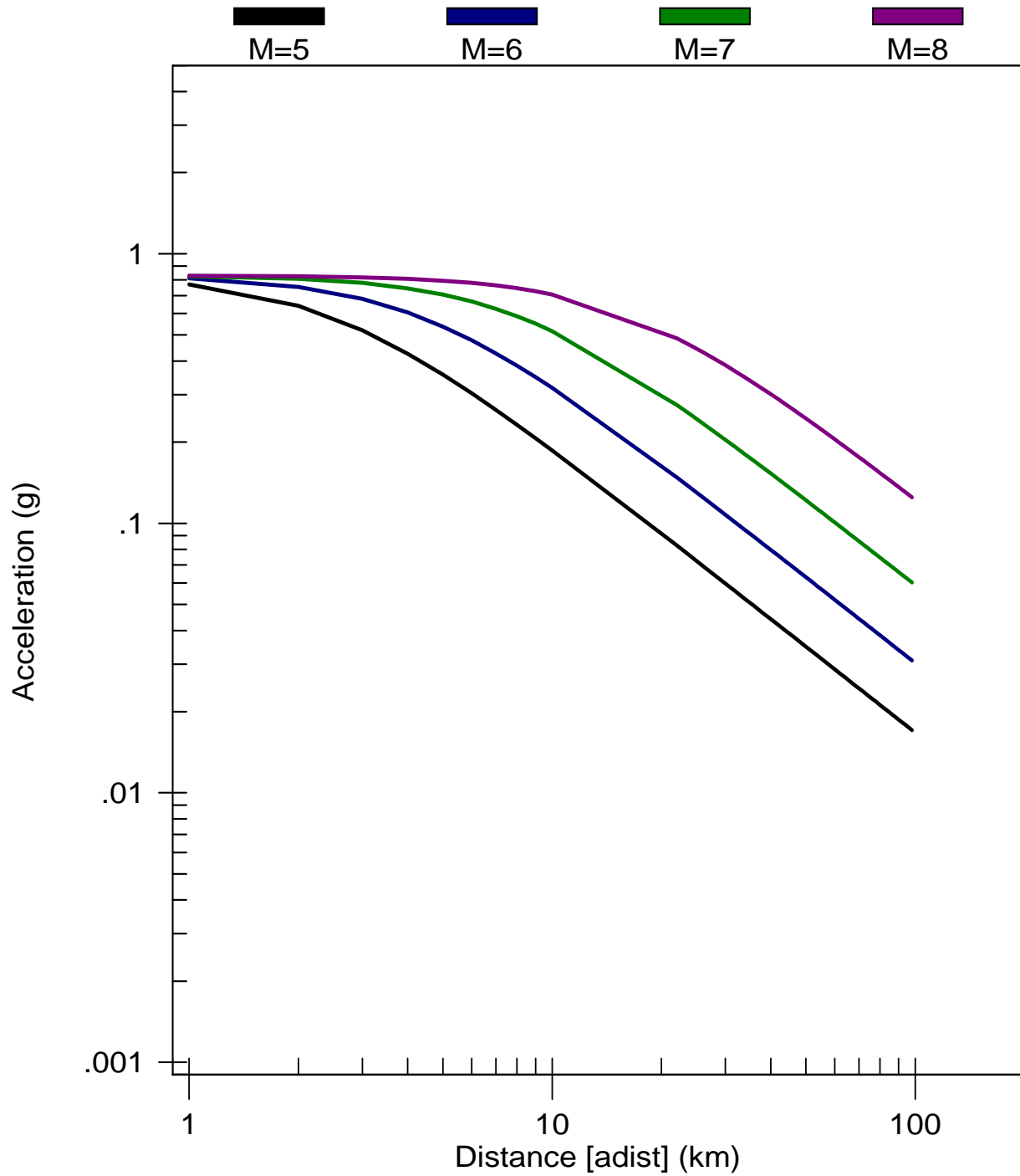
EARTHQUAKE EPICENTER MAP

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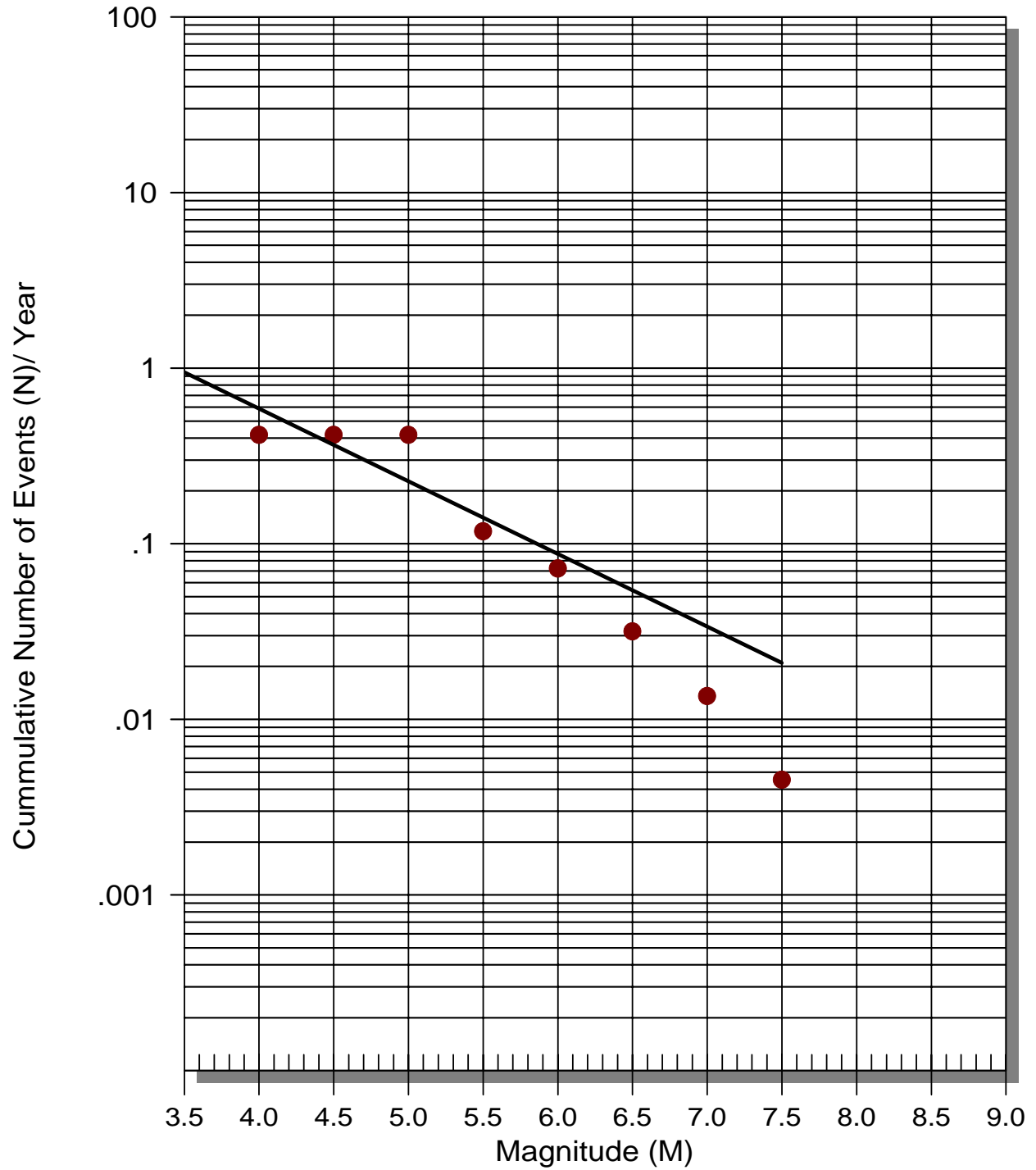
STRIKE-SLIP FAULTS

12) Bozorgnia Campbell Niazi (1999) Hor.-Soft Rock-Cor.



