PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED SKILLED NURSING FACILITY
675 E BRADLEY AVENUE, EL CAJON
SAN DIEGO COUNTY, CALIFORNIA 92021
ASSESSOR'S PARCEL NUMBER (APN) 387-142-36-00

FOR

ARCO CONSTRUCTION COMPANY, INC.
900 N. ROCK HILL ROAD
ST. LOUIS, MISSOURI 63119

W.O. 8096-A-SC **JULY 8, 2021**

SDC PDS RCVD 09-24-24
MUP85-053W2

GeoSoils, Inc.

Geotechnical • Geologic • Coastal • Environmental

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July 8, 2021

W.O. 8096-A-SC

ARCO Construction Company, Inc.
900 N. Rock Hill Road
St. Louis, Missouri 63119

Attention: Mr. Steve L'Hommedieu, Senior Project Manager

Subject: Preliminary Geotechnical Investigation, Proposed Skilled Nursing Facility, 675 E Bradley Avenue, El Cajon, San Diego County, California 92021, Assessor's Parcel Number (APN) 387-142-36-00

Dear Mr. L'Hommedieu:

In accordance with your request, GeoSoils, Inc. (GSI), has performed a preliminary geotechnical investigation of the subject site (see Figure 1, Site Location Map). The purpose of our investigation was to evaluate the subsurface geologic and general geotechnical conditions of the site, relative to the proposed development, and to present recommendations for grading and foundation design, and construction from a geotechnical viewpoint.

EXECUTIVE SUMMARY

Based on our review of the available data (see Appendix A), field exploration, laboratory testing, geologic and engineering analysis, as well as our experience on nearby sites, the proposed development of the property appears to be feasible from a geotechnical viewpoint, provided the recommendations presented in the text of this report are properly incorporated into the design and construction of the project. The site (site, building 2) of building no. 2 is within the southern portion of the property (Figure 1) and will be the focus of this report. Based on our report and past evaluations, the most significant elements of this study are summarized below:

- Based on our evaluation, the site was graded in the past, and likely contains a discontinuous, relatively thin mantle of undocumented artificial fill, primarily associated with the existing structure and improvements (see the attached Geotechnical Map [Plate 1]). The remainder of the site generally appears to be native, or previously cut, and mantled by Quaternary-age colluvium, which in turn is underlain, and outcropped in the south east corner, by Cretaceous-age granitic bedrock (tonalite in composition). In areas that were previously cut during grading, weathered granitic bedrock was observed at the surface and becomes less weathered and more dense with depth, based on seismic refraction survey line data across the building pad.

- Based on our review of 1949, 1953, and 1964 historical aerial photographs obtained from Environmental Data Resources, Inc. (EDR), grading of the site and development of the major existing structures appears to have occurred sometime between 1953 and 1964. Prior to this stage of development, the site appears to have been used for dry farming from at least 1949 to 1953. Based on the available aerial photography, the site has likely received only one stage of grading (during 1953-1964), and the majority of existing onsite structures were likely built prior to 1965.

- GSI understands that the proposed development will include the construction of a new skilled nursing facility and a new concrete generator pad, along with associated underground utilities, landscaping/hardscaping, and an access route for the trash enclosure. The existing building on the east side of the site will remain.

- Soils considered unsuitable for the support of proposed settlement-sensitive improvements (i.e., buildings, walls, underground utilities, pavements, etc.) and new planned fills consist of all undocumented artificial fill, Quaternary-age colluvium, and any highly weathered bedrock (if encountered). For remedial treatment, these earth materials should be removed to expose the underlying granitic bedrock and may be reused as engineered fill, per the recommendations contained herein. Based on the available subsurface data, remedial grading excavations are anticipated to range between approximately 2½ and 3 feet below existing grades (B.E.G.), but may be deeper, locally. Relatively unweathered granitic bedrock is considered suitable bearing material for supporting the proposed improvements and new planned fills in its existing state.

- It should be noted that the 2019 CBC (CBSC, 2019) indicates that removals of unsuitable soils be performed across all areas to be graded, under the purview of the grading permit, and not just within the influence of the proposed facility building. Relatively deep removals may also necessitate a special zone of consideration, on perimeter/confining areas. Grading will be limited in the zone adjacent to the existing building. This zone would be approximately equal to the depth of removals, if removals cannot be performed onsite or offsite. In general, any planned settlement-sensitive improvement located above a 1:1 (h:v) projection up from the bottom, outboard edges of the remedial grading excavations at the property boundaries would be affected by perimeter conditions. On a preliminary basis, any planned settlement-sensitive improvements located within approximately 2½ and 3 feet from the property boundaries would require deepened foundations or additional reinforcement by means of ground improvement or specific structural design. Otherwise, these improvements may be subject to distress and a reduced service life.

- Subsurface water, including seeps or springs, was not encountered within the property during field work performed in preparation of this report. However, a review of the groundwater monitoring report performed by Kahl Environmental

Services ([KES], 2007) appears to show groundwater occurring on an adjacent site at elevations ranging from 431.0 feet MSL to 434.0 feet MSL. As such, groundwater is anticipated to occur at depths greater than 25 feet below the lowest existing grade on the property, and at least 30 feet below the proposed Building 2 floor elevation. Regional groundwater is not expected to be a major factor in the currently proposed site development, provided that the recommendations presented in this report are incorporated into the design and construction of the project. However, the potential for perched water conditions to develop both during and following site development cannot be precluded and should be anticipated. Perched water would likely manifest along zones of contrasting permeabilities/densities (i.e., along fill lifts, fill/bedrock contacts, etc.) and along discontinuities within the granitic bedrock, at any elevation or depth. Likely origins of perched water may include infiltrated precipitation and irrigation waters, broken or damaged wet underground utilities, or water sources.

- The presence of landslide deposits, scarps, slumps, hummocky terrain, or other indications of significant mass wasting were not observed within the existing building pad area or along the north facing slope descending from the existing pad area. In addition, our document and air photo review as well as subsurface investigation did not reveal evidence of deep-seated instability at the subject site. Owing to the erodible nature of the onsite earth materials, a properly designed and maintained surface drainage plan is recommended, as indicated herein. Some nested oversized granitic rock was present in the extreme southeast corner of the site, but will likely not affect the proposed building 2 development.

- Holocene (active) faults at the subject site were not noted during our document review nor during our site exploration. Therefore, the potential for surface rupture to affect the proposed development is considered low. However, due to the proximity of the site to regional fault strands exhibiting movement within the last 11,700 years, the proposed development is subject to moderate to strong ground shaking should an earthquake occur on any of the active faults within the region. Therefore, the herein presented seismic acceleration values and design parameters should be incorporated into the proposed project. The potential for liquefaction, or densification to adversely affect this site following development is considered very low.

- The susceptibility of the site to experience damaging deformations from adverse settlement, or seismically-induced densification will be mitigated to acceptable levels based on the existing site geology and the recommended earthwork presented herein. The recommendations for foundation design and construction should further mitigate significant settlement potential. Some surface manifestations of shrink/swell effects from the expansive soils, noted onsite, may occur over the life of the project, and are discussed herein.

- Expansion Index (E.I.) testing performed on a sample of the onsite soils generally indicates very low expansive soil conditions (E.I. of <21), for soils derived from weathered bedrock. However, it is possible that soils with higher expansion potentials may be encountered locally. Based on the distribution of soils, including colluvium and highly weathered bedrock, etc., blending/mixing of any minor amounts of expansive soil may be performed during grading to reduce the overall expansive character of site soil. Thus, in general, surficial deposits of fill/colluvium and granitic bedrock are generally not considered detrimentally expansive.

- Representative samples of site earth materials were evaluated for corrosion potential, soluble sulfates, and soluble chlorides. Laboratory testing indicates that the tested samples are mildly alkaline with respect to soil acidity/alkalinity; are mildly corrosive to exposed, buried metals when saturated; present elevated sulfate exposure to concrete ("Exposure Class S1" per Table 19.3.1.1 of American Concrete Institute (ACI) 318-14); and have slightly elevated concentrations of soluble chlorides. It should be noted that GSI does not consult in the field of corrosion engineering. Therefore, additional comments and recommendations may be obtained from a qualified corrosion engineer based on the level of corrosion protection required for the project, as determined by the Project Architect, Civil Engineer, and/or Structural Engineer.

- As observed during our investigation, the granitic bedrock contains corestones at the surface in the southeast corner of the site, and additional corestones and boulders within the subsurface cannot be precluded. These conditions imply that some level of screening will be needed during grading so that these earth materials can be used as underground utility trench backfill and under foundations. If oversized granitic corestones cannot be readily reduced onsite, a rock disposal area can be established onsite, and/or oversized fill material can be disposed of within parking areas (outside of underground utility corridors) or in landscape areas, if incorporated into planned and remedial fills.

