PRELIMINARY GEOTECHNICAL EVALUATION
YORK INDEPENDENT LIVING PROJECT
YORK DRIVE AND SOUTH SANTA FE AVENUE
CITY OF VISTA, SAN DIEGO COUNTY, CALIFORNIA 92084

FOR

BALBAS CONSTRUCTION
3189 AIRWAY AVENUE, SUITE D
COSTA MESA, CALIFORNIA 92626

W.O. 7746-A-SC   MARCH 12, 2020

GeoSoils, Inc.

Geotechnical • Geologic • Coastal • Environmental

5741 Palmer Way • Carlsbad, California 92010 • (760) 438-3155 • FAX (760) 931-0915 • www.geosoilsinc.com

March 12, 2020

W.O. 7746-A-SC

Balbas Construction
3189 Airway Avenue, Unit D
Costa Mesa, California 92626

Attention: Mr. Joe Balbas

Subject: Preliminary Geotechnical Evaluation, York Independent Living Project, York Drive and South Santa Fe Avenue, City of Vista, San Diego County, California 92084

Dear Mr. Balbas:

In accordance with your request and authorization, GeoSoils, Inc. (GSI) is pleased to present the results of our preliminary geotechnical evaluation at the subject site. The purpose of our study was to evaluate the geologic and geotechnical conditions at the site, in order to develop preliminary recommendations for site earthwork and the design of foundations, walls, and pavements related to the proposed re-development of the property.

EXECUTIVE SUMMARY

Based upon our field exploration, geologic, and geotechnical engineering analysis, the proposed development appears feasible from a soils engineering and geologic viewpoint, provided that the recommendations presented in the text of this report are properly incorporated into the design and construction of the project. The most significant elements of our study are summarized below:

- In general, the site may be characterized as an irregular, gently sloping (west portion) to relatively flat-lying (east portion) site, approximately 6 acres in size. The site is underlain with surficial deposits of undocumented fill, colluvial topsoils, alluvium, and older alluvium, underlain at depth by Cretaceous-age granitic bedrock. Site drainage is generally directed eastward, toward an existing, south flowing drainage, located near the eastern property line.

- Due to their relatively low density and lack of uniformity, all surficial deposits of existing undocumented fill, colluvium, alluvium, near surface older alluvium, and near surface, weathered bedrock (if present) are not considered unsuitable for the support of settlement-sensitive improvements (i.e., building foundations, concrete slab-on-grade floors, site walls, exterior hardscape, etc.) and/or engineered fill in their existing state. Based on the available data, the thickness of these soils across the site is anticipated to vary between approximately 3 feet to about 9 feet (generally from west to east across the site), with the thicker deposits located toward the eastern property line. However, localized thicker sections of unsuitable soils cannot be precluded, and should be anticipated. Conversely, deposits of older alluvium at depth, and the underlying granitic bedrock formation is generally considered suitable for the support of settlement-sensitive improvements and/or engineered fill.

- The 2019 California Building Code ([2019 CBC], California Building Standards Commission [CBSC], 2019a) indicates that removals of unsuitable soils be performed across all areas to be graded, under the purview of the grading permit, not just within the influence of the building. Relatively deep removals may also necessitate a special zone of consideration, on perimeter/confining areas. This zone would be approximately equal to the depth of removals, if removals cannot be performed onsite or offsite. Thus, any settlement-sensitive improvements (walls, curbs, flatwork, etc.), constructed within this zone may require deepened foundations, reinforcement, etc., or will retain some potential for settlement and associated distress. This will also require proper disclosure to any owners and all interested/affected parties should this condition exist at the conclusion of grading.

- Expansion index (E.I.) testing performed on a representative sample of the onsite soils indicates a range of expansion potentials from very low expansive (E.I. < 21), to medium expansive (E.I. range of 51 to 90). Thus, on a preliminary basis, site soils are anticipated to vary from non-detrimentally expansive to detrimentally expansive. As this site requires a significant amount of import to construct the proposed building pad, the as-built, expansive character of soils underlying the building pad is currently unknown, and will be dependent on the nature of soils imported to the site. The use of very low expansive import for the support of structures will not require specific structural design for the mitigation of shrink/swell effects. Conversely, importing expansive soils with require such design. Additional evaluations regarding the expansion potential of the onsite soils should be performed during site grading, and prior to foundation construction, in order to evaluate the "as-built" expansive character of soils underlying the planned building pad.

- An evaluation of the site soils corrosion potential indicates that site soils are mildly alkaline with respect to pH, moderately corrosive to exposed buried metals when saturated, present negligible sulfate exposure to concrete, and are below the action level for chloride exposure. Site soils are classified as "Exposure Class C1." Import soils will need to be evaluated for the ultimate "as-built" condition.

- A "desk top" review of storm water infiltration indicates that a <u>no</u> infiltration design is recommended.

- A perched water table within younger alluvium was encountered at a shallow depth during our subsurface work completed in preparation of this report. Previous sitework, completed on an adjacent property to the east, also encountered zones of perched groundwater within near surface alluvial soils overlying granitic bedrock

(GSI, 1995). The regional groundwater table was not encountered during this, or the previous 1995 study. Perched groundwater, or the regional groundwater table at depth, is not anticipated to significantly affect the planned improvements, provided that the conclusions and recommendations presented herein, are properly incorporated into the design and construction of the project. Perched water may occur in the future along zones of contrasting permeability and/or density. This potential should be disclosed to all interested/affected parties.

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

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

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

- Site soils are not considered suitable for storm water infiltration, due to the proximity of BMP's to planned graded slopes, the presence of a shallow perched groundwater table within the adjacent alluvial drainage, the underlying dense, granitic bedrock, and the potential adverse affect on existing offsite improvements. The Hydrologic Soil Group for this site is "D," and a no-infiltration design is recommended.

- The recommendations presented in this report should be incorporated into the design and construction considerations of the project.

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

Respectfully submitted,

GeoSoils, Inc.

PROFESSIONAL GEOLOGIST
ROBERT G. CRISMAN
No. 1934
Certified
Engineering
Geologist
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
DAVID W. SKELLY
No. RCE 47857
Exp. 12/31/21
CIVIL
STATE OF CALIFORNIA

Robert G. Crisman
Engineering Geologist, CEG 1934

David W. Skelly
Civil Engineer, RCE 47857

RGC/JPF/DWS/mn

Distribution: (3) Addressee

TABLE OF CONTENTS

PRELIMINARY GEOTECHNICAL EVALUATION
YORK INDEPENDENT LIVING PROJECT
YORK DRIVE AND SOUTH SANTA FE AVENUE
CITY OF VISTA, SAN DIEGO COUNTY, CALIFORNIA 92084

SCOPE OF SERVICES

The scope of our services has included the following:

1. Review of readily available published literature, aerial photographs, and geologic maps of the vicinity (see Appendix A), including proprietary in-house geologic/geotechnical reports for other nearby sites.

2. Reconnaissance geologic mapping, and the excavation of eight (8) exploratory test pits with a track mounted excavator, to evaluate the soil and rock profiles, sample representative soils, and delineate the horizontal and vertical extent of earth material units (see Appendix B).

3. General areal seismicity evaluation (see Appendix C).

4. Appropriate laboratory testing of relatively undisturbed and representative bulk soil samples collected during our geologic mapping and subsurface exploration program.

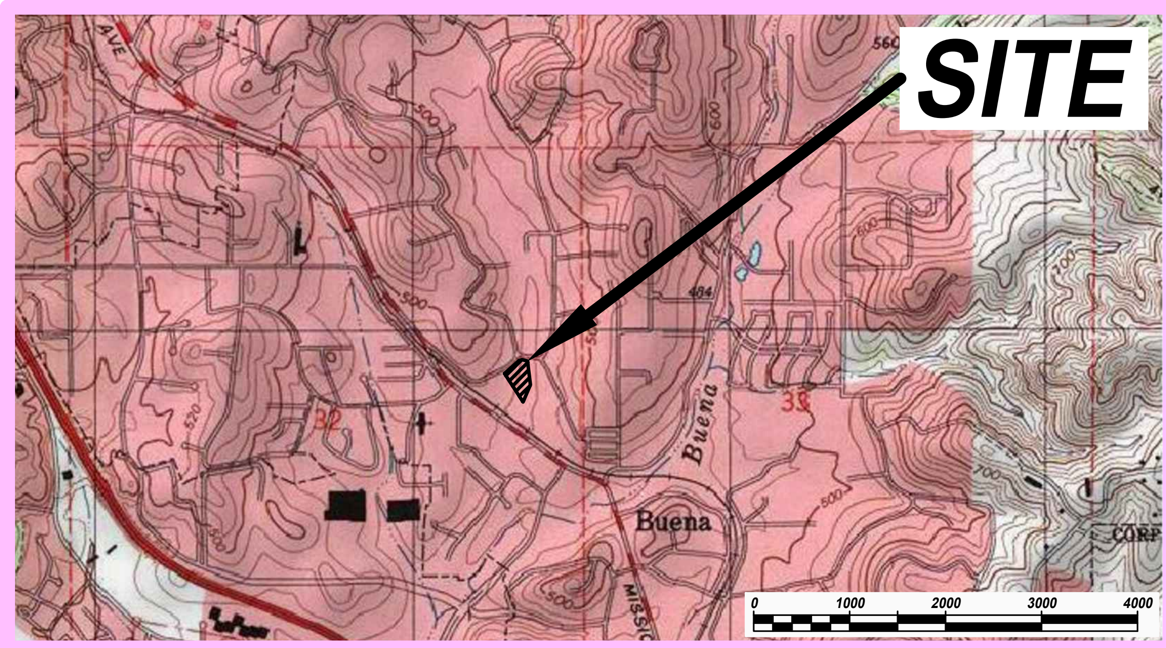
5. Completion of a "desktop" evaluation for storm water infiltration (see Appendix E).

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

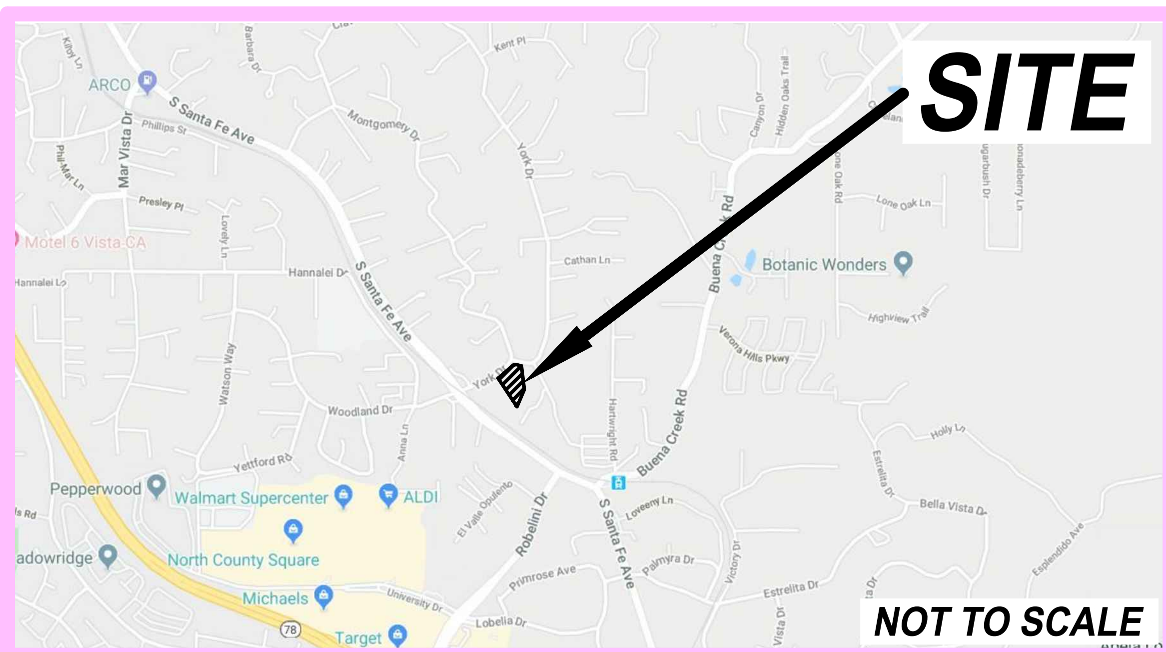
7. Appropriate engineering and geologic analyses of data collected, and the preparation of this summary report and accompaniments.

SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The subject site consists of an irregular shaped parcel, approximately 6 acres in size, located east of the intersection of York Drive and South Santa Fe Avenue, in the City of Vista, San Diego County, California (see Site Location Map, Figure 1). The property is bounded by York Drive to the northwest, S. Santa Fe Avenue and a North County Transit District easement (railway) on the southwest, the existing Devon Park Nursing facility on the east/southeast, and existing residential development on the northeast. Access to subject property is via York Drive. Topographically, the site generally slopes very gently to the southeast, toward an existing south to southwest flowing drainage located near the east/southeast property line. Site elevations appear to range from a high of about 454 feet MSL within the western corner of the site, to a low of about 428 feet MSL at the southern corner of the site, for a total relief of about 26 feet. Drainage appears to be generally directed southeastward across the site as sheet flow towards the existing drainage near



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



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

N

GeoSoils, Inc.	W.O. 7746-A-SC

SITE LOCATION MAP

Figure 1

the eastern/southeastern property line, where runoff ultimately exists near the southern corner of the site. Existing improvements generally consist of about four (4) residential structures and several ancillary out-buildings, scattered throughout the site, with demolition work in progress during our field work. Vegetation generally consists of scattered weeds and grasses, with few trees located near the existing drainage and some of the existing structures.

Based on a review of the reference documents and clients' communication, the proposed development will generally consist of preparing the site for the construction of a large building pad for the support of a four (4) story, senior residential care facility. Site grading is anticipated to consist of the placement of up to about 15 feet of "plan" fill in order to create the main building pad, with graded fill slopes ranging up to about 8 feet in height for cut slopes, and up to about 20 feet in height for fill slope. All slopes are planned at gradients of 2:1 (h:v), or flatter. Based on a review of preliminary architectural plans prepared by Knitter Partners International, Inc. (KPI, 2020), proposed construction is planned to consist of a large residential complex consisting of ground floor facilities, such as offices, kitchen, restaurant, game room, theater, etc., and residential units, with the upper three floors primarily consisting of residential units. Construction is planned to utilize wood frames with typical foundations and slab-on-grade ground floors. Building loads are assumed to be typical for this type of relatively light construction. Sewage disposal is anticipated to be connected into the regional, municipal system. Storm water may be treated onsite prior to its delivery into the municipal system.

FIELD STUDIES

Site-specific field studies were conducted by GSI during December 2019, and consisted of reconnaissance geologic mapping and the excavation of eight (8) exploratory test excavations with a track mounted excavator, for an evaluation of near-surface soil and geologic conditions onsite. The test excavations were logged by a representative of this office who collected representative bulk soil samples for appropriate laboratory testing. The logs of the test excavations are presented in Appendix B. The approximate location of the test excavations are presented on the Geotechnical Map, which uses the "Site Study" plan prepared by KPI (2020), as a base (see Plate 1).

Field work was also performed for the adjacent Devon Park nursing facility in 1995 (GSI, 1995) and included the completion of two (2) soil borings near the subject sites eastern/southeastern property line. Soil information from these borings was reviewed in preparation of this report.

REGIONAL GEOLOGY

The subject property lies within the coastal plain physiographic region of the Peninsular Ranges Geomorphic Province of southern California. This region consists of dissected, mesa-like terraces that transition inland to rolling hills. The encompassing Peninsular Ranges Geomorphic Province is characterized as elongated mountain ranges and valleys that trend northwesterly. This geomorphic province extends from the base of the east-west aligned Santa Monica - San Gabriel Mountains, and continues south into Baja California. The mountain ranges within this province are underlain by basement rocks consisting of pre-Cretaceous metasedimentary rocks, Jurassic metavolcanic rocks, and Cretaceous plutonic (granitic) rocks.

In the southern California region, deposition occurred during the Cretaceous Period and Cenozoic Era in the continental margin of a forearc basin. Sediments, derived from Cretaceous-age plutonic rocks and Jurassic-age volcanic rocks, were deposited during the Tertiary Period (Eocene-age) into the narrow, steep, coastal plain and continental margin of the basin. These rocks have been uplifted, eroded, and deeply incised. During early Pleistocene time, a broad coastal plain was developed from the deposition of marine terrace deposits. During mid to late Pleistocene time, this plain was uplifted, eroded and incised. Alluvial deposits have since filled the lower valleys, and young marine sediments are currently being deposited/eroded within coastal and beach areas. Regional geologic mapping by Kennedy and Tan (2007) indicates the site is underlain by Cretaceous-age granitic bedrock mapped as “undivided tonalite,” with Quaternary-age alluvium mapped along the eastern-southeastern property line.

SITE GEOLOGIC UNITS

General

The earth material units that were observed and/or encountered at the subject site consist of surficial deposits of undocumented fill, colluvium (topsoil), alluvium, and older alluvium, overlying Cretaceous-age granitic bedrock (formation), consisting of “undivided tonalite.” A general description of each material type is presented as follows, from youngest to oldest.

Undocumented Fill (Map Symbol - Afu)

Deposits of undocumented fill occur throughout the site, being intimately associated with existing improvements as building pads and embankments, on the order of about 2 to 3 feet in thickness. Locally, these embankments may potentially be on the order of about 6 feet in thickness. Where encountered, undocumented fill consists of dark brown, silty to clayey sand, and gray sand, typically observed to be moist to wet, and loose, with areas containing abundant organic (plant matter), wood, concrete, and plastic construction debris. Existing undocumented fill is considered potentially compressible in their existing state. As such, undocumented fill should not be used for the support of settlement-sensitive improvements and/or any planned fill, unless adequately remediated, including the removal of deleterious material (wood, organic debris, plastic, etc.).

Colluvium (not mapped)

As observed, existing Quaternary-age colluvium occurs within the upper elevations of the site (where alluvial deposits were not encountered) as a surficial soil layer on the order of about 2 to 2½ feet in thickness. Where encountered, colluvium consists of brown to dark brown, silty to clayey sands, typically observed to be slightly moist to wet, loose, and porous, with few roots. Existing deposits of colluvium are considered potentially compressible in their existing state. As such, colluvium should not be used for the support of settlement-sensitive improvements and/or any planned fill, unless adequately remediated.

Quaternary Alluvium (Map Symbol - Qa)

As observed, existing Quaternary-age alluvium occurs in and near the existing drainage located along the east-southeast side of the site as a surficial soil layer on the order of about 2 to 3 feet in thickness. Where encountered, alluvium consists of dark brown, silty to clayey sand, and sandy clay, typically observed to be wet/saturated, loose (sands), firm (clays), and porous, with roots. Existing deposits of alluvium are considered potentially compressible in their existing state. As such, alluvium should not be used for the support of settlement-sensitive improvements and/or any planned fill, unless adequately remediated.

Quaternary Older Alluvium (Map Symbol - Qalo)

As observed, existing Quaternary-age deposits of older alluvium occurs within the lower elevations of the site, as a near surface soil layer on the order of about 3 to 9 feet in thickness, underlying deposits of undocumented fill, colluvium, and alluvium. Where encountered, older alluvium varies from dark brown silty sand w/clay, brown clayey sand, and dark brown sandy clay, typically observed to be moist to wet, loose to dense (sands) and stiff (clays), with density increasing with depth. Existing near surface deposits of older alluvium are considered potentially compressible near the surface, becoming suitable for the support of settlement sensitive improvements at depth. Near surface deposits of older alluvium should not be used for the support of settlement sensitive improvements and/or any planned fill, unless adequately remediated.

Cretaceous Undivided Tonalite (Map Symbol - Kt)

Bedrock, consisting of granitic rock belonging to the Cretaceous-age southern California Batholith. It was encountered near the surface and at depth. Where encountered, this bedrock disintegrates to yellowish brown, light brown, and brown silty sands and sands, with local production of coarse gravel to cobble size rock fragments. Bedrock is typically slightly moist to moist and dense. Bedrock is generally considered suitable for the support of settlement sensitive improvements and planned fills. The weathered zone may require remediation, based on conditions exposed in the field during grading.

GROUNDWATER

Water seepage into the upper sections of our exploratory excavations was observed within test pits TP-1, TP-2, and TP-3, located within the lower, alluviated areas of the site. The seepage appears to be perched within alluvial soils, overlying older alluvium and granitic bedrock at depth. Water seepage was also observed within test pits TP-4 and TP-6 and appears to be related to a recent rainstorm which occurred just prior to site excavation. Within test pit TP-6, storm water appears to have collected with a relatively permeable zone of undocumented fill perched above underlying colluvium, while subsurface water in test pit TP-4 was noted from a shallow, perforated PVC pipe, embedded within undocumented fill. In preparation of GSI (1995) zones of seepage within adjacent, offsite borings, were observed at depths ranging from about 3 to 10 feet below grade within near surface alluvial soils. GSI did not observe evidence of a regional groundwater table at depth and a regional groundwater table is not anticipated to significantly affect proposed site development, provided that the recommendations contained in this report are properly incorporated into final design and construction. These observations reflect site conditions at the time of our investigation and do not preclude future changes in local groundwater conditions from excessive irrigation, precipitation, or that were not obvious, at the time of our investigation.

Seeps, springs, or other indications of subsurface water were not noted on the subject property during the time of our field investigation. However, perched water seepage may occur locally (as the result of heavy precipitation and/or irrigation, or damaged wet utilities) along zones of contrasting permeabilities/densities (fill/formation contacts, sandy/clayey fill lifts, etc.) or along geologic discontinuities. This potential should be anticipated and disclosed to all interested/affected parties. Dependant upon the time of year site grading occurs, perched groundwater may adversely affect site development. Effects may include, but not limited to special handling of fill soils (drying, spreading, mixing, etc.) and\or specialized excavation equipment. These observations reflect site conditions at the time of this geotechnical study and do not preclude changes in local groundwater conditions in the future.

Due to the potential for post-development perched water to manifest near the surface, owing to as-graded permeability/density contrasts, more onerous slab design is necessary for any new slab-on-grade floor (State of California, 2020). Recommendations for reducing the amount of water and/or water vapor through slab-on-grade floors are provided in the “Soil Moisture Considerations” sections of this report.

GEOLOGIC HAZARDS EVALUATION

Mass Wasting/Landslide Susceptibility

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

According to regional landslide susceptibility mapping by Tan and Giffen (1995), the site is located within landslide susceptibility Subarea 3-1, which is characterized as being "generally susceptible" to landsliding. However, geomorphic expressions indicative of past mass wasting events (i.e., scarps and hummocky terrain) were not observed on the property during our field studies nor our review of regional geologic mapping. Further, no adverse geologic structures were encountered during our subsurface exploration. Regional geologic maps do not indicate the presence of landslides on the property.

The onsite soils are considered erosive. Therefore, slopes comprised of these materials may be subject to rilling, gullying, sloughing, and surficial slope failures depending on rainfall severity and surface drainage practices. Such risks can be minimized through properly designed, and regularly and periodically maintained surface drainage.

FAULTING AND REGIONAL SEISMICITY

Regional Faults

Our review indicates that there are no known active faults crossing the project and the site is not within an Alquist-Priolo Earthquake Fault Zone (California Geological Survey, 2018). However, the site is situated in an area of active faulting. The Rose Canyon fault is the closest known active fault to the site (located at a distance of approximately 11.6 miles [18.6 kilometers]) and should have the greatest effect on the site in the form of strong ground shaking, should the design earthquake occur. A list and the location of the Rose Canyon fault and other major faults relative to the site is provided in Appendix C. The possibility of ground acceleration, or shaking at the site, may be considered as approximately similar to the southern California region as a whole.

Local Faulting

Although active faults lie within a few miles of the site, no local active faulting was noted in our review, nor observed to specifically transect the site during the field investigation.

Additionally, a review of available regional geologic maps does not indicate the presence of local active faults crossing the specific project site.

Seismicity

It is our understanding that site-specific seismic design criteria from the 2019 California Building Code ([2019 CBC], California Building Standards Commission [CBSC], 2019a), are to be utilized for foundation design. Much of the 2016 CBC relies on the American Society of Civil Engineers (ASCE) Minimum Design Loads for Buildings and Other Structures (ASCE Standard 7-16). The seismic design parameters provided herein are based on the 2019 CBC.

The acceleration-attenuation relation of Bozorgnia, Campbell, and Niazi (1999) has been incorporated into EQFAULT (Blake, 2000a). EQFAULT is a computer program developed by Thomas F. Blake (2000a), which performs deterministic seismic hazard analyses using digitized California faults as earthquake sources. The program estimates the closest distance between each fault and a given site. If a fault is found to be within a user-selected radius, the program estimates peak horizontal ground acceleration that may occur at the site from an upper bound (formerly "maximum credible earthquake"), on that fault. Upper bound refers to the maximum expected ground acceleration produced from a given fault. Site acceleration (g) was computed by one user-selected acceleration-attenuation relation that is contained in EQFAULT. Based on the EQFAULT program, a peak horizontal ground acceleration from an upper bound event on the Rose Canyon fault may be on the order of 0.36 g. The computer printouts of pertinent portions of the EQFAULT program are included within Appendix C.

Historical site seismicity was evaluated with the acceleration-attenuation relation of Bozorgnia, Campbell, and Niazi (1999), and the computer program EQSEARCH (Blake, 2000b, updated to September 2019). This program performs a search of the historical earthquake records for magnitude 5.0 to 9.0 seismic events within a 100-kilometer radius, between the years 1800 through September 2019. Based on the selected acceleration-attenuation relationship, a peak horizontal ground acceleration is estimated, which may have affected the site during the specific event listed. Based on the available data and the attenuation relationship used, the estimated maximum (peak) site acceleration during the period 1800 through September 2019 was about 0.207 g. A historic earthquake epicenter map and a seismic recurrence curve are also estimated/generated from the historical data. Computer printouts of the EQSEARCH program are presented in Appendix C.

Seismic Shaking Parameters

Based on the site conditions, the following table summarizes the updated site-specific design criteria obtained from the 2019 CBC (CBSC, 2019a), Chapter 16 Structural Design, Section 1613, Earthquake Loads. The computer program "U.S. Seismic Design Maps, (https://seismicmaps.org/) provided by the Structural Engineer's Association of California and California Office of Statewide Health Planning and Development ([OSHPD], 2020) was utilized for design parameters. The short spectral response utilizes a period of 0.2 seconds.

2019 CBC SEISMIC DESIGN PARAMETERS		
PARAMETER	**VALUE**	**2019 CBC or REFERENCE**
Risk Category	II	Table 1604.5
Site Class	C	Section 1613.2.2/ASCE 7-16 (Chap. 20 p. 203-204)
Spectral Response - (0.2 sec), S_s	0.91 g	Section 1613.2.1 Figure 1613.2.1(1)
Spectral Response - (1 sec), S_1	0.336 g	Section 1613.2.1 Figure 1613.2.1(2)
Site Coefficient, F_a	1.2	Table 1613.2.3(1)
Site Coefficient, F_v	1.5	Table 1613.2.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec), S_{MS}	1.092 g	Section 1613.2.3 (Eqn 16-36)
Maximum Considered Earthquake Spectral Response Acceleration (1 sec), S_{M1}	0.503 g	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	0.728 g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.336 g	Section 1613.2.4 (Eqn 16-39)
PGA_M	0.472 g	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 DESIGN PARAMETERS	
PARAMETER	**VALUE**
Distance to Seismic Source (Rose Canyon)	11.6 mi (18.6 km)*
Upper Bound Earthquake (Rose Canyon)	M_w 7.2**
* Blake (2000a), **Cao, et al. (2003)	

Conformance to the criteria above for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur in the event of a large earthquake. The primary goal of seismic design is to protect life, not to eliminate all damage, since such design may be economically prohibitive. Cumulative effects of seismic events are not addressed in the 2019 CBC (CBSC, 2019) and regular maintenance and repair following locally significant seismic events (i.e., M_w5.5) will likely be necessary, as is the case in all of southern California.

SECONDARY SEISMIC HAZARDS

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

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

SLOPE STABILITY

Based on site conditions and planned improvements, planned 2:1 (h:v) cut and/or fill slopes are anticipated to be stable, assuming that these slopes are properly constructed and maintained over the life of the project. In order to improve surficial slope stability, if feasible, granular, cohesionless soils (i.e., clean sands), should not be placed within 10 feet from the face of any fill slope. Temporary slopes for construction (i.e., trenching, etc.) are discussed in subsequent sections of our report.

LABORATORY TESTING

Laboratory tests were performed on representative samples of site earth materials collected during our subsurface exploration in order to evaluate their physical characteristics. Test procedures used and results obtained are presented below.

Classification

Soils were visually classified with respect to the Unified Soil Classification System (U.S.C.S.) in general accordance with ASTM D 2487 and D 2488. The soil classifications of the onsite soils are provided on the Test Pit and Boring Logs in Appendix B.

Moisture-Density Relations

The field moisture content and dry density of relatively undisturbed soil samples were evaluated in the laboratory, in general accordance with ASTM D 2216 and ASTM D 2937. The results of these tests are shown on the Test Pit and Boring Logs in Appendix B.

Maximum Laboratory Standard

The laboratory maximum dry density and optimum moisture content for a representative soil type onsite was evaluated in general accordance with test method ASTM D 1557. Test results are presented in the following table.

SOIL TYPE	MAXIMUM DENSITY (PCF)	MOISTURE CONTENT (PERCENT)
Brown Clayey SAND (TP-1 @ 3½' - 6')	125.7	10.6

Expansion Index

A test was performed on representative soil samples in general accordance with ASTM D 4829. Test results and the soil's expansion potential are presented in the following table:

SAMPLE LOCATION	DESCRIPTION	EXPANSION INDEX	EXPANSION POTENTIAL
TP-1 @ 3½' - 6'	Clayey SAND	58	Medium
TP-8 @ 0' - 2'	Silty SAND	<21	Very Low

Atterberg Limits

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

SAMPLE LOCATION	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
TP-1 @ 3½' - 6'	35	12	23

Particle-Size Analysis

A particle-size evaluation was performed on a representative sample of surficial, non-bedrock soils (TP-1 @ 5') in general accordance with ASTM D 422-63. The testing was utilized to evaluate the soil classification in accordance with the Unified Soil Classification System (USCS). The results of the particle-size evaluation indicate that the sample consisted of 0.5% Gravel, 55.9% sand and 43.6% fines (i.e., silt and clay). Per the USCS, site soils are classified as a clayey sand (USCS symbol SC). The grain-size distribution curve is presented in Appendix D.

Consolidation Test

Consolidation tests were performed on selected undisturbed soil samples. Testing was performed in general accordance with ASTM Test Method D 2435. Test results are presented as in Appendix D. In general, the testing indicated that representative samples of older alluvial soils exhibited both minor (1%, or less) hydrocollapse and/or swell, following inundation. Testing also demonstrated that the compression of soil at anticipated plan fill loads (2 kips) exhibited strains ranging between approximately ½ to 2½ percent. Samples did not indicate significant elastic behavior when rebounded (unloaded).