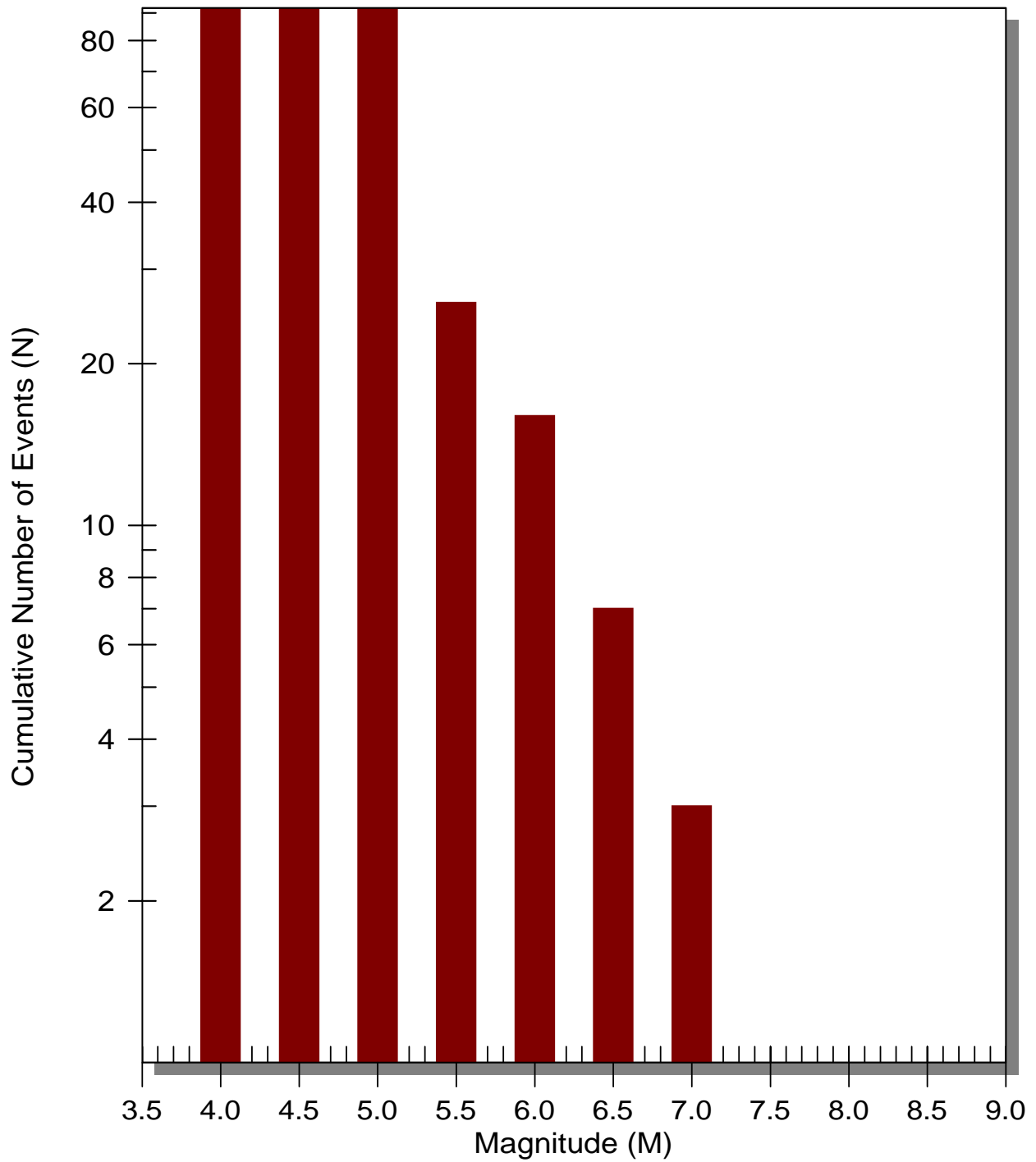
EARTHQUAKE RECURRENCE CURVE

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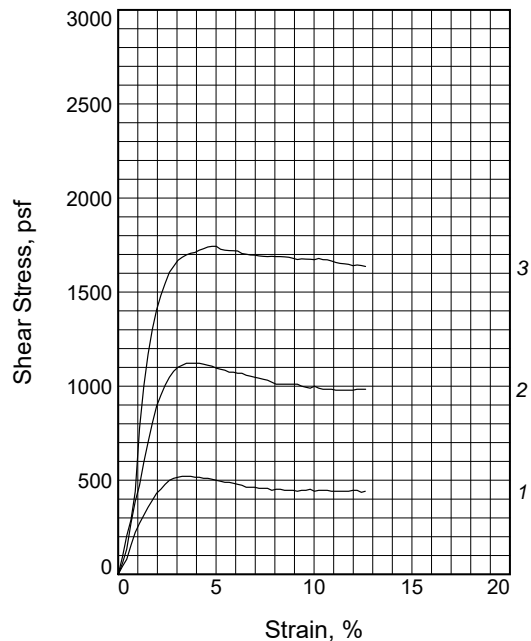
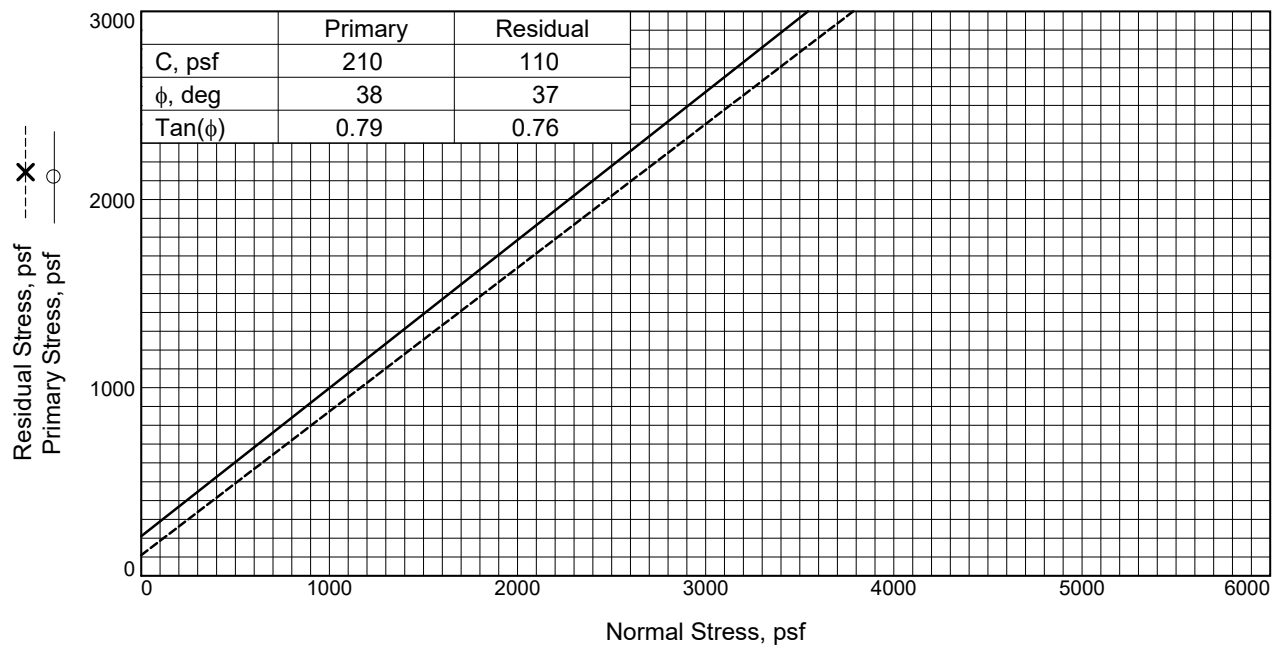
Number of Earthquakes (N) Above Magnitude (M)

FPG - Farm Bureau



APPENDIX D

LABORATORY TEST RESULTS



Sample No.		1	2	3
Initial	Water Content, %	6.9	6.9	6.9
	Dry Density, pcf	121.9	121.9	121.7
	Saturation, %	51.2	51.2	50.9
	Void Ratio	0.3575	0.3575	0.3593
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	13.1	12.9	12.6
	Dry Density, pcf	122.1	123.1	123.4
	Saturation, %	98.0	99.1	98.2
	Void Ratio	0.3548	0.3439	0.3402
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	0.99	0.99
Normal Stress, psf		500	1000	2000
Primary Stress, psf		521	1122	1743
Strain, %		3.5	3.7	5.0
Residual Stress, psf		441	994	1658
Strain, %		9.2	9.6	11.1
Strain rate, in./min.		0.004	0.004	0.004

Sample Type: Remolded
Description: Brown Silty Sand

Specific Gravity= 2.65

Remarks:

Figure _____

Client: Foremost Pacific Group

Project: Farm Bureau - Box Springs

Source of Sample: B-3

Depth: 0-5

Sample Number: B-3

Proj. No.: 8676-A-SC

Date Sampled:

GeoSoils, Inc.

Tested By: TR

Checked By: TR

W.O. 8676-A-SC
 Plate D-1



5741 Palmer Way, Carlsbad CA 92010

Phone (760) 438-3155

CORROSION REPORT SUMMARY

Project No: 8676-A-SC

Project Name: Foremost Pacific Group

Report Date: September 28, 2023

SAMPLE ID	pH (H+)	Minimum Resistivity (ohm/cm)	Sulfate Content (wt%)	Chloride Content (mg/kg)
B-3, 0-5ft	7.5	8800	<0.003	43

Sample testing in accordance with:

pH - CTM 643, Resistivity - CTM 643

Sulfate - CTM 417, Chloride - CTM 422

Remarks: _____

APPENDIX E

FIELD PERCOLATION DATA SHEETS

Leach Line Percolation Data Sheet

Project: <u>Foremost Pacific Group - Tom Bureau</u>	W.O. Number: <u>8678-A-SC</u>
Test Hole No.: <u>P-1/B-7</u>	Date Excavated: <u>9/18/2023</u>
Depth of Test Hole: <u>5' (in pipe) 2' 2" (outside)</u>	Soil Classification: <u>SM</u>
Check for Sandy Soil Criteria Tested by: <u>MAN</u>	Date: <u>9/18/2023</u> Presoak: <u>@ 12:23 PM 9/18/23</u>
Actual Percolation Tested by: <u>MAN</u>	Date: <u>9/19/2023</u>

Sandy Soil Criteria Test

Trial No.	Time	Time Interval (Min.)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ in Water Level (Inches)
1	8:57	25	21.5	16	5.5
	9:22				
2	9:23	25	22.5	19.5	3.0
	9:48				

Use: Normal Sandy (Circle One) Soil Criteria

	Time	Time Interval (min)	Total Elapsed Time (Min.)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ in Water Level (Inches)	Percolation Rate (min/inch)
First Hour	9:49	30	30	22	16.5	5.5	5.45
	10:19						
	10:20	30	60	22.25	16.75	5.5	5.45
	10:50						
Second Hour	10:51	30	90	21	14.75	6.25	4.8
	11:21						
	11:23	30	120	22	15.75	6.25	4.8
	11:53						
Third Hour	11:53	30	150	21.5	16.25	5.25	5.71
	12:23						
	12:24	30	180	21.5	16.0	5.5	5.45
	12:54						
Fourth Hour	12:56	30	210	21.25	16	5.25	5.71
	1:26						
	1:28	30	240	21.25	15.5	5.75	5.22
	1:58						
Fifth Hour	2:00	30	270	21.5	16.25	5.25	5.71
	2:30						
	2:31	30	300	21.25	17.75	3.5	8.57
	3:01						
Sixth Hour	3:01	30	330	21.25	18.25	3	10
	3:31						
	3:33	30	360	21	18	3	10
	4:03						

Leach Line Percolation Data Sheet

Project: <u>Foremost Pacific Group - Farm Bureau</u>	W.O. Number: <u>8676-A-SC</u>
Test Hole No.: <u>P-2/B-6</u>	Date Excavated: <u>9/18/2023</u>
Depth of Test Hole: <u>5'1" (in pipe)</u>	Soil Classification: <u>SM</u>
Check for Sandy Soil Criteria Tested by: <u>MMH</u>	Date: <u>9/19/2023</u> Presoak: <u>@ 10:03 AM 9/18/23</u>
Actual Percolation Tested by: <u>MMH</u>	Date: <u>9/19/2023</u>

Sandy Soil Criteria Test

Trial No.	Time	Time Interval (Min.)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ in Water Level (Inches)
1	9:08	25	35	30	5
	9:33				
2	9:36	25	33	30	3
	10:01				

Use: Normal Sandy (Circle One) Soil Criteria

	Time	Time Interval (min)	Total Elapsed Time (Min.)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ in Water Level (Inches)	Percolation Rate (min/inch)
First Hour	10:02	30	30	30	28	2	15
	10:32						
	10:33	30	60	29	25.25	3.75	8
	10:03						
Second Hour	11:05	30	90	30.25	28	2.25	13.33
	11:35						
	11:36	30	120	30.75	28	2.75	10.91
	12:06						
Third Hour	12:07	30	150	29.25	26.5	2.75	10.91
	12:37						
	12:39	30	180	28	25	3	10
	1:09						
Fourth Hour	1:10	30	210	28	24	4	7.5
	1:40						
	1:41	30	240	28	24.25	3.75	8
	2:11						
Fifth Hour	2:13	30	270	27.5	23.5	4	7.5
	2:43						
	2:45	30	300	28	24.5	3.5	8.57
	3:15						
Sixth Hour	3:16	30	330	28	24.5	3.5	8.57
	3:46						
	3:48	30	360	28	25	3	10

APPENDIX F

GENERAL EARTHWORK AND GRADING GUIDELINES

GENERAL EARTHWORK AND GRADING GUIDELINES

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 alluvium or rock materials, as evaluated by the geotechnical consultant as being unsuitable, should be removed and recompacted 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. 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, 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 then trimmed 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:

1. An extra piece of equipment consisting of a heavy, short-shanked sheep'sfoot should be used to roll (horizontal) parallel to the slopes continuously as fill is placed. The sheep'sfoot 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, after compaction operations.
4. After completion of the slope, the slope face should be shaped with a small tractor and then grid-rolled to achieve compaction to the slope face. Final testing should be used to evaluate compaction after grid-rolling.
5. Where testing indicates less than adequate compaction, the contractor will be responsible to rip, water, mix, and recompact the slope material as necessary to achieve compaction. Additional testing should be performed to evaluate compaction.

SUBDRAIN INSTALLATION

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

EXCAVATIONS

Excavations and cut slopes should be examined during grading by the geotechnical consultant. If directed by the geotechnical consultant, further excavations or overexcavation and refilling of cut areas should be performed, 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.

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

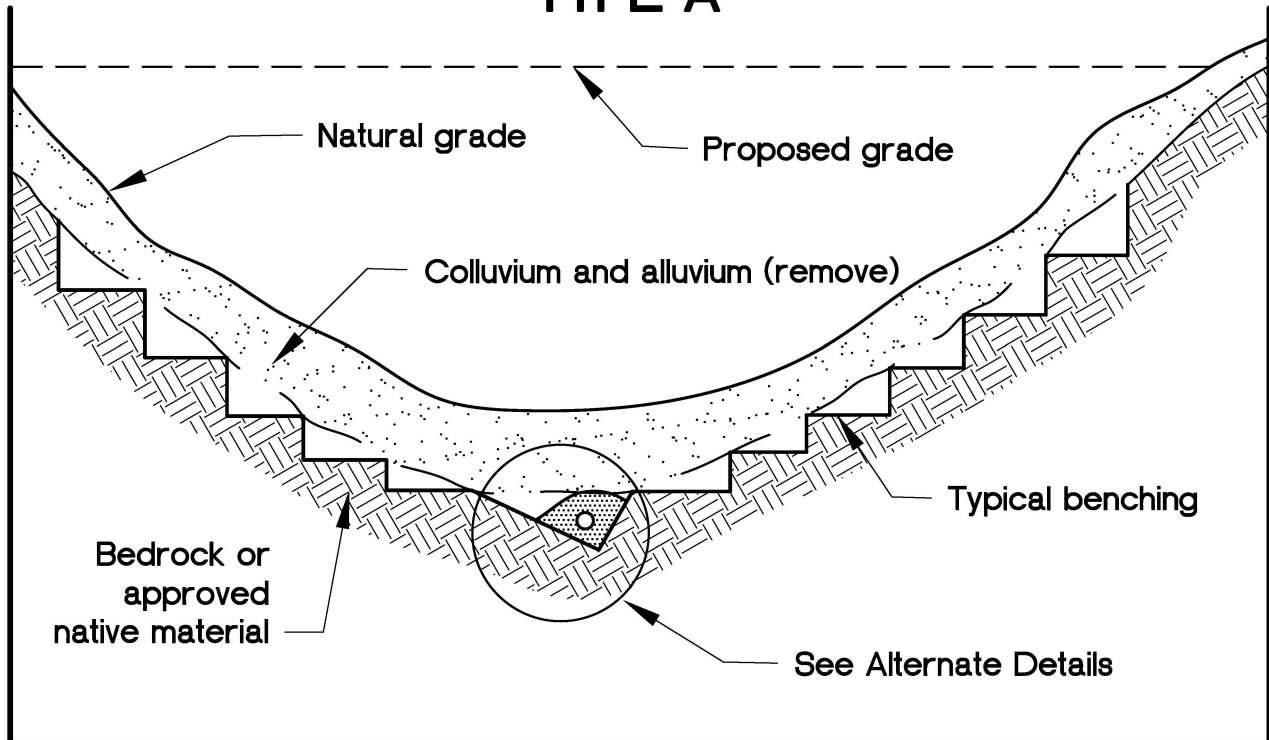
It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Our personnel are directed not to enter any excavation or vertical cut which: 1) is 5 feet or deeper unless shored or laid back; 2) displays any evidence of instability, has any loose rock or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