- A moderate to high amount of difficulty was encountered when excavating the test pits with a rubber-tire backhoe. Thus, on a preliminary basis, GSI anticipates productive ripping with a Caterpillar D-9R bulldozer (or equivalent) and productive trenching with an equivalent excavator to depths of about ±3 to ±5 feet B.E.G. This assumes excavation equipment in good working order. It should be noted that non-productive excavation into the granitic bedrock may occur below this depth. Thus, all excavation equipment should be appropriately sized and powered for the required excavation task. Zones of dense granitic bedrock could require the use of rock breaking equipment such as hoe rams. Blasting should not be considered on this site due to the surrounding development and adjacent, offsite retaining wall on the southwest corner of the site.

- Retaining wall design and construction recommendations are provided herein. While most onsite soils may be suitable for retaining wall backfill, this would require testing and stockpiling of soils during grading. In this regard, the generation of wall backfill materials from the onsite soils could potentially cause construction delays and increased cost. As such, the importation of some select backfill may be necessary.

- Recommendations for Portland cement concrete (PCC) and asphaltic concrete (AC) pavements are provided herein. Due to the relatively sandy nature of the near-surface soils onsite (i.e., low to moderate clay/silt content), pavement subgrades composed of these earth materials are anticipated to perform adequately under traffic loads (i.e., generally moderate to high subgrade resistance values [R-values]). Therefore, County of San Diego minimum pavement sections should be anticipated.

- Graded slopes are generally anticipated to be stable, assuming proper construction, maintenance, and normal climatic conditions. Improvements within the Code setback zone for foundations of H/3 (where H is the height of the slope), may be subject to creep and associated distress, unless such improvements utilize either deepened foundations and/or ground improvements, as discussed herein.

- Owing to the relatively shallow, indurated, and dense nature of the granitic bedrock, as well as the poor hydraulic conductivity rating offered by the USDA for onsite soils, permanent storm water Best Management Practices (BMPs)/Low Impact Development (LID) devices that rely on infiltration into the onsite earth materials have a high potential to contribute to perched water conditions and possibly slope instability, as well as adverse affects on planned onsite, and existing offsite improvements. Therefore, storm water infiltration into the onsite soils is not recommended.

- Existing site improvements that are to remain in service and underlain by left-in-place undocumented fill will retain the potential to undergo settlement-related deformations and associated distress. In addition, given their age, these existing improvements may require increased maintenance and repairs, and perhaps replacement in time frames that are much shorter than the currently proposed improvements. This should be disclosed to all interested/affected parties.

- The recommendations presented herein 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.

John P. Franklin
Engineering Geologist, CEG 1340

Andrew T. Guatelli
Geotechnical Engineer, GE 2320

Matthew J. Smelski
Staff Geologist

MJS/JPF/ATG/sh

Distribution: (3) Addressee (wet signed)

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PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED SKILLED NURSING FACILITY
675 E BRADLEY AVENUE, EL CAJON
SAN DIEGO COUNTY, CALIFORNIA 92021
ASSESSOR'S PARCEL NUMBER (APN) 387-142-36-00

SCOPE OF SERVICES

The scope of our services has included the following:

1. Review of readily available geotechnical and geologic data, and aerial photographs (Appendix A), including a review of San Diego County and City of El Cajon files pertaining to the project and vicinity.

2. Geologic reconnaissance mapping and subsurface exploration consisting of the excavation of two (2) exploratory test pits with a rubber-tire backhoe for geotechnical logging and sampling (Appendix B), along with the performance of one (1) seismic line for bedrock engineering characteristics (Appendix C). The test pits were backfilled with compacted soil in accordance with the requirements of the current property management company.

3. Preparation of a geotechnical map and geologic cross section, depicting the subsurface conditions encountered across the site both in plan view and profile.

4. Evaluations of bedrock hardness (text and Appendix C) and geologic/seismic hazards and regional seismicity (text and Appendix D).

5. Laboratory testing of representative bulk soil samples collected during our subsurface exploration program (Appendix E).

6. Engineering and geologic analysis of data collected.

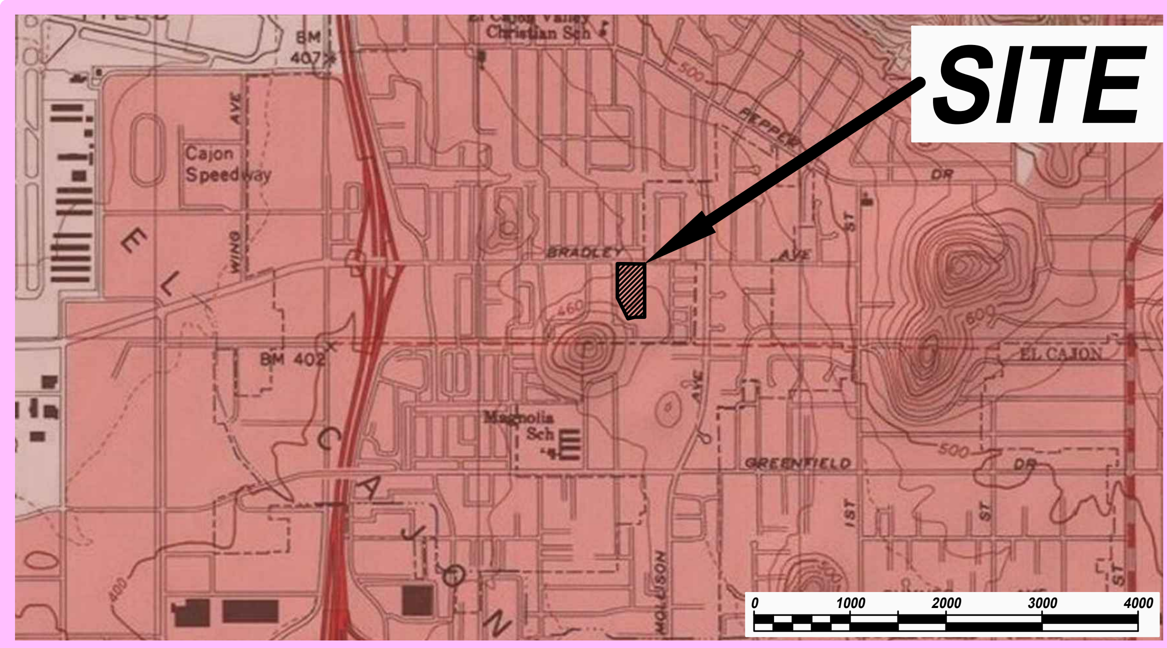
7. Engineering and geologic analysis regarding storm water infiltration (Appendix F)

8. Preparation of this geotechnical report, including conclusions and recommendations regarding: site earthwork (text and Appendix G), foundation design/construction, retaining walls, and improvements.

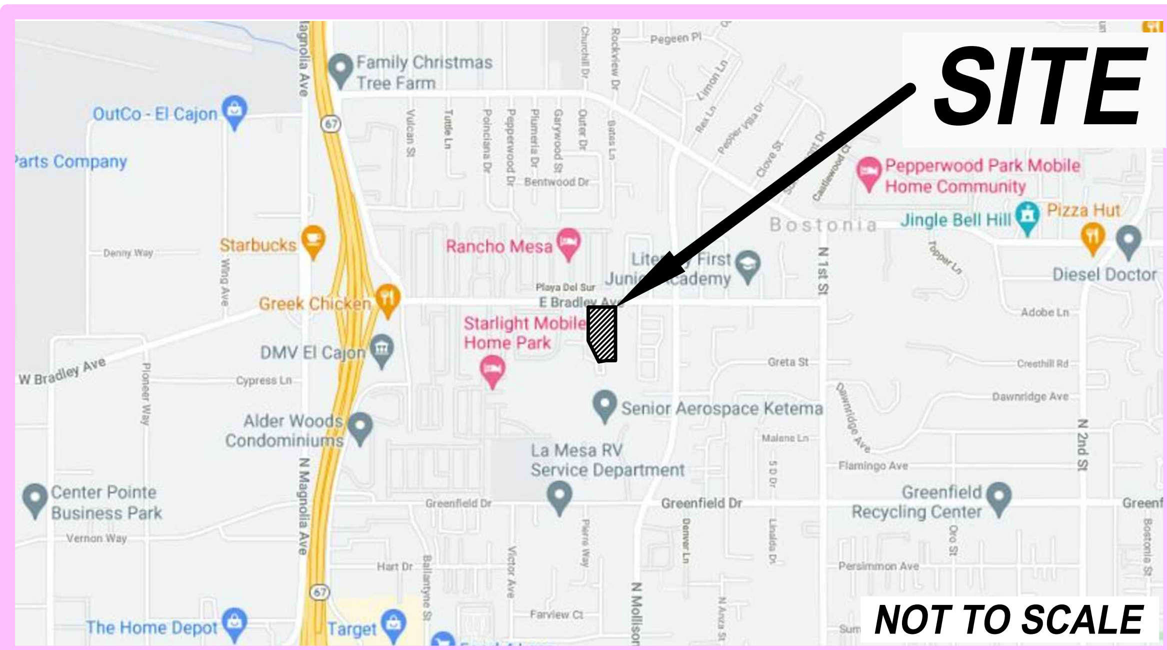
SITE DESCRIPTION AND PROPOSED DEVELOPMENT

Site Description

The subject site consists of approximately the southern one-third (±1 acre) of a 3.4-acre, nearly quadrilateral-shaped property, located at 675 E Bradley Avenue, in the City of El Cajon, San Diego County, California 92021 (see Figure 1 - Site Location Map).