Saturated Resistivity, pH, and Soluble Sulfates, and Chlorides

Based on our experience in the vicinity, onsite soils are likely mildly alkaline with respect to soil acidity/alkalinity, are corrosive to exposed, buried metals when saturated, present negligible (“Exposure Class S0” per ACI 318-14) sulfate exposure to concrete, and chloride levels below the action level for chloride exposure (per State of California Department of Transportation, 2003). For preliminary design purposes, reinforced concrete mix design for foundations, slab-on-grade floors, and pavements should minimally conform to “Exposure Class C1” in Table 19.3.1.1 of ACI 318-14, as concrete would likely be exposed to moisture. However, as this project requires significant import of soil to complete planned pad grades, onsite soils will likely be buried during subsequent grading. As import soils are delivered to the site, corrosive soil evaluations are recommended in order to characterize the as-built corrosive character of the soil. It should be noted that GSI does not consult in the field of corrosion engineering. The client and project architect should agree on the level of corrosion protection required for the project and seek consultation from a qualified corrosion consultant as warranted.

STORM WATER TREATMENT AND HYDROMODIFICATION MANAGEMENT

USDA Study

A review of the United States Department of Agriculture database ([USDA]; 1973, 2020) indicates that site soils located within the upper elevations of the site are classified as

Bonsall sandy loam (2 to 9, and 9-15 percent slopes). Soils within the lower, alluviated areas of the site, near the eastern property boundary (totalling about 0.5 AC) are classified as the Greenfield sandy loam (5 to 9 percent slopes). The USDA study further indicates that the Bonsall series soils are classified as belonging to Hydrologic Soil Group "D," while the minorGreenfield series is classified as belonging to Hydrologic Soil Group "A." It should be noted that based on site geology and Hydrologic soil group mapping by the USDA, Greenfield series soils appear to correlate with the younger, Quaternary-age alluvial deposits onsite (Map Symbol Qa shown on Plate 1), and appear to be underlain at very shallow depths by older alluvial soils and bedrock, which correlate with Hydrologic Soil type "D" soils.

Infiltration Feasibility

A review of USDA (1973 and 2019) indicates that the capacity of the most limiting layer to transmit water (Ksat) within the Bonsall Sandy Loam, is very low (0.00 to 0.06 inches per hour [in/hr]), while the capacity within the Greenfield Sandy Loam is relatively high (1.98 to 5.95 inches per hour [in/hr]). However, during remedial grading of the site, any soils identified as Greenfield sandy loam will be removed and recompacted as engineered fill.

The USDA indicates that Bonsall series soil units fall into Hydrologic Soil Group (HSG) "D," and any basin constructed entirely of compacted fill is also considered as belonging to HSG "D." Per Table B.2-3 of the County (2019), a design infiltration rate of 0.025 inches/hour is provided for HSG "D" soils. This design infiltration rate is generally below the recommended feasibility threshold of 0.52 inches per hour per the EPA (Clar, et al., 2004), and 0.50 inches per hour per the City (2016) for full infiltration. Furthermore, the permeability of the underlying soil/bedrock can be expected to decrease with depth, as the soil/bedrock becomes less weathered, thereby promoting the lateral migration of water in soil. Evidence of this can be seen by the perched water conditions noted across the site.

Proposed fill, and/or moisture-sensitive improvements, such as pavements, and utility trench backfill, foundations, retaining walls, and below grade building walls, would likely be adversely affected by excessive soil moisture, including offsite improvements, causing settlement and distress. Bio-basins can adversely affect the performance of the onsite and offsite structures, foundation systems by: 1) increasing soil moisture transmission rates through concrete flooring, 2) reducing the stability of slopes, and 3) increasing the potential for a loss in bearing strength of soil. Onsite mitigative grading of compressible near-surface soils for the support of structures generally involves removal and recompaction. This is anticipated to create the potential for permeability contrast, and the potential for the development of a shallow "perched" and mounded water table, which can reasonably be anticipated to migrate laterally, beneath the structure(s), or offsite onto adjacent property, causing settlement and associated distress. Based on County (2019), "partial infiltration" is considered feasible, however, based on our review and engineering analysis, we consider the site belonging to HSG D and recommend "no infiltration" BMP design, owing to the potential for associated settlement, distress, and perched groundwater, as well as the close proximity (i.e., potentially within 15 feet) of any planned basin to building foundations, retaining walls, slopes, and other settlement-sensitive improvement. Furthermore, any basin constructed entirely of compacted fill is considered as belonging to HSG D, and a “no infiltration” BMP design is warranted ([EPA,], Clar, et al., 2004). Worksheet Form D.5-1 (City, 2016) are presented in Appendix E.

Onsite Infiltration-Runoff Retention Systems

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

Should onsite infiltration-runoff retention systems (OIRRS) be planned for Best Management Practices (BMPs) or Low Impact Development (LID) principles for the project, some guidelines should be followed in the planning, design, and construction of such systems. Such facilities, if improperly designed or implemented without consideration of the geotechnical aspects of site conditions, can contribute to flooding, saturation of bearing materials beneath site improvements, slope instability, and possible concentration and contribution of pollutants into the groundwater or storm drain and/or utility trench systems.

A key factor in these systems is the infiltration rate (sometimes referred to as the percolation rate) which can be ascribed to, or determined for, the earth materials within which these systems are installed. Additionally, the infiltration rate of the designed system (which may include gravel, sand, mulch/topsoil, or other amendments, etc.) will need to be considered. The project infiltration testing is very site specific, any changes to the location of the proposed OIRRS and/or estimated size of the OIRRS, may require additional infiltration testing. Locally, relatively impermeable residual soils include the underlying bedrock, which is anticipated to have a very low vertical infiltration rate.

The following geotechnical guidelines should be considered when designing onsite infiltration-runoff retention systems:

- It is not good engineering practice to allow water to saturate soils, especially near slopes or improvements; however, the controlling agency/authority may now require this.

- Areas adjacent to, or within, the OIRRS that are subject to inundation should be properly protected against scouring, undermining, and erosion, in accordance with the recommendations of the design engineer.

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

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

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

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

- Storm drain, standpipes, and utilities that cross BMPs should be slurried with a 2-sack mix, to 5 feet outside the structure.

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

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 re-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.
- Temporary slope stability.
- Regional seismic activity.

The recommendations presented herein consider these as well as other aspects of the site. The engineering analyses performed concerning site preparation and the recommendations presented herein have been completed using the information provided and obtained during our field work.

In the event that any significant changes are made to proposed site development, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the recommendations of this report verified or modified in writing by this office. Foundation design parameters are considered preliminary until the foundation design, layout, and structural loads are provided to this office for review.

EARTHWORK CONSTRUCTION RECOMMENDATIONS

General

All earthwork should conform to the guidelines presented in the 2019 CBC (CBSC, 2019a), the requirements of the City of Vista, 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 earthwork schedule. This office should be notified in advance of any fill placement, supplemental regrading of the site, or backfilling underground utility trenches and retaining walls after rough earthwork has been completed. This includes grading for driveway approaches, driveways, and exterior hardscape.

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

Demolition/Grubbing

1. Vegetation and any miscellaneous debris should be removed from the areas of proposed grading.

2. Any existing subsurface structures uncovered during the recommended removal should be observed by GSI so that appropriate remedial recommendations can be provided.

3. Cavities or loose soils remaining after demolition and site clearance should be cleaned out and observed by the soil engineer. The cavities should be replaced with

fill materials that have been moisture conditioned to <u>at least</u> optimum moisture content and compacted to at least 90 percent of the laboratory standard.

4. Onsite septic systems (if encountered) should be removed in accordance with San Diego County Department of Environmental Health standards/guidelines.

Treatment of Existing Ground

1. Removals should consist of all surficial deposits of undocumented fill, colluvium, alluvium, near surface older alluvium, and any highly weathered bedrock (if present). Based on our site work, removals depths on the order of about 2 to potentially 9 feet (near the eastern property boundary) should be anticipated. These soils may be re-used as fill, provided that the soil is cleaned of any deleterious material and moisture conditioned, and compacted to a minimum 90 percent relative compaction per ASTM D 1557. Removals should be completed throughout the entire building/construction area.

2. In addition to removals within the building envelope, and for the mitigation of adverse soil moisture, overexcavation/undercutting of the underlying bedrock soil should be performed in order to provide for at least 5 feet of compacted fill below finish grade, or 2 feet below the bottom of deepest footing; whichever is greater. Undercutting should be completed for a minimum lateral distance of at least 5 feet beyond the building footprint. Once removals and overexcavation is completed, the fill should be cleaned of deleterious materials, moisture conditioned, and recompacted to at least 90 percent relative compaction per ASTM D 1557.

3. Subsequent to the above removals/overexcavation, the exposed bottom should be scarified to a depth of at least 6 to 8 inches, brought to <u>at least</u> optimum moisture content, and recompacted to a minimum relative compaction of 90 percent of the laboratory standard, prior to any fill placement.

4. Onsite soils may be reused as compacted fill provided that major concentrations of vegetation and miscellaneous debris are removed from the site, prior to or during fill placement.

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

Rock Hardness and Rippability/Oversize Materials

Onsite surficial soils (colluvium [topsoil], undocumented fill, alluvium, and older alluvium) can likely be excavated with light to moderate effort using heavy grading/trenching equipment. Regionally, the underlying granitic bedrock is considered rippable with heavy duty equipment (i.e., D9L), to depths (i.e., depth into rock) on the order of 10 to 15 feet. Non-rippable trenching may be encountered at shallower depths, especially with small, rubber tire backhoes. Zones of hard rock requiring blasting/line shooting, or rock breaking may not be precluded, and should be anticipated, and oversize materials will likely be encountered requiring special placement techniques.

Earthwork Balance (Shrinkage/Bulking)

The volume change of excavated materials upon compaction as engineered fill is anticipated to vary with material type and location. The overall earthwork shrinkage and bulking may be approximated by using the following parameters:

Undocumented fill . . . . 8-10% Shrinkage
Colluvium . . . . 10-12% Shrinkage
Alluvium . . . . 10-12% Shrinkage
Older Alluvium (near Surface) . . . . 8-10% Shrinkage
Bedrock (from Church, 1981)
    25% Rock/75% Earth (about 3½ to 8 feet b.e.g.) . . . . 8% Shrinkage
    50% Rock/50% Earth (about 8 to 15 feet b.e.g.) . . . . 5% Shrinkage
    75% Rock/25% Earth (about 15 to 20 feet b.e.g.) . . . . 12% Bulk
    100% Rock (> ±20 -30 feet b.e.g.) . . . . 12-33% Bulk

It should be noted that the above factors are estimates only, based on preliminary data. The colluvium and any weathered bedrock may achieve higher shrinkage if organics or clay content is higher than anticipated, if a high degree of porosity is encountered, or if compaction averages more than 92 percent of the laboratory standard (ASTM D 1557). In addition, due to extensive rodent burrowing, higher shrinkage may be encountered. Final earthwork balance factors could vary. In this regard, it is recommended that balance areas be reserved where grades could be adjusted up or down near the completion of grading in order to accommodate any yardage imbalance for the project.

Fill Suitability

Surficial onsite soils (undocumented fill, colluvium, alluvium, and older alluvium) generally appear to consist of silty to clayey sand with minor amounts of sandy clay, while bedrock is anticipated to generate silty sands and sands with brittle rock fragments. Oversize material (12-inch plus) cannot be precluded from being generated from cut excavation into the bedrock. In order to facilitate shallow onsite trenching, consideration should be given to maintaining a maximum rock fragment size of 6 inches within any future fill areas to be trenched. Materials generated from demolition of structures, such as foundation concrete, should be cleaned of any reinforcing steel and reduced to minus 8-inch size particles before incorporating into the fill. Any asphalt disposal onsite should conform to Code and be placed in street areas, or placed at depth, below the lowest utility invert elevation.

Existing fill soils generally range from very low to medium expansive. Any soil import should be evaluated by this office prior to importing in order to assure compatibility with the onsite site soils and the recommendations presented in this report. An additional discussion of import soils is presented in the following section.

Selective Grading

Foundation design for this project will be primarily based on the bearing capacity and expansive characteristics of soils underlying a given foundation system. When expansive soils underlie the site, foundation design will generally be more onerous than a design based on lessor expansive soils. As this site requires a significant amount of import to build, the type of import soil brought onsite will determine the foundation design.

For preliminary planning purposes, the following criteria should be considered for a conventional foundation design.

- 0 - 7 feet from pad grade - very low expansive soils (E.I. $<$ 21)
- 7 - 15 feet from pad grade - very low to medium expansive soils (E.I. range of 0 to 90)
- 15 feet and greater - very low to highly expansive soils (E.I. Range of 0 to 130)

Import soil should be free of organic debris, deleterious construction debris such as : concrete with rebar, plastic, wood, etc. Plain concrete with a maximum dimension of 12 or less may be placed in fill to no closer than 5 feet from pad grade. Fully cured, substantially hardened, and inelastic asphalt fragments, 6-8 inches in long dimension may be mixed with soil and placed in street areas, below the lowest utility invert elevation, or no closer than 15 feet from pad grade, or any adjacent slope face.

It is also recommended that any potential borrow site be able to produce documentation including the soils expansion potential and documentation such as a Phase I or Phase II environmental report indicating that the import soils do not contain adverse levels of soil contaminants.

Fill Placement

1. Subsequent to ground preparation, fill materials should be brought to <u>at least</u> optimum moisture content, placed in thin 6- to 8-inch lifts, and mechanically compacted to obtain a minimum relative compaction of 90 percent of the laboratory standard.

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

3. Any import materials should be observed and deemed suitable by the soils engineer <u>prior</u> to placement on the site. Foundation designs may be altered if import materials have a greater expansion value than the onsite materials encountered in this investigation.

Graded Slopes

Graded slopes, up to about 20 feet in height are planned, and should be constructed per Code and the recommendations presented herein. In order to improve surficial slope stability, any import consisting of granular, cohesionless soils (i.e., clean sands), should not be placed within 15 feet from the face of any fill slope.

Temporary Slopes

Temporary slopes for excavations greater than 4 feet, but less than 20 feet in overall height should conform to CAL-OSHA and/or OSHA requirements for Type "B" soils. Temporary slopes, up to a maximum height of $\pm$20 feet, may be excavated at a 1:1 (h:v) gradient, or flatter, provided groundwater and/or running sands are not exposed. Construction materials or soil stockpiles should not be placed within 'H' of any temporary slope where 'H' equals the height of the temporary slope. All temporary slopes should be observed by a licensed engineering geologist and/or geotechnical engineer prior to worker entry into the excavation.

Subdrainage

Based on conditions exposed during grading, subdrainage may be recommended along the inside, upper edge of the perimeter fill slope keyway (east side) for the mitigation of perched subsurface water. Additional subdrainage may be necessary where significant permeability contrasts are created within the fill (i.e., sand over clay) and will be evaluated during site grading.

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.

GSI understands that the project is in its very conceptual stages. Thus, the foundation design recommendations, included herein, are based on anticipated average and maximum static column loads of 100 and 250 kips, respectively. Maximum wall loads are anticipated to be on the order of 5 kips per lineal foot. The slabs-on-grade are anticipated to have typical light loads on the order of 50 psf. It is unknown if equipment and elevator pit areas will be included in the design. GSI does not anticipate high vibratory equipment loads on the floor slabs. GSI also does not anticipate highly sensitive electrical equipment mounted on the floor slab.

The foundation design recommendation contained in this report may be modified once actual loading conditions have been provided for GSI review. All foundations should be designed using at a minimum, the parameters and static settlements described herein. All foundations should be evaluated for seismic deformations described herein

Preliminary Conventional Foundation Design

The following foundation construction recommendations are presented as a minimum criteria from a soils engineering viewpoint, where the planned improvements are underlain by at least 7 feet of non-detrimentally expansive soils (i.e., E.I.<21 and P.I. <15). Should foundations be underlain by (detrimentally) expansive soils, as is anticipated for the ground floor portions of the structure, they will require specific design to mitigate expansive soil effects as required in Sections 1808.6.1 or 1808.6.2 of the 2019 CBC.

1. Conventional foundation systems should be designed and constructed in accordance with guidelines presented in the 2019 CBC (CBSC, 2019a).

2. Based on the anticipated foundation loads and preliminary design information provided us, building loads may be supported on continuous or isolated spread footings designed in accordance with the following recommendations.

ALLOWABLE BEARING VALUES FOR FOOTINGS		
DEPTH BELOW LOWEST ADJACENT FINISHED GRADE (INCHES)	ALLOWABLE BEARING CAPACITY FOR SPREAD FOOTINGS (MINIMUM WIDTH = 4 FEET)	ALLOWABLE BEARING CAPACITY FOR CONTINUOUS WALL FOOTINGS (MINIMUM WIDTH = 2 FEET)
24 to 30	2.5 ksf	2.0 ksf
36 to 48	3.5 ksf	3.0 ksf

The above values are for dead plus live loads and may be increased by one-third for short-term wind or seismic loads. Where column or wall spacings are less than twice the width of the footing, some reduction in bearing capacity may be necessary to compensate for the effects of footings with shared bearing soils. GSI should review the foundation plans and overlying building load patterns and evaluate this potential with the structural consultant. Reinforcement should be designed in accordance with local codes and structural considerations.

The recommended allowable bearing capacity provided herein is generally based on maximum static total and differential settlements of up to 2 inches and 1 inch, respectively. Differential settlements are over a distance of 50 lateral feet or between heaviest and lightest foundation loads. Actual settlement can be estimated on the basis that settlement is roughly proportional to the net contact bearing pressure on compacted fill, or formation. The majority of the settlement should occur during construction as building loads are applied. Since settlement is a function of footing size and contact bearing pressure, some static differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists. However, for most cases, differential settlements are considered unlikely to exceed 1¼ inches in 50 feet (angular distortion = 1/480). With increased footing depth/width ratios, differential settlement should be less. The anticipated total vertical deformation (post-earthquake) for the design seismic event may be on the order of ±1 inch with a potential seismic differential settlement of approximately ¼ inch to ¾ inch over 50 feet horizontally (i.e., angular distortion approximately 1/800) under the basement/garage structure. Other settlement-sensitive improvements (i.e., underground utilities, pavements, flatwork) are susceptible to seismic settlement outside the footprint of the basement/garage structure.

3. Foundation embedment depth excludes concrete slabs-on-grade, and/or slab underlayment. Foundations should bear entirely on a minimum 5-foot thick layer of approved engineered fill overlying suitable formation (i.e., dense older alluvium, or bedrock). All isolated pad footings should be tied to the perimeter foundation in at least one direction to reduce the potential for lateral drift.

4. For foundations deriving passive resistance from engineered fill, prepared in accordance with the recommendations provided in this report, a pressure of 250 pcf may be used if the footing face is embedded entirely in engineered fill, and the embedment is 24 to 48 inches.

5. The upper 6 inches of passive pressure should be neglected if not confined by slabs or pavement.

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

7. When combining passive pressure and frictional resistance, the passive pressure

component should be reduced by one-third.

8. Although not anticipated, given our understanding of the proposed development, all footing setbacks from slopes should comply with Figure 1808.7.1 of the 2019 CBC. GSI recommends a minimum horizontal setback distance of 7 feet as measured from the bottom, outboard edge of the footing to the slope face. Foundations should also extend below a 1:1 (h:v) projection up from the bottom outside edge of remedial grading excavations.

9. Footings for structures adjacent to retaining/privacy 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.

10. Footings constructed below a 1:1 projection from adjacent property lines should be designed for any applicable surcharge.

PRELIMINARY CONVENTIONAL FOUNDATION CONSTRUCTION RECOMMENDATIONS

Current laboratory testing indicates that some onsite soils meet the criteria of detrimentally expansive soils as defined in Section 1803.5.3 of the 2019 CBC. The following foundation construction recommendations are presented as a minimum criteria from a soils engineering viewpoint, where the planned improvements are underlain by at least 7 feet, and perhaps more (as determined during grading), of non-detrimentally expansive soils (i.e., E.I.<21 and P.I. <15). Should foundations be underlain by expansive soils, such as is anticipated for the ground floor portions of the structure, they will require specific design to mitigate expansive soil effects as required in Sections 1808.6.1 or 1808.6.2 of the 2019 CBC (CBSC, 2019a).

1. Exterior and interior footings should be founded into approved engineered fill, as indicated in the previous "Preliminary Foundation Design" section of this report. Reinforcement should be designed in accordance with local codes and structural considerations, and per the project structural engineer. At a minimum, 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 footing should be provided by the projects structural engineer.

2. All interior and exterior column footings, and perimeter wall footings, should be tied together via grade beams in at least two directions. The grade beam should be at least 24 inches square in cross section, and the base of the reinforced grade beam should be at the same elevation as the adjoining footings. At a minimum, grade beams should be minimally reinforced with four No. 4 reinforcing bars, two placed

near the top and two placed near the bottom of the footing. Reinforcement should be designed in accordance with local codes and structural considerations, and per the projects structural engineer.

3. A grade beam, reinforced as previously recommended and at least 24 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. Non-vehicular slab-on-grade floors should have a minimum thickness of 5 inches with steel reinforcement consisting of No. 3 reinforcing bars positioned at 18 inches on center in two perpendicular directions (i.e., long axis and short axis). All slab reinforcement should be supported to ensure proper mid-slab height positioning during placement of the concrete. "Hooking" of reinforcement is not an acceptable method of positioning. The actual thickness and steel reinforcement for concrete slab-on-grade floors should be determined by the project structural engineer, based on the anticipated loading conditions and building use. However, the slab thickness and steel reinforcement recommendations, contained herein, are considered minimum guidelines.

5. Slab subgrade pre-soaking may be required for the onsite soil conditions, should medium expansive soils be present near finish grade. If this is the case, the subgrade soils should be moisture conditioned to 2 percent over optimum moisture content (or 1.2 x optimum moisture, whichever is greater). This will need to be verified within 2 hours of the placement of underlayment sand and gravel and the vapor retarder. However, for very low to low expansive soils, the developer should consider moisture conditioning slab subgrade materials to at least optimum moisture content to a minimum depth of 12 inches, within 72 hours of the placement of underlayment sand and gravel and the vapor retarder.

6. Soils generated from footing excavations to be used onsite should be compacted to a minimum relative compaction of 90 percent of the laboratory standard (ASTM D 1557), whether the soils are to be placed inside the foundation perimeter or in other areas of the site. This material must not alter positive drainage patterns that direct drainage away from the structural areas and toward the street.

7. Reinforced concrete mix design should conform to recommendations contained in the “Soil Moisture Transmission Considerations” section of this report, and should consider the site’s proximity to the Pacific Ocean corrosive environment, and the elevated chloride concentrations found in some of the onsite soils.

8. Specific slab subgrade pre-soaking is recommended for these soil conditions. Prior to the placement of underlayment sand and vapor retarder, GSI recommends that the slab subgrade materials be moisture conditioned to at least optimum moisture content to a minimum depth of 12 inches. Slab subgrade pre-soaking should be evaluated by the geotechnical consultant within 72 hours of the placement of the

underlayment sand and vapor retarder.

9. Soils generated from footing excavations to be used onsite should be compacted to a minimum relative compaction of 90 percent of the laboratory standard (ASTM D 1557), whether the soils are to be placed inside the foundation perimeter or in the yard/right-of-way areas. This material must not alter positive drainage patterns that direct drainage away from the structural areas and toward the street.

10. Reinforced concrete mix design should conform to "Exposure Class S0, W0, and C1" in Table 19.3.2.1 of ACI 318R.

Foundations for Expansive Soils

Should foundations be underlain by expansive soils (as defined in Section 1803.5.3 of the CBC [CBSC, 2019a]) at depths of less than 7 feet, foundations will require specific design to mitigate expansive soil effects as required in Sections 1808.6.1 or 1808.6.2 of the 2019 CBC (CBSC, 2019a). Foundation design would typically include post tension slab foundations, structural mat slabs, and/or slabs designed using WRI methodology, and may be provided upon request. It is recommended that site conditions are periodically reviewed during grading operations, including expansive soil evaluations, to assess the impact of as-built fills on foundation design, including updated foundation design recommendations, as necessary.

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, 2020). These recommendations may be exceeded or supplemented by a water "proofing" specialist, project architect, or structural consultant. Thus, the client will need to evaluate the following in light of a cost vs. benefit analysis (owner expectations and repairs/replacement), along with disclosure to all interested/affected parties. It should also be noted that vapor transmission will occur in new slab-on-grade floors as a result of chemical reactions taking place within the curing concrete. Vapor transmission through concrete floor slabs as a result of concrete curing has the potential to adversely affect sensitive floor coverings depending on the thickness of the concrete floor slab and the duration of time between the placement of concrete, and the floor covering. It is possible that a slab moisture sealant may be needed prior to the placement of sensitive floor coverings if a thick slab-on-grade floor is used and the time frame between concrete and floor covering placement is relatively short.

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

- Concrete slabs should be increased in thickness.

- Concrete slab underlayment should consist of a 15-mil vapor retarder, or equivalent, with all laps sealed per the 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) shall be installed per the recommendations of the manufacturer, including <u>all</u> penetrations (i.e., pipe, ducting, rebar, etc.).

- Concrete slabs, including the garage areas, shall be underlain by 2 inches of clean, washed sand (SE $\geq$ 30) above a 15-mil vapor retarder (ASTM E-1745 - Class A, per Engineering Bulletin 119 [Kanare, 2005]) installed per the recommendations of the manufacturer, including all penetrations (i.e., pipe, ducting, rebar, etc.). The manufacturer shall provide instructions for lap sealing, including minimum width of lap, method of sealing, and either supply or specify suitable products for lap sealing (ASTM E 1745), and per Code.

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

- The vapor retarder shall be underlain by 2 inches of sand (SE $\geq$ 30) placed directly on the prepared, moisture conditioned, subgrade and should be sealed to provide a continuous retarder under the entire slab, as discussed above. As discussed previously, GSI indicated this layer of import sand may be eliminated below the vapor retarder, <u>if</u> laboratory testing indicates that the slab subgrade soil have a sand equivalent (SE) of 30 or greater.

- Concrete should have a maximum water/cement ratio of 0.50. This does not supercede Table 19.3.2.1 of ACI (2014) for corrosion or other corrosive requirements. Additional concrete mix design recommendations should be provided by the structural consultant and/or waterproofing specialist. Concrete finishing and workablity should be addressed by the structural consultant and a waterproofing specialist.

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

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

- Additional recommendations regarding water or vapor transmission should be provided by the architect/structural engineer/slab or foundation designer and should be consistent with the specified floor coverings indicated by the architect.

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

WALL DESIGN PARAMETERS

General

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

Conventional Retaining Walls

The design parameters provided below assume that either very low expansive soils (typically Class 2 permeable filter material or Class 3 aggregate base) or native onsite materials with an expansion index up to 20 are used to backfill any retaining wall. Please note that the onsite soils likely meet this criteria. 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 and/or seismic loads.

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

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

Backfill Soil Density - Soil densities ranging between 110 pcf and 115 pcf may be used in the design of retaining wall foundations. This assumes an average engineered fill compaction of at least 90 percent of the laboratory standard (ASTM D 1557).

Any retaining wall footings near the perimeter of the site will likely need to be deepened into unweathered Santiago Formation for adequate vertical and lateral bearing support. All retaining wall footing setbacks from slopes should comply with Figure 1808.7.1 of the 2019 CBC. GSI recommends a minimum horizontal setback distance of 7 feet as measured from the bottom, outboard edge of the footing to the slope face.

Restrained Walls

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

Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls up to 10 feet high. Design parameters for walls less than 3 feet in height may be superceded by City regional standard design. Active earth pressure may be used for retaining wall design, provided the top of the wall is not restrained from minor deflections. An equivalent fluid pressure approach may be used to compute the horizontal pressure against the wall. Appropriate fluid unit weights are given below for specific slope gradients of the retained material. These do not include other superimposed loading conditions due to traffic, structures, seismic events or adverse geologic conditions. When wall configurations are finalized, the appropriate loading conditions for superimposed loads can be provided upon request.

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

SURFACE SLOPE OF RETAINED MATERIAL (HORIZONTAL:VERTICAL)	EQUIVALENT FLUID WEIGHT P.C.F. (SELECT BACKFILL)[2]	EQUIVALENT FLUID WEIGHT P.C.F. (NATIVE BACKFILL)[3]
Level[1]	38	50
2 to 1	55	60

[1] Level backfill behind a retaining wall is defined as compacted earth materials, properly drained, without a slope for a distance of 2H behind the wall, where H is the height of the wall.
[2] SE $\geq$ 30, P.I. < 15, E.I. < 21, and $\leq$ 10% passing No. 200 sieve.
[3] E.I. = 0 to 50, SE $\geq$ 30, P.I. < 15, E.I. < 21, and $\leq$ 15% passing No. 200 sieve.

Seismic Surcharge

For engineered retaining walls with more than 6 feet of retained materials, as measured vertically from the bottom of the wall footing at the heel to daylight , GSI recommends that the walls be evaluated for a seismic surcharge (in general accordance with 2019 CBC requirements). The site walls in this category should maintain an overturning Factor-of-Safety (FOS) of approximately 1.25 when the seismic surcharge (increment), is applied. For restrained walls, the seismic surcharge should be applied as a uniform surcharge load from the bottom of the footing (excluding shear keys) to the top of the backfill at the heel of the wall footing. This seismic surcharge pressure (seismic increment) may be taken as 15H where "H" for retained walls is the dimension previously noted as the height of the backfill to the bottom of the footing. The resultant force should be applied at a distance 0.6 H up from the bottom of the footing. For the evaluation of the seismic surcharge, the bearing pressure may exceed the static value by one-third, considering the transient nature of this surcharge. For cantilevered walls, the pressure should be applied as an inverted triangular distribution using 15H. For restrained walls, the pressure should be applied as a rectangular distribution. Please note this is for local wall stability only.