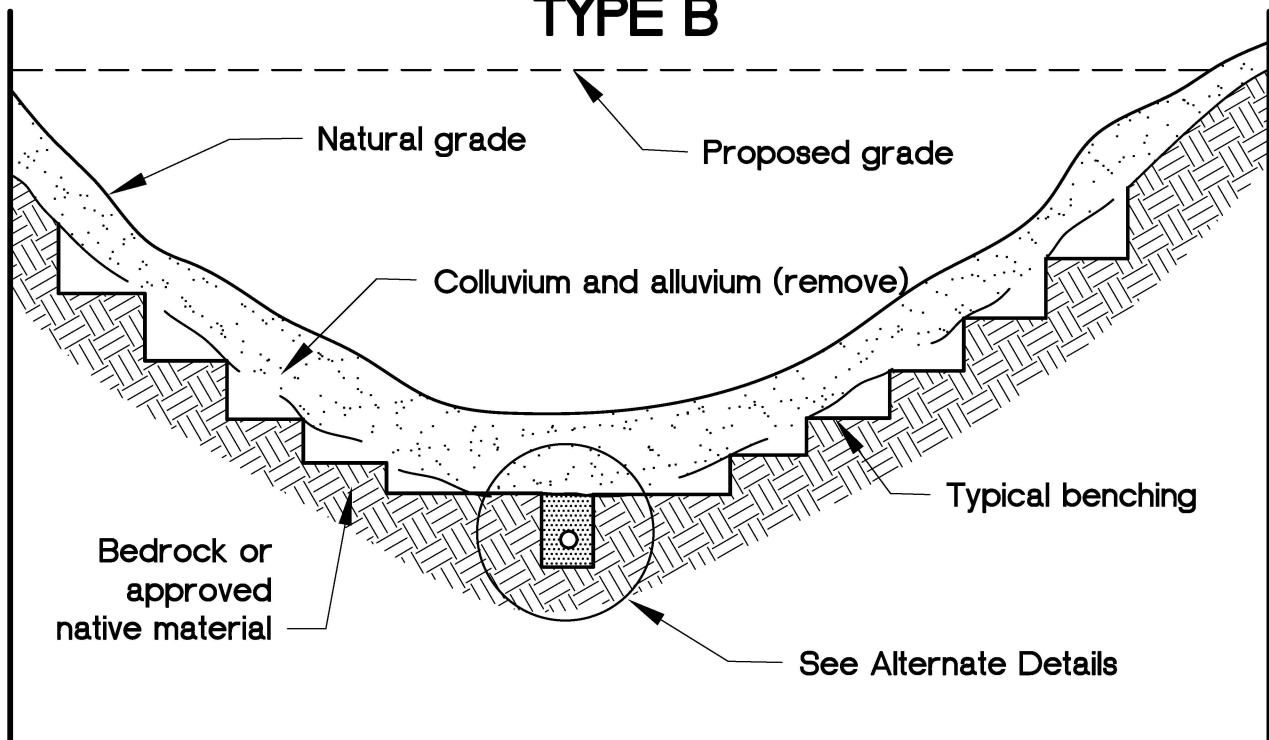
If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraw and notify his/her supervisor. The contractor's representative will be contacted in an effort to affect a solution. All backfill not tested due to safety concerns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

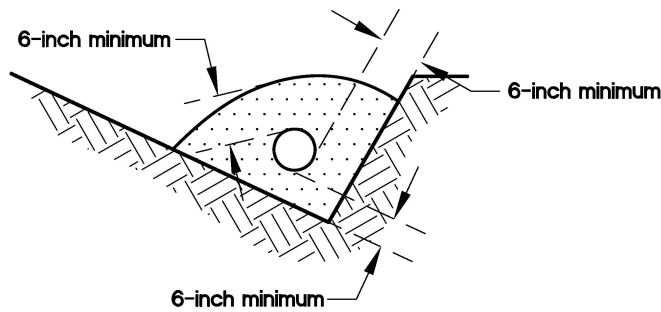
TYPE A



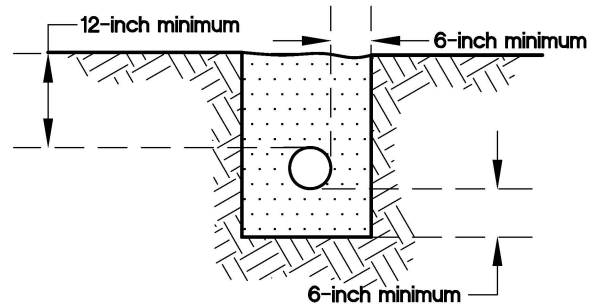
TYPE B



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.

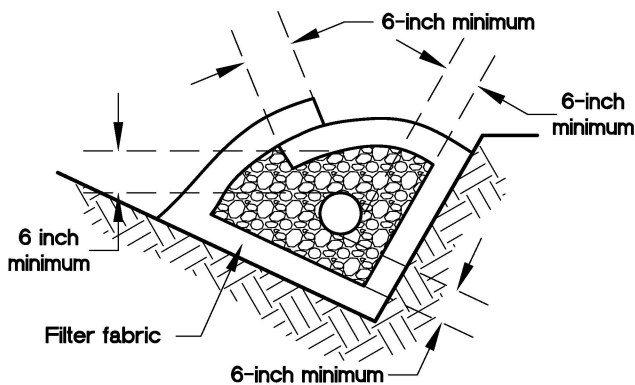
Perforated pipe: 6-inch-diameter ABS or PVC pipe or approved substitute with minimum 8 perforations ($\frac{1}{4}$ -inch diameter) per lineal foot in bottom half of pipe (ASTM D-2751, SDR-35, or ASTM D-1527, Schd. 40).

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

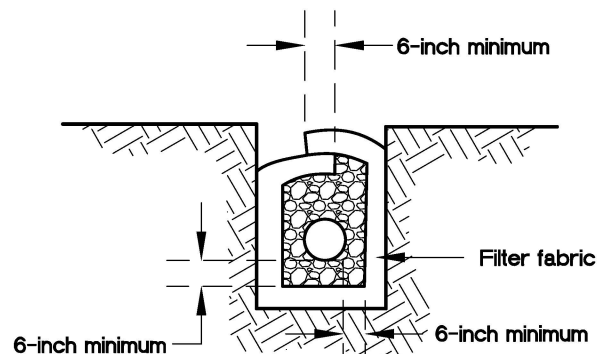
FILTER MATERIAL

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

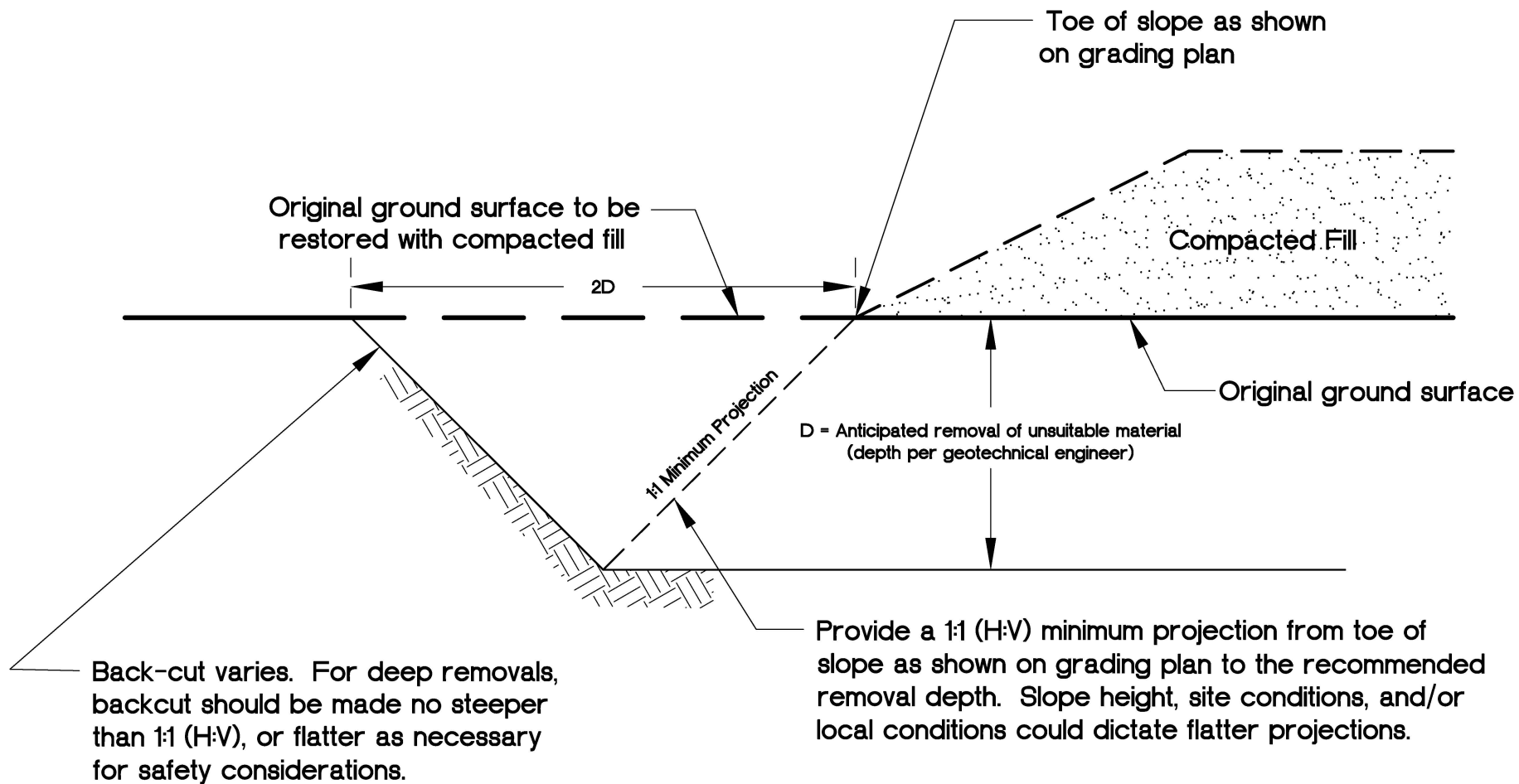
Gravel Material: 9 cubic feet per lineal foot.