Base Map: TOPO!® ©2003 National Geographic, U.S.G.S. El Cajon Quadrangle, California -- San Diego Co., 7.5 Minute, dated 1997, current, 1997.



Base Map: Google Maps, Copyright 2021 Google, Map Data Copyright 2021 Google

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GeoSoils, Inc.	W.O. 8096-A-SC
SITE LOCATION MAP	
Figure 1	

The geographic coordinates of the approximate centroid of the site are 32.8178˚, -116.9522˚. The site is bounded by a single-story 28-bed convalescent facility to the north (there is also a single-story 28-bed facility on the east side of the site), Sams Hill Road and residential development to the west/southwest, and by commercial/residential development to the remaining quadrants. In general, the site appears to consist of a relatively-flat, previously cut area ("building pad") with an existing single-story, 28-bed, healthcare facility located towards the eastern/southeastern portion of the site.

The building no. 2 site (Figure 1) slightly descends to the north/northwest (toward Bradley Avenue), and is located between approximate elevations of 463 feet Mean Sea Level (MSL) and 471 feet MSL (excluding any tops of core stones in the southeast corner). Thus, the overall relief across the building site area is approximately 8 feet. The western/southwestern property line contains an existing offsite, CMU retaining wall that appeared to be approximately ±10 feet in maximum height. An east-facing slope near the eastern property line descends from the pad area to the adjacent residential development, which appears to vary from about 5 to 10 feet in height. The general gradient of this slope is approximately 1.5:1 (horizontal to vertical [h:v]), with steeper and flatter slope areas occurring, locally. The site is generally lightly vegetated, with moderate vegetation occurring locally near the slope, consisting of some surficial grasses and scattered trees/shrubs. Surface drainage within the existing pad area appears to be directed offsite toward the northern portion of the property and Bradley Avenue.

Proposed Development

It is our understanding that the proposed development will consist of preparing the site for the construction of a new 8,215 square-foot, 32-bed, single-story addition to the existing skilled nursing healthcare building, along with a new concrete generator pad, and associated driveways, parking, trash enclosures, and sidewalk improvements. Cut and fill grading techniques are anticipated to have maximum geometric cuts and fills on the order of 5 feet or less, as the current building pad is near design grade. Additional improvements, consisting of underground utilities, typical exterior flatwork, and landscaping are also planned. The approximate limits of the new structure are shown on Plate 1 (Geotechnical Map). Plate 1 has been adapted from the 20-scale preliminary grading exhibit prepared by Excel Engineering ([EXE], 2021).

We have assumed that the structure would utilize typical shallow foundations with a slab-on-grade floor, and would be subject to local building review and compliance with the 2019 California Building Code ([2019 CBC], California Building Standards Commission [CBSC], 2019). We also anticipate that the design and construction of the proposed project will be subject to the State of California Office of Statewide Health Planning and Development's (OSHPD's) oversight. Sewage disposal is understood to be tied into the regional/municipal system. Building loads are also assumed to be typical for these types of single-story structures and the need for import soils is not anticipated at this time.

SITE EXPLORATION

Surface observations and subsurface explorations were performed by GSI in late April 2021, by a geologist in coordination with a California licensed Geotechnical Engineer from this office. Near-surface soil and geologic conditions were explored with two (2) test pits excavated with a rubber-tire backhoe, and one (1) seismic line survey performed with a seismograph. The approximate locations of the test pits and seismic survey are shown on the attached Plate 1 (Geotechnical Map). Logs of the test pits are presented in Appendix B and the seismic survey data is presented in Appendix C. This field study was performed in conjunction with another subsurface exploration of the adjoining project to the north (building no. 1, [Figure 1]), and a Phase 1 environmental site assessment (GSI, 2021).

PHYSIOGRAPHIC AND REGIONAL GEOLOGIC SETTINGS

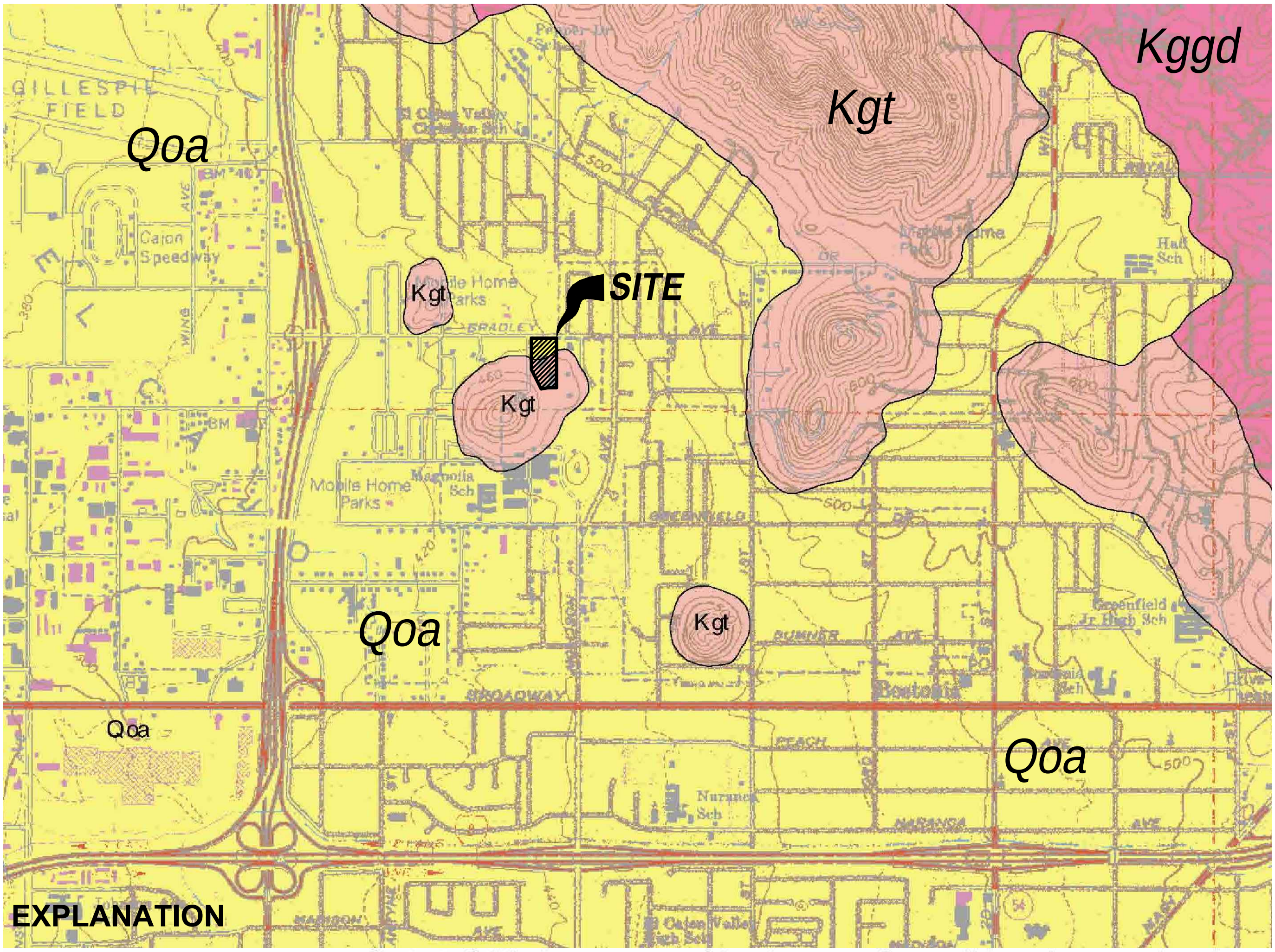
Physiographic Setting

The subject site is located in the central mountain-valley physiographic section of San Diego County. This section is characterized by basins or valleys ranging in elevation from 500 to 5,000 feet that are underlain by moderate thicknesses of alluvium and residuum (colluvium). They are intermittently dissected by stream channels that convey water from the eastern highlands to the Pacific Ocean.

Regional Geologic Setting

The subject property is located within a prominent natural geomorphic province in southwestern California known as the Peninsular Ranges. It is characterized by steep, elongated mountain ranges and valleys that trend northwesterly. The mountain ranges are underlain by basement rocks consisting of pre-Cretaceous metasedimentary rocks, Jurassic metavolcanic rocks, and Cretaceous plutonic rocks of the southern California batholith.