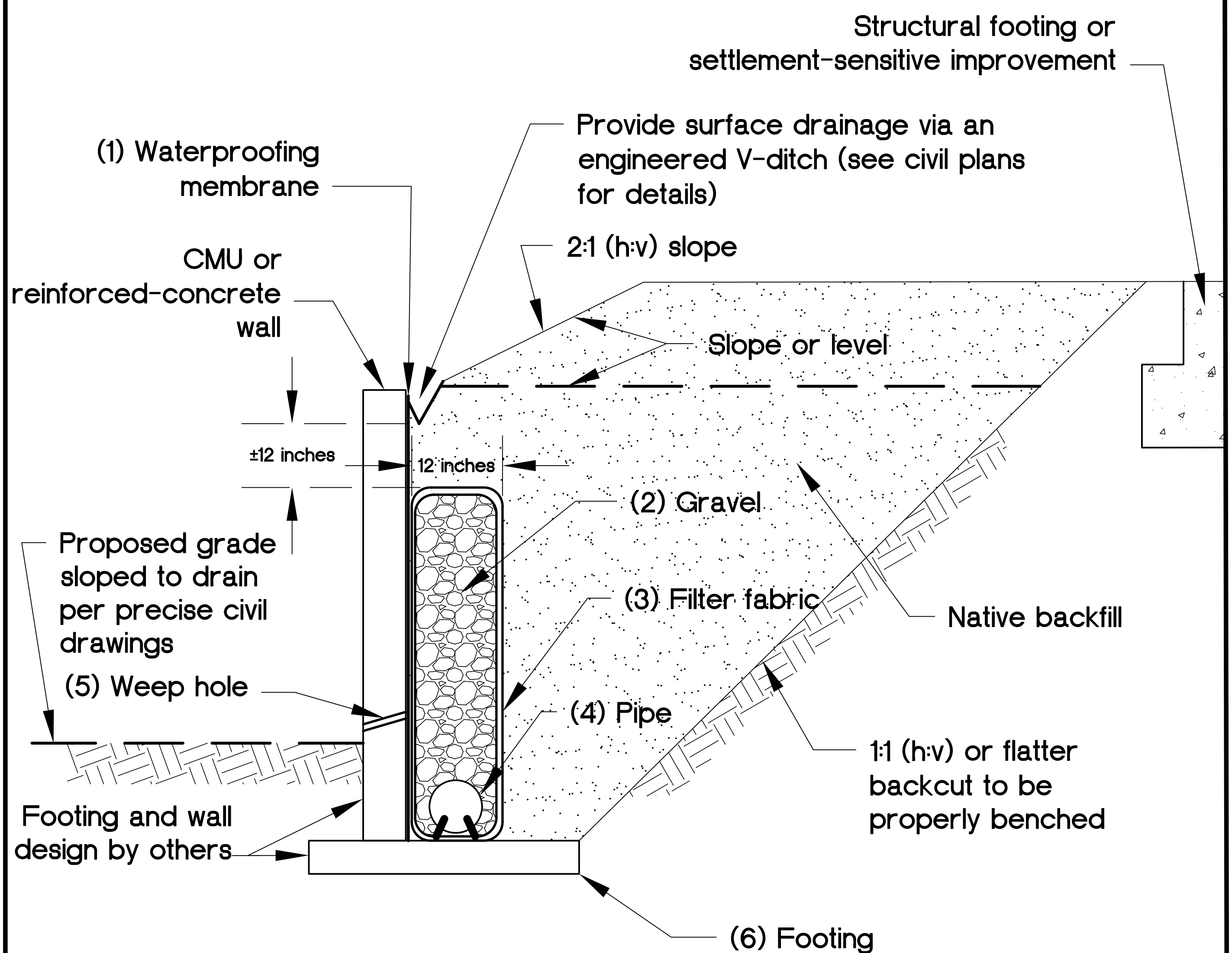
The 15H is derived from a Mononobe-Okabe solution for both restrained cantilever walls. This accounts for the increased lateral pressure due to shakedown or movement of the sand fill soil in the zone of influence from the wall or roughly a 45° - ϕ/2 plane away from the back of the wall. The 15H seismic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

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

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

Retaining Wall Backfill and Drainage

Positive drainage must be provided behind all retaining walls in the form of gravel wrapped in geofabric and outlets. A backdrain system is considered necessary for retaining walls that are 2 feet or greater in height. Details 1, 2, and 3, present the back drainage options discussed below. Backdrains should consist of a 4-inch diameter perforated PVC or ABS pipe encased in either Class 2 permeable filter material or ¾-inch to 1½-inch gravel wrapped in approved filter fabric (Mirafi 140 or equivalent). For low expansive backfill, the filter material should extend a minimum of 1 horizontal foot behind the base of the walls and upward at least 1 foot. For native backfill that has up to medium expansion potential, continuous Class 2 permeable drain materials should be used behind the wall. This material should be continuous (i.e., full height) behind the wall, and it should be constructed in accordance with the enclosed Detail 1 (Typical Retaining Wall Backfill and Drainage Detail). For limited access and confined areas, (panel) drainage behind the wall may be constructed in accordance with Detail 2 (Retaining Wall Backfill and Subdrain Detail Geotextile Drain). Materials with an E.I. potential of greater than 50 should not be used as



(1) Waterproofing membrane.

(2) Gravel: Clean, crushed, $^3/_4$ to $1^1/_2$ inch.

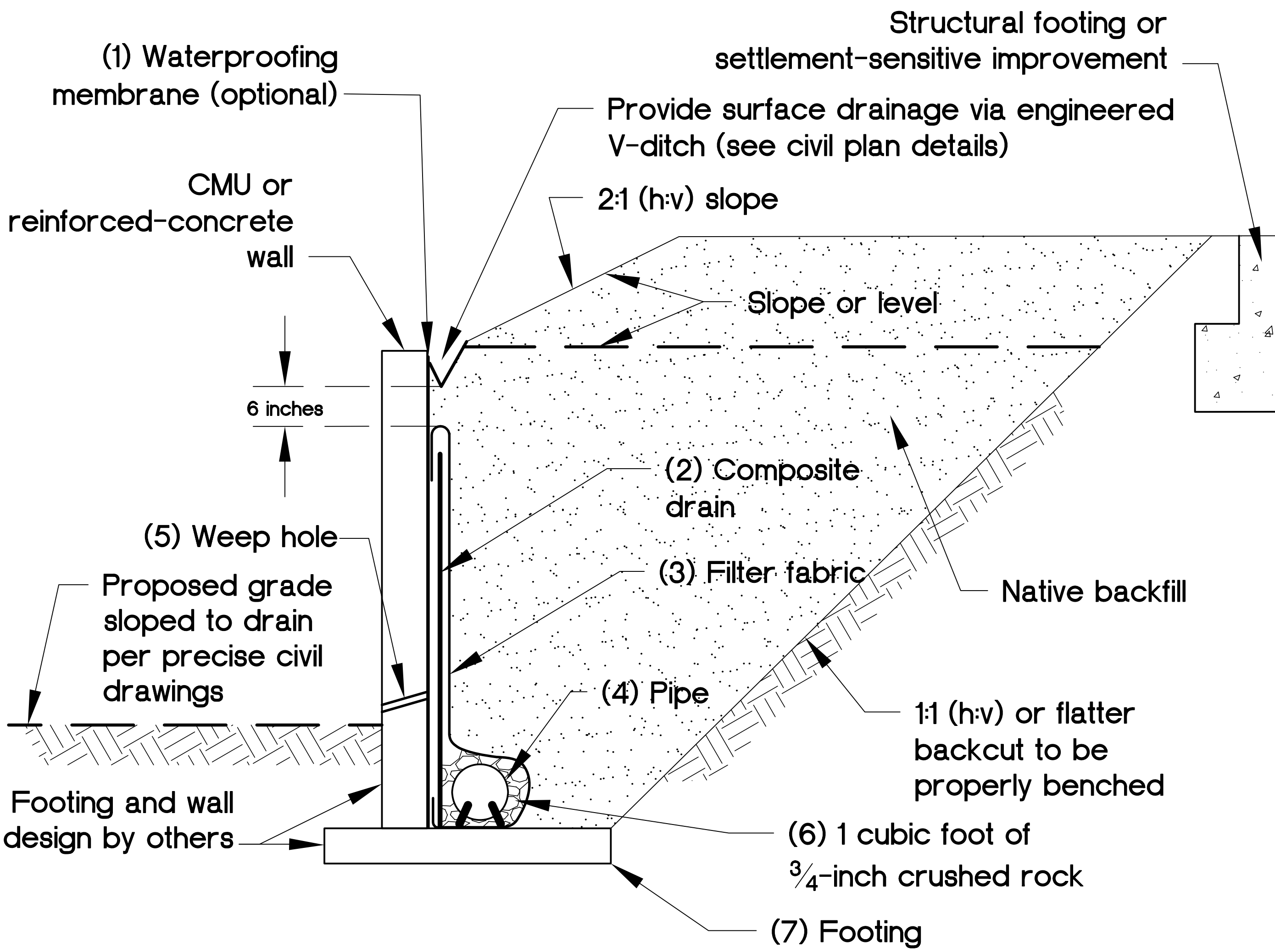
(3) Filter fabric: Mirafi 140N or approved equivalent.

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

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

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

GeoSoils, Inc.	RETAINING WALL DETAIL — ALTERNATIVE A	Detail 1



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

(2) Drain: Miradrain 6000 or J-drain 200 or equivalent for non-waterproofed walls; Miradrain 6200 or J-drain 200 or equivalent for waterproofed walls (all perforations down).

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

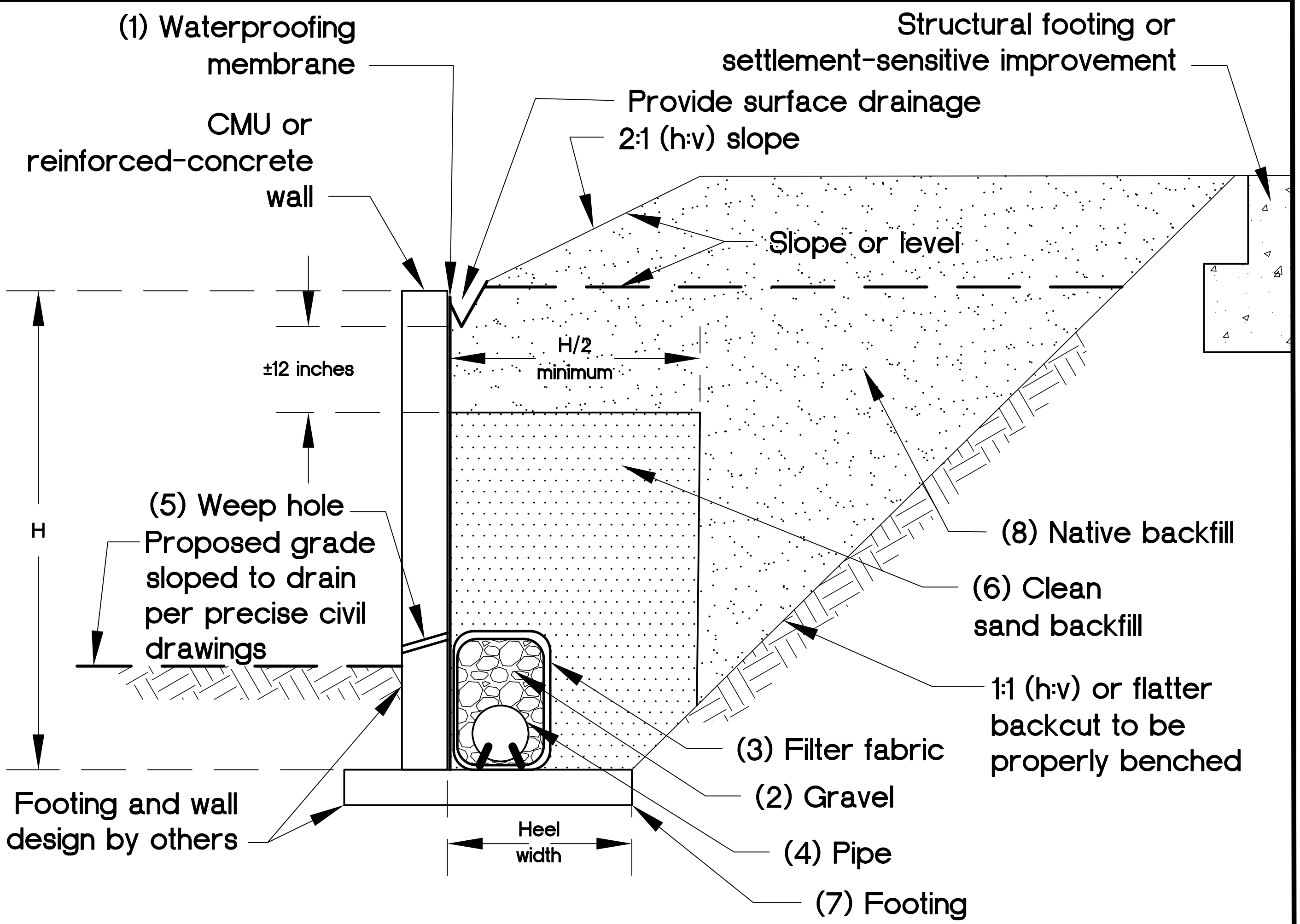
(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

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

(6) Gravel: Clean, crushed, 3/4 to 1½ inch.

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

GeoSoils, Inc.	RETAINING WALL DETAIL — ALTERNATIVE B	Detail 2



(1) Waterproofing membrane: Liquid boot or approved masticequivalent.

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

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

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

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

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

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

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

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

backfill for retaining walls. For more onerous expansive situations, backfill and drainage behind the retaining wall should conform with Detail 3 (Retaining Wall And Subdrain Detail Clean Sand Backfill). Drain outlets should consist of a 4-inch diameter solid PVC or ABS pipe spaced no greater than ±100 feet apart, with a minimum of two outlets, one on each end. The use of weep holes, only, in walls higher than 2 feet, is not recommended. The surface of the backfill should be sealed by pavement or the top 18 inches compacted with native soil (E.I. $\leq$50). Proper surface drainage should also be provided. For additional mitigation, consideration should be given to applying a water-proof membrane to the back of all retaining structures. The use of a waterstop should be considered for all concrete and masonry joints.

Wall/Retaining Wall Footing Transitions

Site walls are anticipated to be founded on footings designed in accordance with the recommendations in this report. Although not anticipated, should wall footings transition from cut to fill, the civil designer may specify either:

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

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

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

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

DRIVEWAY/PARKING, FLATWORK, AND OTHER IMPROVEMENTS

The effects of expansive soils are cumulative, and typically occur over the lifetime of any improvements. On relatively level areas, when the soils are allowed to dry, the dessication and swelling process tends to cause heaving and distress to flatwork and other improvements. The resulting potential for distress to improvements may be reduced, but not totally eliminated. To that end, it is important that the owner be aware of this long-term potential for distress. To reduce the likelihood of distress, the following recommendations are presented for all exterior flatwork:

1. The subgrade area for concrete slabs should be compacted to achieve a minimum 90 percent relative compaction (sidewalks, patios), and 95 percent relative compaction (traffic pavements), and then be presoaked to 120 percent of the soils' optimum moisture content, to a depth of 18 inches below subgrade elevation. If very low expansive soils are present, only optimum moisture content, or greater, is required and specific presoaking is not warranted. The moisture content of the subgrade should be proof tested within 72 hours prior to pouring concrete.Concrete slabs should be cast over a non-yielding surface, consisting of a 4-inch layer of crushed rock, gravel, or clean sand, that should be compacted and level prior to pouring concrete. If very low expansive soils are present, the rock or gravel or sand may be deleted. The layer or subgrade should be wet-down completely prior to pouring concrete, to minimize loss of concrete moisture to the surrounding earth materials.

2. Exterior slabs (non-vehicular sidewalks, patios, etc.) should be a minimum of 4 inches thick.

3. Driveway and parking area slabs and approaches should be at least 6 inches thick. A thickened edge (12 inches) should also be considered adjacent to all landscape areas, to help impede infiltration of landscape water under the slab(s). All pavement construction should minimally be performed in general accordance with industry standards and properly transitioned.

4. Pavements using asphaltic concrete over aggregate base construction may be designed per City of Vista standards. On a preliminary basis, driveways constructed on good to excellent subgrades may be constructed as 4 inches asphaltic concrete over 8 inches aggregate base, while parking areas may be constructed as 3 inches of asphaltic concrete over 6 inches aggregate base. Final pavement sections will be based on an evaluation of as-built subgrades upon the completion of site grading.

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

6. In order to reduce the potential for unsightly cracks, slabs should be reinforced at mid-height with a minimum of No. 3 bars placed at 18 inches on center, in each direction. If subgrade soils within the top 7 feet from finish grade are very low expansive soils (i.e., E.I. $\leq$20), then 6x6-W1.4xW1.4 welded-wire mesh may be substituted for the rebar, provided the reinforcement is placed on chairs, at slab mid-height. The exterior slabs should be scored or saw cut, ½ to ⅜ inches deep, often enough so that no section is greater than 10 feet by 10 feet. For sidewalks or narrow slabs, control joints should be provided at intervals of every 6 feet. The slabs should be separated from the foundations and sidewalks with expansion joint filler material.

7. No traffic should be allowed upon the newly poured concrete slabs until they have been properly cured to within 75 percent of design strength. Concrete compression strength should be a minimum of 2,500 psi for sidewalks and patios, and a minimum 3,250 psi for traffic pavements.

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

9. Planters and walls should not be tied to the structure.

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

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

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

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

14. Air conditioning (A/C) units should be supported by slabs that are incorporated into the building foundation or constructed on a rigid slab with flexible couplings for plumbing and electrical lines. A/C waste water lines should be drained to a suitable non-erosive outlet.

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

DEVELOPMENT CRITERIA

Slope Maintenance and Planting

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

Drainage

Adequate surface drainage is a very important factor in reducing the likelihood of adverse performance of foundations, hardscape, and slopes. Surface drainage should be sufficient to mitigate ponding of water anywhere on the property, and especially near structures and tops of slopes. Surface drainage should be carefully taken into consideration during fine grading, landscaping, and building construction. Therefore, care should be taken that future landscaping or construction activities do not create adverse drainage conditions. Positive site drainage within the property should be provided and maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and tops of slopes, and not allowed to pond and/or seep into the ground. In general, site drainage should conform to Section 1804.3 of the 2019 CBC. Consideration should be given to avoiding construction of planters adjacent to structures (buildings, pools, spas, etc.). Building pad drainage should be directed toward the street or other approved area(s). Although not a geotechnical requirement, roof gutters, down spouts, or other appropriate means may be utilized to control roof drainage. Down spouts, or drainage devices should outlet a minimum of 5 feet from structures or into a subsurface drainage system. Areas of seepage may develop due to irrigation or heavy rainfall, and should be anticipated. Minimizing irrigation will lessen this potential. If areas of seepage develop, recommendations for minimizing this effect could be provided upon request.

Erosion Control

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

Landscape Maintenance

Only the amount of irrigation necessary to sustain plant life should be provided. Over-watering the landscape areas will adversely affect proposed site improvements. We would recommend that any proposed open-bottom planters adjacent to proposed structures be eliminated for a minimum distance of 10 feet. As an alternative, closed-bottom type planters could be utilized. An outlet placed in the bottom of the planter, could be installed to direct drainage away from structures or any exterior concrete flatwork. If planters are constructed adjacent to structures, the sides and bottom of the planter should be provided with a moisture retarder to prevent penetration of irrigation water into the subgrade. Provisions should be made to drain the excess irrigation water from the planters without saturating the subgrade below or adjacent to the planters. Graded slope areas should be planted with drought resistant vegetation. Consideration should be given to the type of vegetation chosen and their potential effect upon surface improvements (i.e., some trees will have an effect on concrete flatwork with their extensive root systems). From a geotechnical standpoint leaching is not recommended for establishing landscaping. If the surface soils are processed for the purpose of adding amendments, they should be recompacted to 90 percent minimum relative compaction.

Gutters and Downspouts

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

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

Site Improvements

If in the future, any additional improvements (e.g., pools, spas, etc.) are planned for the site, recommendations concerning the geological or geotechnical aspects of design and construction of said improvements could be provided upon request. Pools and/or spas should not be constructed without specific design and construction recommendations from GSI, and this construction recommendation should be provided to all interested/affected parties. This office should be notified in advance of any fill placement, grading of the site, or trench backfilling after rough grading has been completed. This includes any grading, utility trench and retaining wall backfills, flatwork, etc.

Tile Flooring

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

Additional Grading

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

Footing Trench Excavation

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

Trenching/Temporary Construction Backcuts

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

Utility Trench Backfill

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

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

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

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

SUMMARY OF RECOMMENDATIONS REGARDING GEOTECHNICAL OBSERVATION AND TESTING

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

- During grading/recertification.

- During excavation.

- During placement of subdrains or other subdrainage devices, prior to placing fill and/or backfill.
- After excavation of building footings, retaining wall footings, and free standing walls footings, prior to the placement of reinforcing steel or concrete.

- Prior to pouring any slabs or flatwork, after presoaking/presaturation of building pads and other flatwork subgrade, before the placement of concrete, reinforcing steel, capillary break (i.e., sand, pea-gravel, etc.), or vapor retarders (i.e., visqueen, etc.).

- During retaining wall subdrain installation, prior to backfill placement.

- During placement of backfill for area drain, interior plumbing, utility line trenches, and retaining wall backfill.

- During slope construction/repair.

- When any unusual soil conditions are encountered during any construction operations, subsequent to the issuance of this report.

- When any homeowner improvements, such as flatwork, spas, pools, walls, etc., are constructed, prior to construction.

- A report of geotechnical observation and testing should be provided at the conclusion of each of the above stages, in order to provide concise and clear documentation of site work, and/or to comply with code requirements.

OTHER DESIGN PROFESSIONALS/CONSULTANTS

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

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

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

PLAN REVIEW

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

LIMITATIONS

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

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

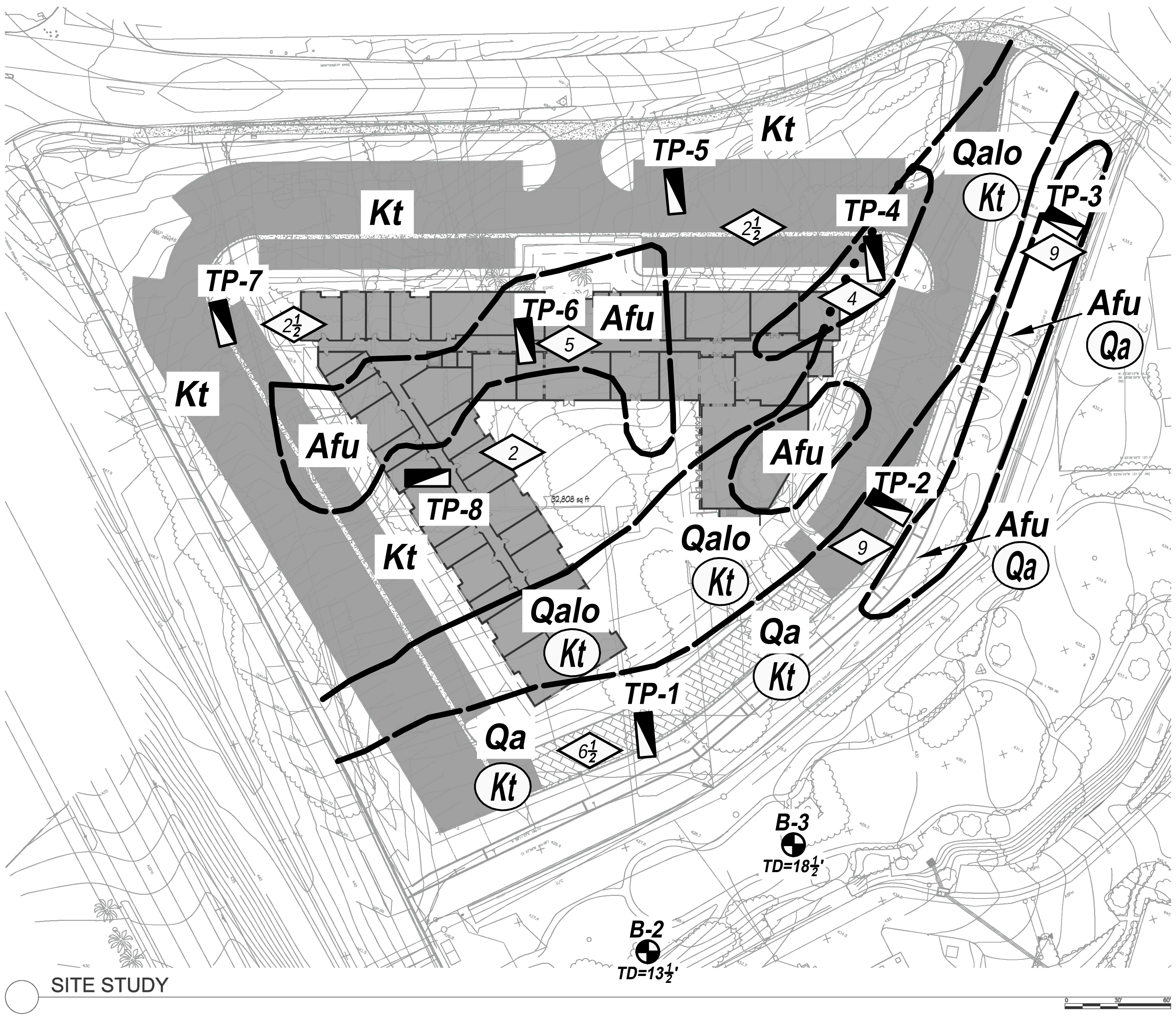
BASE MAP FROM:

KPI
KNITTER PARTNERS INTERNATIONAL, INC.
architecture & planning
17752 MITCHELL NORTH, SUITE C
IRVINE, CALIFORNIA 92614-6802
949.752.1177 www.knitter.com

NO. DATE REVISION

York Independent Living

York Independent Living
1822 - 1864 York Drive
San Diego County, California



GSI LEGEND

Afu — ARTIFICIAL FILL – UNDOCUMENTED

Qa — QUATERNARY ALLUVIUM, CIRCLED WHERE BURIED

Qalo — QUATERNARY OLDER ALLUVIUM, CIRCLED WHERE BURIED

Kt — UNDIFFERENTIATED CREATCEOUS – AGE GRANITIC BEDROCK, CIRCLED WHERE BURIED

TP-8 — APPROXIMATE LOCATION OF EXPLORATORY TEST PIT (THIS STUDY)

B-3 TD=18½' — APPROXIMATE LOCATION OF EXPLORATORY BORING WITH TOTAL DEPTH IN FEET (GSI, 1995)

9 — APPROXIMATE REMOVAL DEPTH (FEET) AT THIS APPROXIMATE SPOT LOCATION

N

GRAPHIC SCALE

80 0 40 80 160

1" = 80'

TABULATION

		FIRST FLOOR	SECOND FLOOR	THIRD FLOOR	FOURTH FLOOR	TOTAL
STUDIO	±381 sq ft	13 UNITS	34 UNITS	35 UNITS	35 UNITS	117 UNITS
ONE BEDROOM	±605 sq ft	9 UNITS	16 UNITS	16 UNITS	16 UNITS	57 UNITS
TWO BEDROOM	±940 sq ft	2 UNITS	2 UNITS	2 UNITS	2 UNITS	8 UNITS
		24 UNITS	52 UNITS	53 UNITS	53 UNITS	182 UNITS
BUILDING AREA		±32,808 sq ft	±32,464 sq ft	±32,357 sq ft	±32,041 sq ft	±129,670 sq ft

PROJECT INFO

PROJECT NUMBER: 20013

PROJECT MANAGER: KPI

DRAWN BY: SAS

SHEET ISSUE DATE: 3/10/20

SHEET TITLE

SITE STUDY

SHEET NUMBER

A-0.01

ALL LOCATIONS ARE APPROXIMATE

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

GeoSoils, Inc.

GEOTECHNICAL MAP

Plate 1

W.O. 7746-A-SC	DATE: 03/20	SCALE: 1" = 80'

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

American Concrete Institute, 2014a, Building code requirements for structural concrete (ACI 318-14), and commentary (ACI 318R-14): reported by ACI Committee 318, dated September.

_____, 2014b, Building code requirements for concrete thin shells (ACI 318.2-14), and commentary (ACI 318.2R-14), dated September.

_____, 2004, Guide for concrete floor and slab construction: reported by ACI Committee 302; Designation ACI 302.1R-04, dated March 23.

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

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

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

American Society of Civil Engineers, 2018a, Supplement 1 to Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE/SEI 7-16), first printing, dated December 13.

_____, 2018b, Errata for Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE/SEI 7-16), by ASCE, dated July 9.

_____, 2017, Minimum design loads and associated criteria and other structures, ASCE Standard ASCE/SEI 7-16, published online June 19.

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

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

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

California Building Standards Commission, 2019a, California Building Code, California Code of Regulations, Title 24, Part 2, Volume 2 of 2, based on the 2018 International Building Code, effective January 1, 2020.

_____, 2019b, California Building Code, California Code of Regulations, Title 24, Part 2, Volume 1 of 2, Based on the 2018 International Building Code, effective January 1, 2020.

_____, 2016c, California green building standard code of regulations, Title 24, Part 11, ISBN 978-1-60983-462-3.

California Geological Survey, 2018, Earthquake Fault Zones, a guide for government agencies, property owners/developers, and geoscience practitioners for assessing fault rupture hazards in California, Special Publication 42, revised.

California Office of Statewide Health Planning and Development (OSHPD), 2020, Seismic design maps, https://seismicmaps.org/.

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

City of Carlsbad, 2016, Carlsbad storm water design manual (BMP design manual) for permanent site design with respect to storm water treatment and hydromodification management, dated February 16.

_____, 1993a, Standards for design and construction of public works improvements in the City of Carlsbad.

_____, 1993b, Technical guidelines for geotechnical reports, dated December, 1992

_____, 1992, Geotechnical hazards analysis and mapping study, dated November.

Church, H.K., 1981, Excavation handbook, 1,024 pp., McGraw-Hill.

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

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

GeoSoils, Inc., 1995, Preliminary geotechnical study, Devon Park Nursing Home - APN 184-040-45 & 46, Vista, County of San Diego, California, W.O. 1813-A-SC, dated April 25.

Knitter Partners International, Inc., Architecture & Planning, 2020, Site study and floor plans, Sheets A-0.01 through A-1.04, Project No. 20013, dated March 10.

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

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

Kennedy, M.P., and Tan, SS., 2007, Geologic map of the Oceanside 30' by 60' quadrangle, California, regional geologic map series, scale 1:100,000, California Geologic Survey Map No. 2.

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

Romanoff, M., 1957, Underground corrosion, originally issued April 1.

San Diego County, 2019, County of San Diego BMP design manual, for permanent site design, storm water treatment and hydromodification management, storm water requirements for development applications, update to February 2016 manual, dated January 1.

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

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

State of California, 2020, Civil Code, Sections 895 et seq.

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

Structural Engineers Association of California and California Office of Statewide Health Planning and Development, 2019, Seismic design maps, https://seismicmaps.org/

Tan, S.S., and Giffen, D.G., 1995, Landslide hazards in the northern part of the San Diego Metropolitan area, San Diego County, California, Landslide hazard identification map no. 35, Plate 35B, Department of Conservation, Division of Mines and Geology, DMG Open File Report 95-04.

United States Department of Agriculture, National Resources Conservation Service, 2020, Custom soils report for San Diego County area, York Drive, dated February.

United States Department of Agriculture, 1973, Soil survey, San Diego area, California, Part I and Part II.

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

Vista, City of, 2016, City of Vista BMP design manual for permanent site design, stormwater treatment and hydromodification management, with appendicies, dated February.

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

APPENDIX B

TEST PIT LOGS
AND
BORING LOGS (GSI, 1995)

UNIFIED SOIL CLASSIFICATION SYSTEM

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

CONSISTENCY OR RELATIVE DENSITY

CRITERIA

Standard Penetration Test

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

Standard Penetration Test

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

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

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

MOISTURE CONDITIONS

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

MATERIAL QUANTITY

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

OTHER SYMBOLS

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

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

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

GeoSoils, Inc.

W.O. 7746-A-SC
Balbas
York Drive
Logged By: RGC
12-5-19

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-1	429.5	0-2	SM	Ring @ 1.5	11.5	109.6	**ALLUVIUM:** SILTY SAND with CLAY, dark brown, wet, loose; very pourous, few roots. @1' becomes slightly moist, some seepage around some of the longer roots.
		2-3½	CL				**OLDER ALLUVIUM:** SANDY CLAY, dark brown, moist, stiff.
		3½-6½	SC	Bulk, 3½-6½ Ring@ 5	9.1	125.7	CLAYEY SAND, brown, moist, medium dense to dense.
		6½-9	SM	Bulk @ 7-8			**BEDROCK:** GRANITIC ROCK breaking to SILTY SAND upon excavation, brown (mixed color), moist, dense; 1' weathered zone of sandy clay at alluvial contact.
							Total Depth = 9' Water Seepage Encountered @ 1'-2' Backfilled 12-5-2019

GeoSoils, Inc.