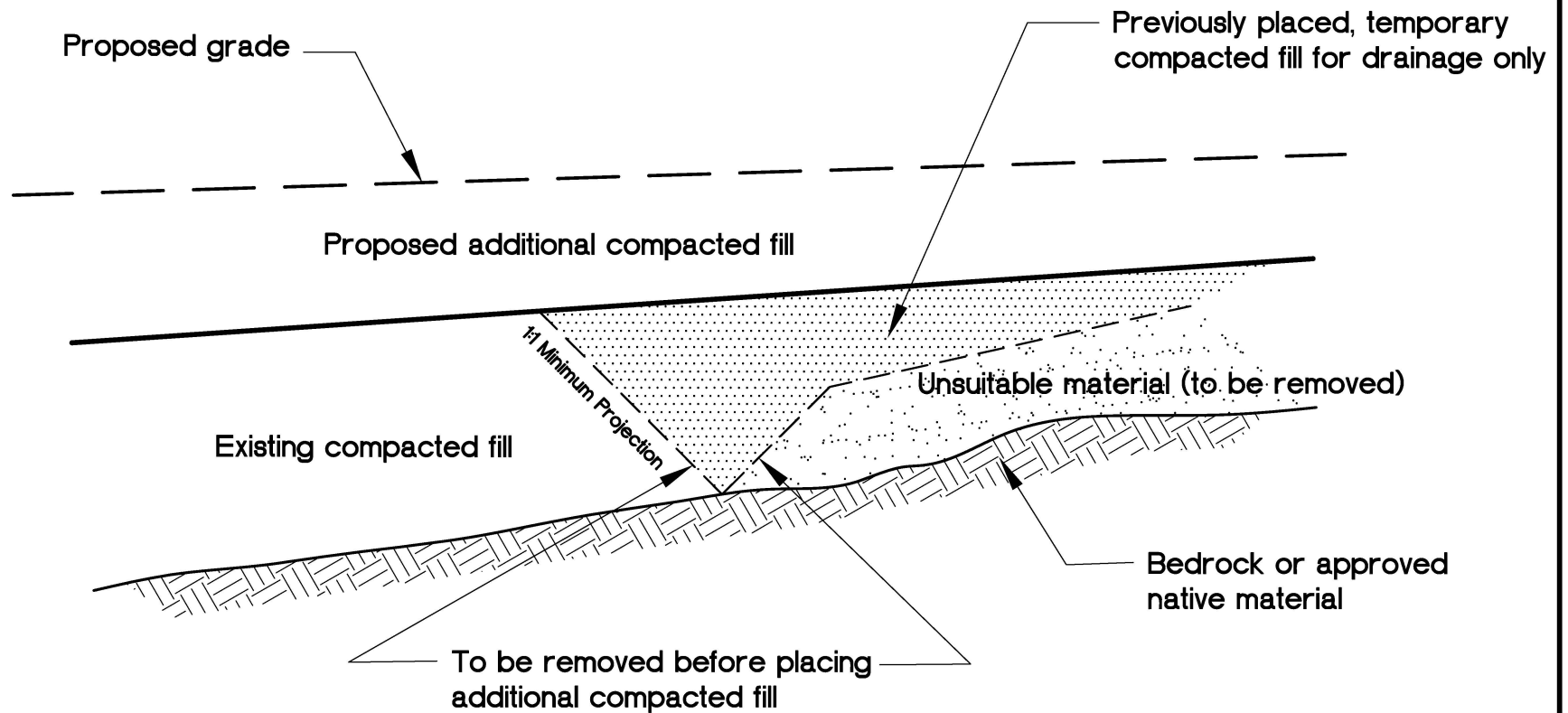
Perforated Pipe: See Alternate 1

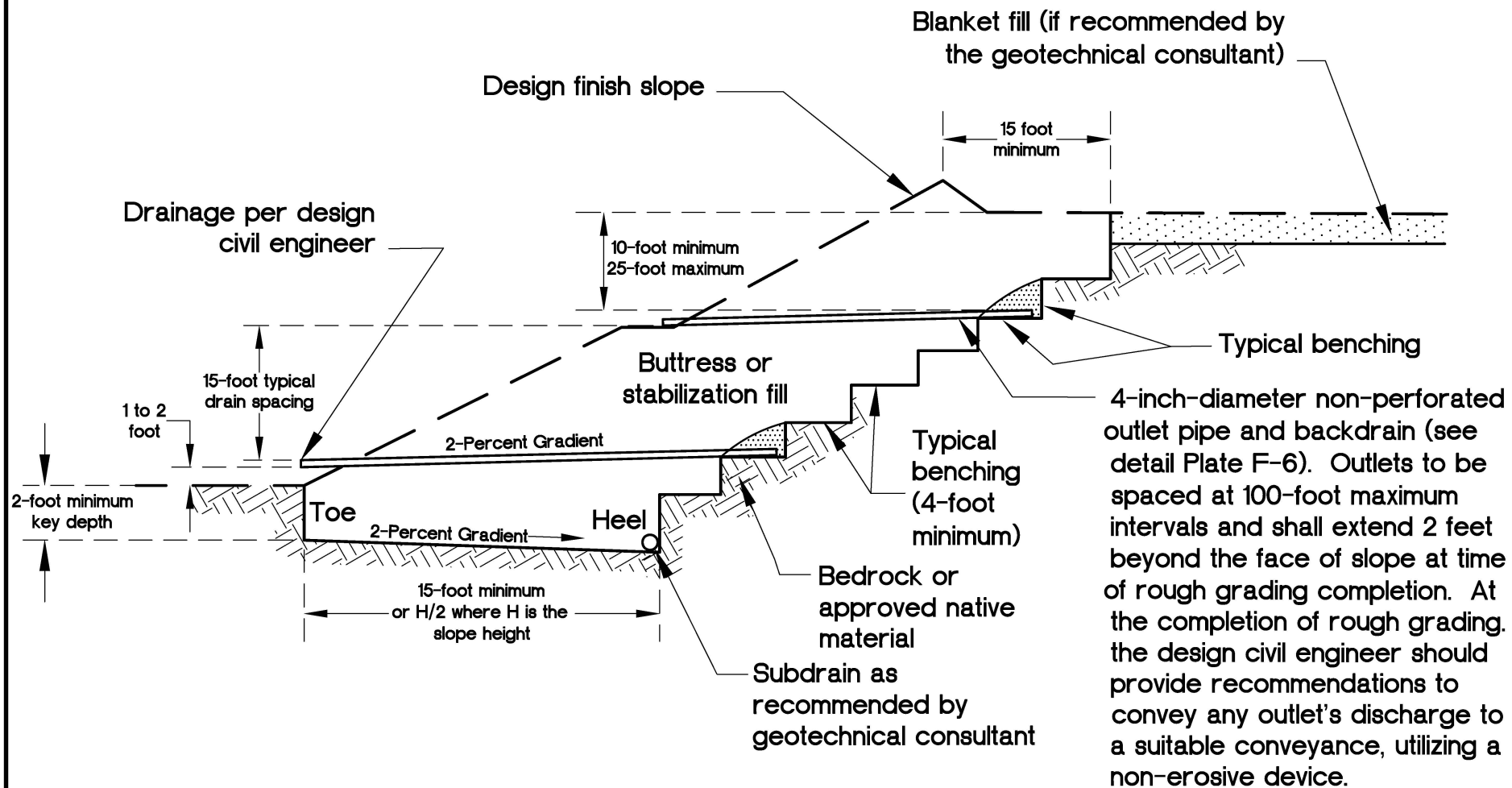
Gravel: Clean $\frac{3}{4}$ -inch rock or approved substitute.

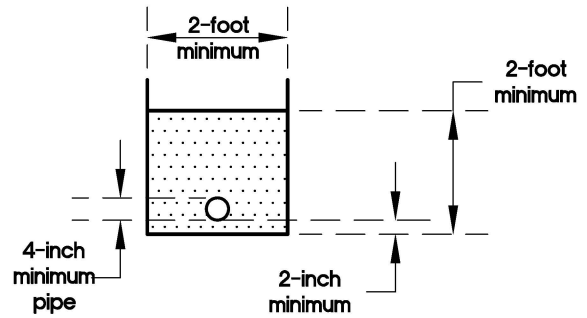
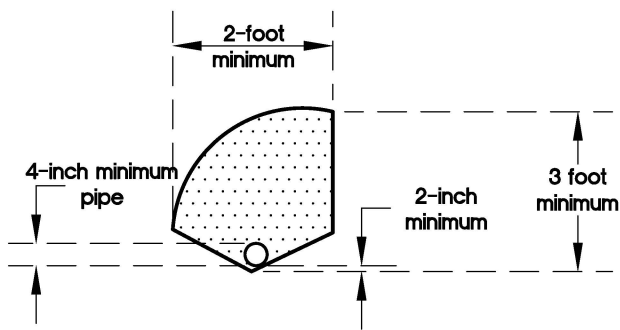
Filter Fabric: Mirafi 140 or approved substitute.

ALTERNATE 2: PERFORATED PIPE, GRAVEL, AND FILTER FABRIC









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.

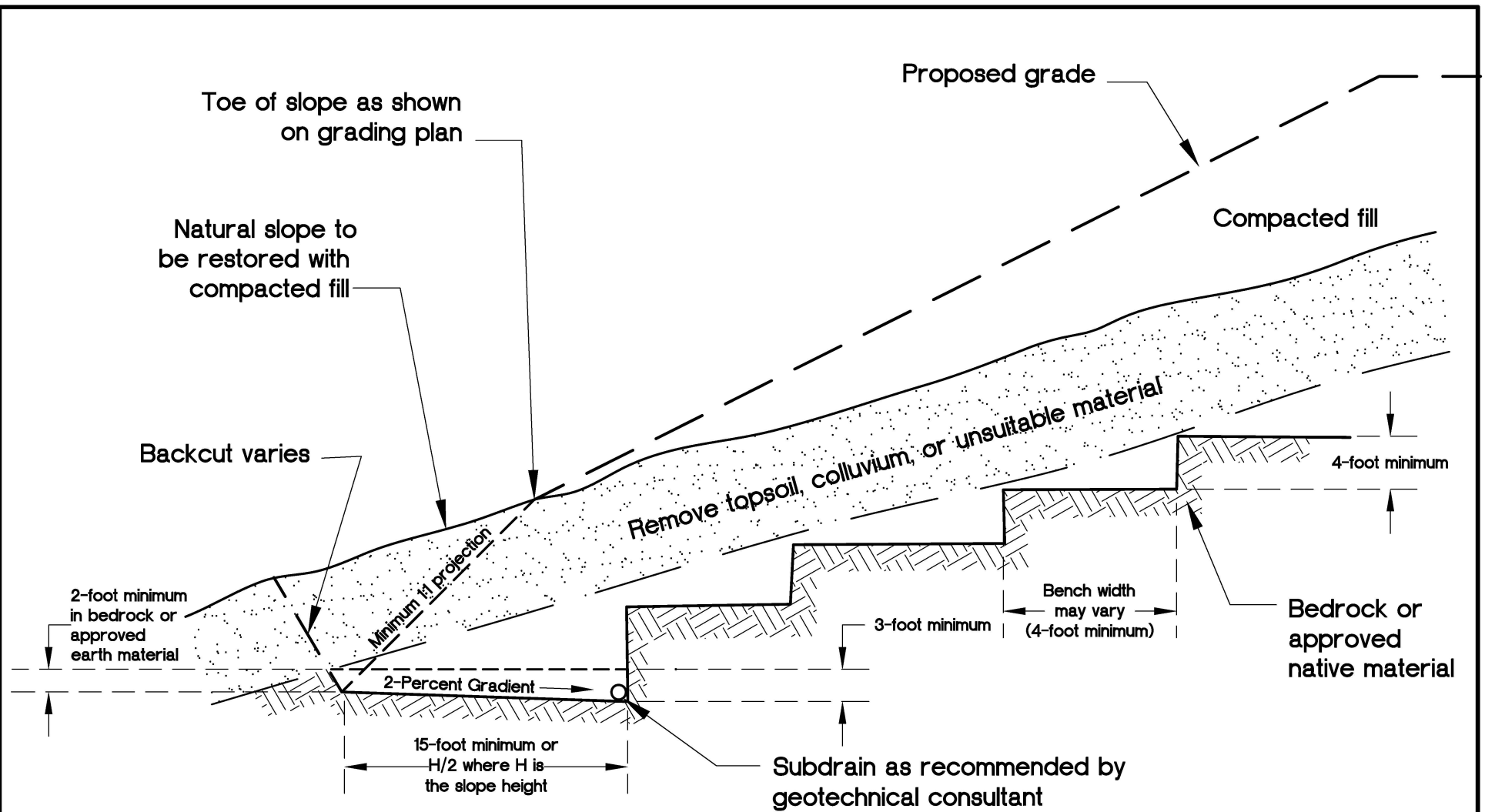
- Notes:**
1. Trench for outlet pipes to be backfilled and compacted with onsite soil.
 2. Backdrains and lateral drains shall be located at elevation of every bench drain. First drain located at elevation just above lower lot grade. Additional drains may be required at the discretion of the geotechnical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

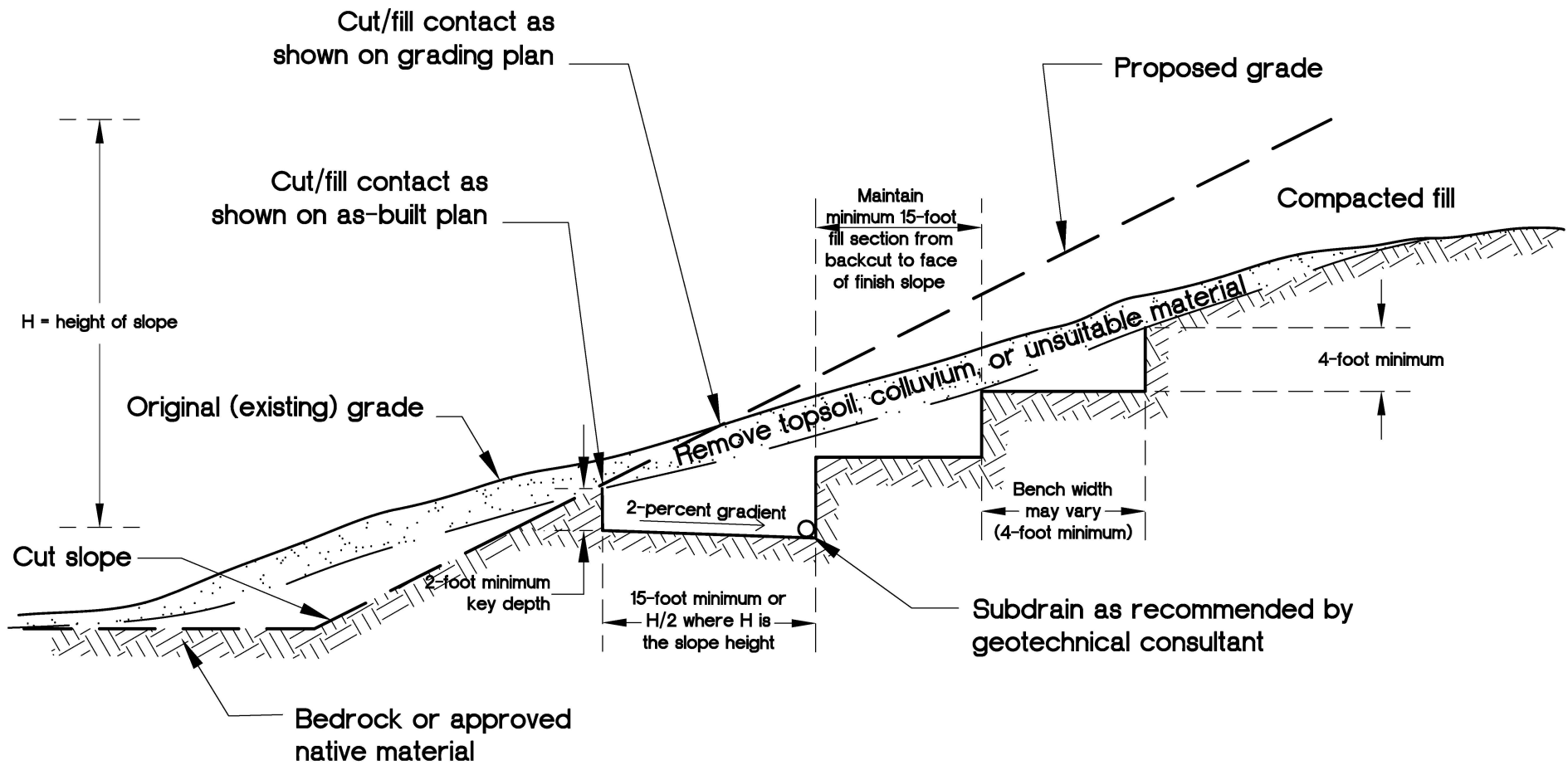
Gravel shall be of the following specification or an approved equivalent.