In the San Diego County region, deposition occurred during the Cretaceous Period and Cenozoic Era (Tertiary-age) in the continental margin of a forearc basin. Sediments, derived from Cretaceous-age plutonic rocks and Jurassic-age volcanic rocks, were deposited into the narrow, steep, coastal plain and continental margin of the basin. These rocks have been uplifted, tilted, faulted, 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. Based on our review of Tan (2002), which shows the geologic conditions at the subject site and near vicinity (Figure 2 - Regional Geologic Map), the site appears to be underlain with Cretaceous-age Tonalite (granitic bedrock at this site).



EXPLANATION

Contact—Solid where accuracy of location ranges from well located to approximately located; dashed where very poorly located or inferred. Color change without a contact shown is a scratch boundary

MAP UNITS

Qoa	Late Pleistocene alluvial deposits; moderately consolidated, poorly-sorted flood plain deposits consisting of gravelly, sandy silt and clay.
Kgt	Tonalite (Cretaceous); includes some granodiorite and quartz diorite; medium-grained; generally dark colored and severely weathered.
Kggd	Granodiorite (Cretaceous); includes some tonalite and monzogranite; medium-to coarse-grained.

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GRAPHIC SCALE

2000 0 1000 2000 4000

1" = 2000'

BASE MAP FROM:

GEOLOGIC MAP OF THE
EL CAJON 7.5' QUADRANGLE
SAN DIEGO COUNTY, CALIFORNIA:
A DIGITAL DATABASE

by

Siang S. Tan

Digital Preparation by
Jonathan Williams and Sybil Jorgensen

2002

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.

REGIONAL GEOLOGIC MAP

Figure 2

W.O. 8096-A-SC	*DATE: 07/21*	*SCALE: 1" = 2000'*

ONSITE GEOLOGIC UNITS

The onsite geologic units, encountered or observed during our subsurface exploration and site reconnaissance, included quaternary-age colluvium/slopewash deposits, and Cretaceous-age granitic bedrock (tonalite in composition). Undocumented artificial fill was not observed, but is likely present near the existing improvements. The general distribution of these materials is shown in plan view on Plate 1, and in profile on Plates 2 and 3.

Undocumented Artificial Fill (Map Symbol - Afu)

Although not encountered in our explorations during the study, it is anticipated that undocumented fill exists in the southeast corner of the site, likely associated with the development of the existing Building 2 (see Plate 1) and auxiliary improvements. Based on the surficial appearance of the site, these fill soils are assumed to have been locally derived and likely consisted of dry to moist, reddish brown and yellowish brown silty/clayey sand. As a result of the potentially compressible nature of these undocumented fill soils, they are considered unsuitable for the support of settlement sensitive structures and/or improvements in their existing state, if encountered. These materials should be removed, moisture-conditioned, and recompacted and/or processed in place, should settlement-sensitive improvements be proposed.

Colluvium - Slopewash (Map Symbol - Qcol)

A relatively thin, surficial layer of colluvium, or "slopewash," was noted across the natural areas of the site. Where observed, the colluvium appeared to be on the order of about 2½ feet in thickness, and is anticipated to be thicker towards the north (downhill). The colluvium consisted of moderate to dark reddish brown, dry to damp, and medium dense to dense clayey sand with sporadic granitic gravels and visible porosity near the surface. Surficial deposits of colluvium are considered potentially compressible in their existing state and will require removal and recompaction, if engineered fills, and/or settlement-sensitive improvements are proposed within its influence.

Cretaceous-age Granitic Bedrock (Map Symbol - Kgt)

Granitic bedrock (tonalite in composition) was observed at the surface in the upper portions of the site, and underlying the colluvium in the lower portions, up to a depth of roughly 2½ feet below existing grade (B.E.G.). Where encountered, the granitic bedrock is weathered and generally disintegrates to a silty sand, with some gravels, upon excavation. This weathered bedrock is typically moderate to dark yellowish brown and olive brown, dry to damp, and very dense. The weathered granitic bedrock encountered onsite is considered suitable in its existing state for the support of engineered fills and settlement-sensitive improvements. In the event that any near-surface, highly weathered bedrock is encountered (friable with visible voids, etc.), it is not considered suitable for the support of settlement-sensitive improvements and/or planned fill, in its existing state.

Structural Geology

No adverse geologic structures were observed on the site; the granite bedrock is typically massive and jointed with high angle fractures.

BEDROCK HARDNESS AND RIPPABILITY/TRENCHABILITY

Based on our review of the available subsurface data and our understanding of the planned grading shown on EXE (2021), it is the opinion of GSI that excavations into the onsite earth materials with typical heavy-duty earth moving equipment in good working condition will range between easy and difficult. There is a potential that dense areas of the granitic bedrock and bedrock corestones could require the use of rock breaking equipment. Earth moving equipment with ripper attachments should be considered. The use of relatively lightweight excavation and trenching equipment, such as small bulldozers, mini-excavators and rubber-tire backhoes could encounter non-productive excavation from the surface to depths as shallow as about 2½ feet B.E.G. This should be considered during project planning and budgetary evaluations. Excavation equipment should be properly sized and powered for the required excavation task. If additional information regarding rock hardness and excavation feasibility are necessary, this office could perform supplemental seismic refraction surveys upon request.

Rock Hardness Evaluation

A seismic refraction survey was performed within a selected area of the site where the proposed Building 2 is generally located (see Plate 1). The survey consisted of one (1) seismic refraction line (or traverse), conducted using a Geometrics SmartSeis 12-channel exploration seismograph with a hammer and plate energy source. The approximate seismic line location is shown on Plate 1, and the velocity and depth interval results are graphically shown, and included in Appendix C. An example of the raw seismic data is also included in Appendix C, and illustrates a forward and split spread shot from the same line.

The first arrival information, shot point locations, geophone locations, and line geometry from each survey are utilized in the computer programs SIPwin (Rimrock Geophysics, 2002) which produces time-distance plots for each of the survey lines (see example, Appendix C). The graphic curves reflect the actual time-distance plots generated by the program, showing the shot points and phone locations. The first curve, from left to right shows the forward spread from the first shot. The second, or split spread shot point creates two curves in opposite directions from the shot in the middle of the spread. The third curve represents the reverse shot from the distal end of the spread.

The data for the survey performed generally show a three-layer case. The uppermost layer is generally thin as would be expected, reflecting the surficial materials (i.e., undifferentiated colluvium/undocumented fill/highly weathered bedrock). Undulations in time-distance curves can be attributed to a lack of elevation corrections to the raw data, possible minor disturbances from noise (e.g., wind or traffic), decreased energy at distant geophones, and discontinuities in the subsurface.

The velocity-depth model (or cross-section) generated is included as Plate 4, for seismic traverses SL-1. As can be seen on these plates, the boundaries between the upper most seismic velocity layers (soil over rock) appear to be relatively undulatory, with boundary conditions (boundaries between different rock densities) within the bedrock becoming more flat-lying at depth.

Layer boundaries tend to mimic the surface topography, although variations are common depending upon the depth of weathering, fracturing, presence of core stones, etc. In general, the survey indicated a near-surface layer (Layer 1) thickness (i.e., colluvium/highly weathered bedrock) on the order of about ±3 to ±5 feet. The average velocity of Layer 1 material is about ±3,053 fps, and is considered relatively dense for such near surface material. The depth to the Layer 1/Layer 2 transition (weathered bedrock) is approximately 3 to 5 feet below existing grades. The average velocity of Layer 2 is about ±4,506 fps, and is considered representative of slightly weathered to weathered bedrock. The depth to the Layer 2/Layer 3 transition (weathered bedrock/bedrock) ranges from about 27 to 29 feet B.E.G. The average velocity of Layer 3 is about ±8,793 fps. Oversize material will likely be routinely generated in Layer 2 and primarily in Layer 3.

At depths where velocities are greater than about 6,000 fps, rippability is ambiguous and blasting usually is required. An evaluation has been made of the seismic refraction line data to estimate the approximate depth to non-rippable trenching (i.e., utility excavation) and to non-rippable bedrock. Approximate cut-off velocities of ±3,800 and ±6,000 fps are generally used as a basis for non-rippable trenching (assuming a Cat 235 Hoe [a large trackhoe], or equivalent), and non-rippable bedrock (assuming a D9L, or equivalent), respectively.

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

Rock Hardness Summary

Based upon our experience in this area, and the seismic refraction data obtained, the following reflects our estimates of the rippability and trenchability at the location of the seismic refraction survey line; other interpretations are possible. In general, utilizing the seismic data, it appears that the area in the vicinity of our seismic line may be characterized as being underlain by surficial soils (colluvium/highly weathered bedrock) to depths of about ±3 to ±5 feet B.E.G. At depths below about 3 to 5 feet, non-trenchable and very hard ripping to non-rippable conditions are anticipated to depths ranging from about 27 to 29 feet B.E.G., with dense, non rippable rock below these depths. Some dense, surficial outcrops of bedrock (i.e., corestones) were also noted throughout the site.