W.O. 7746-A-SC
Balbas
York Drive
Logged By: RGC
12-5-19

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-2	430	0-2	SC				**UNDOCUMENTED FILL:** CLAYEY SAND, dark brown, wet, loose; wood, concrete, plastic debris, few roots.
		2-5	SC				**ALLUVIUM:** CLAYEY SAND, dark brown, wet/saturated, loose; few roots, porous, water seepage along basal contact.
		5-9	SM				**OLDER ALLUVIUM:** SILTY SAND with CLAY, dark brown, moist, loose to medium dense; no visible porosity.
		9-14	SM	Chunk Sample @ 8-10	11.0	105.0	SILTY SAND with CLAY, brown, moist, medium dense to dense; no visible porosity.
		14-15	SP/SM				**BEDROCK:** GRANITIC ROCK breaking to SAND and SILTY SAND upon excavation, brown, moist, dense.
							Total Depth = 15' Water Seepage Encountered @ 4½-5' Backfilled 12-5-2019

GeoSoils, Inc.

W.O. 7746-A-SC
Balbas
York Drive
Logged By: RGC
12-5-19

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-3	434	0-3	SM				**UNDOCUMENTED FILL:** SILTY SAND, dark brown, wet, loose; some wood, concrete, and plastic debris.
		3-6	SM				**ALLUVIUM:** SILTY SAND, dark brown, wet, loose; many roots, water seepage along basal contact, at about 5-6'.
		6-9	SM/SC				**OLDER ALLUVIUM:** SILTY SAND with CLAY, dark brown, wet, medium dense, no visible porosity, @ 7' becomes very moist.
		9-12	SM/SC				SILTY SAND with CLAY, dark brown to brown, moist, dense.
		12-14	SM/SW				**BEDROCK:** GRANITIC ROCK breaking to SAND and SILTY SAND upon excavation, brown (mixed color), moist, dense.
							Total Depth = 14' Water Seepage Encountered @ 5'-6' Backfilled 12-5-2019

GeoSoils, Inc.

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-4	436	0-2	SC				**UNDOCUMENTED FILL:** CLAYEY SAND, dark brown, wet, loose; pockets of organic debris, PVC pipe, gravel, water seepage from gravel/perforated PVC pipe @ 1.5'.
		2-4	SC				**COLLUVIUM:** CLAYEY SAND, brown, moist, loose; porous.
		4-7	SM/SC				**OLDER ALLUVIUM:** SILTY SAND with CLAY, brown, moist, medium dense; no visible porosity.
		7-9	SM/SP				**BEDROCK:** GRANITIC ROCK breaking to SAND and SILTY SAND upon excavation, brown, moist, dense.
							Total Depth = 9' Local Seepage Encountered @ 1.5' (from perforated PVC pipe) Backfilled 12-5-2019

GeoSoils, Inc.

W.O. 7746-A-SC
Balbas
York Drive
Logged By: RGC
12-5-19

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-5	443.5	0-2½	SM	Ring @ 1	18.7	102.8	**COLLUVIUM:** SILTY SAND with clay, dark brown, moist, loose; few roots.
		2½-7	SM/SP				**BEDROCK:** GRANITIC ROCK breaking to SILTY SAND and SAND upon excavation, with brittle, coarse gravel to cobble size rock fragments, brown, slightly moist, dense.
							Total Depth = 7' No Groundwater Encountered Backfilled 12-5-2019

GeoSoils, Inc.

W.O. 7746-A-SC
Balbas
York Drive
Logged By: RGC
12-5-19

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-6	442	0-1½	SP				**UNDOCUMENTED FILL:** SAND, gray, moist, loose.
		½-2½	SM				SILTY SAND, brown, wet, loose, many roots. Water Seepage locally along basal contact.
		2½-5	SC				**COLLUVIUM:** CLAYEY SAND, brown to dark brown, slightly moist, loose; porous.
		5-7	SM/SP				**BEDROCK:** GRANITIC ROCK breaking to SAND and SILTY SAND upon excavation, light brown to brown, slightly moist, dense.
							Total Depth = 7' Groundwater/Seepage @ 2½' Backfilled 12-5-2019

GeoSoils, Inc.

W.O. 7746-A-SC
Balbas
York Drive
Logged By: RGC
12-5-19

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-7	446	0-1	SM				**COLLUVIUM:** SILTY SAND, dark brown, wet, loose; few roots.
		1-2½	SC				CLAYEY SAND, brown, moist, loose; porous.
		2½-10	SM/SP				**BEDROCK:** GRANITE ROCK breaking to SAND and SILTY SAND upon excavation, yellowish brown, slightly moist, dense.
							Total Depth = 10' No Groundwater Encountered Backfilled 10-5-2019

GeoSoils, Inc.

W.O. 7746-A-SC
Balbas
York Drive
Logged By: RGC
12-5-19

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-8	438.5	0-1	SM				**COLLUVIUM:** SILTY SAND, dark brown, wet, loose; few roots, porous.
		1-2	SC				SILTY SAND with CLAY, brown, slightly moist, loose; porous.
		2-4	SM/SP				**BEDROCK:** GRANITIC ROCK breaking to SILTY SAND and SAND upon excavation, brown to yellowish brown, slightly moist, dense.
							Total Depth = 4' No Groundwater Encountered Backfilled 12-5-2019

BORING LOG

GeoSoils, Inc.

W.O. 1813-SC

PROJECT: TROY BROWNLEE
1863 DEVON PLACE, VISTA

BORING B-1 SHEET 1 OF 1

DATE EXCAVATED 4-11-95

SAMPLE METHOD: Drive Tube 140 lb/30" drop

Standard Penetration Test

Water Seepage into hole

Undisturbed, Ring Sample

Depth (ft.)	Sample: Bulk	Sample: Undis-turbed	Sample: Blows/6"	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Description of Material
							0-6' +/- **ALLUVIUM (Qal)**: Brown-orange brown, silty SAND, loose to medium dense, decreasing porosity from 3' +/-, moist.
			8/ 17	SM	109.9	17.6	
5			33/ 17 2"	SC	121.4	11.5	Note: Moderately difficult drilling from 5' +/-. @5' Brownish orange, clayey fine to coarse grain SAND, medium dense, poorly sorted, occasional to common coarse sandy pockets, moist.
							@6' +/- - 8' **WEATHERED BEDROCK (Kgr)**: Medium dark brownish gray, weathered GRANITIC rock, medium dense, medium friable, moist.
10							
							Total depth = 8 feet Trace seepage at 4 +/- feet Hole backfilled 4-11-95
15							
20							
25							

GeoSoils, Inc.

BORING LOG

W.O. 1813-SC

PROJECT: TROY BROWNLEE
1863 DEVON PLACE, VISTA

BORING B-2 *SHEET* 1 *OF* 1

DATE EXCAVATED 4-11-95

SAMPLE METHOD: Drive Tube 140 lb/30" drop

Standard Penetration Test

Water Seepage into hole

Undisturbed, Ring Sample

Depth (ft.)	Sample: Bulk	Sample: Undis-turbed	Blows/6"	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Description of Material
0							0-13' +/- **ALLUVIUM (Qal)**: Brown, silty clayey SAND, loose, locally porous, moist. Note: Free water in sample at 3 feet.
3		X	3/4	SC	111.2	17.4	
5	X	X	10/22		119.7	14.9	@5' Medium dark brown, silty clayey fine to coarse SAND, loose to medium dense, poorly sorted, micaceous, moist with no free water.
8		X	13/20		118.5	17.0	
9							Note: Free water at 9 +/- feet.
13			50-1.5		PR		@13'-13 1/2'- **WEATHERED BEDROCK (Kgr)**: Medium to dark gray, weathered GRANITIC rock, medium dense, moderately friable, moist.

Total depth = 13 1/2 feet
Seepage at 9 +/- feet
Hole backfilled
*PR = Poor recovery

GeoSoils, Inc.

BORING LOG

W.O. 1813-SC

PROJECT: TROY BROWNLEE
1863 DEVON PLACE, VISTA

BORING B-3 *SHEET* 1 *OF* 1

DATE EXCAVATED 4-11-95

SAMPLE METHOD: Drive Tube 140 lb/30" drop

Standard Penetration Test

Undisturbed, Ring Sample

Water Seepage into hole

Depth (ft.)	Sample: Bulk	Sample: Undis-turbed	Sample: Blows/6"	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Description of Material
0							0-18' +/- ~~**ALLUVIUM (Qal)**~~: ~~Red brown, clayey fine to coarse~~ grain SAND, locally very coarse, loose, slightly permeable, moist with free water at 3' +/-.
3			6/ 8			8.3	
5			7/ 11	SC	113.7	16.3	@5' Dark brown, clayey SAND, medium stiff, pliable, moist.
8			22/ 32	CL	118.8	16.4	@8' Mottled dry and brown sandy CLAY, stiff, pliable, moist.
10			19/ 25		116.6	16.4	@10' Mottled gray-orange-brown, sandy CLAY, stiff, locally very sandy, pliable, moist with free water.
15			25/ 25-2" 14/ 15/ 17	SC	108.5	21.1	@15' Brown, bedded silty SAND and sandy CLAY, medium dense, moist.
18 1/2			50-2"		PR		@18 1/2' <u>WEATHERED BEDROCK (Kgr)</u>: Gray, black and brown, weathered GRANITIC rock, dense, moist.
20							
25							

Practical refusal at 18 1/2 feet
Seepage at 3 and 10 feet +/-
Hole backfilled
*PR= Poor recovery

APPENDIX C

SEISMICITY

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

DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 7746A

DATE: 02-05-2020

JOB NAME: Balbas

CALCULATION NAME: Test Run Analysis

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

SITE COORDINATES:
   SITE LATITUDE: 33.1759
   SITE LONGITUDE: 117.2112

SEARCH RADIUS: 62.4 mi

ATTENUATION RELATION: 12) Bozorgnia Campbell Niazi (1999) Hor.-Soft Rock-Cor.
   UNCERTAINTY (M=Median, S=Sigma): S   Number of Sigmas: 1.0
   DISTANCE MEASURE: cdist
   SCOND: 0
   Basement Depth: .10 km   Campbell SSR: 1   Campbell SHR: 0
   COMPUTE PEAK HORIZONTAL ACCELERATION

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

MINIMUM DEPTH VALUE (km): 3.0

W.O. 7746-A-SC
PLATE C-1

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
ROSE CANYON	11.6(18.6)	7.2	0.360	IX
NEWPORT-INGLEWOOD (Offshore)	12.8(20.6)	7.1	0.310	IX
ELSINORE (TEMECULA)	18.1(29.1)	6.8	0.185	VIII
ELSINORE (JULIAN)	18.1(29.1)	7.1	0.225	IX
CORONADO BANK	27.3(43.9)	7.6	0.210	VIII
ELSINORE (GLEN IVY)	33.3(53.6)	6.8	0.099	VII
EARTHQUAKE VALLEY	36.5(58.7)	6.5	0.074	VII
SAN JOAQUIN HILLS	39.5(63.6)	6.6	0.103	VII
SAN JACINTO-ANZA	40.9(65.9)	7.2	0.105	VII
SAN JACINTO-SAN JACINTO VALLEY	42.6(68.5)	6.9	0.082	VII
PALOS VERDES	42.8(68.9)	7.3	0.108	VII
SAN JACINTO-COYOTE CREEK	45.1(72.6)	6.6	0.063	VI
CHINO-CENTRAL AVE. (Elsinore)	49.3(79.4)	6.7	0.086	VII
NEWPORT-INGLEWOOD (L.A.Basin)	50.9(81.9)	7.1	0.078	VII
ELSINORE (COYOTE MOUNTAIN)	51.3(82.5)	6.8	0.063	VI
WHITTIER	53.3(85.8)	6.8	0.060	VI
SAN JACINTO-SAN BERNARDINO	58.2(93.6)	6.7	0.051	VI
SAN JACINTO - BORREGO	58.9(94.8)	6.6	0.047	VI
SAN ANDREAS - Whole M-1a	61.5(99.0)	8.0	0.124	VII
SAN ANDREAS - San Bernardino M-1	61.5(99.0)	7.5	0.085	VII
SAN ANDREAS - SB-Coach. M-1b-2	61.5(99.0)	7.7	0.099	VII
SAN ANDREAS - SB-Coach. M-2b	61.5(99.0)	7.7	0.099	VII

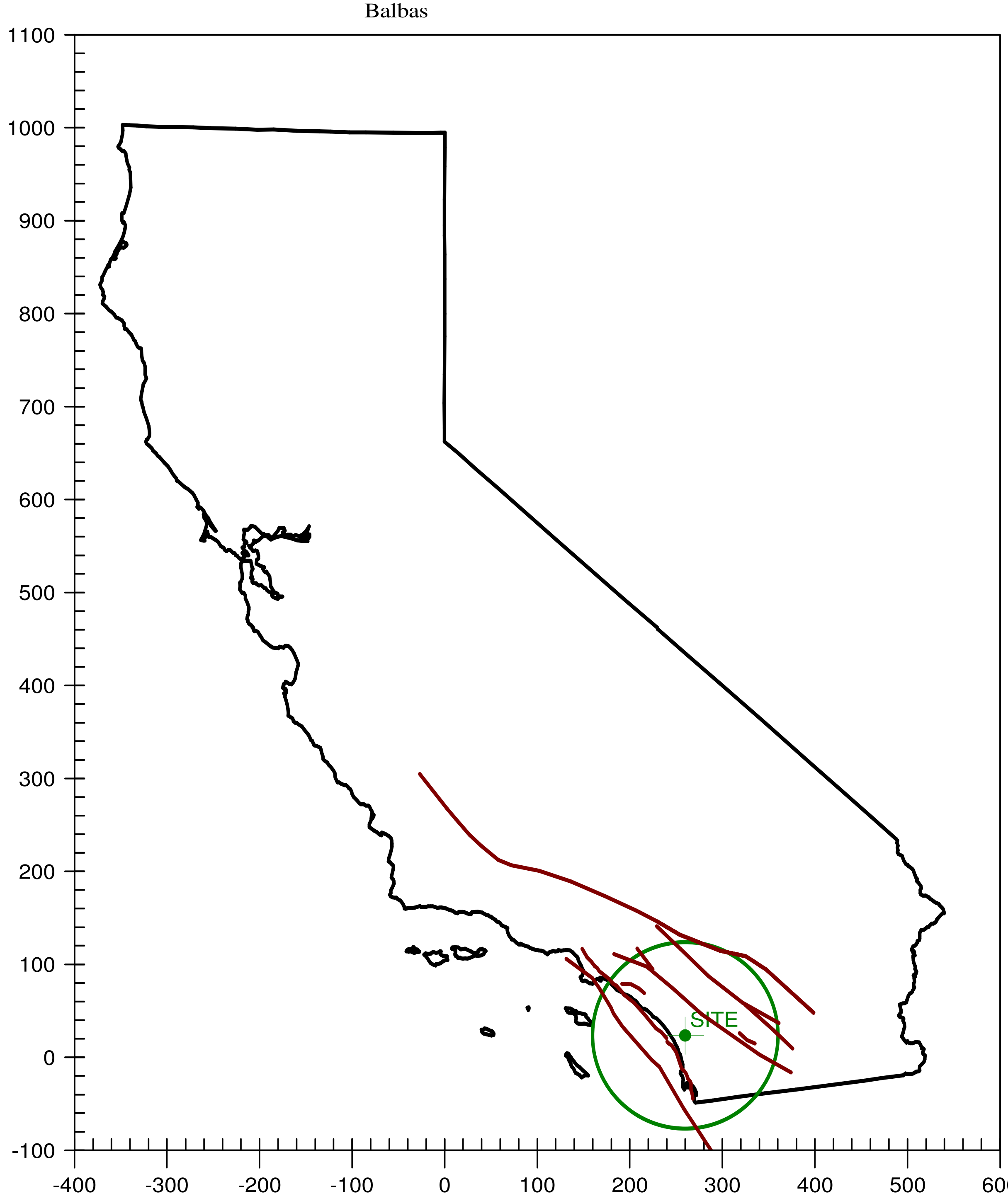
****

-END OF SEARCH- 22 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ROSE CANYON FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 11.6 MILES (18.6 km) AWAY.

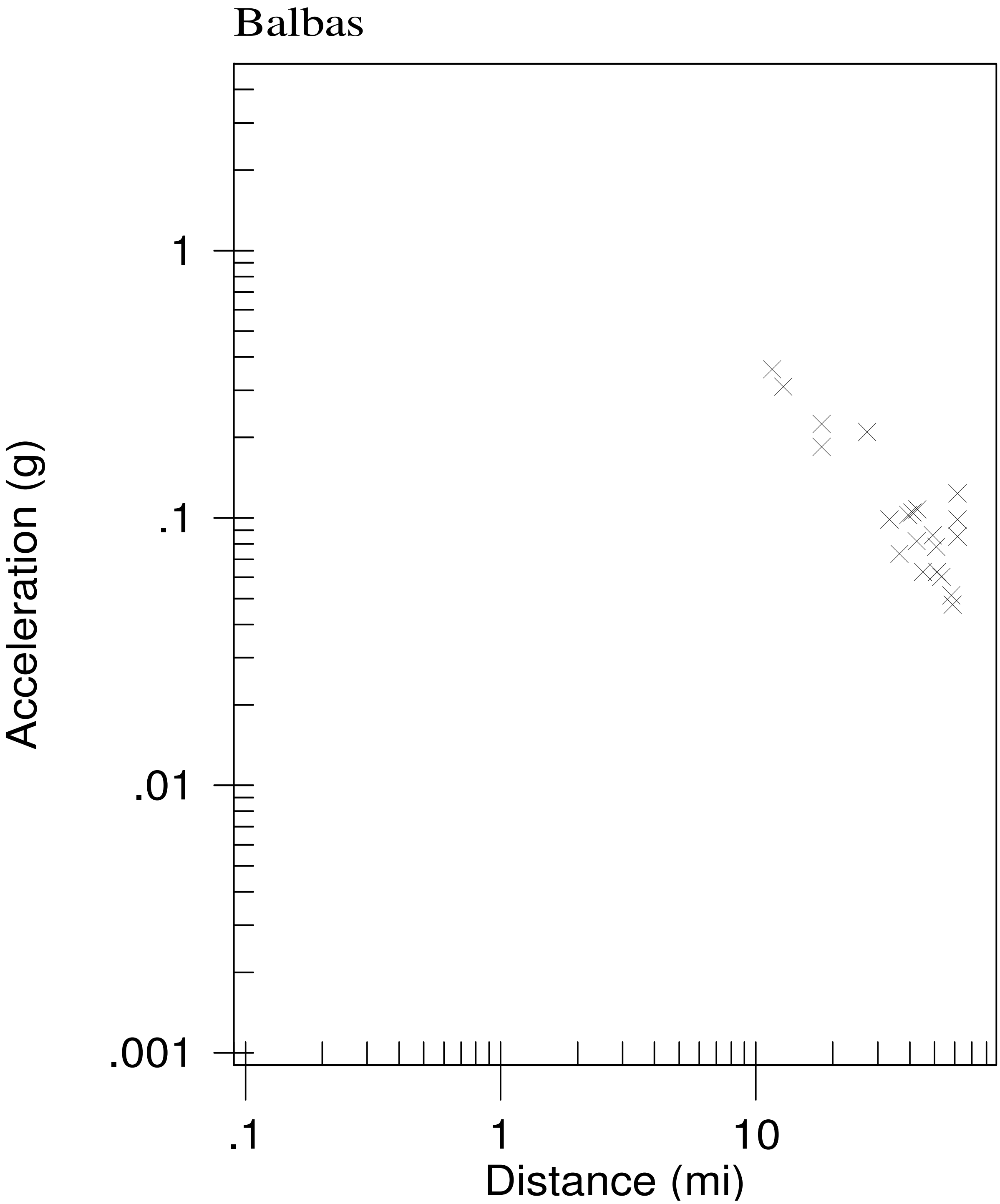
LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.3597 g

CALIFORNIA FAULT MAP



W.O. 7746-A-SC
PLATE C-3

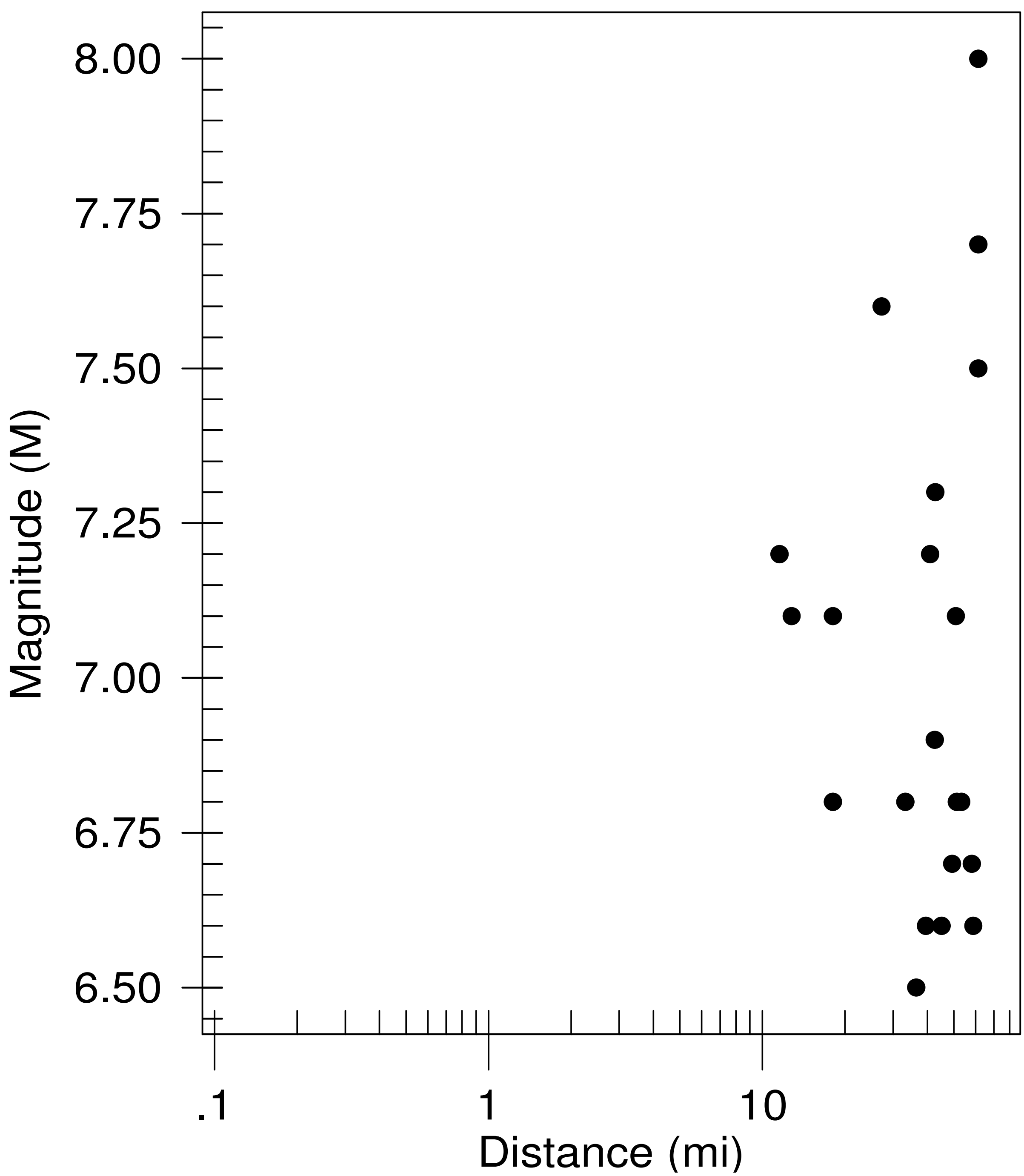
MAXIMUM EARTHQUAKES



W.O. 7746-A-SC
PLATE C-4

EARTHQUAKE MAGNITUDES & DISTANCES

Balbas



W.O. 7746-A-SC
PLATE C-5

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

ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 7746a

DATE: 02-05-2020

JOB NAME: Balbas

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

MAGNITUDE RANGE:
   MINIMUM MAGNITUDE: 5.00
   MAXIMUM MAGNITUDE: 9.00

SITE COORDINATES:
   SITE LATITUDE: 33.1759
   SITE LONGITUDE: 117.2112

SEARCH DATES:
   START DATE: 1800
   END DATE: 2020

SEARCH RADIUS:
   62.4 mi
   100.4 km

ATTENUATION RELATION: 12) Bozorgnia Campbell Niazi (1999) Hor.-Soft Rock-Cor.
   UNCERTAINTY (M=Median, S=Sigma): S   Number of Sigmas: 1.0
   ASSUMED SOURCE TYPE: SS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
   SCOND: 1 Depth Source: A
   Basement Depth: .10 km   Campbell SSR: 1   Campbell SHR: 0
   COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km): 3.0

EARTHQUAKE SEARCH RESULTS

Page 1

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	33.0000	117.3000	11/22/1800	2130 0.0	0.0	6.50	0.207	VIII	13.2(21.2)
MGI	33.0000	117.0000	09/21/1856	730 0.0	0.0	5.00	0.063	VI	17.2(27.7)
MGI	32.8000	117.1000	05/25/1803	0 0 0.0	0.0	5.00	0.040	V	26.7(43.0)
DMG	33.2000	116.7000	01/01/1920	235 0.0	0.0	5.00	0.036	V	29.6(47.6)
DMG	32.7000	117.2000	05/27/1862	20 0 0.0	0.0	5.90	0.056	VI	32.9(52.9)
T-A	32.6700	117.1700	05/24/1865	0 0 0.0	0.0	5.00	0.030	V	35.0(56.3)
T-A	32.6700	117.1700	10/21/1862	0 0 0.0	0.0	5.00	0.030	V	35.0(56.3)
T-A	32.6700	117.1700	12/00/1856	0 0 0.0	0.0	5.00	0.030	V	35.0(56.3)
DMG	32.8000	116.8000	10/23/1894	23 3 0.0	0.0	5.70	0.046	VI	35.2(56.7)
MGI	33.2000	116.6000	10/12/1920	1748 0.0	0.0	5.30	0.036	V	35.4(56.9)
DMG	33.7000	117.4000	04/11/1910	757 0.0	0.0	5.00	0.028	V	37.8(60.8)
DMG	33.7000	117.4000	05/15/1910	1547 0.0	0.0	6.00	0.051	VI	37.8(60.8)
DMG	33.7000	117.4000	05/13/1910	620 0.0	0.0	5.00	0.028	V	37.8(60.8)
DMG	33.6990	117.5110	05/31/1938	83455.4	10.0	5.50	0.035	V	40.0(64.4)
DMG	33.7100	116.9250	09/23/1963	144152.6	16.5	5.00	0.026	V	40.4(65.0)
PAS	32.9710	117.8700	07/13/1986	1347 8.2	6.0	5.30	0.031	V	40.7(65.4)
DMG	33.7500	117.0000	06/06/1918	2232 0.0	0.0	5.00	0.026	V	41.5(66.7)
DMG	33.7500	117.0000	04/21/1918	223225.0	0.0	6.80	0.079	VII	41.5(66.7)
GSP	33.5290	116.5720	06/12/2005	154146.5	14.0	5.20	0.027	V	44.2(71.1)
DMG	33.8000	117.0000	12/25/1899	1225 0.0	0.0	6.40	0.055	VI	44.8(72.0)
GSG	33.4200	116.4890	07/07/2010	235333.5	14.0	5.50	0.031	V	45.0(72.3)
PAS	33.5010	116.5130	02/25/1980	104738.5	13.6	5.50	0.031	V	46.1(74.2)
GSP	33.5080	116.5140	10/31/2001	075616.6	15.0	5.10	0.024	V	46.3(74.5)
DMG	33.0000	116.4330	06/04/1940	1035 8.3	0.0	5.10	0.024	IV	46.6(75.0)
DMG	33.5000	116.5000	09/30/1916	211 0.0	0.0	5.00	0.023	IV	46.7(75.2)
GSP	33.4315	116.4427	06/10/2016	080438.7	12.3	5.19	0.025	V	47.7(76.8)
MGI	33.8000	117.6000	04/22/1918	2115 0.0	0.0	5.00	0.022	IV	48.6(78.1)
DMG	33.9000	117.2000	12/19/1880	0 0 0.0	0.0	6.00	0.038	V	50.0(80.5)
DMG	33.3430	116.3460	04/28/1969	232042.9	20.0	5.80	0.033	V	51.3(82.5)
DMG	33.5750	117.9830	03/11/1933	518 4.0	0.0	5.20	0.022	IV	52.3(84.2)
DMG	33.6170	117.9670	03/11/1933	154 7.8	0.0	6.30	0.043	VI	53.2(85.5)
DMG	33.4000	116.3000	02/09/1890	12 6 0.0	0.0	6.30	0.042	VI	54.8(88.2)
DMG	33.6170	118.0170	03/14/1933	19 150.0	0.0	5.10	0.020	IV	55.5(89.4)
DMG	34.0000	117.2500	07/23/1923	73026.0	0.0	6.25	0.039	V	56.9(91.6)
DMG	33.4080	116.2610	03/25/1937	1649 1.8	10.0	6.00	0.033	V	57.1(91.9)
DMG	33.9500	116.8500	09/28/1946	719 9.0	0.0	5.00	0.018	IV	57.3(92.3)
DMG	33.2000	116.2000	05/28/1892	1115 0.0	0.0	6.30	0.039	V	58.4(94.1)
MGI	34.0000	117.5000	12/16/1858	10 0 0.0	0.0	7.00	0.062	VI	59.3(95.4)
DMG	33.6830	118.0500	03/11/1933	658 3.0	0.0	5.50	0.023	IV	59.7(96.0)
DMG	33.2830	116.1830	03/19/1954	95556.0	0.0	5.00	0.017	IV	59.8(96.3)
DMG	33.2830	116.1830	03/23/1954	41450.0	0.0	5.10	0.018	IV	59.8(96.3)
DMG	33.2830	116.1830	03/19/1954	95429.0	0.0	6.20	0.036	V	59.8(96.3)
DMG	33.2830	116.1830	03/19/1954	102117.0	0.0	5.50	0.023	IV	59.8(96.3)

DMG	33.7000	118.0670	03/11/1933	85457.0	0.0	5.10	0.018	IV	61.2(98.4)
DMG	33.7000	118.0670	03/11/1933	51022.0	0.0	5.10	0.018	IV	61.2(98.4)
DMG	33.9760	116.7210	06/12/1944	104534.7	10.0	5.10	0.018	IV	62.0(99.8)
DMG	32.7000	116.3000	02/24/1892	720 0.0	0.0	6.70	0.048	VI	62.2(100.1)
GSG	33.9530	117.7610	07/29/2008	184215.7	14.0	5.30	0.020	IV	62.3(100.2)
DMG	33.2170	116.1330	08/15/1945	175624.0	0.0	5.70	0.025	V	62.4(100.3)

****

-END OF SEARCH- 49 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 2020

LENGTH OF SEARCH TIME: 221 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 13.2 MILES (21.2 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.207 g