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

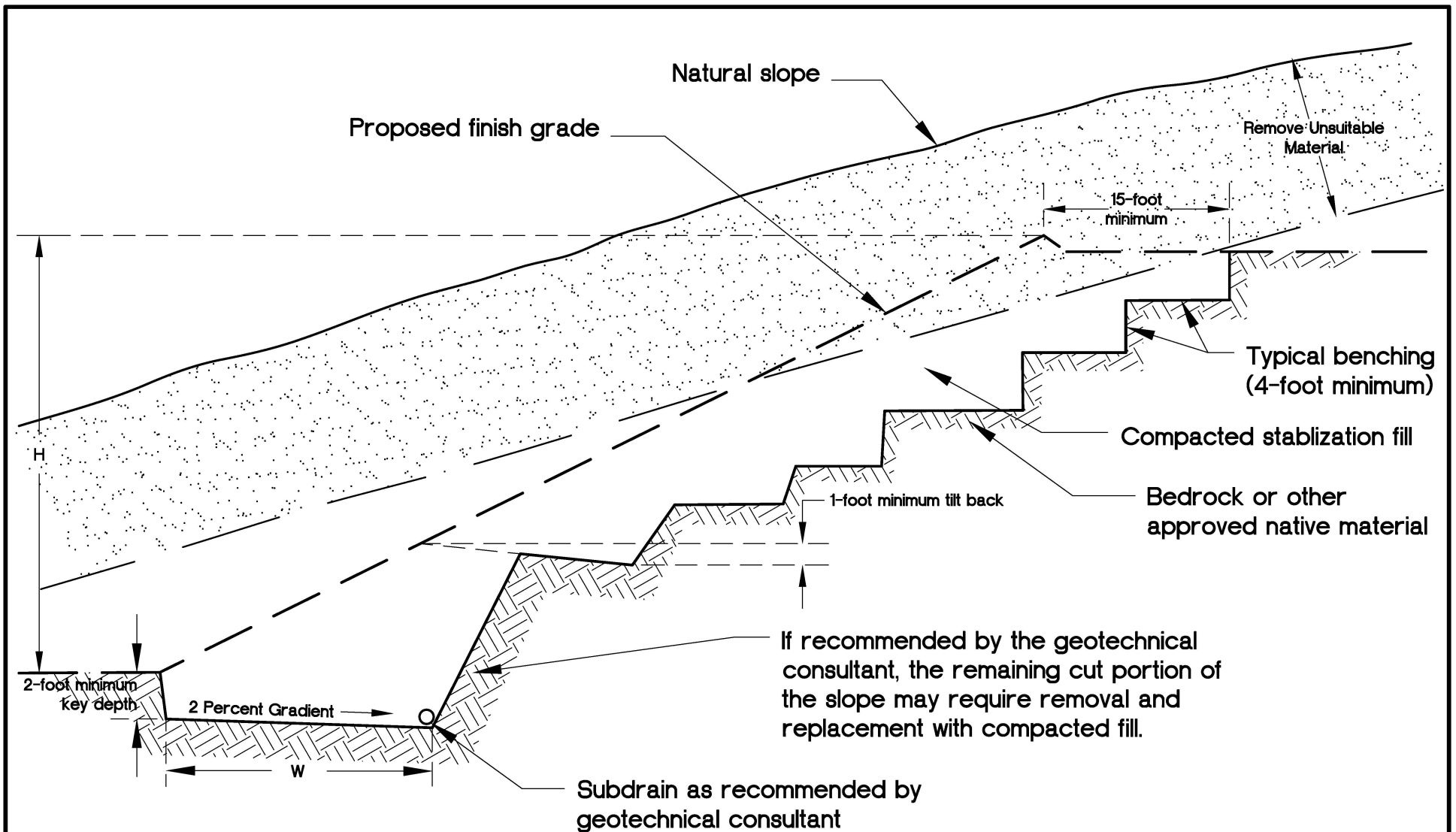


NOTES:

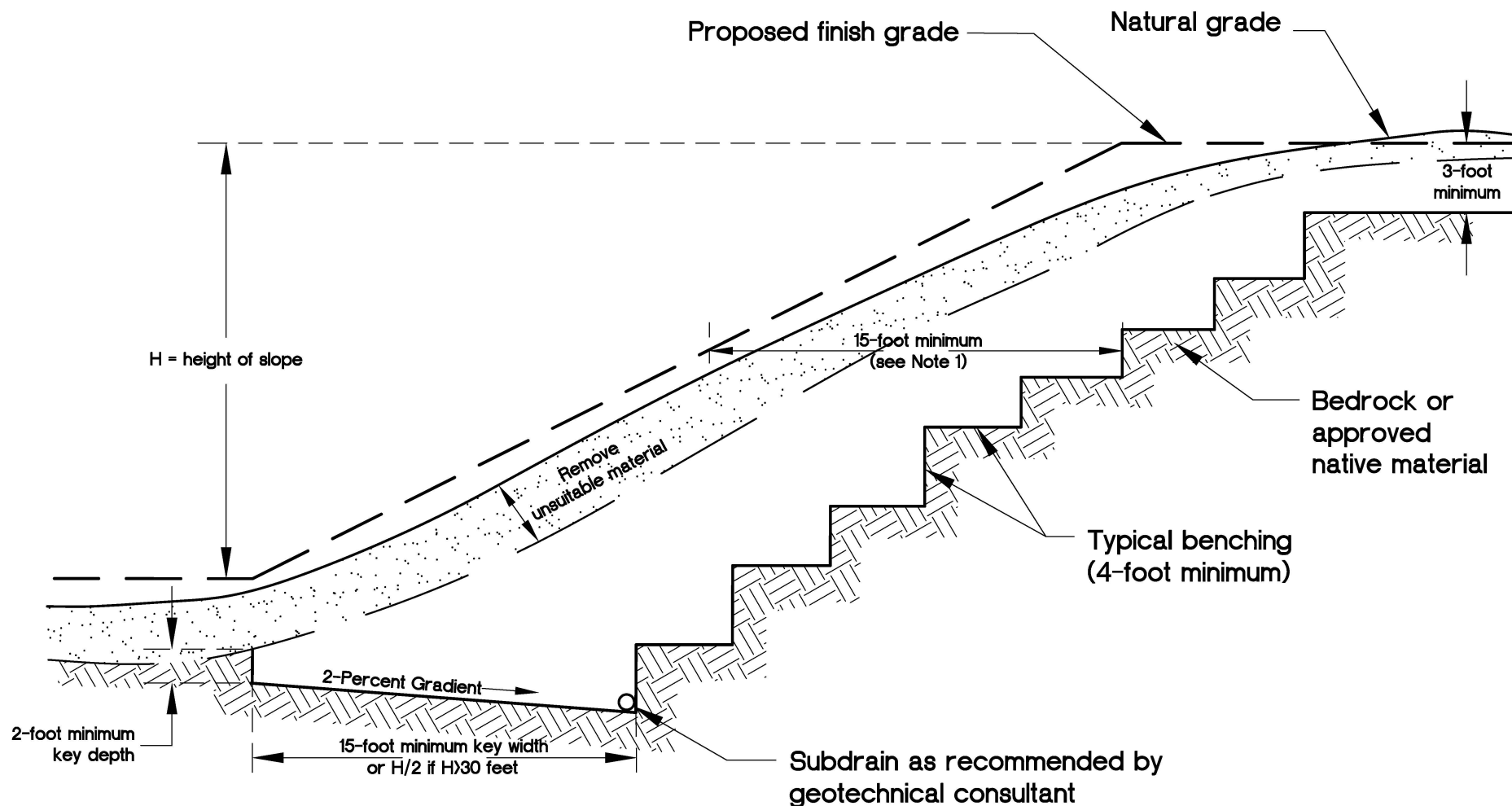
1. Where the natural slope approaches or exceeds the design slope ratio, special recommendations would be provided by the geotechnical consultant.
2. The need for and disposition of drains should be evaluated by the geotechnical consultant, based upon exposed conditions.



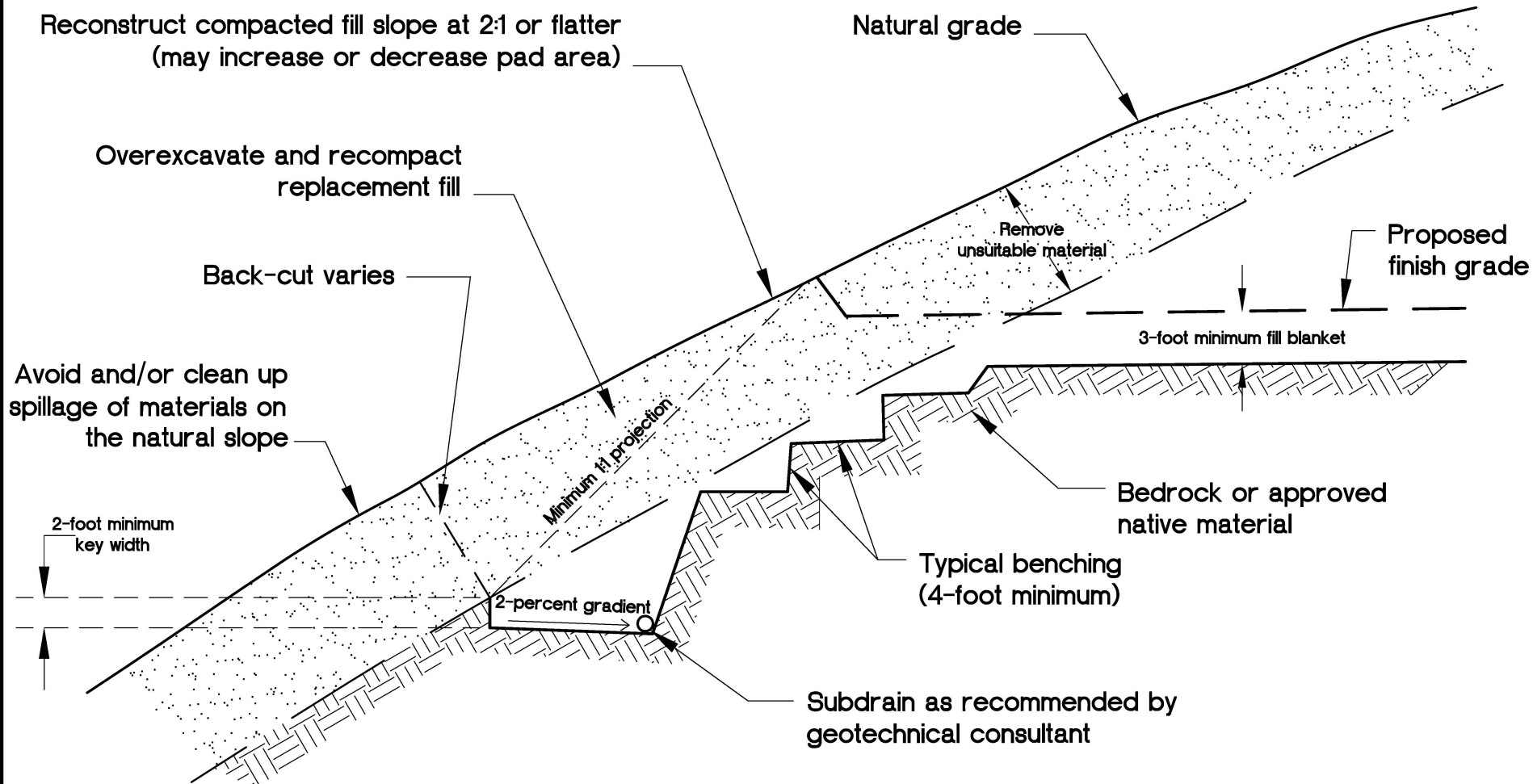
NOTE: The cut portion of the slope should be excavated and evaluated by the geotechnical consultant prior to construction of the fill portion.



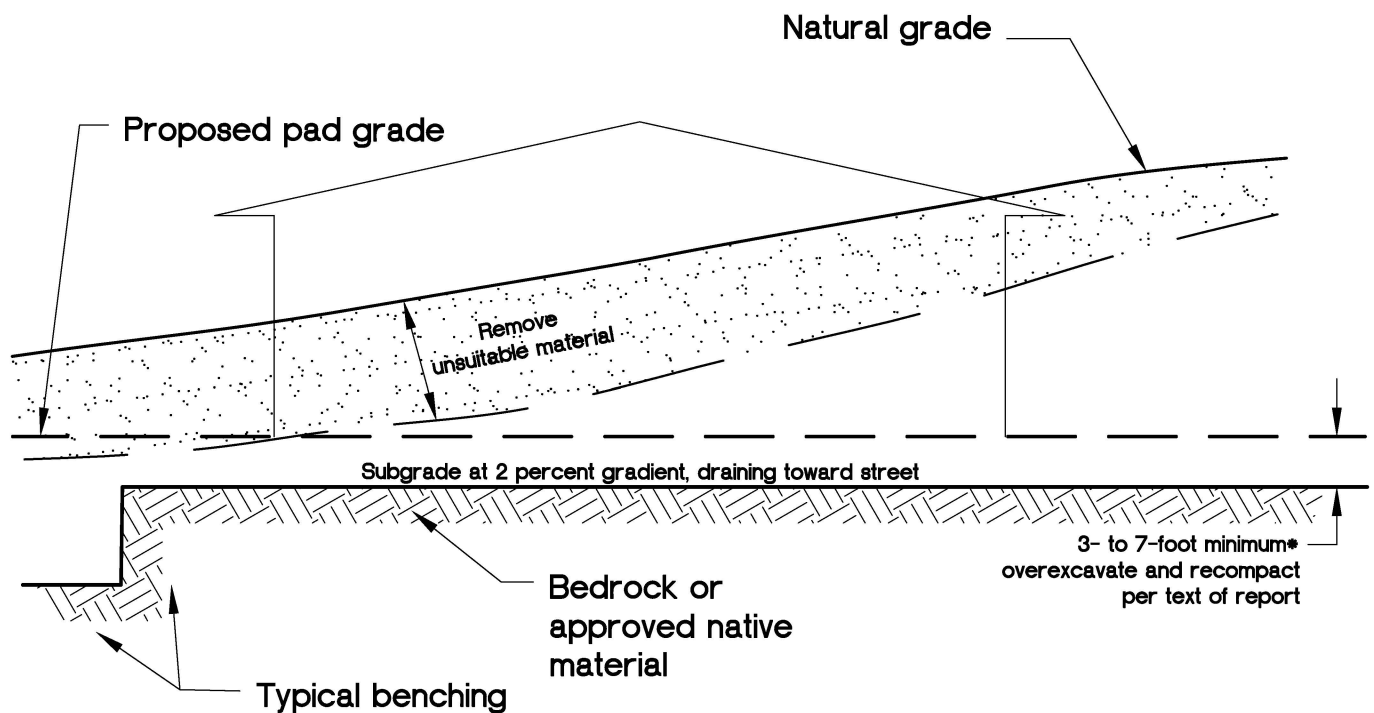
- NOTES:
1. Subdrains may be required as specified by the geotechnical consultant.
 2. W shall be equipment width (15 feet) for slope heights less than 25 feet. For slopes greater than 25 feet, W shall be evaluated by the geotechnical consultant. At no time, shall W be less than $H/2$, where H is the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