Based on all of the above, including plan cut/fill, the need for blasting and/or line shooting for building foundations does not appear necessary. However, the plans prepared by EXE (2021) do not appear to show planned improvements for the proposed building, so the need for blasting should be reevaluated once civil utility drawings have been produced. In the event that planned improvements are to occur at depths greater than 3 to 5 feet B.E.G., blasting and/or line shooting may be necessary. It should also be noted that due to the variability of bedrock weathering, and the potential for local boulders, or less weathered bedrock, very difficult ripping, rock breaking, etc. should be anticipated at shallower depths. Based on the rock hardness data, undercutting the street section during grading in order to facilitate potential utility construction is recommended.

GROUNDWATER

Subsurface water, including seeps or springs, was not encountered within the property during field work performed in preparation of this report. However, a review of the groundwater monitoring report performed by Kahl Environmental Services ([KES], 2007) appears to show groundwater occurring on an adjacent site at elevations ranging from 431.0 feet MSL to 434.0 feet MSL. As such, groundwater is anticipated to occur at depths greater than 25 feet below the lowest existing grade/improvement onsite, and at least 30 feet below the proposed building floor elevation. Regional groundwater is not expected to adversely affect the proposed site development, provided that the recommendations contained in this report are incorporated into project design and construction, and that prudent surface and subsurface drainage practices are incorporated into the construction plans. 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, damaged wet underground utilities, or other factors that were not obvious, at the time of our investigation.

Due to the nature of the onsite earth materials, perched groundwater conditions cannot be precluded from occurring in the future, and would likely manifest along zones of contrasting permeabilities/densities (i.e., along fill lifts, geologic contacts between the bedrock and fills [existing and proposed], and natural overburden soils), and along geologic discontinuities (joints, fractures, etc.), at any elevation/depth. This potential will need to be disclosed to the owner/developer and/or any other interested/affected parties.

GEOLOGIC HAZARDS EVALUATION

Mass Wasting/Landslide Susceptibility

Mass wasting refers to the various processes by which earth materials are moved downslope 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).

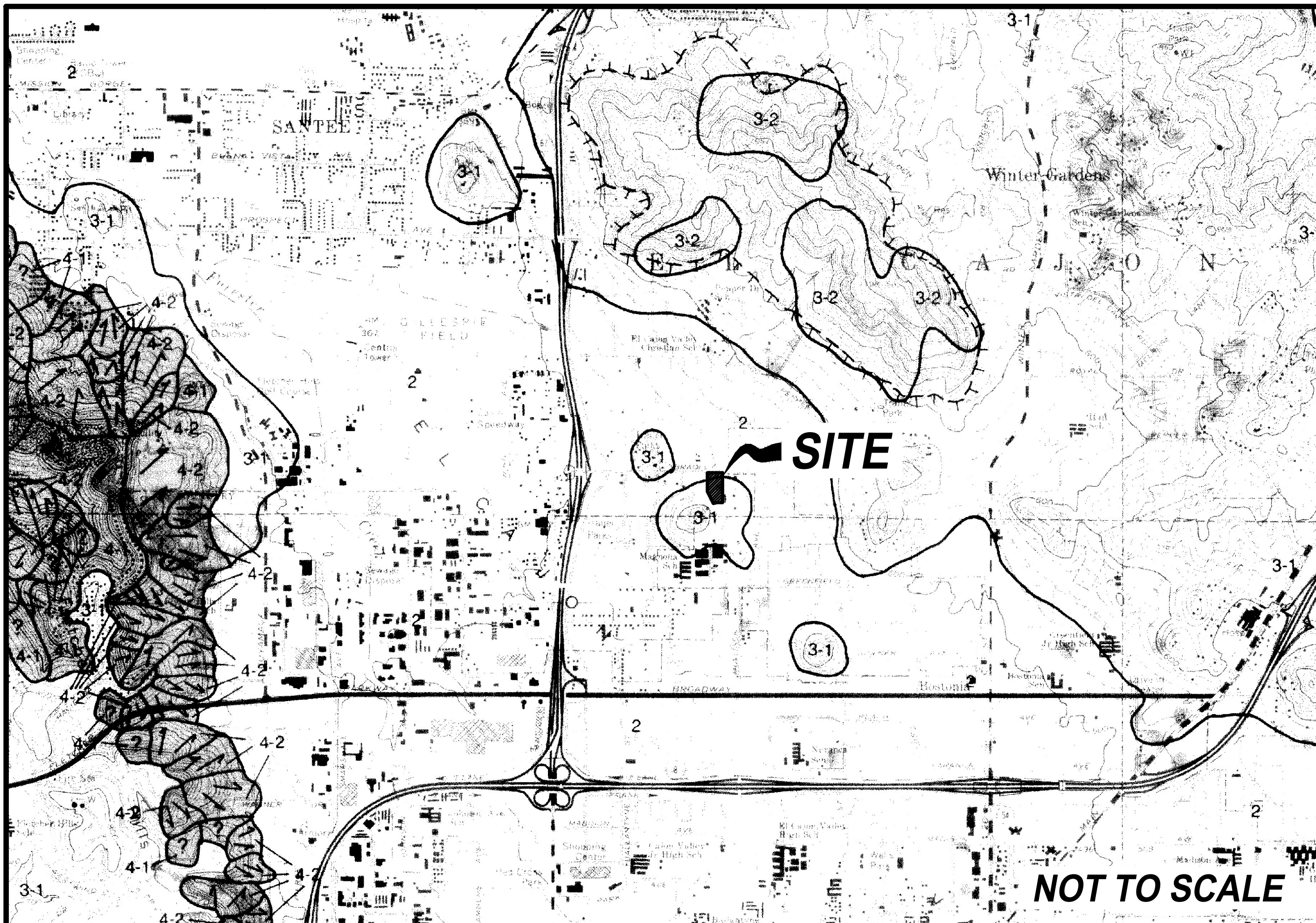
According to regional landslide susceptibility mapping by Tan (1995), the site is located within landslide susceptibility Subarea 3-1 which is characterized as being "generally susceptible" to landsliding. However, Tan (2002, 1995) and Todd (2004) do not show the presence of landslides within the property, and geomorphic expressions indicative of past mass wasting events (i.e., scarps, hummocky terrain, arcuate drainage courses, etc.) were not observed on the property during our field studies nor our review of stereoscopic aerial photographs. Lastly, GSI did not observe evidence of landslide debris nor adverse geologic structures indicative of deep-seated instability, such as shear zones, clay seams, or out-of-slope discontinuities.

Based on the relatively dense and crystalline nature of the underlying granitic bedrock, and observations made during our field investigation , the potential for landsliding is considered low. The site location on the regional landslide susceptibility maps prepared by Tan (1995) is presented as Figure 3.

The onsite earth materials 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 frequency, and irrigation and surface drainage practices. Such risks can be minimized through properly designed, and regularly and periodically maintained surface drainage, the use of deep-rooted plant species capable of surviving the prevailing semi-arid climate with little to no irrigation water, and the avoidance of over-irrigating landscaping on slopes.

Other Geologic Hazards

- Expansive soils do not appear to occur onsite, as reported herein.
- Corrosive soils do not appear to occur onsite, as reported herein.



Base Map: Relative Landslide Susceptibility and Lamdslide Distribution Map, El Cajon 7.5' Quadrangle, San Diego County, California (Tan, 1995)

LANDSLIDE HAZARDS IN THE SOUTHERN PART OF THE SAN DIEGO METROPOLITAN AREA, SAN DIEGO COUNTY, CALIFORNIA

by

Siang S. Tan
Geologist

1995

RELATIVE LANDSLIDE SUSCEPTIBILITY AND LANDSLIDE DISTRIBUTION MAP
EL CAJON QUADRANGLE
(PLATE C)

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LANDSLIDE SUSCEPTIBILITY MAP

Figure 3

- Natural occurring methane gas, hydrogen sulfide gas, and tar seeps, were not noted, and are not anticipated, based on the underlying geology.

- The site is not within the immediate influence of a potential volcanic eruption.

- The site is located at elevations ranging from about 461 to 473 feet MSL within gentle to moderate hillside terrain, and is located within Zone “X” or an “area of minimal flood hazard” (Federal Emergency Management Agency [FEMA], 2012).

- The lithology of granitic bedrock is not conducive to the presence of naturally occurring asbestos.

- Alluvial deposits are not present onsite. Existing fill will be removed as a result of geometric grading or mitigation during removal and recompaction. As such, the potential for soil hydrocollaspe is considered nil subsequent to grading.

- Granitic bedrock is not locally known for bedrock heave and the planned removal depths are generally less than 10 feet.

- Tsunami and seiche inundation potential are discussed in the following section, and are not significant at this location.

- Regional subsidence is not considered significant with respect to site development, and is not known at this location.