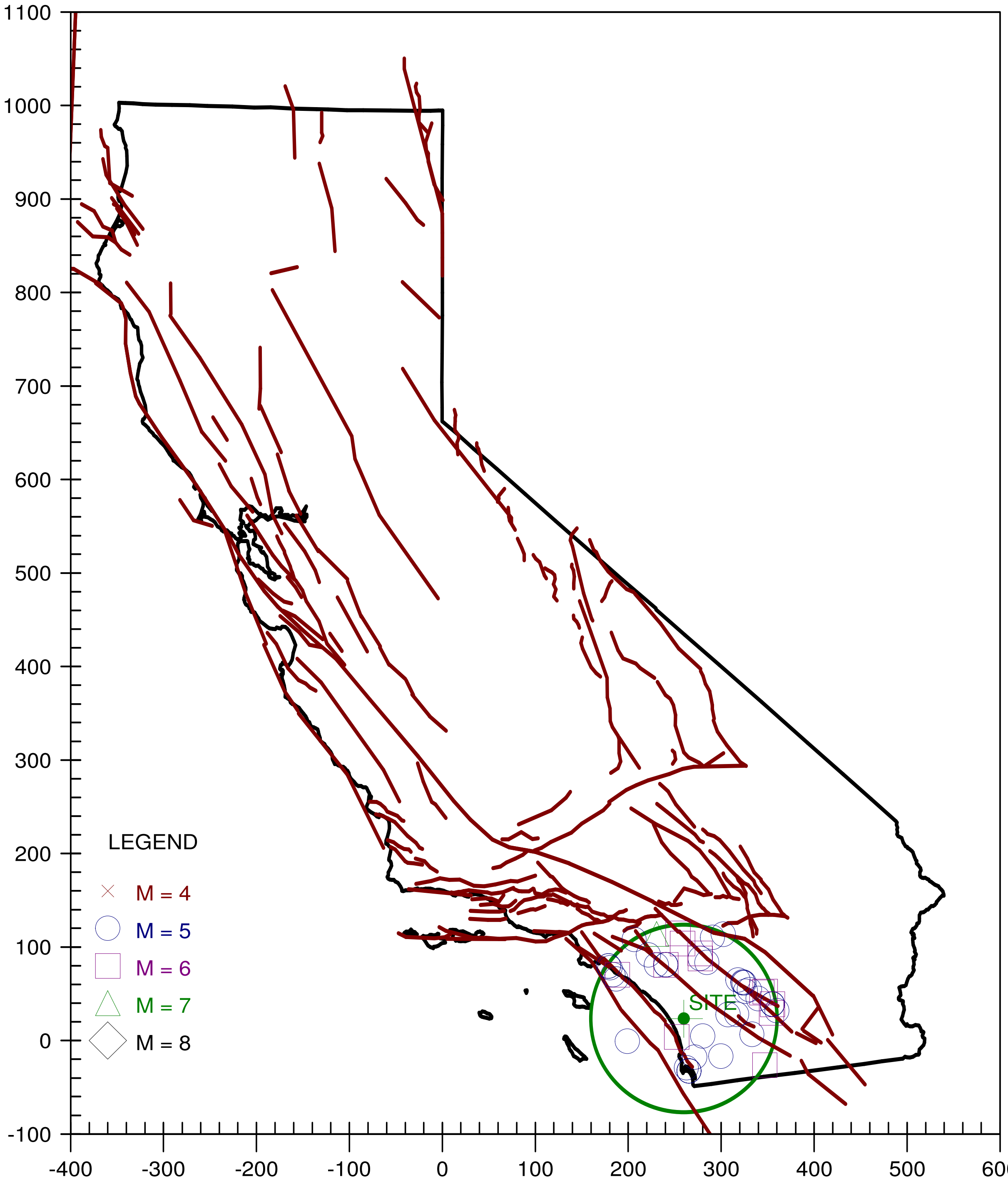
COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
  a-value= 0.661
  b-value= 0.300
  beta-value= 0.691

TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	49	0.22172
4.5	49	0.22172
5.0	49	0.22172
5.5	22	0.09955
6.0	13	0.05882
6.5	4	0.01810
7.0	1	0.00452

EARTHQUAKE EPICENTER MAP

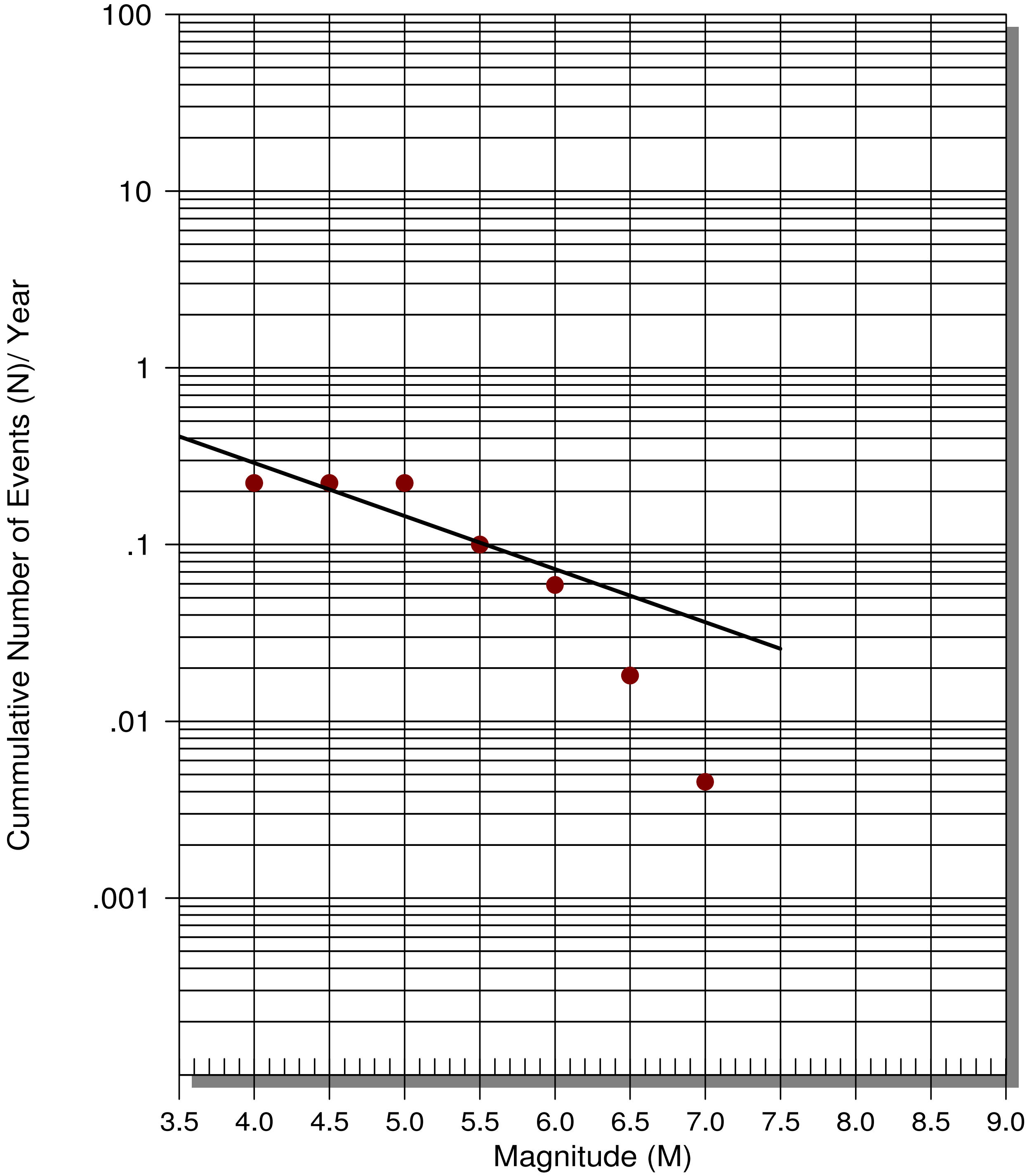
Balbas



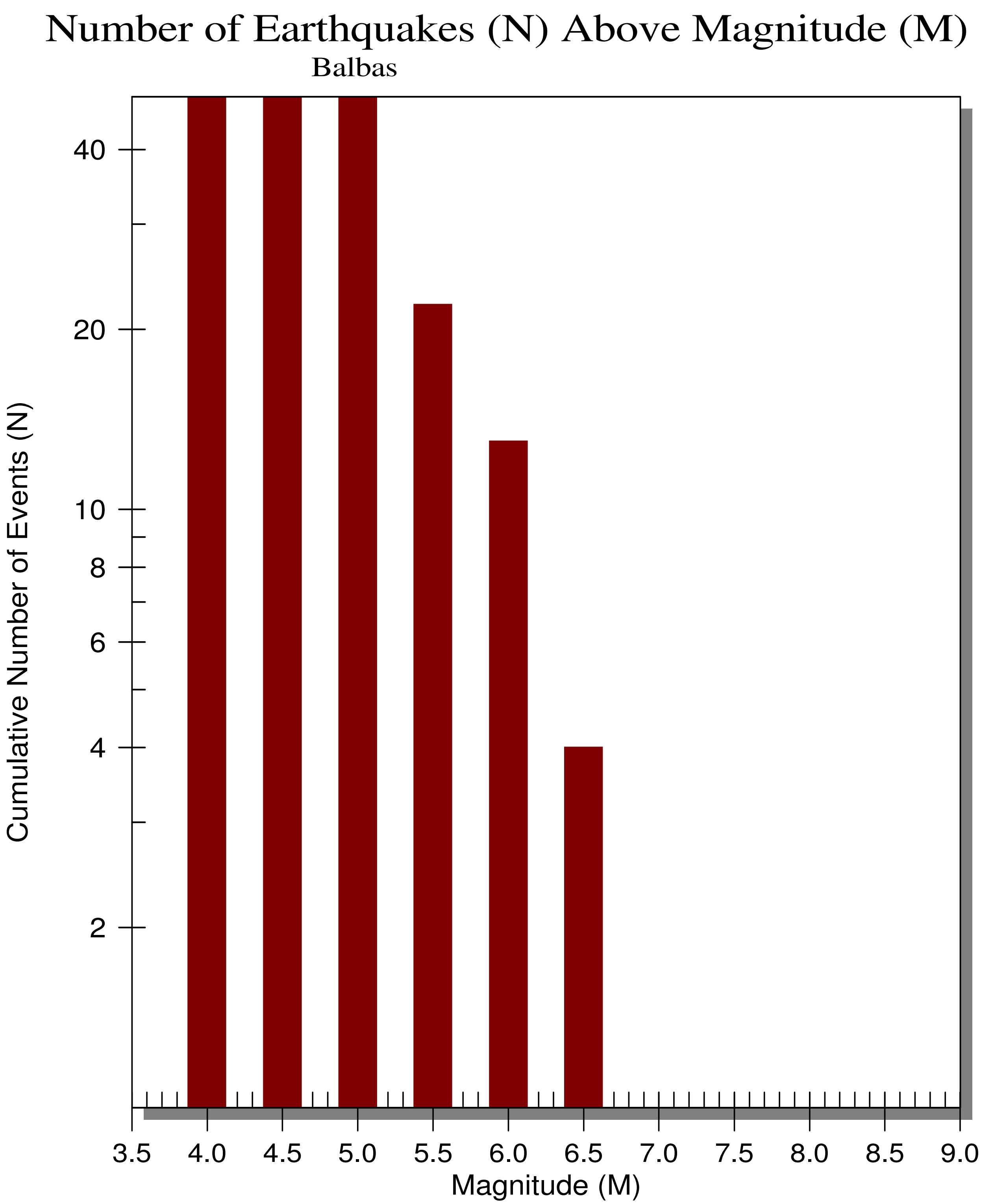
W.O. 7746-A-SC
PLATE C-9

EARTHQUAKE RECURRENCE CURVE

Balbas



W.O. 7746-A-SC
PLATE C-10

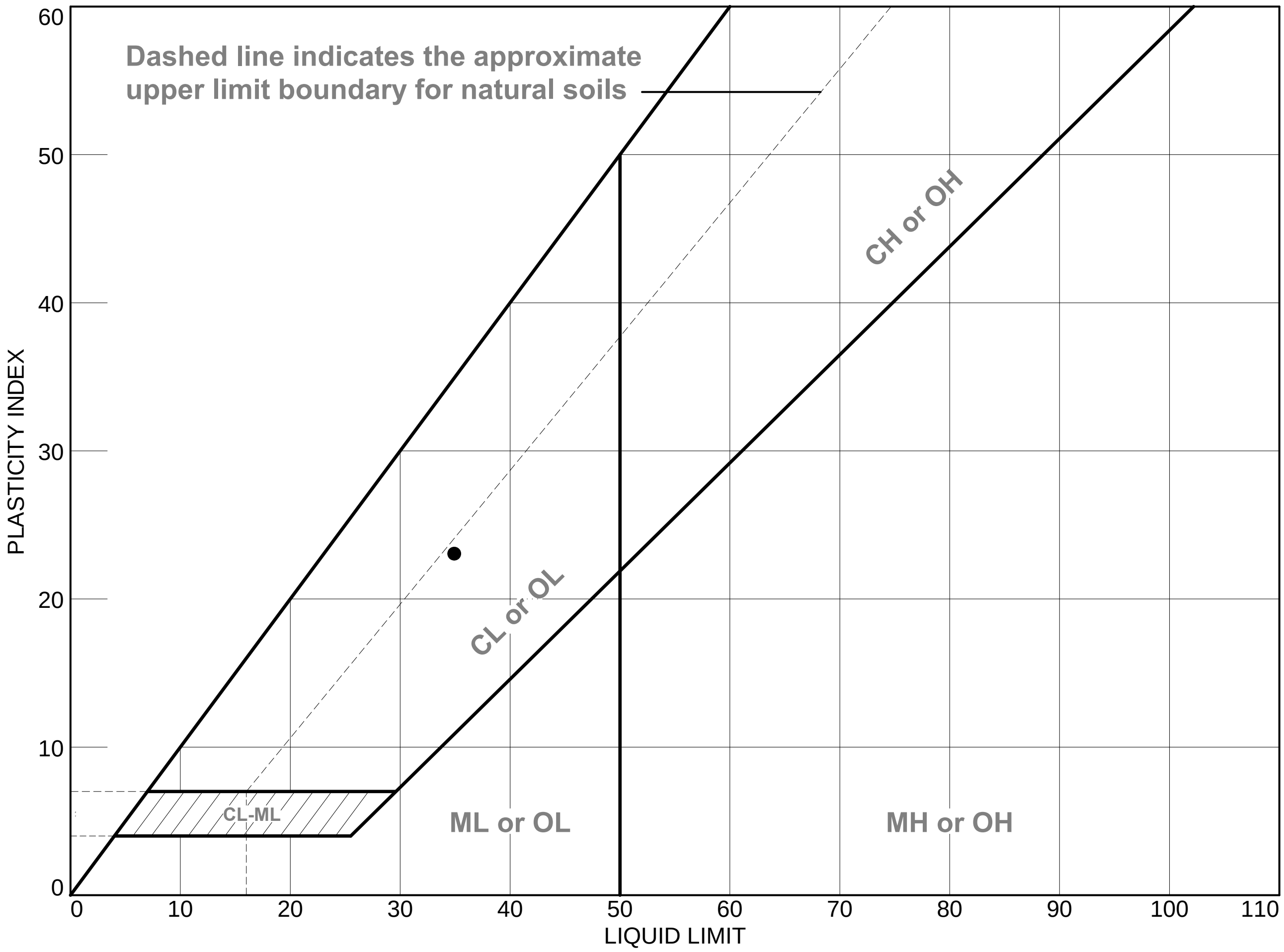


W.O. 7746-A-SC
PLATE C-11

APPENDIX D

LABORATORY RESULTS

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	TP-1	TP-1	3.5-6	-	12	35	23	CL

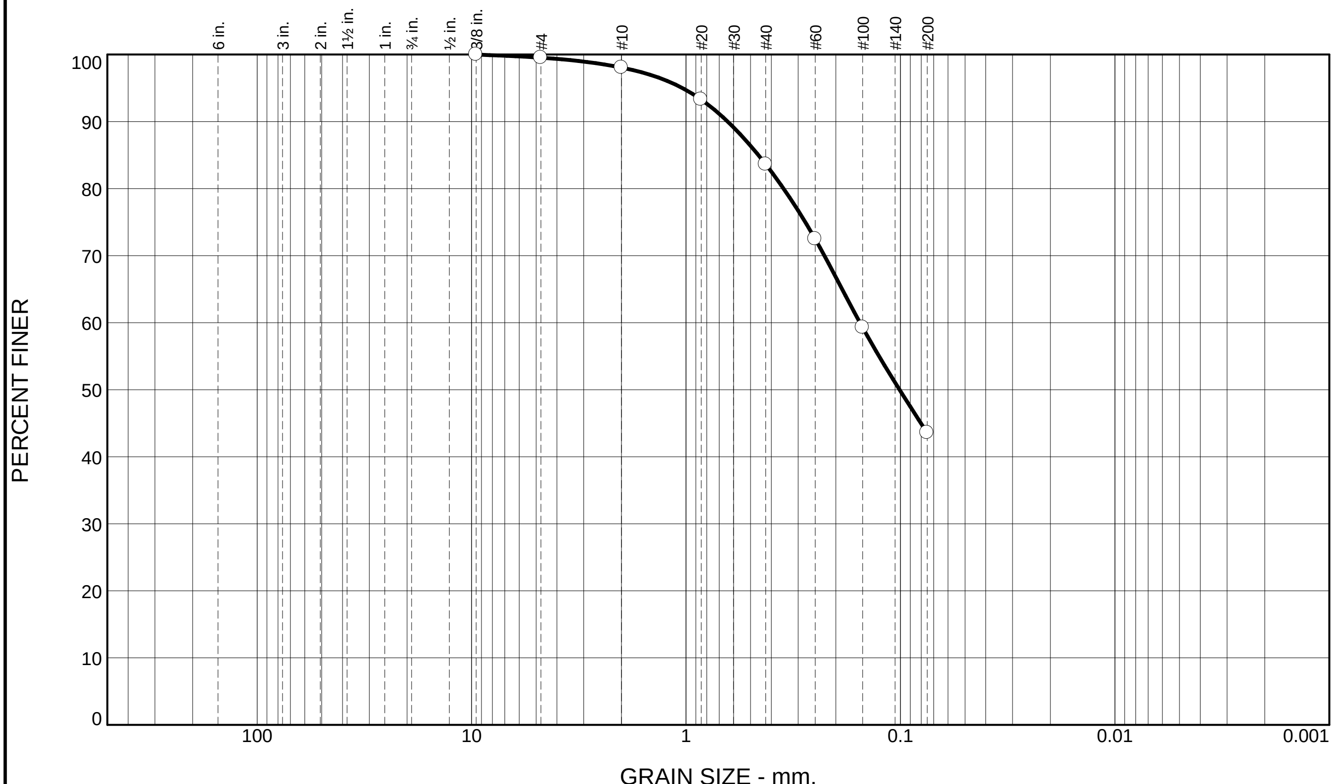
GeoSoils, Inc.

Client: Balbas
Project: York Drive

Project No.: 7746-A-SC **Figure** D-1

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

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.5	1.4	14.5	40.0	43.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375	100.0		
#4	99.5		
#10	98.1		
#20	93.3		
#40	83.6		
#60	72.5		
#100	59.3		
#200	43.6		

* (no specification provided)

Soil Description

Dark Reddish Brown Clayey Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.6391 D_{85}= 0.4598 D_{60}= 0.1544
D_{50}= 0.1010 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

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

Date: 12-9-19

GeoSoils, Inc.

Client: Balbas
Project: York Drive
Project No: 7746-A-SC **Figure** D-2

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

APPENDIX E

STORM WATER BMP CHECKLISTS/FORMS

From "Appendix C: Geotechnical and Groundwater Investigation Requirements" included in: **"City of Vista BMP Design Manual for Permanent Site Design, Stormwater Treatment and Hydromodification Management."**

Worksheet C.4-1 Page 2 of 4			
Criteria	**Screening Question**	**Yes**	**No**
3	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensible evaluation of the factors presented in Appendix C.3.		X
Provide basis: N/A. See our response to Criteria No. 1. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
4	**Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as a change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.		X
Provide basis: N/A. See our response to Criteria No. 1. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
Part 1 Result*	In the answers to rows 1-4 are "**Yes**" a full infiltration design is potentially feasible. The feasibility screening category is **Full Infiltration** If any answer from row 1-4 is "**No**", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design. Proceed to Part 2		**No**

* To be completed using gathered site information and best professional judgement considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by [City Engineer] to substantiate findings.

From "Appendix C: Geotechnical and Groundwater Investigation Requirements" included in: **"City of Vista BMP Design Manual for Permanent Site Design, Stormwater Treatment and Hydromodification Management."**

<table>
<tr><th colspan="4">Worksheet C.4-1 Page 3 of 4</th></tr>
<tr><td colspan="4"><u>Part 2 - Partial Infiltration vs. No Infiltration Feasibility Screening Criteria</u>

Would infiltration of water in an appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?</td></tr>
<tr><td>Criteria</td><td>Screening Question</td><td>Yes</td><td>No</td></tr>
<tr><td>5</td><td>Do soil and geologic conditions allow for infiltration in any appreciable rate or volume? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:

Yes. USDA data indicates a range of infiltration rates between 0.00 and 0.06 inches per hour. This rate does not support partial infiltration." Per Table B.2-3 of the County (2019), a design infiltration rate of 0.025 inches/hour is provided for HSG "D" soils.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>6</td><td>Can infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis:

Due to the occurrence of granitic bedrock at depth within the subject site, storm water infiltration has the potential to create shallow perched groundwater conditions. Currently planned BMP's shown on KPI (2020) are also located alongt he toe of a planned graded slope. Any infiltration would create a negative impact on the overall stability and performance of the slope.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
</table>

From "Appendix C: Geotechnical and Groundwater Investigation Requirements" included in: **"City of Vista BMP Design Manual for Permanent Site Design, Stormwater Treatment and Hydromodification Management."**

<table>
<tr><th colspan="4">Worksheet C.4-1 Page 4 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>7</td><td>Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis:

The site currently demonstrates the capacity to develop shallow zones of perched groundwater. Contributions from any storm water BMP would tend to further exacerbate this condition. Subsurface exploration, and the current site pla (KPI, 2020) indicates less than 10 feet of vertical separation between the infiltration surface elevation and know areas of perched groundwater. Thus, there is a potential that storm water pollutants could impact groundwater quality.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>8</td><td>Can infiltration be allowed without violating downstream water rights? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:

Downstream water rights are considered a legal matter and typically do not fall within the purview of geotechnical engineering. However, GSI is not aware of any significant downstream water rights issues of concern on the adjoining properties. Given the slow soil infiltration rates onsite, infiltration should not significantly affect downstream water rights, from a geotechnical perspective.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>Part 2 Result*</td><td colspan="2">If all answers from row 5-8 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration.

If any answer from row 5-8 is no, then infiltration of any volume is considered to be infeasible within the drainage area. The feasibility screening category is No Infiltration.</td><td>No Infiltration</td></tr>
</table>

* To be completed using gathered site information and best professional judgement considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by Agency/Jurisdictions to substantiate findings.

APPENDIX F

GENERAL EARTHWORK, GRADING GUIDELINES AND PRELIMINARY CRITERIA

GENERAL EARTHWORK, GRADING GUIDELINES, AND PRELIMINARY CRITERIA

General

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

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

EARTHWORK OBSERVATIONS AND TESTING

Geotechnical Consultant

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

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

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

Laboratory and Field Tests

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

Contractor's Responsibility

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

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

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

SITE PREPARATION

All major vegetation, including brush, trees, thick grasses, organic debris, and other deleterious material, should be removed and disposed of off-site. These removals must be concluded prior to placing fill. In-place existing fill, soil, alluvium, colluvium, or rock materials, as evaluated by the geotechnical consultant as being unsuitable, should be removed prior to any fill placement. Depending upon the soil conditions, these materials may be reused as compacted fills. Any materials incorporated as part of the compacted fills should be approved by the geotechnical consultant.

Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipelines, or other structures not located prior to grading, are to be removed or treated in a manner recommended by the geotechnical consultant. Soft, dry, spongy, highly fractured, or otherwise unsuitable ground, extending to such a depth that surface processing cannot adequately improve the condition, should be overexcavated down to firm ground and approved by the geotechnical consultant before compaction and filling operations continue. Overexcavated and processed soils, which have been properly mixed and moisture conditioned, should be re-compacted to the minimum relative compaction as specified in these guidelines.

Existing ground, which is determined to be satisfactory for support of the fills, should be scarified (ripped) to a minimum depth of 6 to 8 inches, or as directed by the geotechnical consultant. After the scarified ground is brought to optimum moisture content, or greater and mixed, the materials should be compacted as specified herein. If the scarified zone is greater than 6 to 8 inches in depth, it may be necessary to remove the excess and place the material in lifts restricted to about 6 to 8 inches in compacted thickness.

Existing ground which is not satisfactory to support compacted fill should be overexcavated as required in the geotechnical report, or by the on-site geotechnical consultant. Scarification, disc harrowing, or other acceptable forms of mixing should continue until the soils are broken down and free of large lumps or clods, until the working surface is reasonably uniform and free from ruts, hollows, hummocks, mounds, or other uneven features, which would inhibit compaction as described previously.

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical [h:v]), the ground should be stepped or benched. The lowest bench, which will act as a key, should be a minimum of 15 feet wide and should be at least 2 feet deep into firm material, and approved by the geotechnical consultant. In fill-over-cut slope conditions, the recommended minimum width of the lowest bench or key is also 15 feet, with the key founded on firm material, as designated by the geotechnical consultant. As a general rule, unless specifically recommended otherwise by the geotechnical consultant, the minimum width of fill keys should be equal to ½ the height of the slope.

Standard benching is generally 4 feet (minimum) vertically, exposing firm, acceptable material. Benching may be used to remove unsuitable materials, although it is understood that the vertical height of the bench may exceed 4 feet. Pre-stripping may be considered for unsuitable materials in excess of 4 feet in thickness.

All areas to receive fill, including processed areas, removal areas, and the toes of fill benches, should be observed and approved by the geotechnical consultant prior to placement of fill. Fills may then be properly placed and compacted until design grades (elevations) are attained.

COMPACTED FILLS

Any earth materials imported or excavated on the property may be utilized in the fill provided that each material has been evaluated to be suitable by the geotechnical consultant. These materials should be free of roots, tree branches, other organic matter, or other deleterious materials. All unsuitable materials should be removed from the fill as directed by the geotechnical consultant. Soils of poor gradation, undesirable expansion potential, or substandard strength characteristics may be designated by the consultant as unsuitable and may require blending with other soils to serve as a satisfactory fill material.

Fill materials derived from benching operations should be dispersed throughout the fill area and blended with other approved material. Benching operations should not result in the benched material being placed only within a single equipment width away from the fill/bedrock contact.

Oversized materials defined as rock, or other irreducible materials, with a maximum dimension greater than 12 inches, should not be buried or placed in fills unless the location of materials and disposal methods are specifically approved by the geotechnical consultant. Oversized material should be taken offsite, or placed in accordance with recommendations of the geotechnical consultant in areas designated as suitable for rock disposal. GSI anticipates that soils to be utilized as fill material for the subject project may contain some rock. Appropriately, the need for rock disposal may be necessary during grading operations on the site. From a geotechnical standpoint, the depth of any rocks, rock fills, or rock blankets, should be a sufficient distance from finish grade. This depth is generally the same as any overexcavation due to cut-fill transitions in hard rock areas, and generally facilitates the excavation of structural footings and substructures. Should deeper excavations be proposed (i.e., deepened footings, utility trenching, swimming pools, spas, etc.), the developer may consider increasing the hold-down depth of any rocky fills to be placed, as appropriate. In addition, some agencies/jurisdictions mandate a specific hold-down depth for oversize materials placed in fills. The hold-down depth, and potential to encounter oversize rock, both within fills, and occurring in cut or natural areas, would need to be disclosed to all interested/affected parties. Once approved by the governing agency, the hold-down depth for oversized rock (i.e., greater than 12 inches) in fills on this project is provided as 10 feet, unless specified differently in the text of this report. The governing agency may require that these materials need to be deeper, crushed, or reduced to less than 12 inches in maximum dimension, at their discretion.

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

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

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

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

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

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

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

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

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

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

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

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

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

SUBDRAIN INSTALLATION

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

EXCAVATIONS

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

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

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

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

COMPLETION

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

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

PRELIMINARY OUTDOOR POOL/SPA DESIGN RECOMMENDATIONS

The following preliminary recommendations are provided for consideration in pool/spa design and planning. Actual recommendations should be provided by a qualified geotechnical consultant, based on site specific geotechnical conditions, including a subsurface investigation, differential settlement potential, expansive and corrosive soil potential, proximity of the proposed pool/spa to any slopes with regard to slope creep and lateral fill extension, as well as slope setbacks per Code, and geometry of the proposed improvements. Recommendations for pools/spas and/or deck flatwork underlain by expansive soils, or for areas with differential settlement greater than ¼-inch over 40 feet horizontally, will be more onerous than the preliminary recommendations presented below.

The 1:1 (h:v) influence zone of any nearby retaining wall site structures should be delineated on the project civil drawings with the pool/spa. This 1:1 (h:v) zone is defined as a plane up from the lower-most heel of the retaining structure, to the daylight grade of the nearby building pad or slope. If pools/spas or associated pool/spa improvements are constructed within this zone, they should be re-positioned (horizontally or vertically) so that they are supported by earth materials that are outside or below this 1:1 plane. If this is not possible given the area of the building pad, the owner should consider eliminating these improvements or allow for increased potential for lateral/vertical deformations and associated distress that may render these improvements unusable in the future, unless they are periodically repaired and maintained. The conditions and recommendations presented herein should be disclosed to all homeowners and any interested/affected parties.

General

1. The equivalent fluid pressure to be used for the pool/spa design should be 60 pounds per cubic foot (pcf) for pool/spa walls with level backfill, and 75 pcf for a 2:1 sloped backfill condition. In addition, backdrains should be provided behind pool/spa walls subjacent to slopes.

2. Passive earth pressure may be computed as an equivalent fluid having a density of 150 pcf, to a maximum lateral earth pressure of 1,000 pounds per square foot (psf).

3. An allowable coefficient of friction between soil and concrete of 0.30 may be used with the dead load forces.

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

5. Where pools/spas are planned near structures, appropriate surcharge loads need to be incorporated into design and construction by the pool/spa designer. This includes, but is not limited to landscape berms, decorative walls, footings, built-in barbeques, utility poles, etc.

6. All pool/spa walls should be designed as "free standing" and be capable of supporting the water in the pool/spa without soil support. The shape of pool/spa in cross section and plan view may affect the performance of the pool, from a geotechnical standpoint. Pools and spas should also be designed in accordance with the latest adopted Code. Minimally, the bottoms of the pools/spas, should maintain a distance H/3, where H is the height of the slope (in feet), from the slope face. This distance should not be less than 7 feet, nor need not be greater than 40 feet.

7. The soil beneath the pool/spa bottom should be uniformly moist with the same stiffness throughout. If a fill/cut transition occurs beneath the pool/spa bottom, the cut portion should be overexcavated to a minimum depth of 48 inches, and replaced with compacted fill, such that there is a uniform blanket that is a minimum of 48 inches below the pool/spa shell. If very low expansive soil is used for fill, the fill should be placed at a minimum of 95 percent relative compaction, at optimum moisture conditions. This requirement should be 90 percent relative compaction at over optimum moisture if the pool/spa is constructed within or near expansive soils. The potential for grading and/or re-grading of the pool/spa bottom, and attendant

potential for shoring and/or slot excavation, needs to be considered during all aspects of pool/spa planning, design, and construction.

8. If the pool/spa is founded entirely in compacted fill placed during rough grading, the deepest portion of the pool/spa should correspond with the thickest fill on the lot.

9. Hydrostatic pressure relief valves should be incorporated into the pool and spa designs. A pool/spa under-drain system is also recommended, with an appropriate outlet for discharge.

10. All fittings and pipe joints, particularly fittings in the side of the pool or spa, should be properly sealed to prevent water from leaking into the adjacent soils materials, and be fitted with slip or expandible joints between connections transecting varying soil conditions.