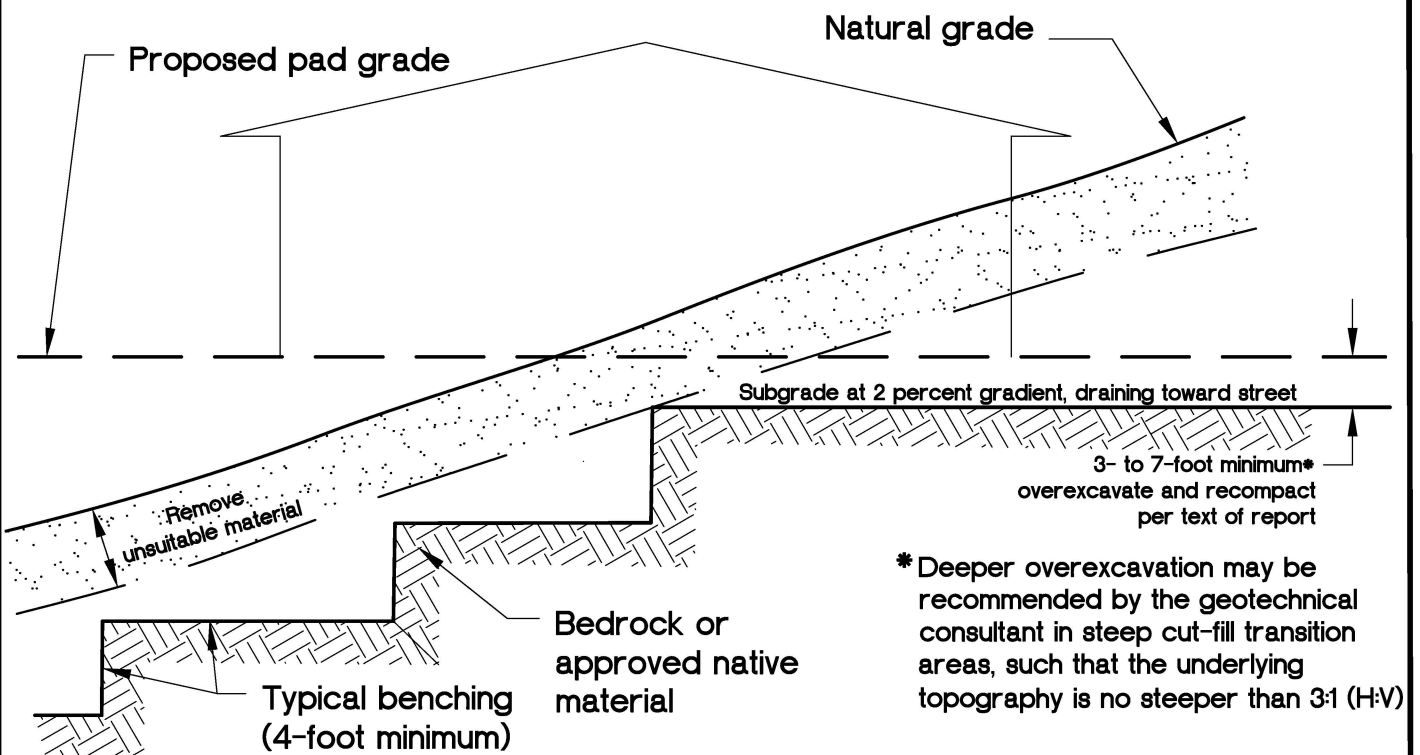
- NOTES:
1. 15-foot minimum to be maintained from proposed finish slope face to backcut.
 2. The need and disposition of drains will be evaluated by the geotechnical consultant based on field conditions.
 3. Pad overexcavation and recompaction should be performed if evaluated to be necessary by the geotechnical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

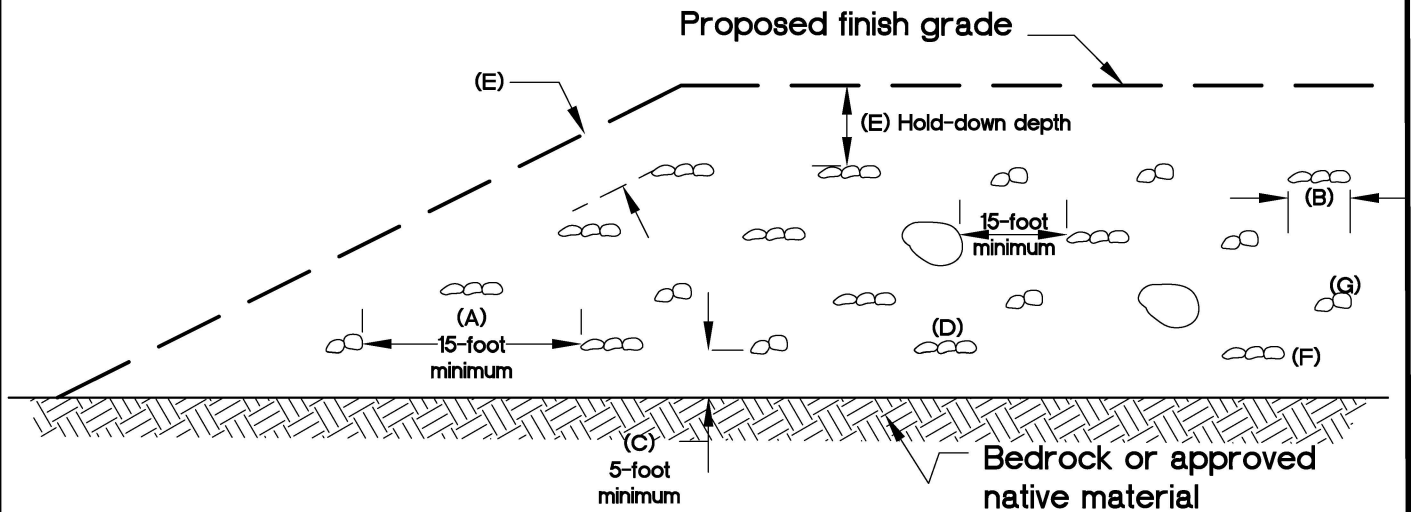


CUT LOT OR MATERIAL-TYPE TRANSITION

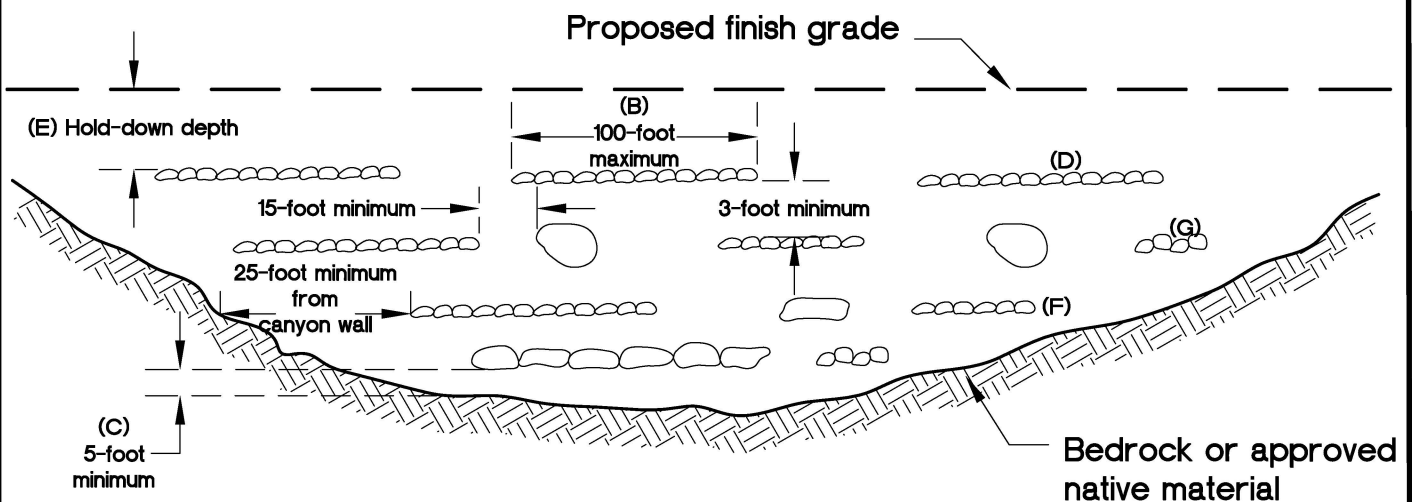


CUT-FILL LOT (DAYLIGHT TRANSITION)

VIEW NORMAL TO SLOPE FACE



VIEW PARALLEL TO SLOPE FACE



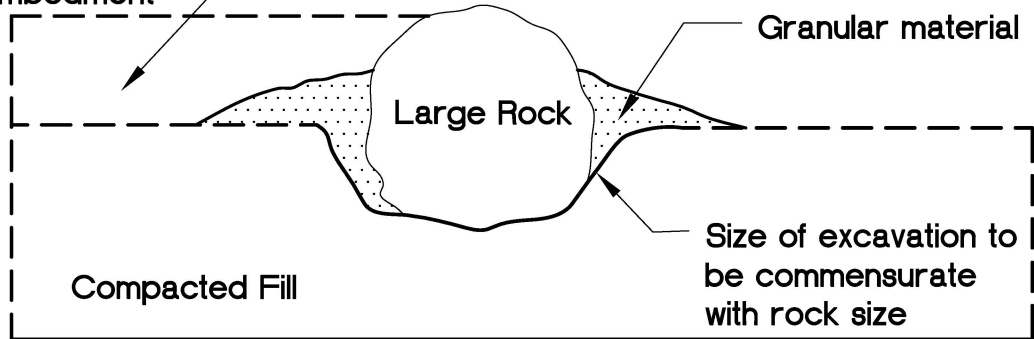
NOTES:

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

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

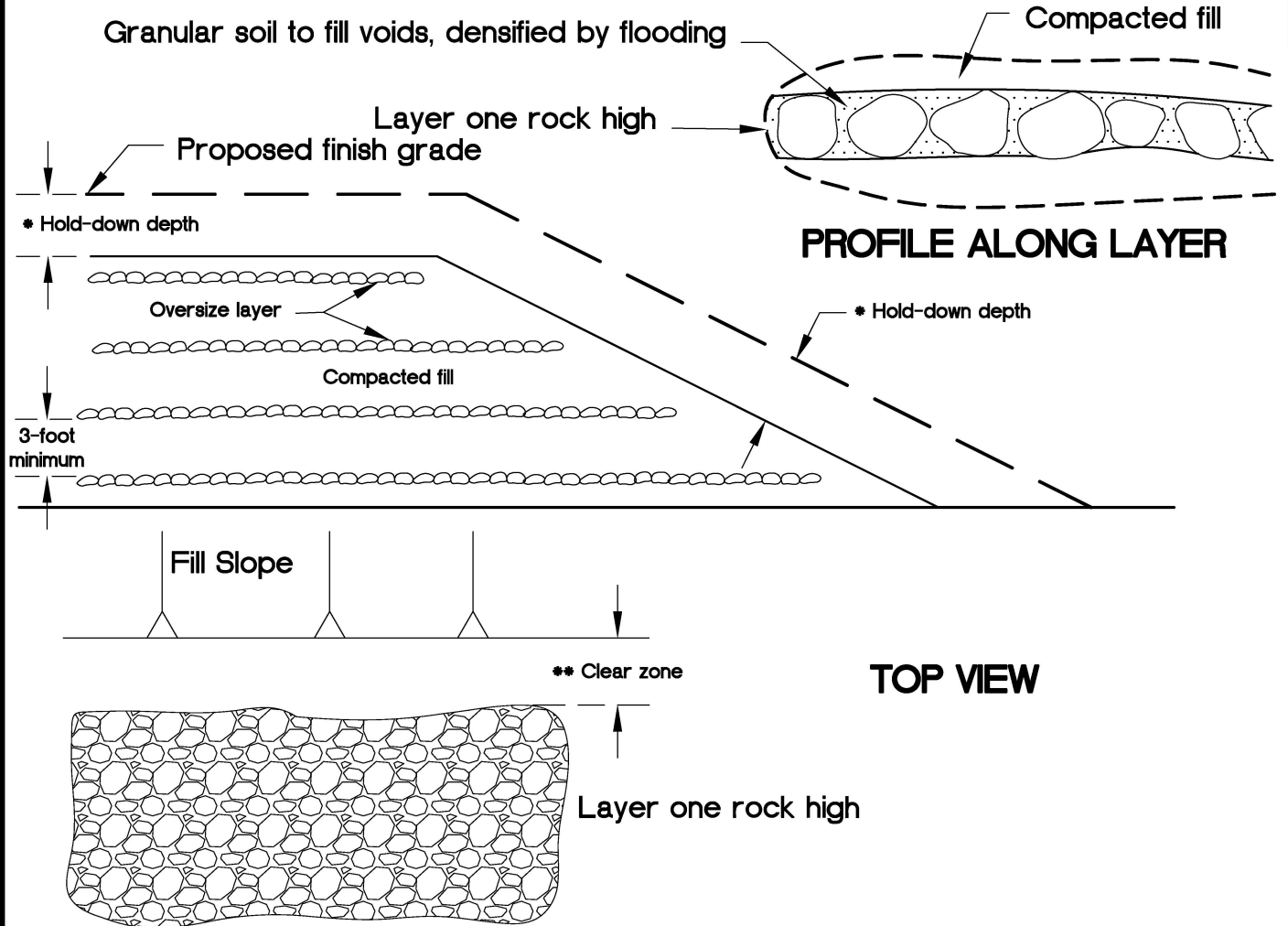
ROCK DISPOSAL PITS

Fill lifts compacted over rock after embedment



ROCK DISPOSAL LAYERS

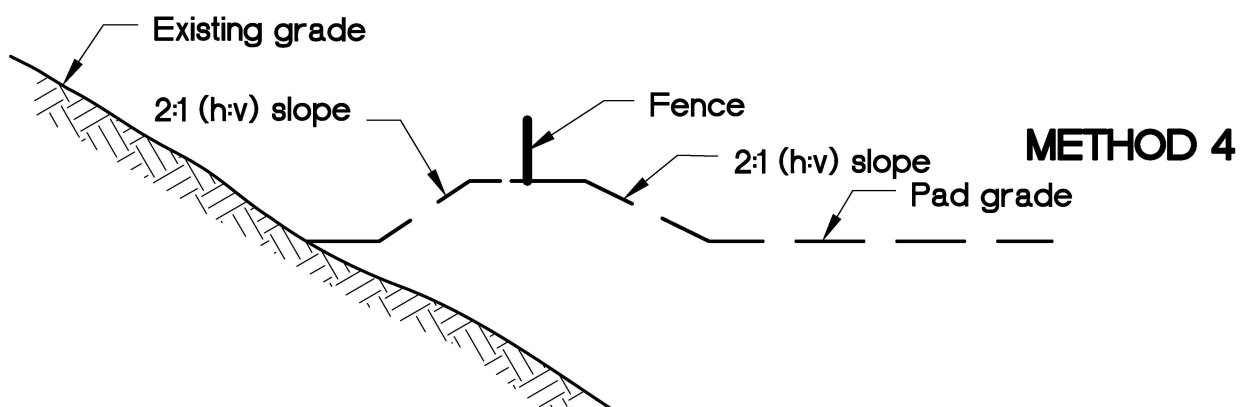
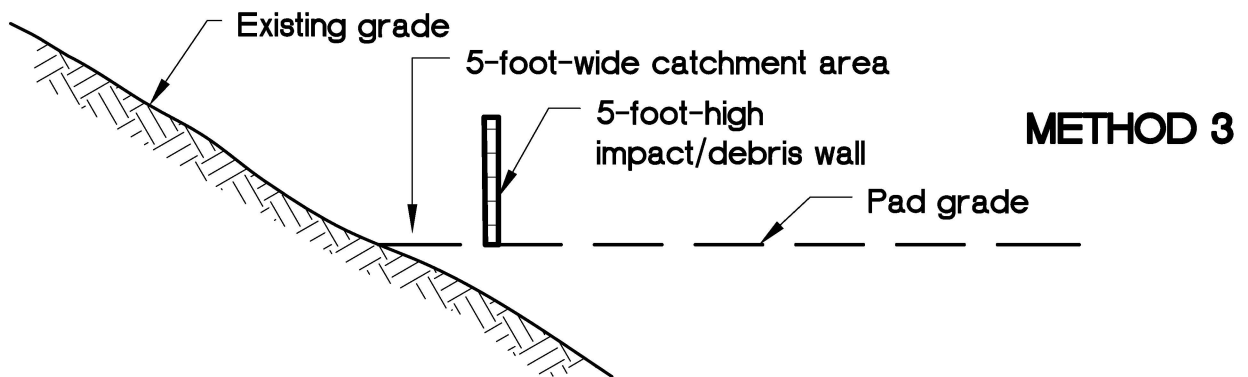
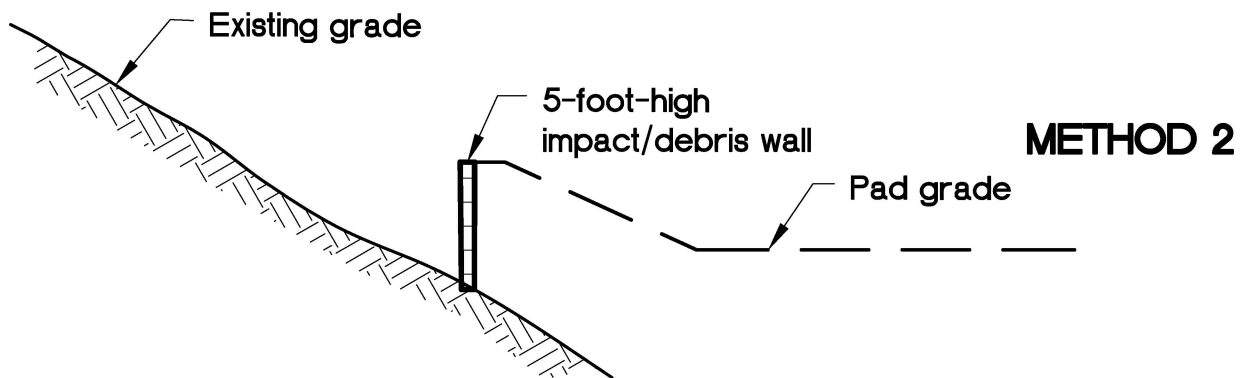
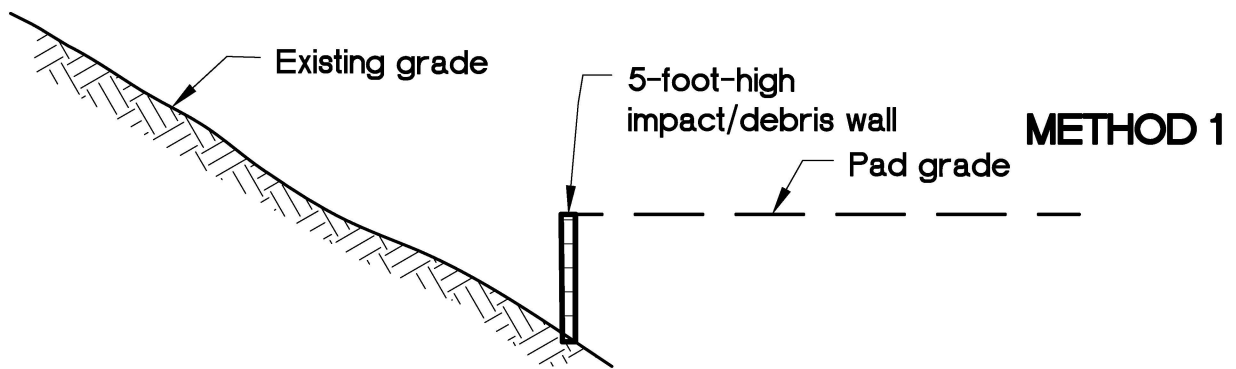
Granular soil to fill voids, densified by flooding



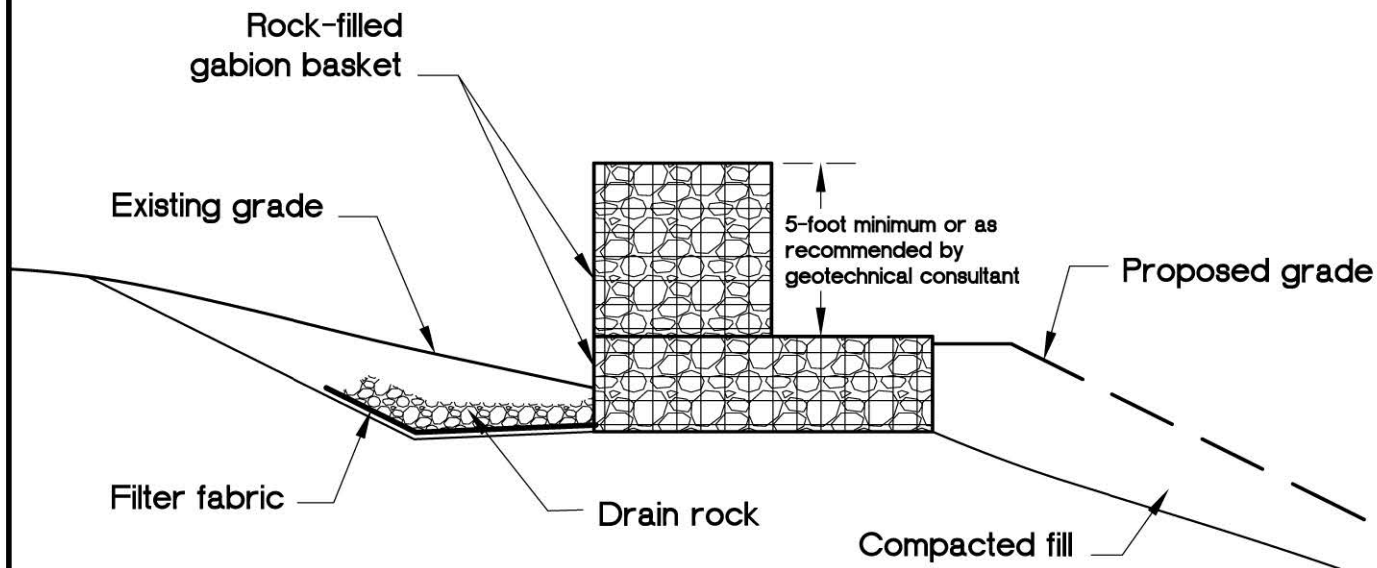
* Hold-down depth or below lowest utility as specified in text of report, subject to governing agency approval.

** 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

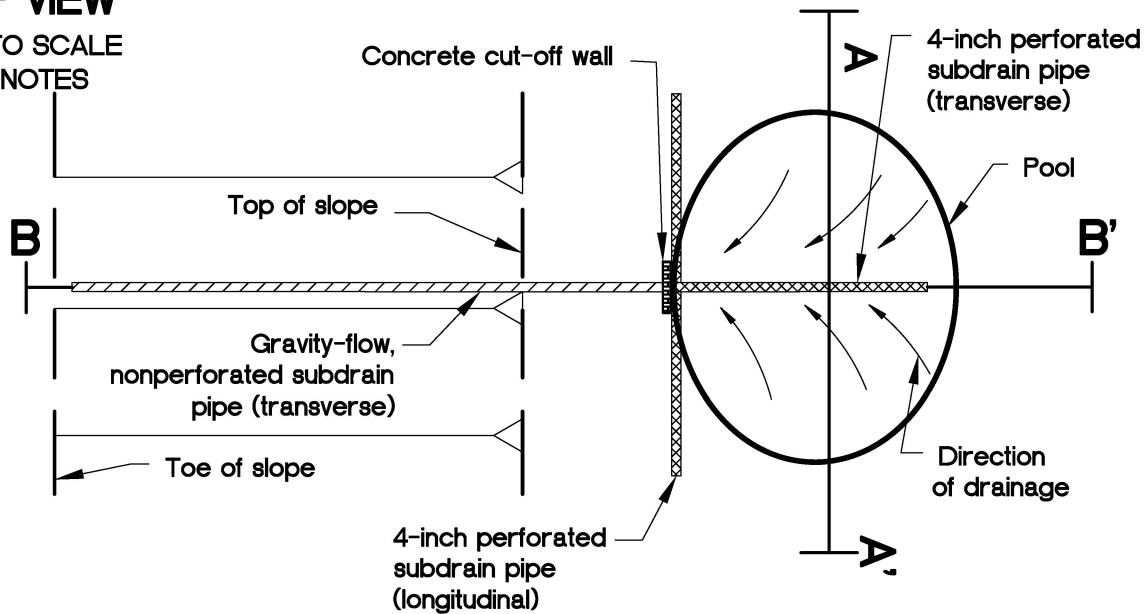


Gabion impact or diversion wall should be constructed at the base of the ascending slope subject to rock fall. Walls need to be constructed with high segments that sustain impact and mitigate potential for overtopping, and low segment that provides channelization of sediments and debris to desired depositional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