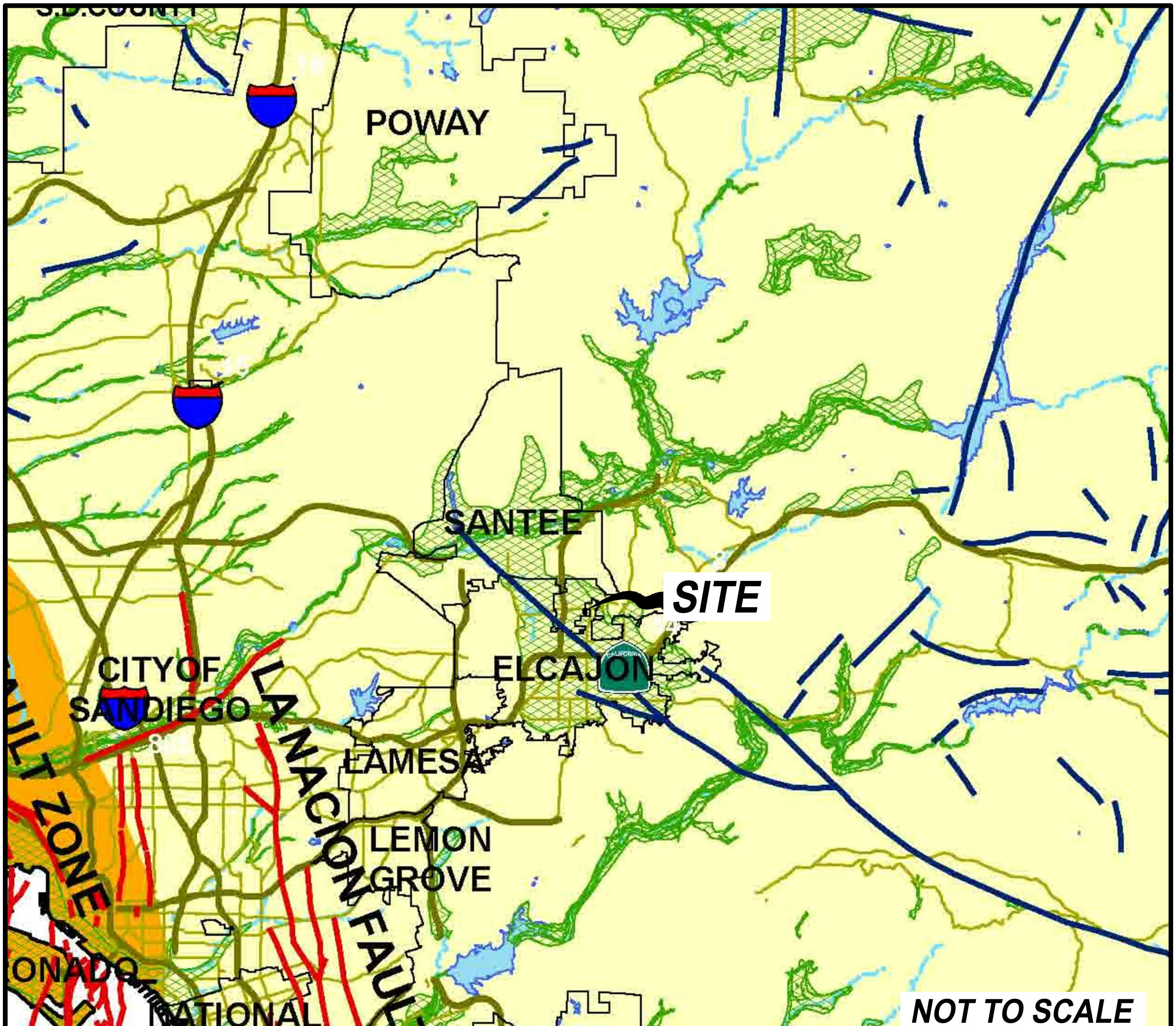
- The potential for cyclic strain softening of earth material due to seismic loading is considered very low due to the dense conditions of the granitic bedrock, and the granular nature of the fills generated from this bedrock.

- Based on a review of liquefaction potential per the San Diego County’s (2017) Liquefaction Hazard Mitigation Planning exhibit (see Figure 4), the site is mapped within a “liquefaction layer” zone. However, due to the crystalline structure and dense nature of the granitic bedrock, GSI believes that the site has been incorrectly mapped as being susceptible to liquefaction. Seismic densification and liquefaction due to strong ground shaking during the design-basis earthquake will be mitigated by near-surface grading and the dense conditions of the granitic bedrock; and therefore, not a significant geotechnical concern.

FAULTING AND REGIONAL SEISMICITY

Regional Faults

Our review indicates that there are no known Holocene-active faults (i.e., faults demonstrating movement within the last 11,700 years [California Geological Survey, 2018])



Base Map:

DRAFT - LIQUEFACTION
COUNTY OF SAN DIEGO
HAZARD MITIGATION PLANNING

LEGEND:

Earthquake Faults:
- Fault
- Zoned Earthquake Fault

Liquefaction Layers
- Liquefaction Layers

Peak Ground Acceleration (2% in 50 yrs)
- 0.18 - 0.5 (Low Liquefaction Risk)
- 0.51 - 1.60 (High Liquefaction Risk)

Base Layers
- Incorporated City Boundary
- Freeways
- Major Roads
- Streams
- Lakes

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POTENTIAL LIQUEFACTION AREA MAP

Figure 4

crossing the property and the site is not within an Alquist-Priolo Earthquake Fault Zone (California Geological Survey, 2018; Jennings and Bryant, 2010). However, the site is situated in a region subject to periodic earthquakes along active faults. The Rose Canyon fault (RCF), part of the Newport-Inglewood - Rose Canyon (NIRC) fault system, is the closest known Holocene-active fault to the site (located at a distance of approximately 14.2 miles [22.8 kilometers]) to the west, and should have the greatest effect on the site in the form of strong ground shaking, should the design earthquake occur.

Cao, et al. (2003) indicate the slip rate on the Rose Canyon fault (RCF) is 1.5 (±0.5) millimeters per year (mm/yr) and the fault is capable of a maximum magnitude 7.2 earthquake. Rockwell (2010), however, indicates a slip rate of 2 mm/yr, with a recurrance interval of about 1,000 years, for the RCG. The location of the RCF and other major faults relative to the site is shown on the "California Fault Map" in Appendix D. The possibility of ground acceleration, or shaking at the site, may be considered as approximately similar to the Southern California region as a whole. The approximate location of the site relative to mapped Quaternary-age faults is shown on Figure 5.

Local Faulting

Although active faults lie within the region, 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 (Tan 2002; Todd, 2004) does not indicate the presence of older faults crossing the specific project site.