11. An elastic expansion joint (flexible waterproof sealant) should be installed to prevent water from seeping into the soil at all deck joints.

12. A reinforced grade beam should be placed around skimmer inlets to provide support and mitigate cracking around the skimmer face.

13. In order to reduce unsightly cracking, deck slabs should minimally be 4 inches thick, and reinforced with No. 3 reinforcing bars at 18 inches on-center. All slab reinforcement should be supported to ensure proper mid-slab positioning during the placement of concrete. Wire mesh reinforcing is specifically not recommended. Deck slabs should not be tied to the pool/spa structure. Pre-moistening and/or pre-soaking of the slab subgrade is recommended, to a depth of 12 inches (optimum moisture content), or 18 inches (120 percent of the soil's optimum moisture content, or 3 percent over optimum moisture content, whichever is greater), for very low to low, and medium expansive soils, respectively. This moisture content should be maintained in the subgrade soils during concrete placement to promote uniform curing of the concrete and minimize the development of unsightly shrinkage cracks. Slab underlayment should consist of a 1- to 2-inch leveling course of sand (S.E.>30) and a minimum of 4 to 6 inches of Class 2 base compacted to 90 percent. Deck slabs within the H/3 zone, where H is the height of the slope (in feet), will have an increased potential for distress relative to other areas outside of the H/3 zone. If distress is undesirable, improvements, deck slabs or flatwork should not be constructed closer than H/3 or 7 feet (whichever is greater) from the slope face, in order to reduce, but not eliminate, this potential.

14. Pool/spa bottom or deck slabs should be founded entirely on competent bedrock, <u>or</u> properly compacted fill. Fill should be compacted to achieve a minimum 90 percent relative compaction, as discussed above. Prior to pouring concrete, subgrade soils below the pool/spa decking should be throughly watered to achieve a moisture content that is at least 2 percent above optimum moisture content, to a

depth of at least 18 inches below the bottom of slabs. This moisture content should be maintained in the subgrade soils during concrete placement to promote uniform curing of the concrete and minimize the development of unsightly shrinkage cracks.

15. In order to reduce unsightly cracking, the outer edges of pool/spa decking to be bordered by landscaping, and the edges immediately adjacent to the pool/spa, should be underlain by an 8-inch wide concrete cutoff shoulder (thickened edge) extending to a depth of at least 12 inches below the bottoms of the slabs to mitigate excessive infiltration of water under the pool/spa deck. These thickened edges should be reinforced with two No. 4 bars, one at the top and one at the bottom. Deck slabs may be minimally reinforced with No. 3 reinforcing bars placed at 18 inches on-center, in both directions. All slab reinforcement should be supported on chairs to ensure proper mid-slab positioning during the placement of concrete.

16. Surface and shrinkage cracking of the finish slab may be reduced if a low slump and water-cement ratio are maintained during concrete placement. Concrete utilized should have a minimum compressive strength of 4,000 psi. Excessive water added to concrete prior to placement is likely to cause shrinkage cracking, and should be avoided. Some concrete shrinkage cracking, however, is unavoidable.

17. Joint and sawcut locations for the pool/spa deck should be determined by the design engineer and/or contractor. However, spacings should not exceed 6 feet on center.

18. Considering the nature of the onsite earth materials, it should be anticipated that caving or sloughing could be a factor in subsurface excavations and trenching. Shoring or excavating the trench walls/backcuts at the angle of repose (typically 25 to 45 degrees), should be anticipated. <u>All</u> excavations should be observed by a representative of the geotechnical consultant, including the project geologist and/or geotechnical engineer, prior to workers entering the excavation or trench, and <u>minimally</u> conform to Cal/OSHA ("Type C" soils may be assumed), state, and local safety codes. Should adverse conditions exist, appropriate recommendations should be offered at that time by the geotechnical consultant. GSI does not consult in the area of safety engineering and the safety of the construction crew is the responsibility of the pool/spa builder.

19. It is imperative that adequate provisions for surface drainage are incorporated by the homeowners into their overall improvement scheme. Ponding water, ground saturation and flow over slope faces, are all situations which must be avoided to enhance long term performance of the pool/spa and associated improvements, and reduce the likelihood of distress.

20. Regardless of the methods employed, once the pool/spa is filled with water, should it be emptied, there exists some potential that if emptied, significant distress may occur. Accordingly, once filled, the pool/spa should not be emptied unless evaluated by the geotechnical consultant and the pool/spa builder.

21. For pools/spas built within (all or part) of the Code setback and/or geotechnical setback, as indicated in the site geotechnical documents, special foundations are recommended to mitigate the affects of creep, lateral fill extension, expansive soils and settlement on the proposed pool/spa. Most municipalities or County reviewers do not consider these effects in pool/spa plan approvals. As such, where pools/spas are proposed on 20 feet or more of fill, medium or highly expansive soils, or rock fill with limited "cap soils" and built within Code setbacks, or within the influence of the creep zone, or lateral fill extension, the following should be considered during design and construction:

    OPTION A: Shallow foundations with or without overexcavation of the pool/spa "shell," such that the pool/spa is surrounded by 5 feet of very low to low expansive soils (without irreducible particles greater that 6 inches), and the pool/spa walls closer to the slope(s) are designed to be free standing. GSI recommends a pool/spa under-drain or blanket system (see attached Typical Pool/Spa Detail). The pool/spa builders and owner in this optional construction technique should be generally satisfied with pool/spa performance under this scenario; however, some settlement, tilting, cracking, and leakage of the pool/spa is likely over the life of the project.

    OPTION B: Pier supported pool/spa foundations with or without overexcavation of the pool/spa shell such that the pool/spa is surrounded by 5 feet of very low to low expansive soils (without irreducible particles greater than 6 inches), and the pool/spa walls closer to the slope(s) are designed to be free standing. The need for a pool/spa under-drain system may be installed for leak detection purposes. Piers that support the pool/spa should be a minimum of 12 inches in diameter and at a spacing to provide vertical and lateral support of the pool/spa, in accordance with the pool/spa designers recommendations current applicable Codes. The pool/spa builder and owner in this second scenario construction technique should be more satisfied with pool/spa performance. This construction will reduce settlement and creep effects on the pool/spa; however, it will not eliminate these potentials, nor make the pool/spa "leak-free."

22. The temperature of the water lines for spas and pools may affect the corrosion properties of site soils, thus, a corrosion specialist should be retained to review all spa and pool plans, and provide mitigative recommendations, as warranted. Concrete mix design should be reviewed by a qualified corrosion consultant and materials engineer.

23. All pool/spa utility trenches should be compacted to 90 percent of the laboratory standard, under the full-time observation and testing of a qualified geotechnical consultant. Utility trench bottoms should be sloped away from the primary structure on the property (typically the residence).

24. Pool and spa utility lines should not cross the primary structure's utility lines (i.e., not stacked, or sharing of trenches, etc.).

25. The pool/spa or associated utilities should not intercept, interrupt, or otherwise adversely impact any area drain, roof drain, or other drainage conveyances. If it is necessary to modify, move, or disrupt existing area drains, subdrains, or tightlines, then the design civil engineer should be consulted, and mitigative measures provided. Such measures should be further reviewed and approved by the geotechnical consultant, prior to proceeding with any further construction.

26. The geotechnical consultant should review and approve all aspects of pool/spa and flatwork design prior to construction. A design civil engineer should review all aspects of such design, including drainage and setback conditions. Prior to acceptance of the pool/spa construction, the project builder, geotechnical consultant and civil designer should evaluate the performance of the area drains and other site drainage pipes, following pool/spa construction.

27. All aspects of construction should be reviewed and approved by the geotechnical consultant, including during excavation, prior to the placement of any additional fill, prior to the placement of any reinforcement or pouring of any concrete.

28. Any changes in design or location of the pool/spa should be reviewed and approved by the geotechnical and design civil engineer prior to construction. Field adjustments should not be allowed until written approval of the proposed field changes are obtained from the geotechnical and design civil engineer.

29. Disclosure should be made to homeowners and builders, contractors, and any interested/affected parties, that pools/spas built within about 15 feet of the top of a slope, and/or H/3, where H is the height of the slope (in feet), will experience some movement or tilting. While the pool/spa shell or coping may not necessarily crack, the levelness of the pool/spa will likely tilt toward the slope, and may not be esthetically pleasing. The same is true with decking, flatwork and other improvements in this zone.

30. Failure to adhere to the above recommendations will significantly increase the potential for distress to the pool/spa, flatwork, etc.

31. Local seismicity and/or the design earthquake will cause some distress to the pool/spa and decking or flatwork, possibly including total functional and economic loss.

32. The information and recommendations discussed above should be provided to any contractors and/or subcontractors, or homeowners, interested/affected parties, etc., that may perform or may be affected by such work.

JOB SAFETY

General

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

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

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

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

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

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

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

Test Pits Location, Orientation, and Clearance

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

Test pits should be excavated so that the spoil pile is placed away from oncoming traffic, whenever possible. The technician's vehicle is to be placed next to the test pit, opposite the spoil pile. This necessitates the fill be maintained in a driveable condition. Alternatively, the contractor may wish to park a piece of equipment in front of the test holes, particularly in small fill areas or those with limited access.

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

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

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

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

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

Trench and Vertical Excavation

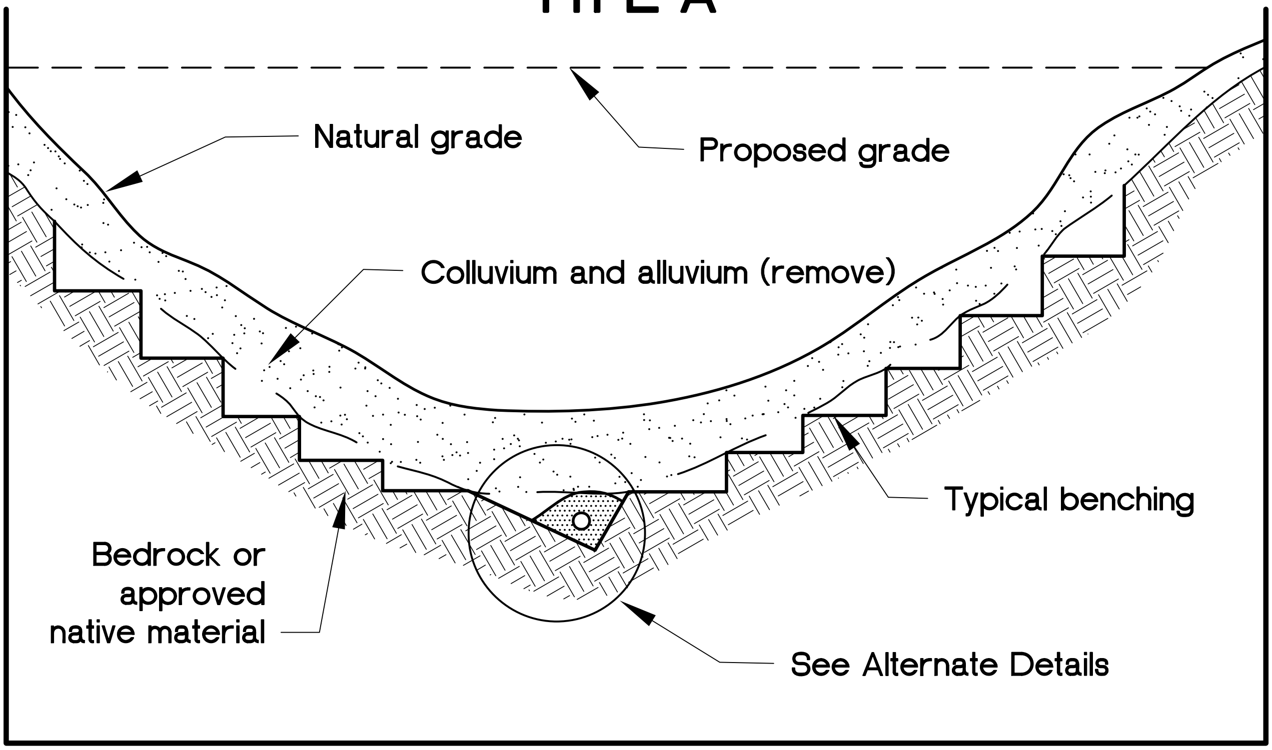
It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Our personnel are directed not to enter any excavation or vertical cut which: 1) is 5 feet or deeper unless shored or laid back; 2) displays any evidence of instability, has any loose rock or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

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

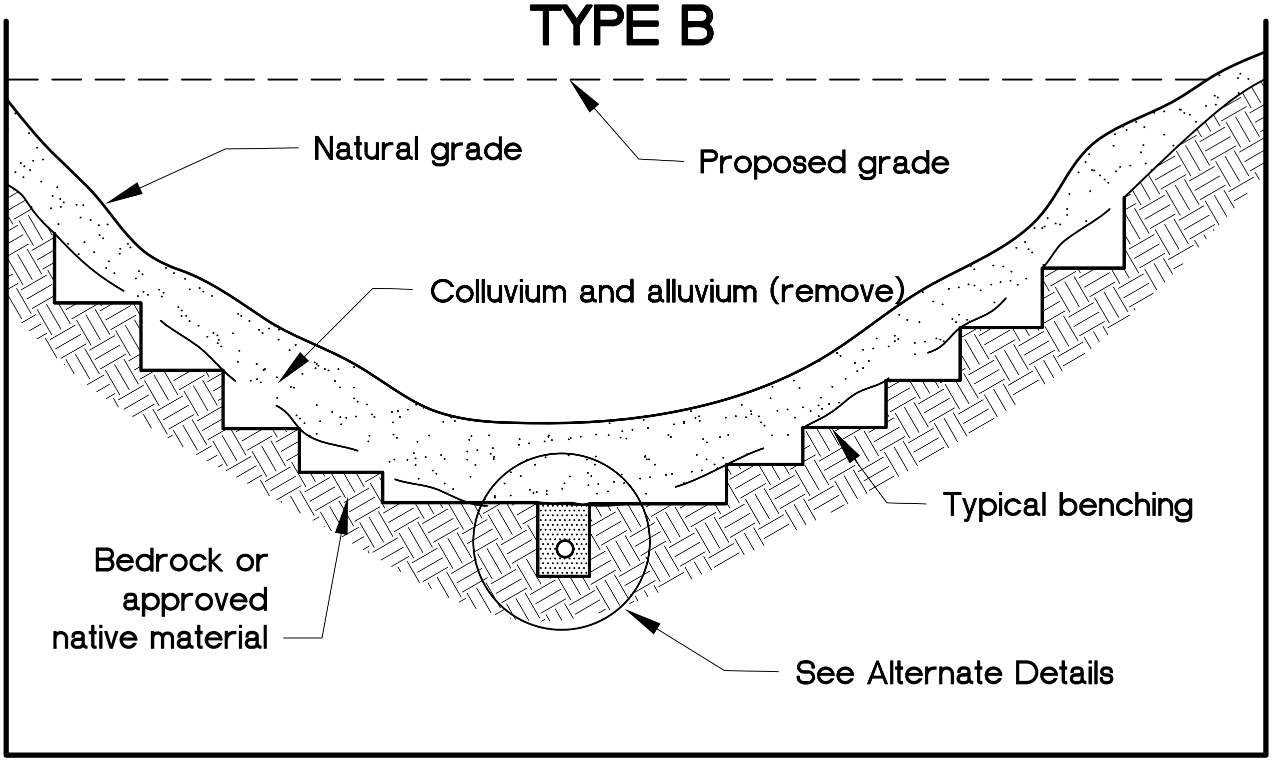
If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraw and notify his/her supervisor. The contractor's representative will be contacted in an effort to affect a solution. All backfill not tested due to safety concerns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

If GSI personnel become aware of anyone working beneath an unsafe trench wall or vertical excavation, we have a legal obligation to put the contractor and owner/developer on notice to immediately correct the situation. If corrective steps are not taken, GSI then has an obligation to notify Cal/OSHA and/or the proper controlling authorities.

TYPE A

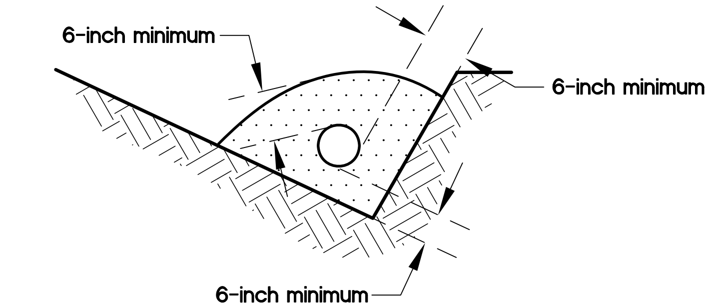


TYPE B

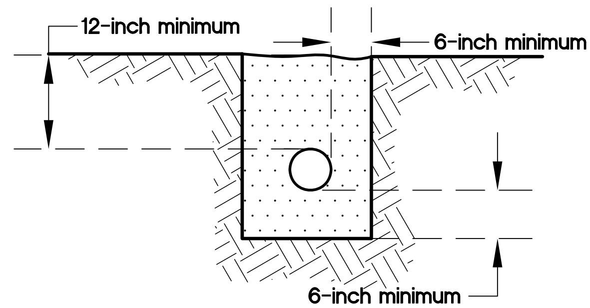


Selection of alternate subdrain details, location, and extent of subdrains should be evaluated by the geotechnical consultant during grading.

GeoSoils, Inc.	CANYON SUBDRAIN DETAIL	Plate F-1



A-1



B-1

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

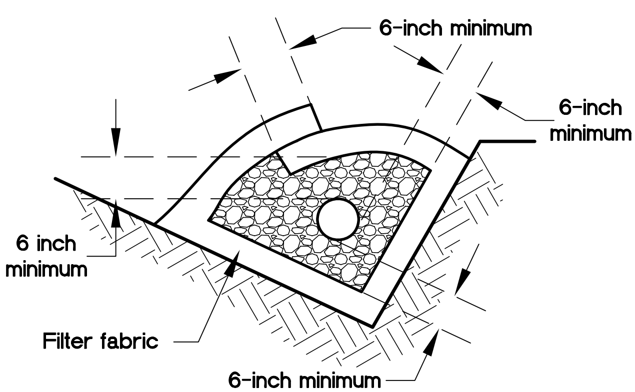
Perforated pipe: 6-inch-diameter ABS or PVC pipe or approved substitute with minimum 8 perforations ($\frac{1}{4}$-inch diameter) per lineal foot in bottom half of pipe (ASTM D-2751, SDR-35, or ASTM D-1527, Schd. 40).

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

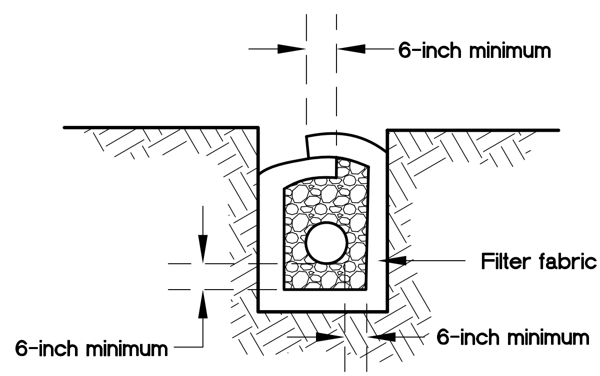
FILTER MATERIAL

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

ALTERNATE 1: PERFORATED PIPE AND FILTER MATERIAL



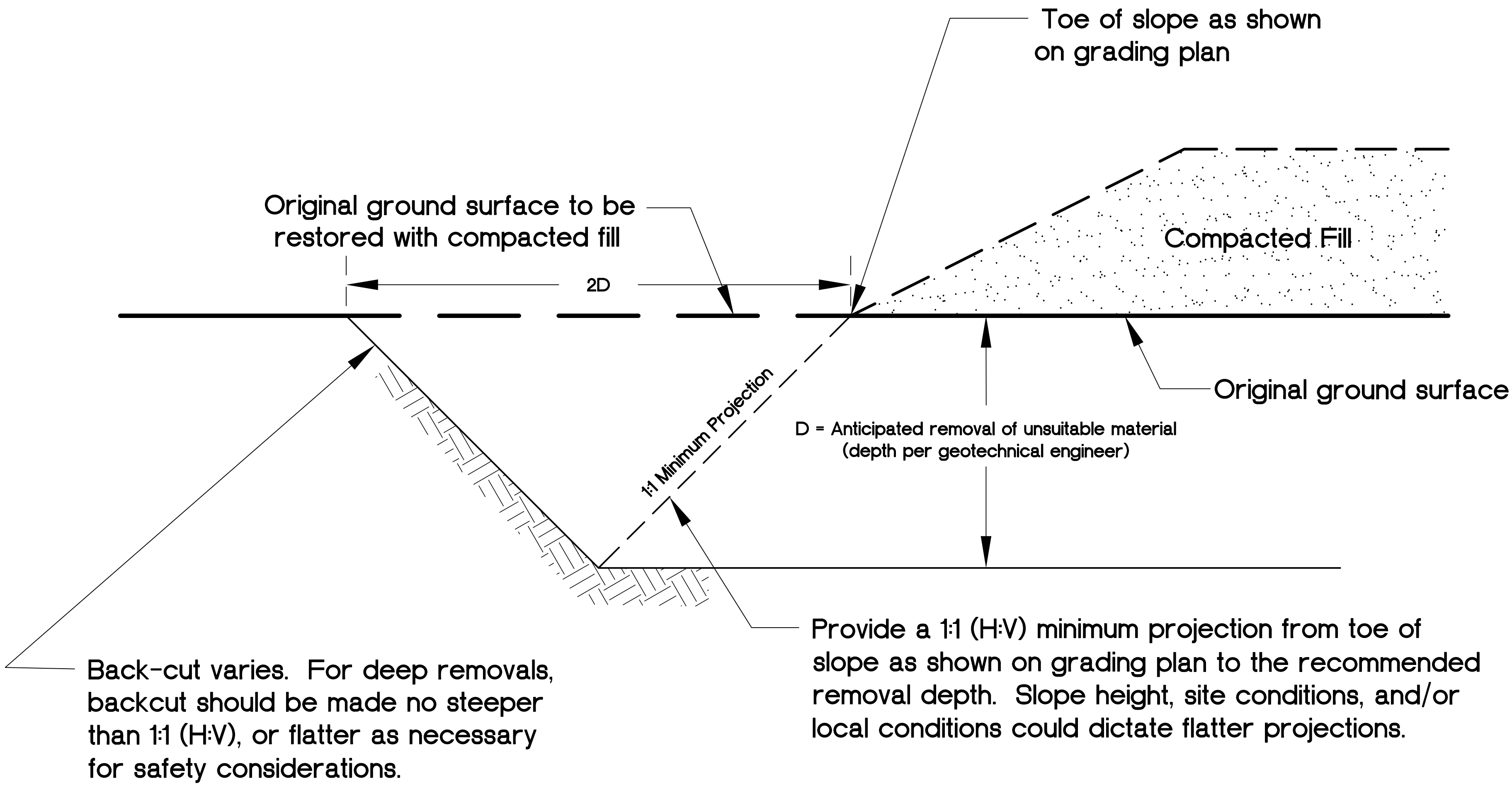
A-2



B-2

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

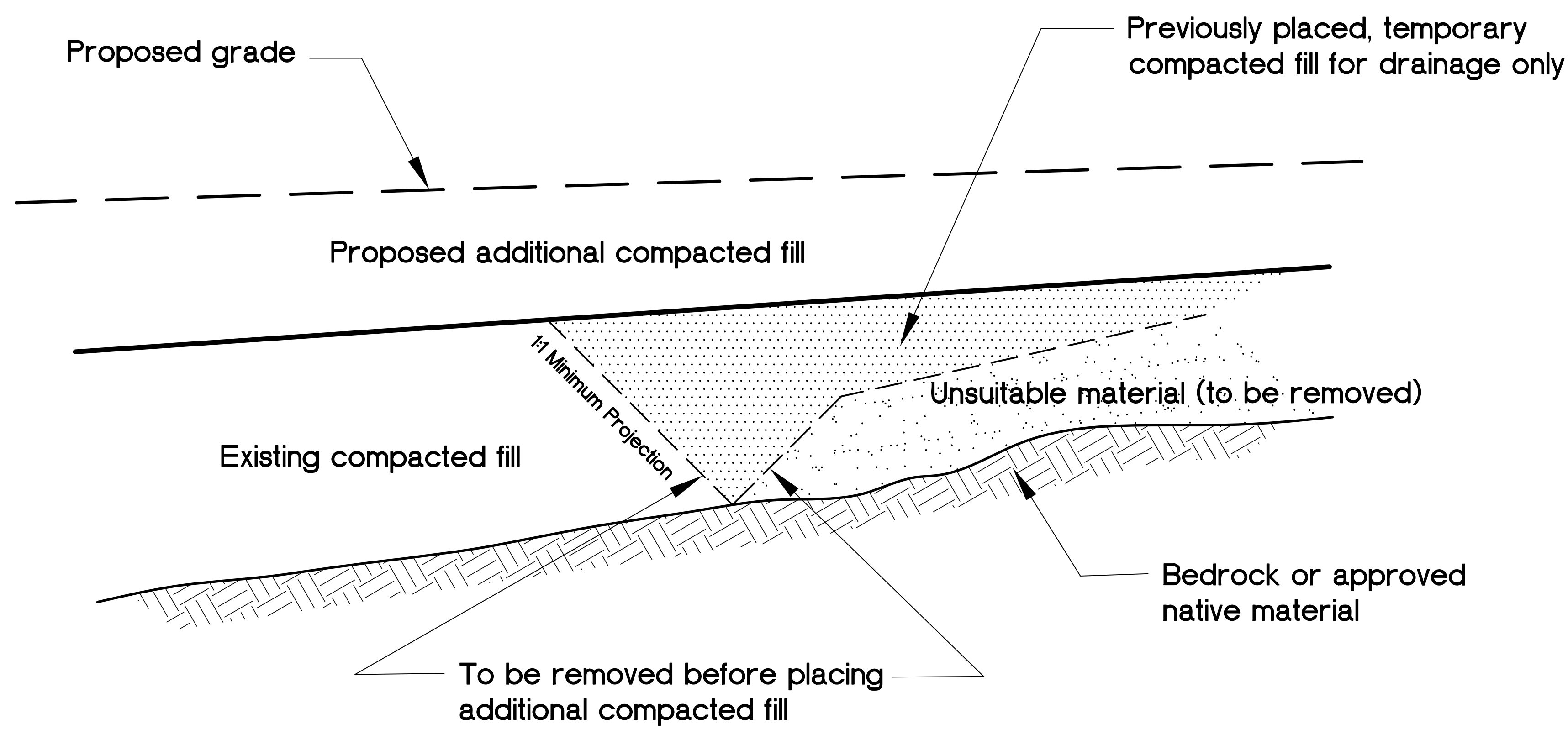
ALTERNATE 2: PERFORATED PIPE, GRAVEL, AND FILTER FABRIC



GeoSoils, Inc.

FILL SLOPE TOEING OUT ON FLAT ALLUVIATED CANYON DETAIL

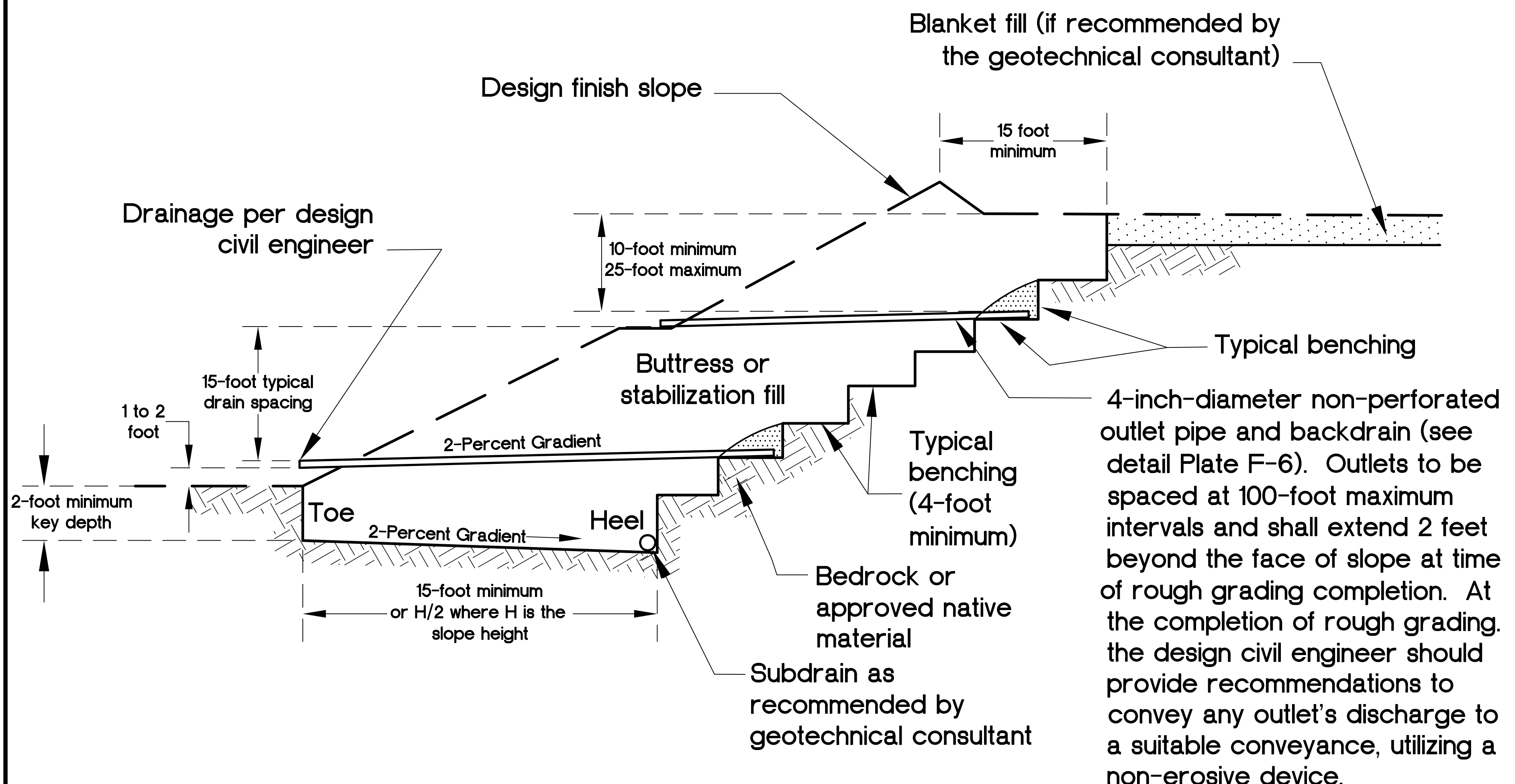
Plate F-3



GeoSoils, Inc.

REMOVAL ADJACENT TO EXISTING FILL ADJOINING CANYON FILL DETAIL

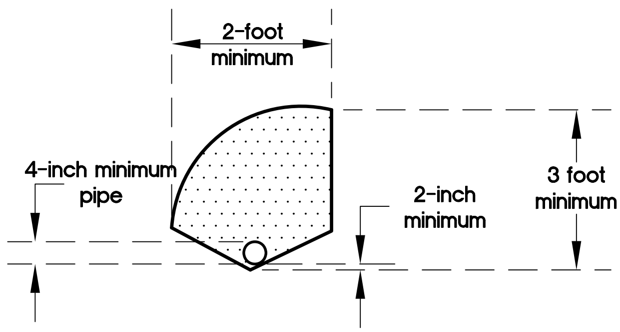
Plate F-4

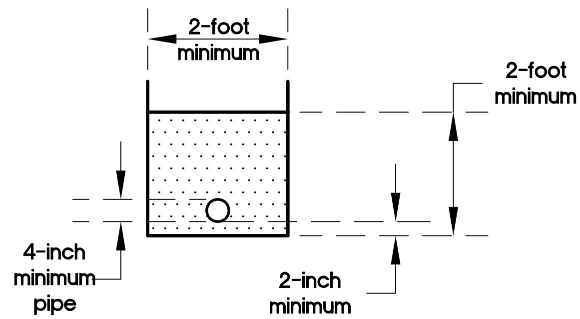


GeoSoils, Inc.