From GSA, 1987

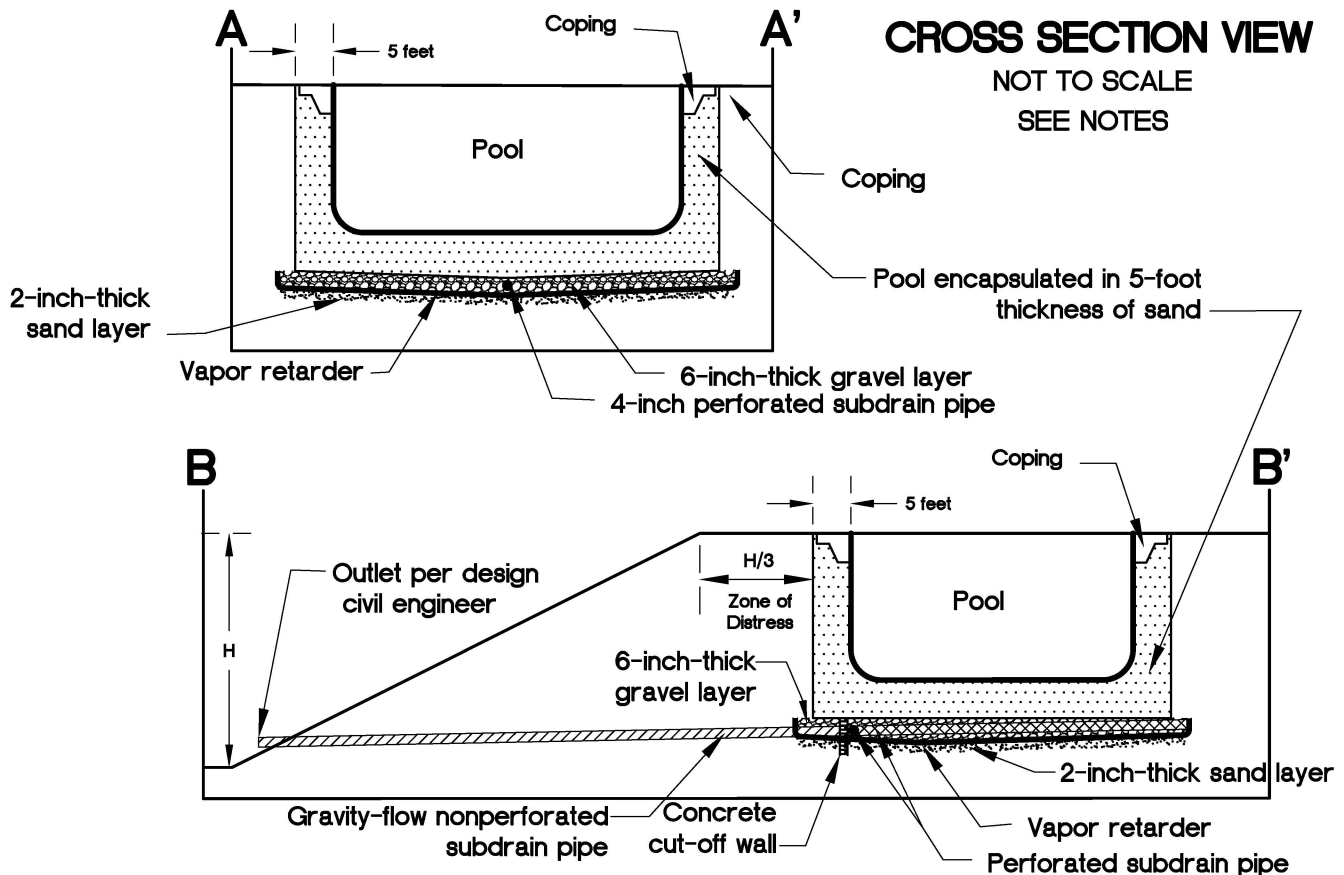
MAP VIEW

NOT TO SCALE
SEE NOTES



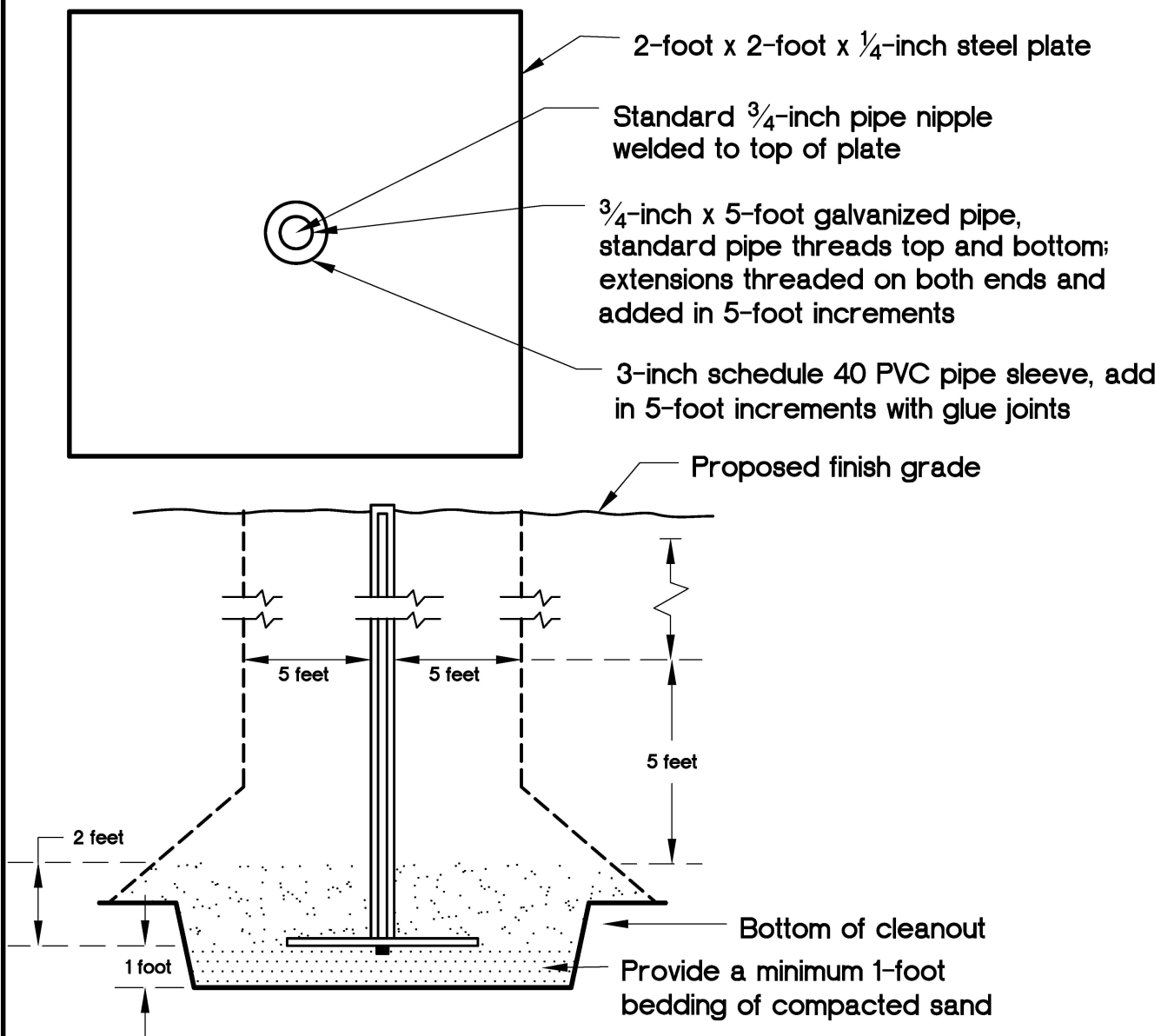
CROSS SECTION VIEW

NOT TO SCALE
SEE NOTES



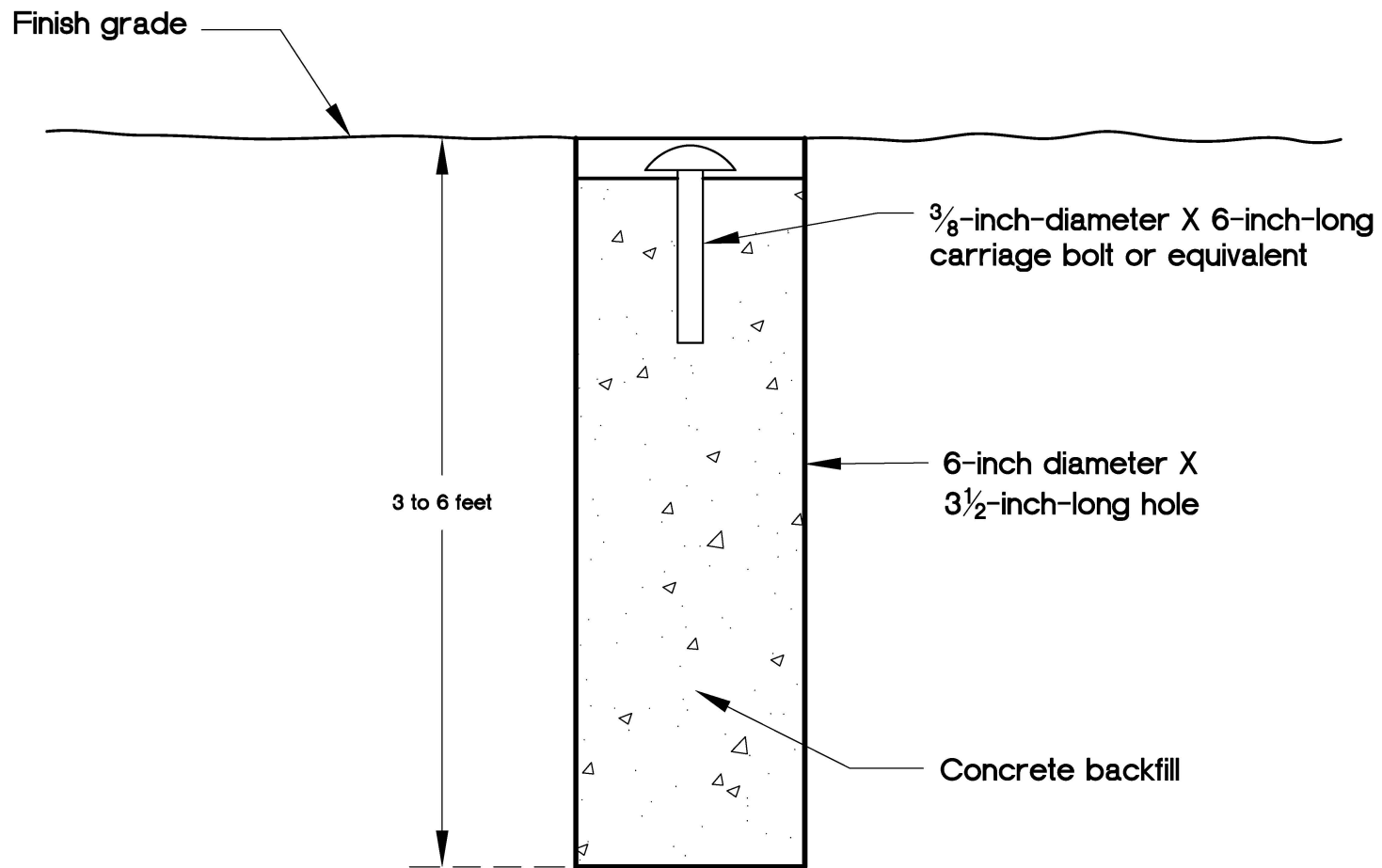
NOTES:

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.

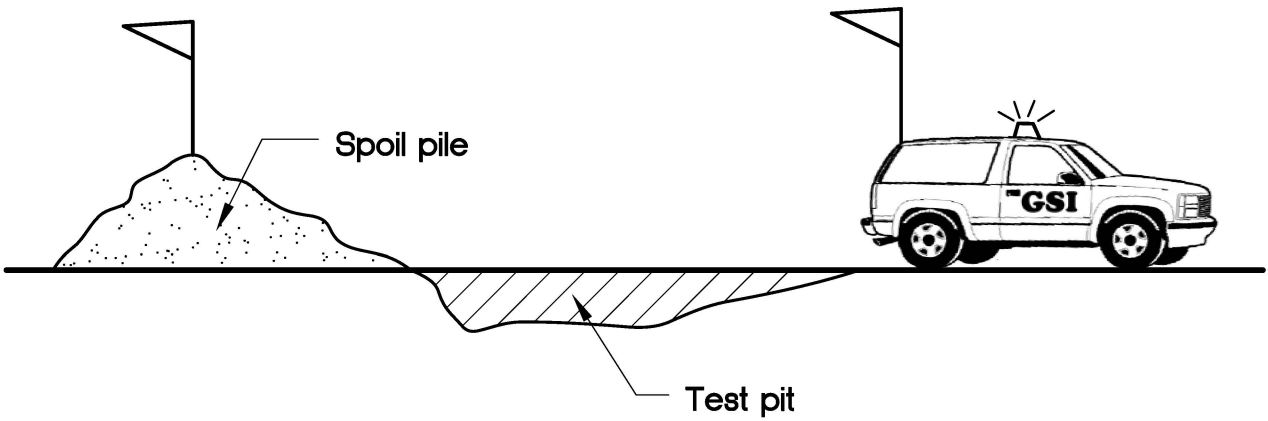


NOTES:

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



SIDE VIEW



TOP VIEW