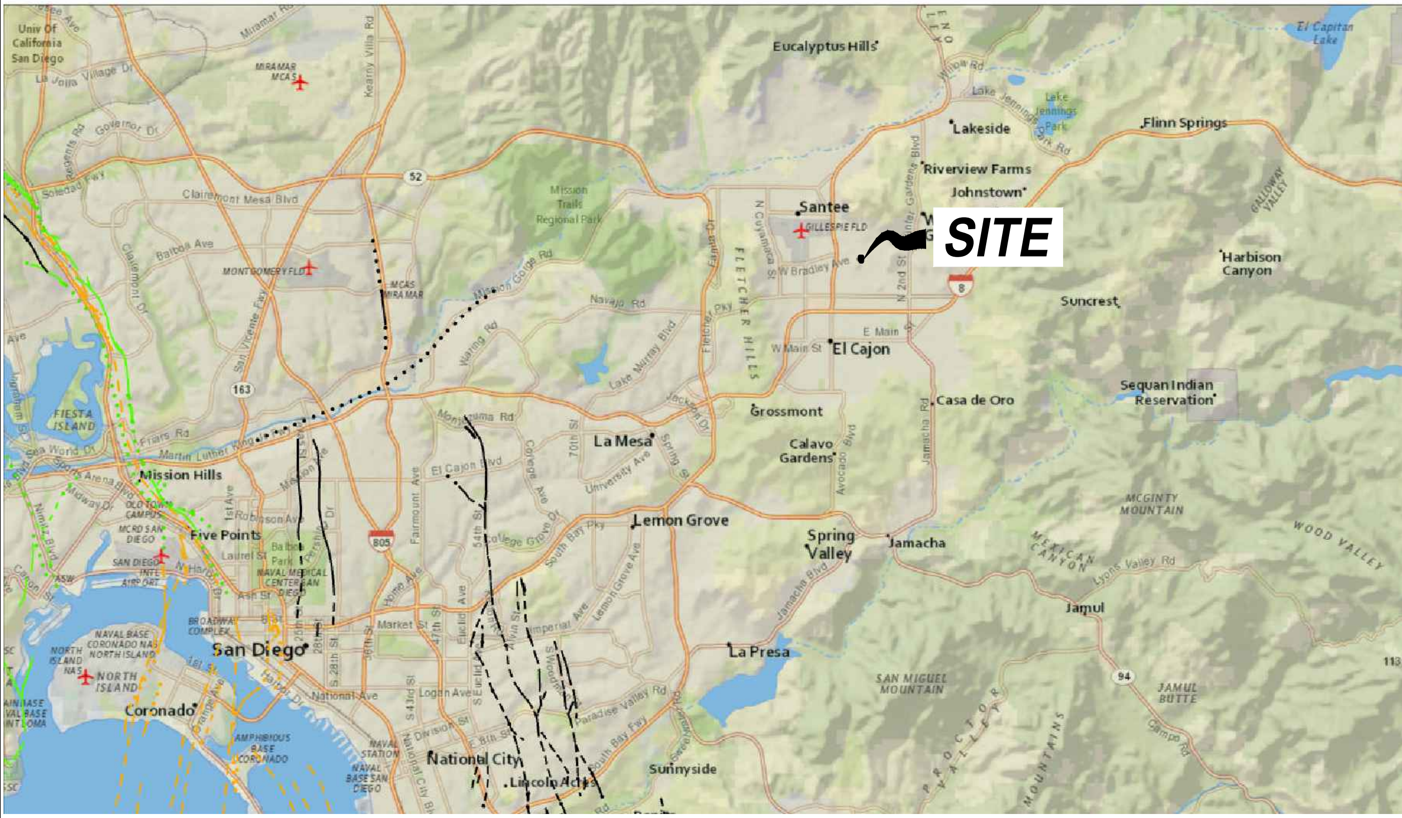
Seismicity

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

The program estimates the closest distance between each fault and a given site. If a fault is found to be within a user-selected radius, the program estimates peak horizontal ground acceleration that may occur at the site from an upper-bound (formerly "maximum credible earthquake"), on that fault. Upper-bound refers to the maximum expected ground acceleration produced from a given fault. Site acceleration (g) was computed by one user-selected acceleration-attenuation relation that is contained in EQFAULT. Based on the EQFAULT program, a peak horizontal ground acceleration from an upper bound event on the Rose Canyon fault may be on the order of 0.272 g. The computer printouts of pertinent portions of the EQFAULT program are included within Appendix D.

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 May 8, 2021). This program performs a search of the historical

U.S. Geological Survey Quaternary Faults



Base Map: Mapped Faults in San DIego County, ©2007 Guidelines for Determining Singnificance Geologic Hazards

1:144,448

0 1.5 3 6 mi

0 2.25 4.5 9 km

National Geographic, Esri, Garmin, HERE, UNEP-WCMC, USGS, NASA, ESA, METI, NRCAN, GEBCO, NOAA, increment P Corp.

Fault Areas

Class B

historic

late Quaternary

latest Quaternary

middle and late Quaternary

National Database

Historic (< 150 years), well constrained location

Historic (< 150 years), moderately constrained location

Historic (< 150 years), inferred location

Latest Quaternary (<15,000 years), well constrained location

Latest Quaternary (<15,000 years), moderately constrained location

Latest Quaternary (<15,000 years), inferred location

Late Quaternary (< 130,000 years), well constrained location

Late Quaternary (< 130,000 years), moderately contrained location

Late Quaternary (< 130,000 years), inferred location

N

Middle and late Quaternary (< 750,000 years), well constrained location

Middle and late Quaternary (< 750,000 years), moderately constrained location

Middle and late Quaternary (< 750,000 years), inferred location

Undifferentiated Quaternary (< 1.6 million years), well constrained location

Undifferentiated Quaternary (< 1.6 million years), moderately constrained location

Undifferentiated Quaternary (< 1.6 million years), inferred location

Unspecified age, well constrained location

Unspecified age, moderately constrained location

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FAULT MAP

Figure 5

earthquake records for magnitude 5.0 to 9.0 seismic events within a 100-kilometer radius, between the years 1800 through May 8, 2021. Based on the selected acceleration-attenuation relationship, a peak horizontal ground acceleration is estimated, which may have affected the site during the specific event listed. Based on the available data and the attenuation relationship used, the estimated maximum (peak) site acceleration during the period 1800 through May 8, 2021 was about 0.158 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 D.

Seismic Shaking Parameters

Based on the site conditions, the following table summarizes the site-specific design criteria obtained from the 2019 CBC (CBSC, 2019), Chapter 16 Structural Design, Section 1613, Earthquake Loads. The computer program "OSHPD Seismic Design Maps" (https://seismicmaps.org/), provided by the Structural Engineers Association of California (SEAOC) and the State of California's Office of Statewide Health Planning and Development (OSHPD) was utilized for design. The short spectral response utilizes a period of 0.2 seconds. The output file and response spectra produced by "OSHPD Seismic Design Maps" for the subject site are included in Appendix D. The table below provides the calculated seismic design parameters.

Based on the anticipated relatively thin fills (i.e., less than 10 feet) below the building foundations and the dense nature of the granitic bedrock (see Appendix C), it is our opinion that a Site Class "B" designation is appropriate for the proposed project (less than 10 feet of proposed fill).

2019 CBC SEISMIC DESIGN PARAMETERS		
PARAMETER	**VALUE**	**2019 CBC or REFERENCE**
Risk Category	III	Table 1604.5
Site Class	B	Section 1613.2.2/Chap. 20 ASCE 7-16 (p. 203-204)
Spectral Response - (0.2 sec), S_s	0.759 g	Section 1613.2.1 Figure 1613.2.1(1)
Spectral Response - (1 sec), S_1	0.28 g	Section 1613.2.1 Figure 1613.2.1(2)
Site Coefficient, F_a	0.9	Table 1613.2.3(1)
Site Coefficient, F_v	0.8	Table 1613.2.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec), S_{MS}	0.683 g	Section 1613.2.3 (Eqn 16-36)

2019 CBC SEISMIC DESIGN PARAMETERS		
PARAMETER	VALUE	2019 CBC or REFERENCE
Maximum Considered Earthquake Spectral Response Acceleration (1 sec), S_{M1}	0.224 g	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	0.455 g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.149 g	Section 1613.2.4 (Eqn 16-39)
PGA_M - Probabilistic Vertical Ground Acceleration may be assumed as about 50% of these values.	0.293 g	ASCE 7-16 (Eqn 11.8.1)
Seismic Design Category	C	Section 1613.2.5/ASCE 7-16 (p. 85: Table 11.6-1 or 11.6-2)

GENERAL SEISMIC PARAMETERS	
PARAMETER	VALUE
Distance to Seismic Source[1] - A fault[2] (Rose Canyon fault)	14.2mi (22.8 km)
Upper Bound Earthquake (Rose Canyon fault)	$M_w = 7.2$ [2]
[1] - From Blake (2000) [2] - 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, ground failure, or surface manifestations will not occur in the event of a large earthquake in this region. The primary goal of seismic design is to protect life, not to eliminate all damage, since such design may be economically prohibitive. Cumulative effects of seismic events are not addressed in the 2019 CBC (CBSC, 2019) and regular maintenance and repair following locally significant seismic events (i.e., M_w 5.0) will likely be necessary.

SECONDARY SEISMIC HAZARDS

Liquefaction

Liquefaction describes a phenomenon in which cyclic stresses, produced by earthquake-induced ground motion, create excess pore pressures in relatively cohesionless soils. These soils may thereby acquire a high degree of mobility, which can lead to vertical deformation, lateral movement, lurching, sliding, and as a result of seismic loading, volumetric strain and manifestation in surface settlement of loose sediments, sand boils and other damaging lateral deformations. This phenomenon occurs only below the water table, but after liquefaction has developed, it can propagate upward into overlying non-saturated soil as excess pore water dissipates.

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

The condition of liquefaction has two principal effects. One is the consolidation of loose sediments with resultant settlement of the ground surface. The other effect is lateral sliding. Significant permanent lateral movement generally occurs only when there is significant differential loading, such as fill or natural ground slopes within susceptible materials. No such loading conditions exist at the site.

Liquefaction susceptibility is related to numerous factors and the following five conditions should be concurrently present for liquefaction to occur: 1) sediments must be relatively young in age and not have developed a large amount of cementation; 2) sediments must generally consist of medium- to fine-grained, relatively cohesionless sands; 3) the sediments must have low relative density; 4) free groundwater must be present in the sediment; and 5) the site must experience a seismic event of a sufficient duration and magnitude, to induce straining of soil particles. Based on our evaluation, it does not appear that at least four of the five necessary concurrent conditions have the potential to affect the site.

It is the opinion of GSI that the susceptibility of the site to experience damaging deformations from seismically-induced liquefaction is very low owing to the dense nature of the granitic bedrock, and the absence of free groundwater. In addition, the herein provided recommendations for remedial earthwork and foundations would further reduce any significant liquefaction potential at the site. The location of the site relative to mapped areas susceptible to liquefaction is shown on Figure 4.

Other Secondary Seismic Hazards

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

- Seismic densification
- Seismic Settlement
- Surface Fault Rupture
- Ground Lurching or Shallow Ground Rupture
- Top-of-Slope Seismic Deformations
- Tsunami
- Seiche

It is important to keep in perspective that in the event of an upper-bound earthquake occurring on any of the nearby major faults, strong ground shaking would occur in the subject site's general area. Potential damage to any structure(s) would likely be greatest from the vibrations and impelling force caused by the inertia of a structure's mass than from those induced by the hazards considered above. Following implementation of remedial earthwork and design of foundations described herein, this potential would be no greater than that for other existing structures and improvements in the immediate vicinity that comply with current and adopted building standards. Evaluation following each significant seismic event $M_W \geq 5.0$ (horizontal site acceleration $\geq$ 0.25g) would be necessary to monitor the long-term performance of site improvements.