TYPICAL STABILIZATION / BUTTRESS FILL DETAIL

Plate F-5





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

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

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

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

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

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

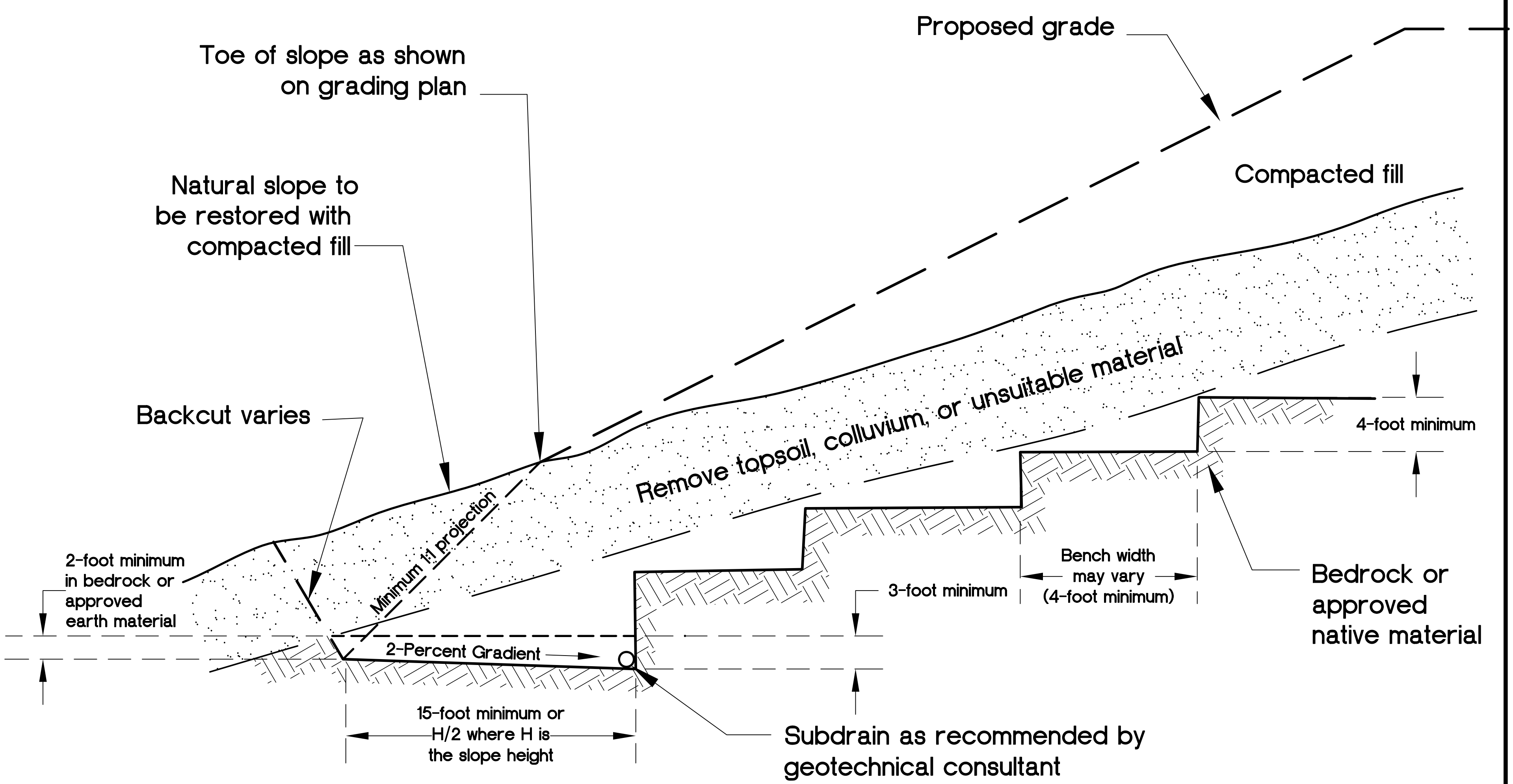
Gravel shall be of the following specification or an approved equivalent.

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

GeoSoils, Inc.

TYPICAL BUTTRESS SUBDRAIN DETAIL

Plate F-6



NOTES:

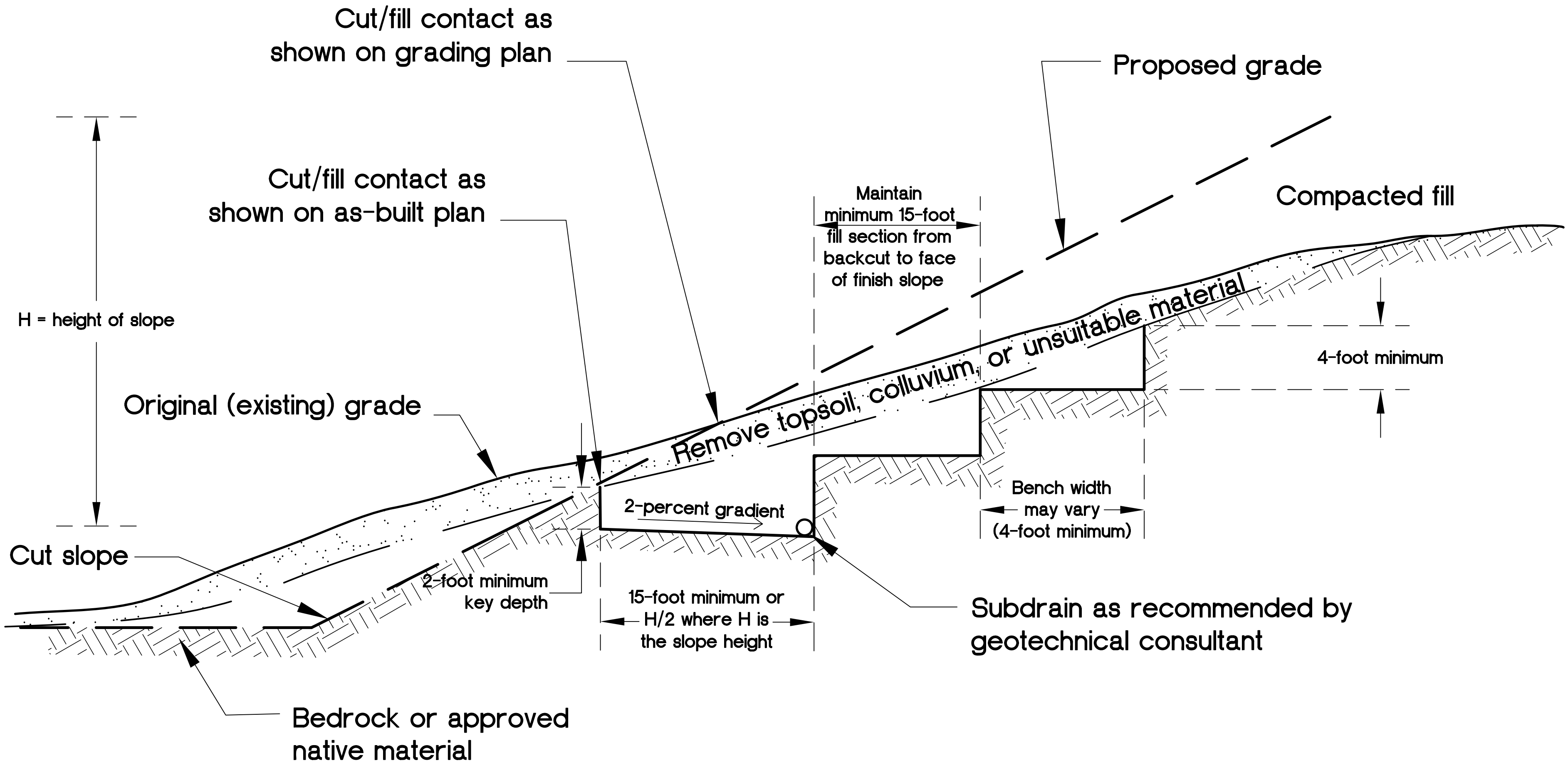
1. Where the natural slope approaches or exceeds the design slope ratio, special recommendations would be provided by the geotechnical consultant.

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

GeoSoils, Inc.

FILL OVER NATURAL (SIDEHILL FILL) DETAIL

Plate F-7

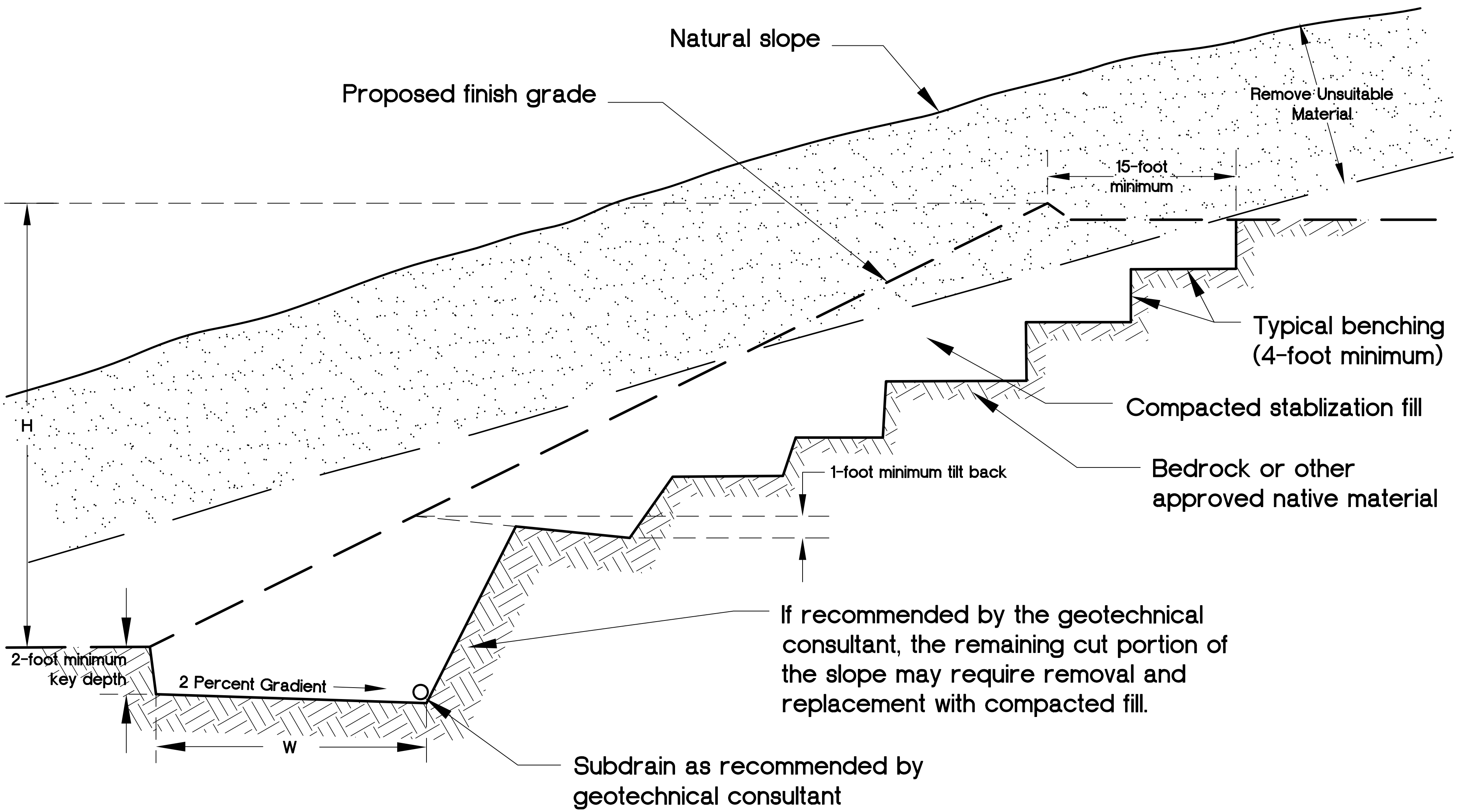


NOTE: The cut portion of the slope should be excavated and evaluated by the geotechnical consultant prior to construction of the fill portion.

GeoSoils, Inc.

FILL OVER CUT DETAIL

Plate F-8

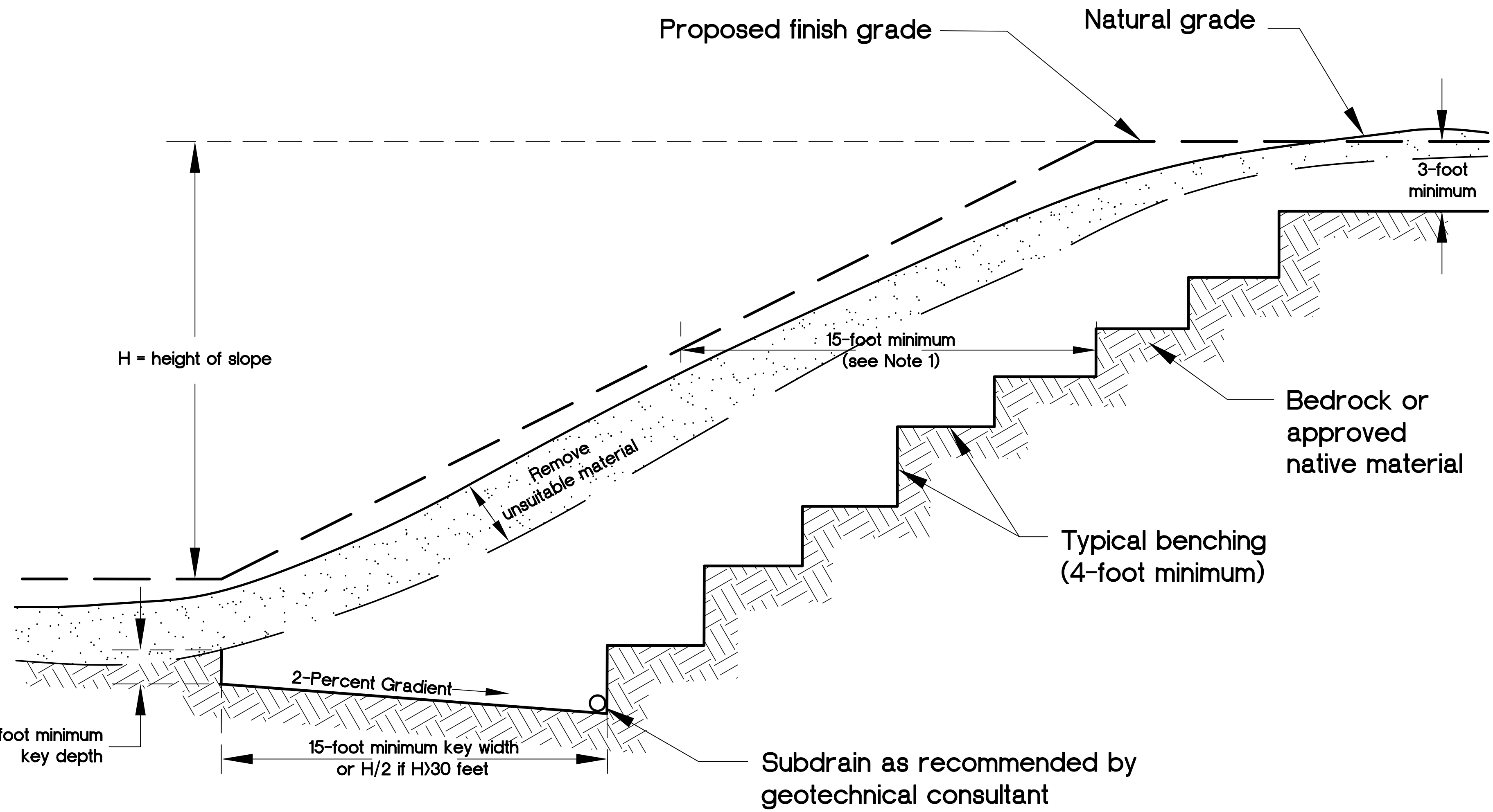


NOTES:

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

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

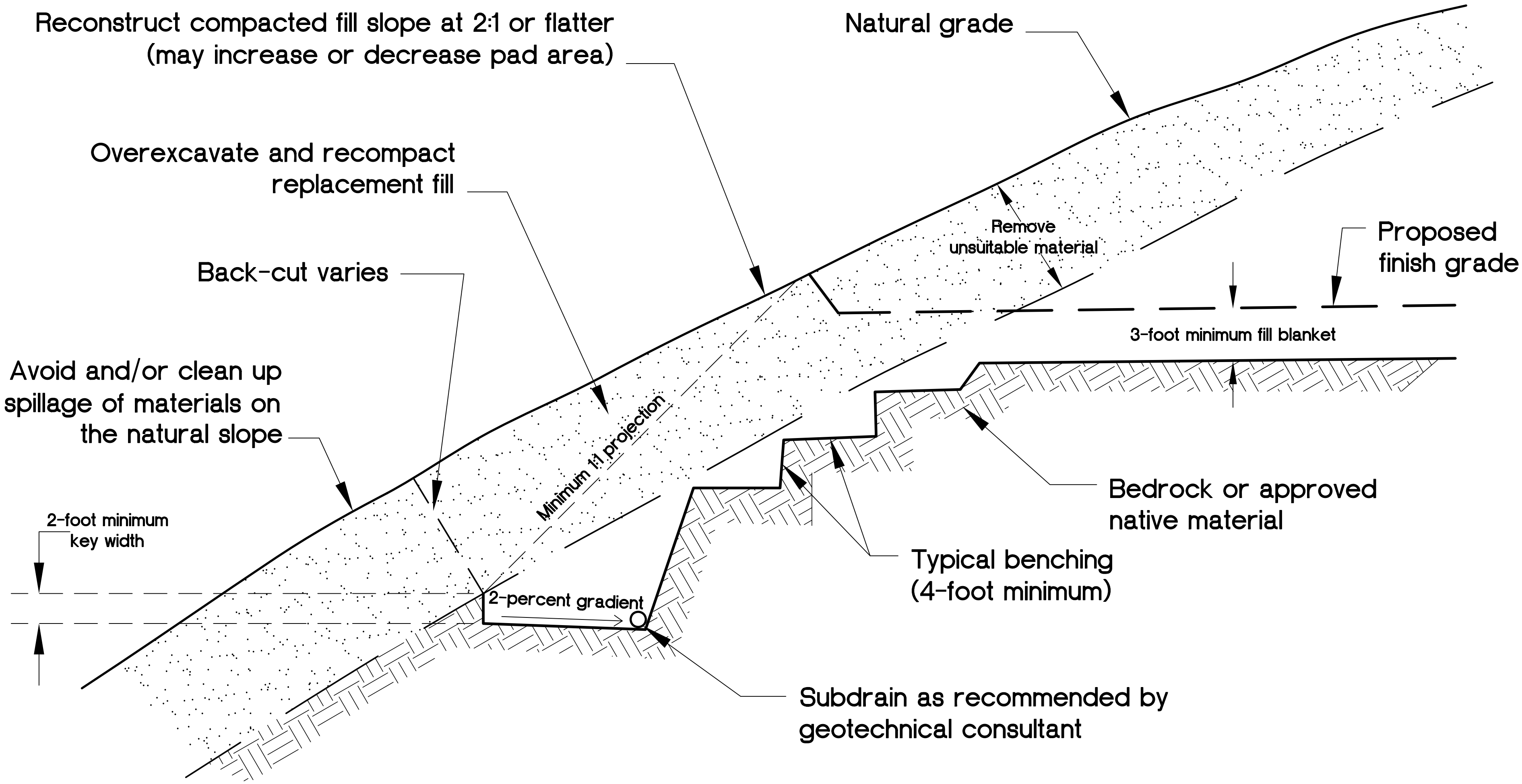
GeoSoils, Inc.	STABLIZATION FILL FOR UNSTABLE MATERIAL EXPOSED IN CUT SLOPE DETAIL	Plate F-9



NOTES:

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

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



NOTES:

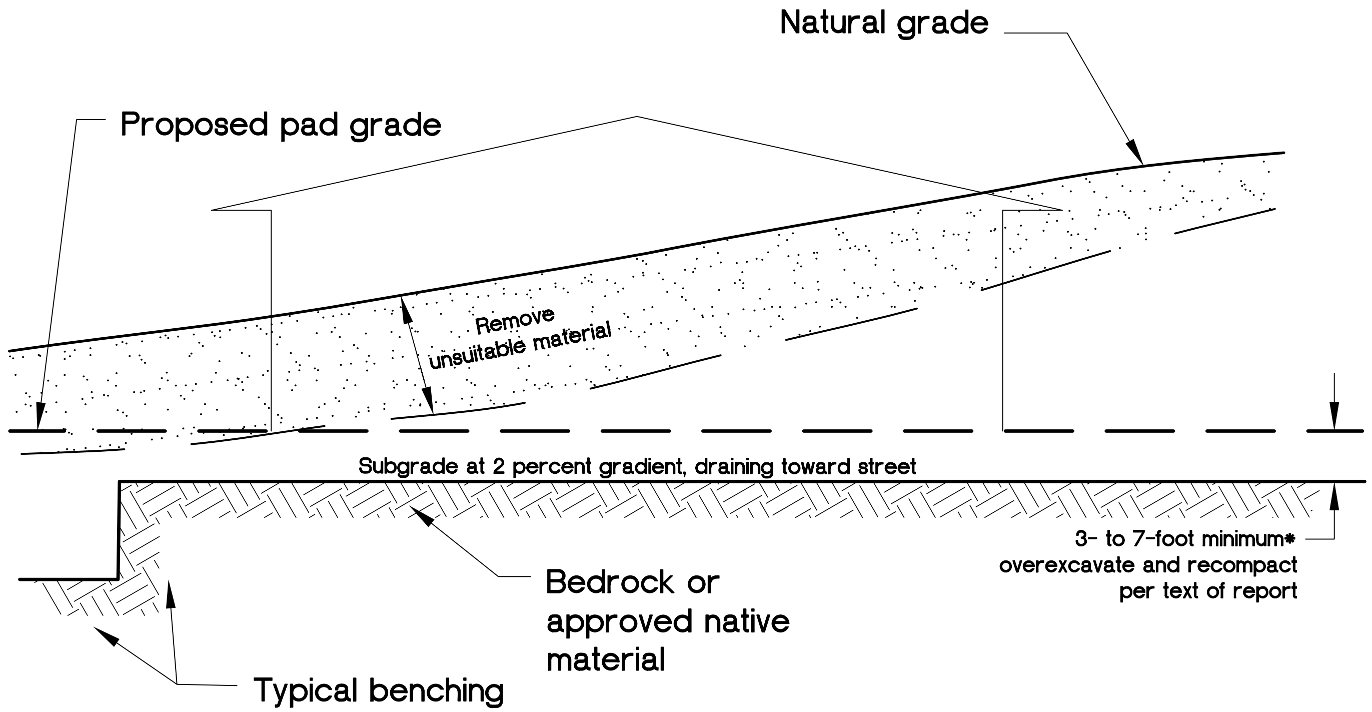
1. Subdrain and key width requirements will be evaluated based on exposed subsurface conditions and thickness of overburden.

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

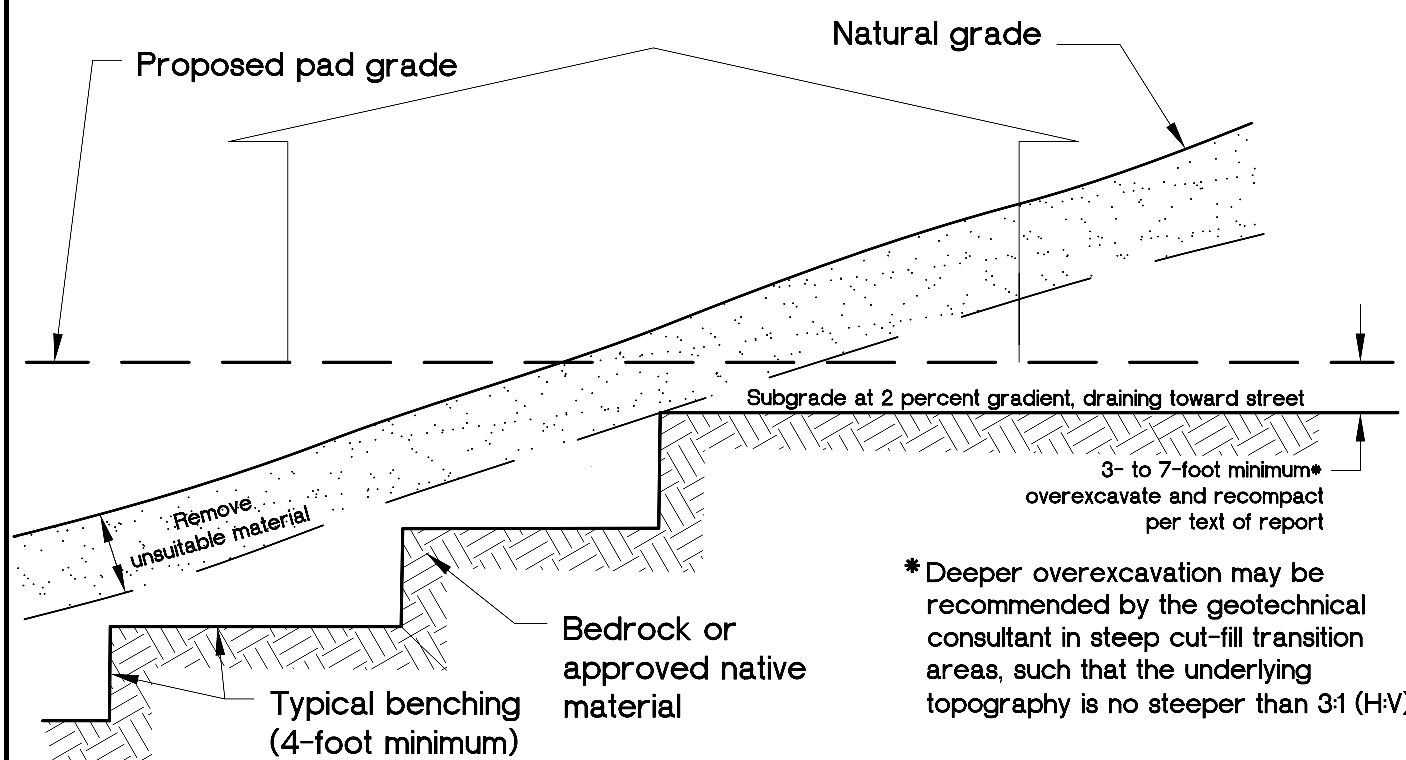
GeoSoils, Inc.

DAYLIGHT CUT LOT DETAIL

Plate F-11



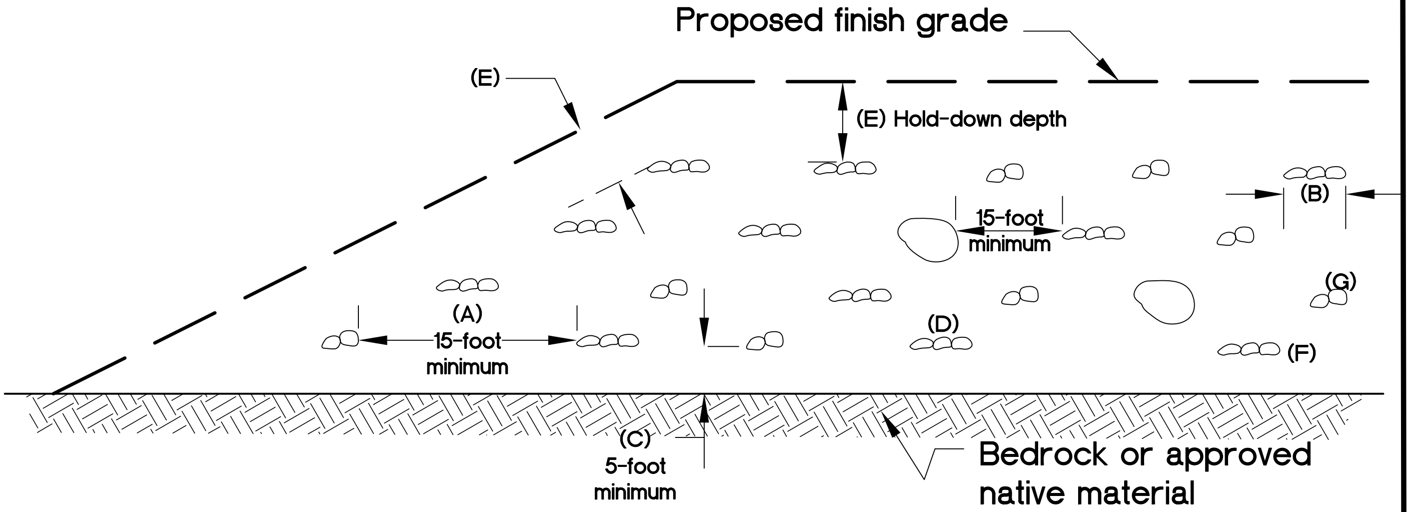
CUT LOT OR MATERIAL-TYPE TRANSITION



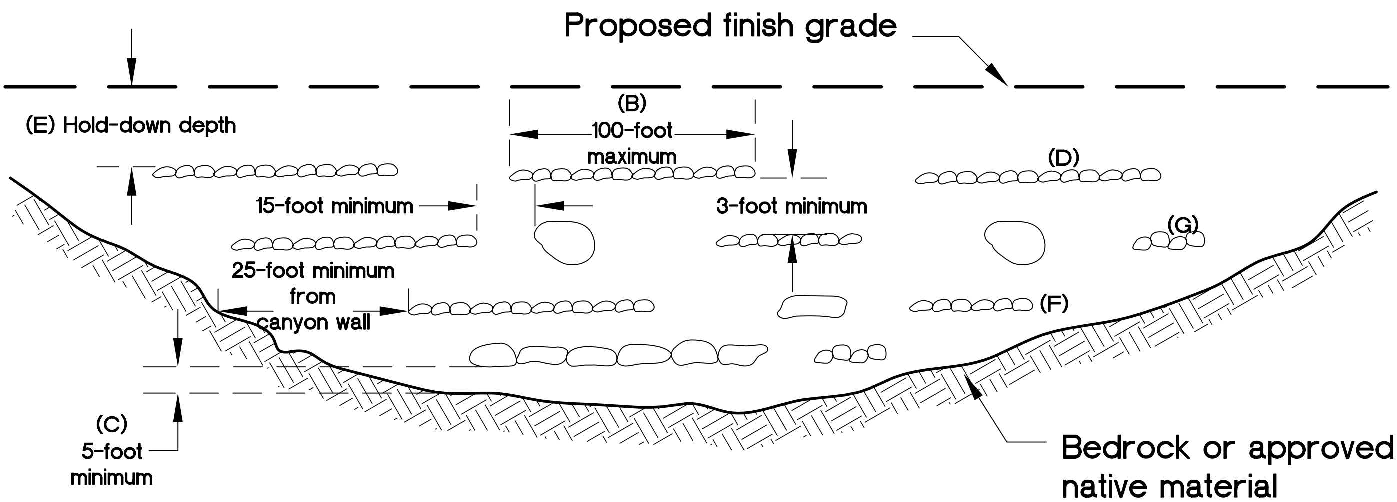
CUT-FILL LOT (DAYLIGHT TRANSITION)

GeoSoils, Inc.	TRANSITION LOT DETAILS	Plate F-12

VIEW NORMAL TO SLOPE FACE



VIEW PARALLEL TO SLOPE FACE



NOTES:

A. One equipment width or a minimum of 15 feet between rows (or windrows).

B. Height and width may vary depending on rock size and type of equipment. Length of windrow shall be no greater than 100 feet.

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

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

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

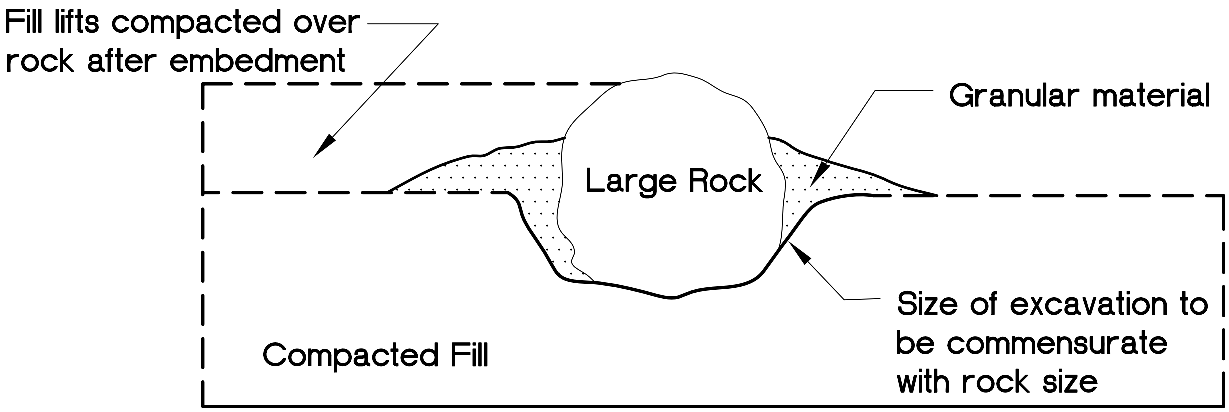
F. All fill over and around rock windrow shall be compacted to at least 90 percent relative compaction or as recommended.

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

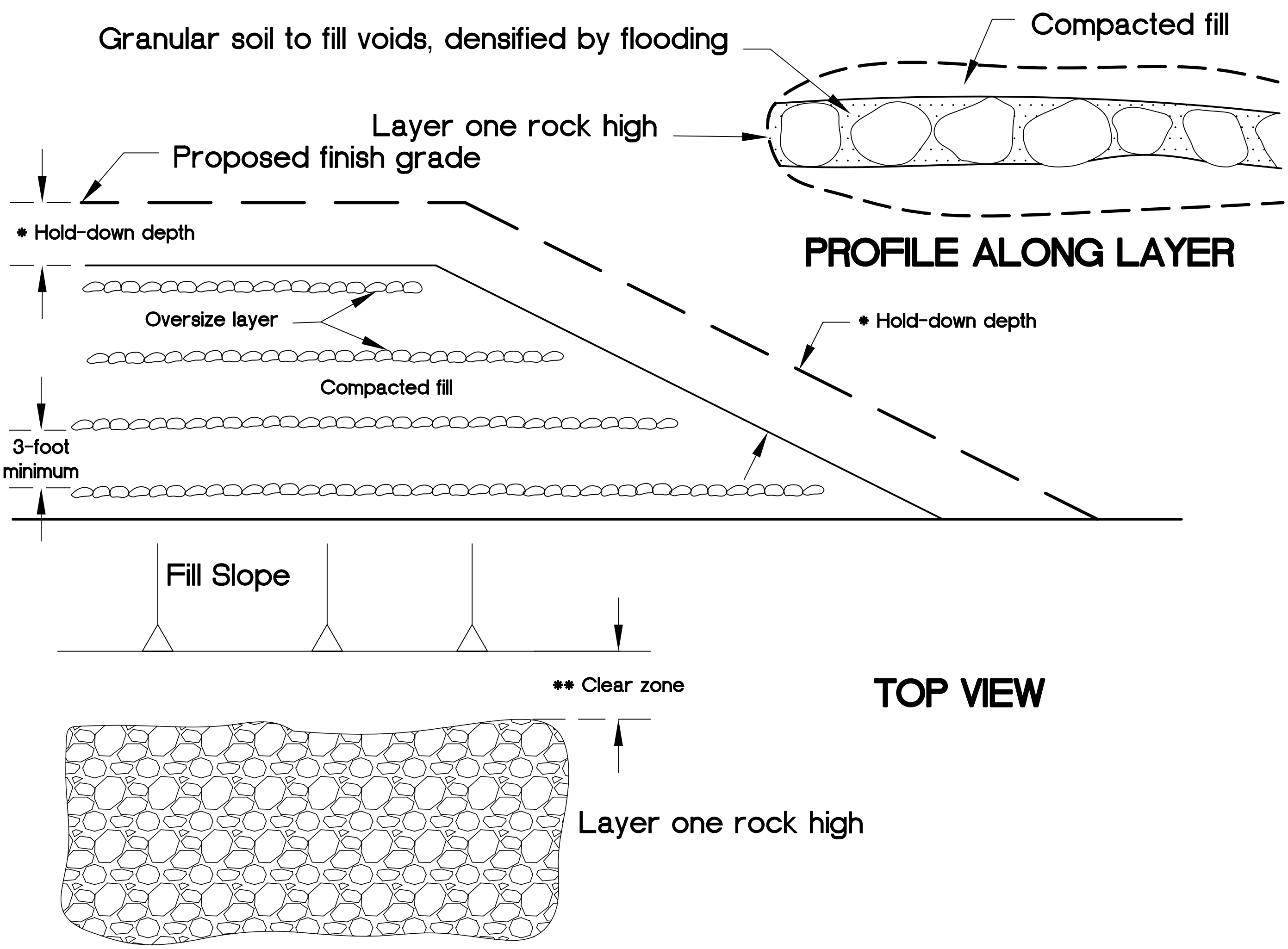
VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE
ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED

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

ROCK DISPOSAL PITS



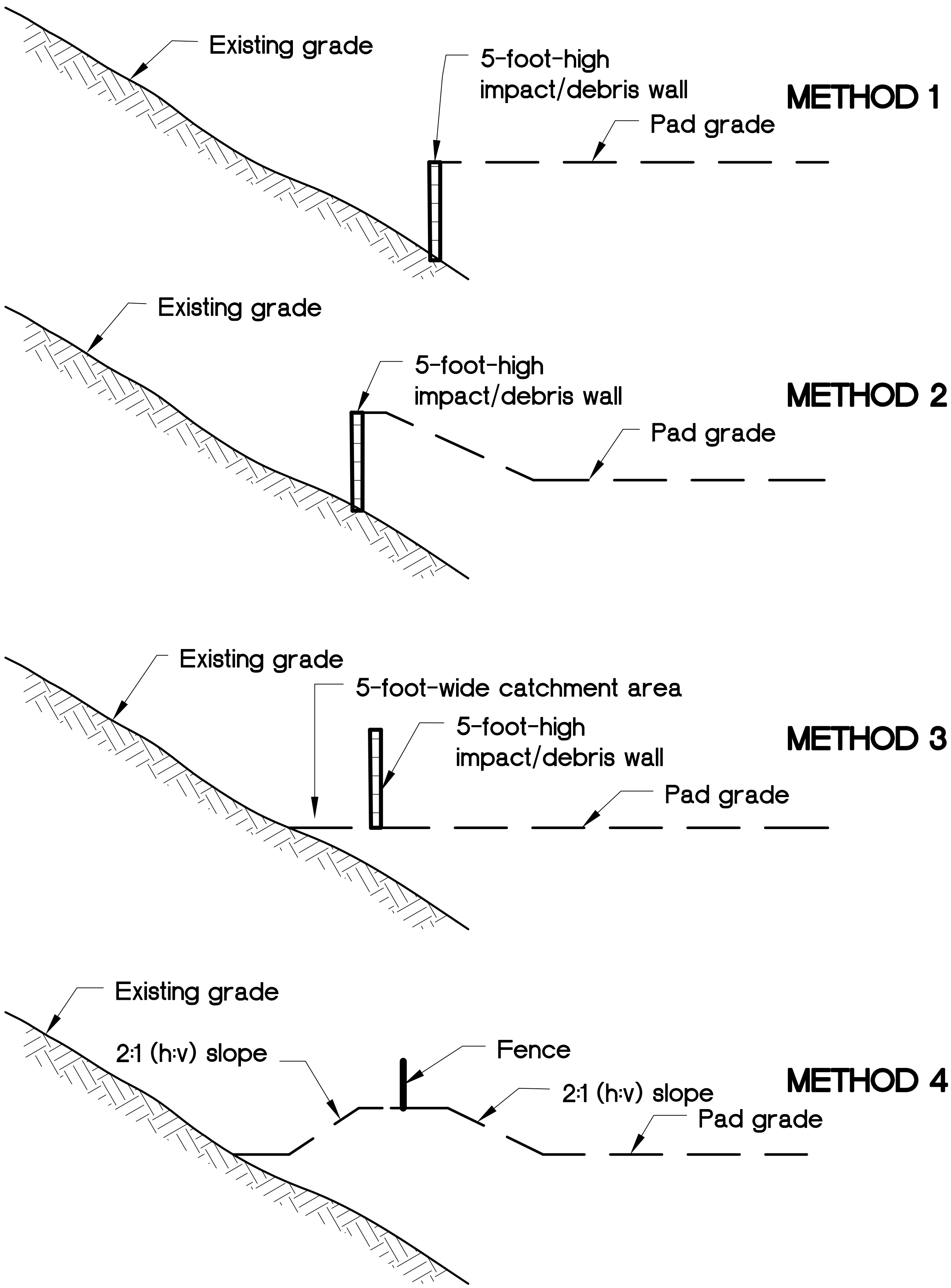
ROCK DISPOSAL LAYERS



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

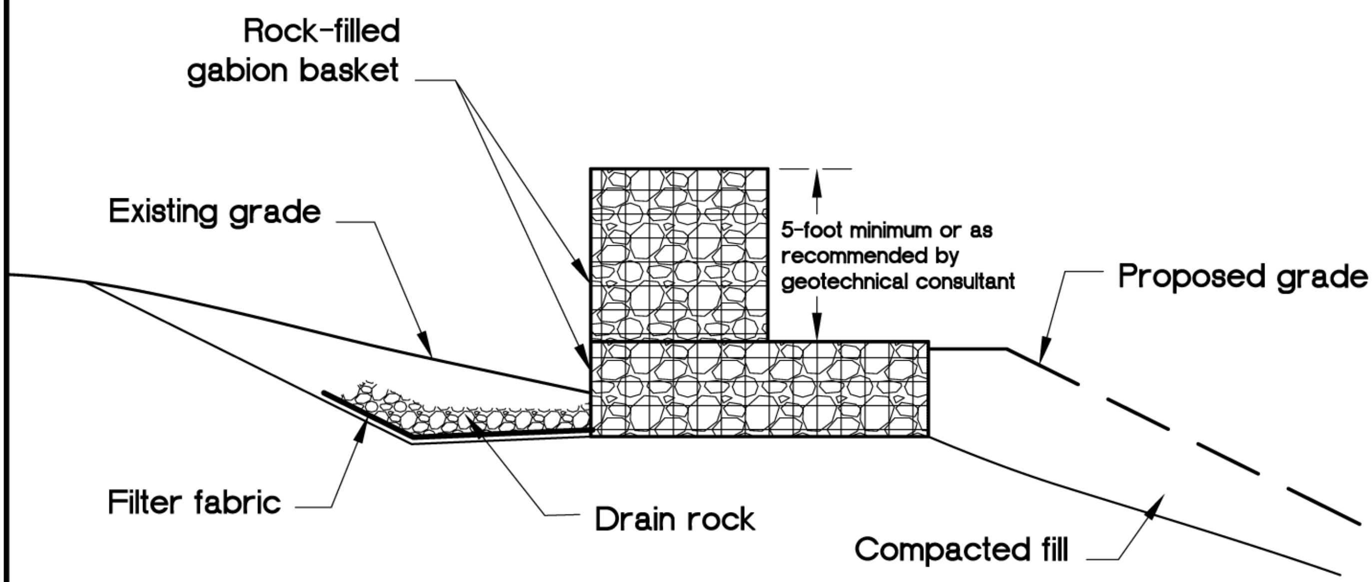
VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE
ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED IN

GeoSoils, Inc.	ROCK DISPOSAL DETAIL	Plate F-14



NOT TO SCALE

GeoSoils, Inc.	DEBRIS DEVICE CONTROL METHODS DETAIL	Plate F-15



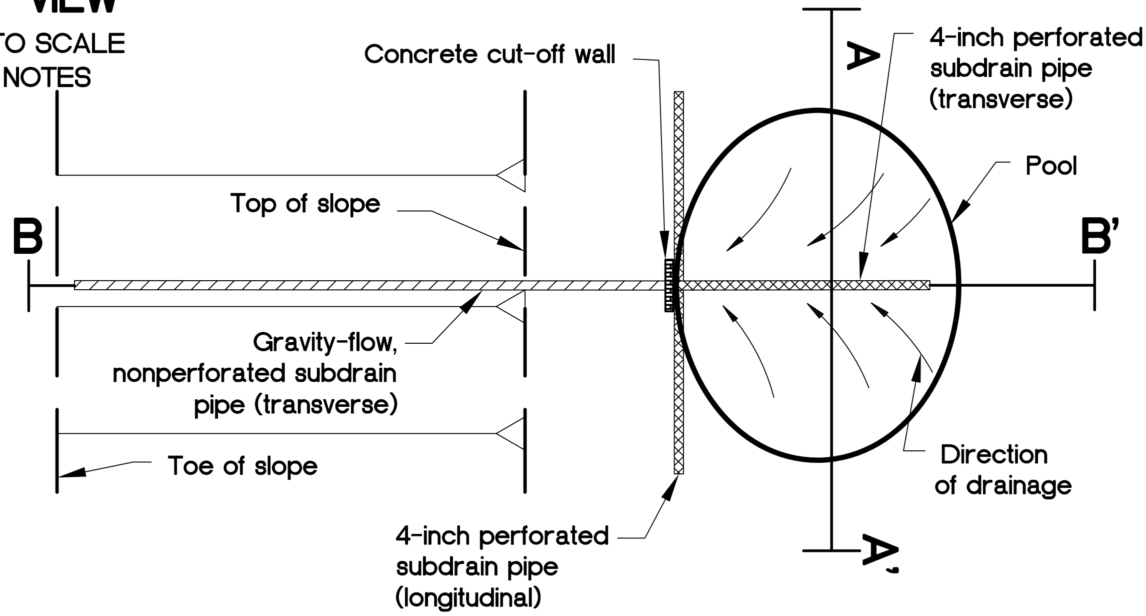
Gabion impact or diversion wall should be constructed at the base of the ascending slope subject to rock fall. Walls need to be constructed with high segments that sustain impact and mitigate potential for overtopping, and low segment that provides channelization of sediments and debris to desired depositional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

From GSA, 1987

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

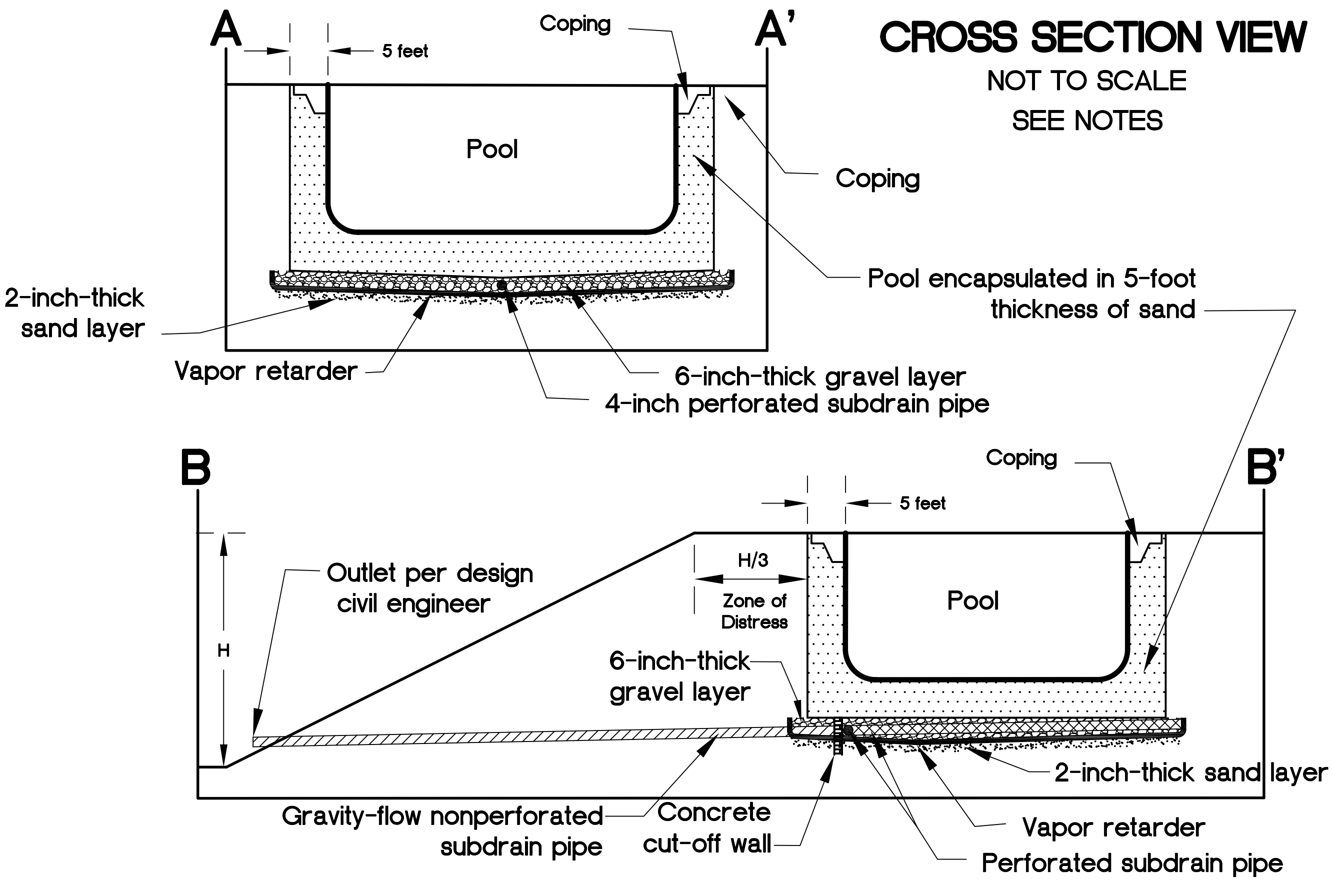
MAP VIEW

NOT TO SCALE
SEE NOTES



CROSS SECTION VIEW

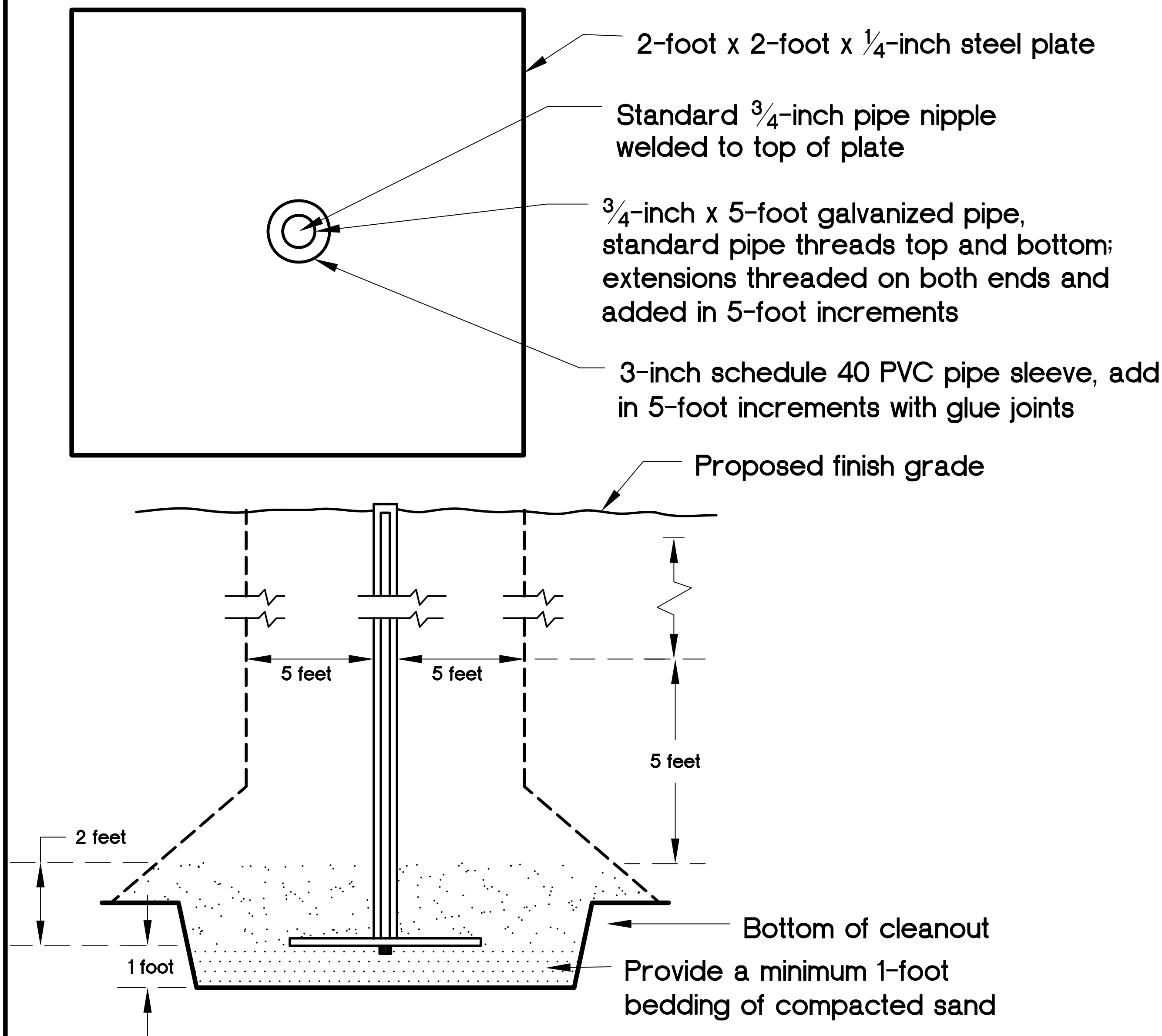
NOT TO SCALE
SEE NOTES



NOTES:

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

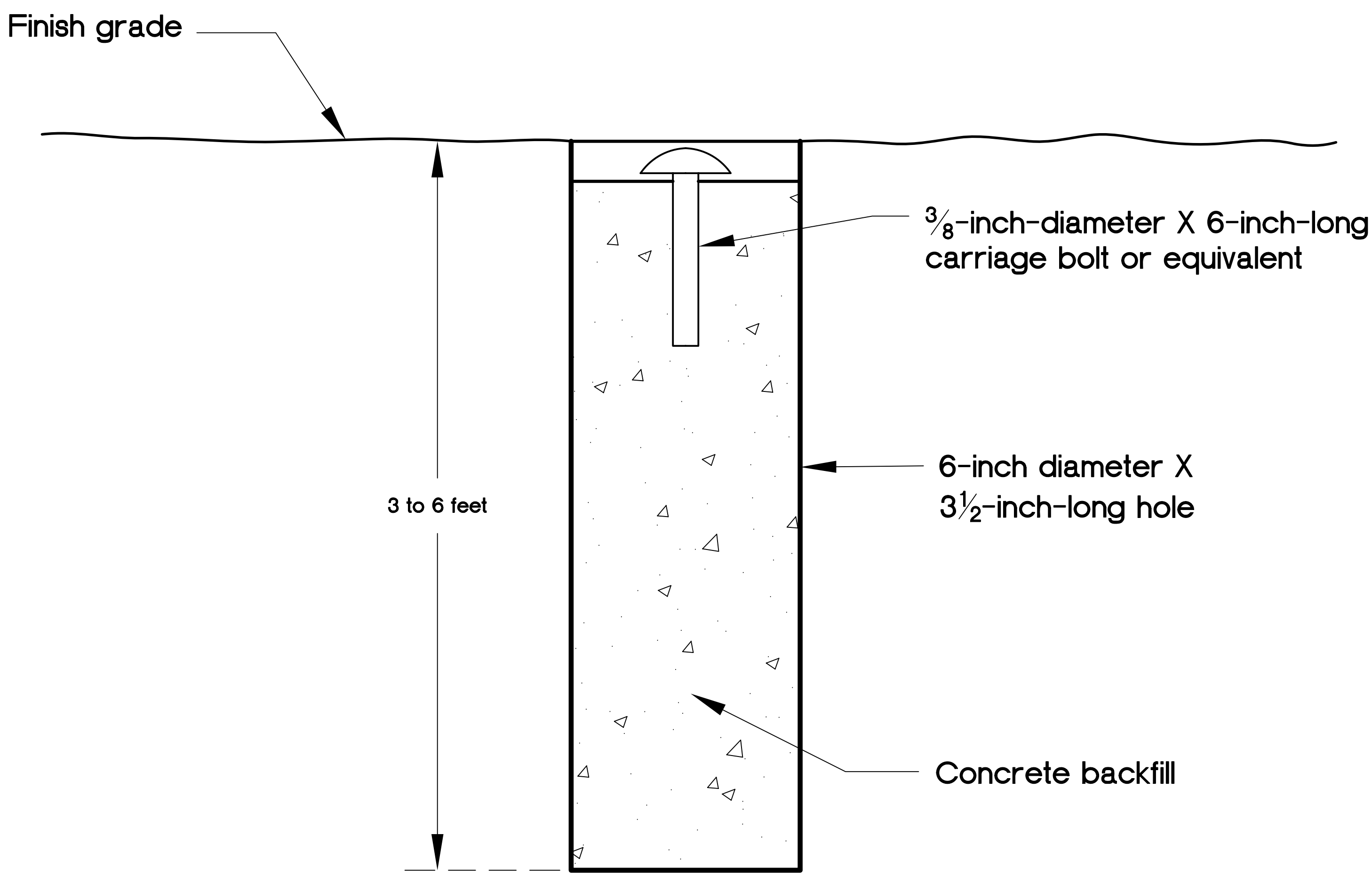
GeoSoils, Inc.	TYPICAL POOL/SPA DETAIL	Plate F-17



NOTES:

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

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

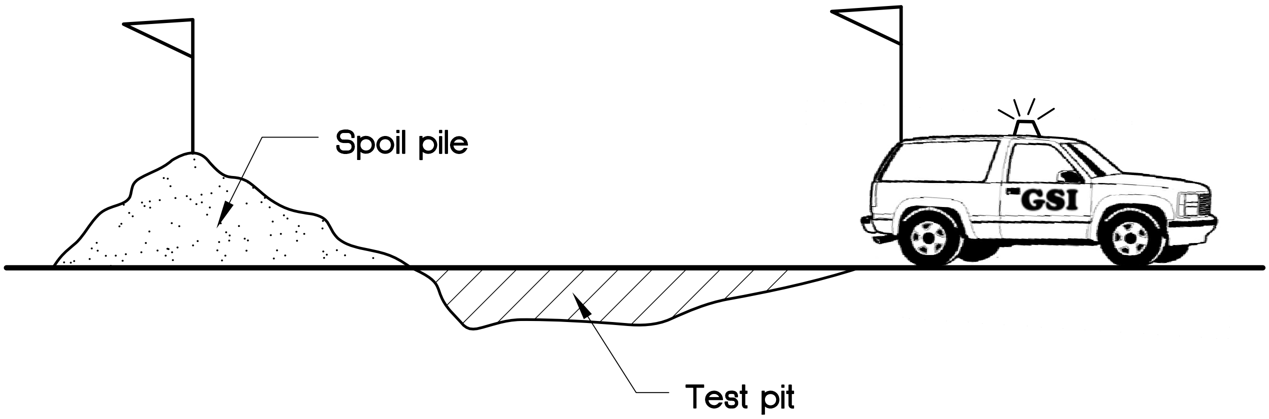


GeoSoils, Inc.

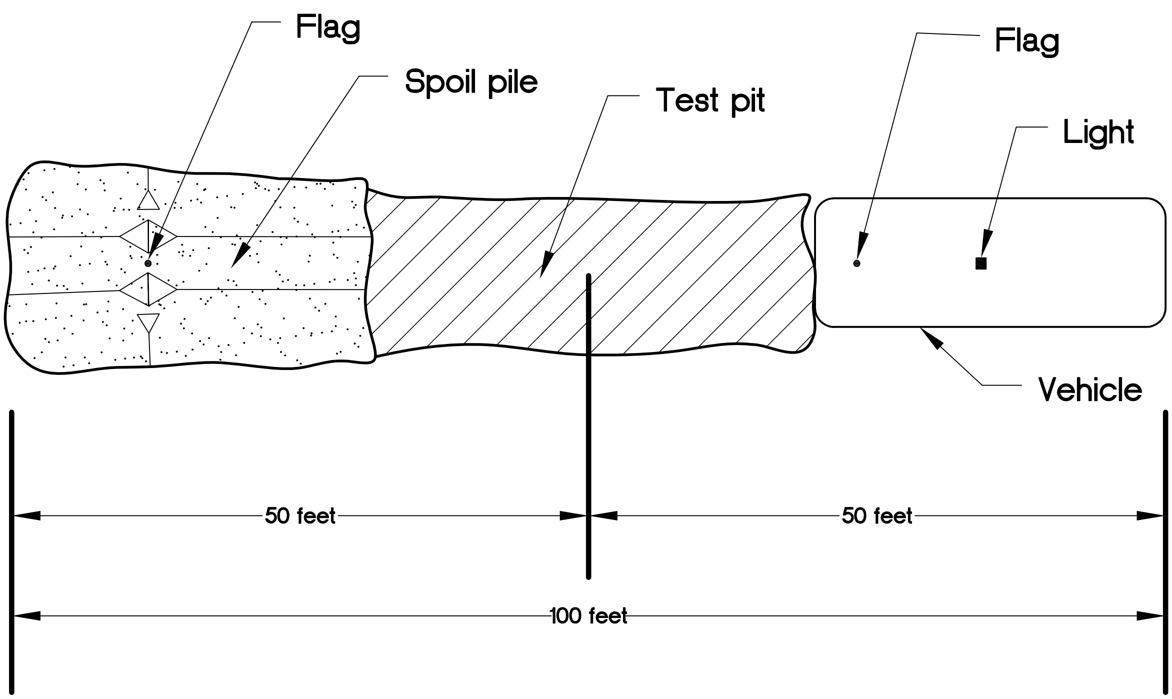
TYPICAL SURFACE SETTLEMENT MONUMENT

Plate F-19

SIDE VIEW



TOP VIEW



GeoSoils, Inc.	TEST PIT SAFETY DIAGRAM	Plate F-20