LABORATORY TESTING

General

Laboratory testing was performed on representative bulk samples of the onsite earth materials collected during our subsurface exploration in order to evaluate their physical characteristics and engineering properties. The test procedures utilized and laboratory results obtained are presented below.

Classification

Soils were visually classified with respect to the Unified Soil Classification System (U.S.C.S.) developed by Sowers and Sowers (1979) in general accordance with ASTM D 2487 and D 2488. The soil classifications of the onsite soils are provided on the Test Pit Logs in Appendix B.

Laboratory Standard

The maximum dry density and optimum moisture content was determined for representative soil samples. The laboratory standard utilized was ASTM D 1557. The moisture-density relationship obtained for this soil is shown below:

SAMPLE LOCATION AND DEPTH (FT)	SOIL TYPE	MAXIMUM DRY DENSITY (PCF)	OPTIMUM MOISTURE CONTENT (%)
TP-7 @ 0-1½	Dark Gray Silty Sand w/ Silt	129.8	7.6

Expansion Potential

A representative sample of near-surface site soil was evaluated for expansion potential. Expansion Index (E.I.) testing and expansion potential classification were performed in general accordance with ASTM Standard D 4829. The results of the expansion index testing are presented in the following table.

SAMPLE LOCATION AND DEPTH (FT)	LABORATORY EXPANSION INDEX (E.I.)	EXPANSION POTENTIAL*
TP-7 @ 0-1½	<21	Very Low
* In accordance with ASTM 4829 (Table 1): E.I. = 0-20 very low; E.I. = 21-50 low; E.I. = 51-90 medium; E.I. = 91-130 high.		

Direct Shear Test

Shear testing was performed on a representative, "remolded" samples of site soil in general accordance with ASTM Test Method D 3080 in a Direct Shear Machine of the strain control type. Prior to testing, the samples were remolded to 90 percent of the laboratory standard (per ASTM D 1557). The shear test results are presented in Appendix E, and the following table:

SAMPLE LOCATION AND DEPTH (FT)	PRIMARY		RESIDUAL	
	COHESION (PSF)	FRICTION ANGLE (DEGREES)	COHESION (PSF)	FRICTION ANGLE (DEGREES)
TP-7 @ 0-1½ (Remolded)	154	39	51	37

Particle-Size Analysis

A particle-size evaluation was performed on representative soil samples 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 are presented in the following table:

SAMPLE LOCATION	COLOR DESCRIPTION	USCS SOIL CLASSIFICATION	GRAIN SIZE PERCENTAGES
TP-7 @ 0-1½	Dark Gray	Silty Sand w/Silt (SW-SM)	2.1% Gravel, 89.0% Sand, 8.9% Fines

Saturated Resistivity, pH, and Soluble Sulfates

GSI conducted sampling and testing of representative samples of the onsite earth materials for general soil corrosivity and soluble sulfates, and chlorides testing. The testing included evaluations of soil pH, soluble sulfates, soluble chlorides, and saturated resistivity. Test results are presented in the following table and Appendix E.

SAMPLE LOCATION AND DEPTH (FT)	pH	SATURATED RESISTIVITY (ohm-cm)	SOLUBLE SULFATES (% by weight)	SOLUBLE CHLORIDE (mg/L)
TP-7 @ 0-1½	7.4	19,000	0.001	50

Corrosion Summary

Laboratory testing indicates that tested samples of the onsite soils are mildly alkaline with respect to soil acidity/alkalinity; are mildly corrosive to exposed, buried metals when saturated; present elevated sulfate exposure to concrete (Exposure Class S1 per Table 19.3.1.1 of ACI 318-14); and contain sightly elevated concentrations of soluble chlorides. It should be noted that GSI does not consult in the field of corrosion engineering. Therefore, additional comments and recommendations may be obtained from a qualified corrosion engineer based on the level of corrosion protection required for the project, as determined by the Project Architect, Civil Engineer, and/or Structural Engineer.

PRELIMINARY SETTLEMENT EVALUATION

GSI has estimated the potential magnitudes of total settlement, differential settlement, and angular distortion for the site. The analyses were based on the subsurface data and the results of laboratory tests performed on representative soil samples, collected from our test pits, and our preliminary assumptions regarding building loads. Site-specific conditions affecting settlement potential include age, compaction effort, contact inclination, grain size and lithology of sediments, cementing agents, stress history, moisture history, material shape, density, void ratio, etc.

In consideration of the above, total static compression and seismic deformations, due to densification of fill and/or deformation of dense earth materials subjected to strong shaking during the design earthquake, should be anticipated. Total static settlement on the order of ¼ to ½ inch may occur during and after grading and as building loads are applied to the foundation. Differential static deformation is anticipated to be on the order of ½ inch in a 50-foot span or between the heaviest or lightest foundation elements (i.e., angular distortion of approximately 1/1,200). Seismic deformation of fill soil compacted to

92 percent is anticipated to be on the order of ¼ to ½ inch in 100 feet (i.e., angular distortion approximately 1/2,400). Static settlement should be incorporated into the foundation system design. Seismic deformations should be included in the design as part of the seismic performance evaluation of the building and other improvements onsite. These estimated settlements assume that the fill will be approximately 10 feet or less within the influence of the building and that the dense granitic bedrock will not significantly add to the static and seismic settlement.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

General

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

- Deformation potential (static and seismic) and uniform support of proposed settlement-sensitive improvements;
- Depth to competent bearing materials below the existing grades;
- On-going expansion and corrosion potentials of onsite soils;
- Seepage, drainage, and moisture transmission through foundations, retaining walls, and slab-on-grade floors;
- Productive ripping and trenching within the granitic bedrock;
- The potential for encountering oversized rock constituents requiring special handling and placement in engineered fills;
- The potential for existing site improvements that are to remain in service and underlain by undocumented artificial fill to undergo settlement-related deformations and associated distress;
- Differences in age and the level of performance between existing site improvements that are to remain in service and the currently proposed improvements; and
- 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 are evaluated or modified in writing by this office. Foundation design parameters are considered preliminary until the foundation design, layout, and structural loads are provided to this office for review.

1. Geotechnical engineering, observation, and testing services should be provided during grading to aid the contractor in removing unsuitable soils, overexcavating the granitic bedrock, and in their effort to compact the fill. Although unlikely, if unexpected subsurface conditions are encountered during grading, supplemental recommendations and earthwork may be warranted.

2. Owing to the presence of surficial colluvium and potential undocumented artificial fill in the area of the site to receive the proposed building, there will be a potential for distress to settlement-sensitive improvements, absent of mitigation. Remedial measures should include the removal and recompaction of any existing colluvium/fill materials and any highly weathered or loose, disturbed zones of granitic bedrock (if encountered) to conform to current industry standards and Code, as recommended herein. Based on the available subsurface data, remedial grading excavations in the areas of proposed development are anticipated to extend to depths on the order of 2½ to 3 feet B.E.G. The actual depths of remedial grading excavations will be based on the subsurface conditions exposed during grading. Should unsuitable soils extend to depths greater than those encountered during our field investigation, deeper remedial excavation would be recommended. Thus, some variability should be anticipated and considered in preliminary earthwork construction estimates.

3. In order to reduce differential settlement potential, the foundation for the proposed healthcare building should be supported by a uniform thickness (overexcavated pad) compacted fill blanket. Based on the available subsurface data, this will require overexcavation (undercutting) and replacing the granitic bedrock with compacted fill materials per the recommendations provided in this report. On a preliminary basis, overexcavation should be completed to the greater of either 3 feet below pad grade or 2 feet below the lowest foundation element. The minimum horizontal extent of the overexcavation should be 5 feet outside the perimeter footprint of the proposed skilled nursing facility building.

4. In general and based upon the available data to date, regional groundwater is not expected to be encountered during the onsite excavations. However, perched water seepage between layers of fill, fill/bedrock and within bedrock discontinuities or bedding, cannot be precluded from being encountered either during or following site development. GSI did not encounter any evidence of perched groundwater with our subsurface explorations.

5. Expansion Index (E.I.) testing performed on a sample of the onsite soils generally indicates very low expansive soil conditions (E.I. of <21), for soils derived from weathered bedrock. However, it is possible that soils with higher expansion potentials may be encountered locally. Based on the distribution of soils, including colluvium and highly weathered bedrock, etc., blending/mixing of any minor amounts of expansive soil may be performed during grading to reduce the overall

expansive character of site soil. Thus, in general, surficial deposits of fill/colluvium and granitic bedrock are generally not considered detrimentally expansive.

6. Representative samples of site earth materials were evaluated for corrosion potential, soluble sulfates, and soluble chlorides. Laboratory testing indicates that the tested samples are mildly alkaline with respect to soil acidity/alkalinity; are mildly corrosive to exposed, buried metals when saturated; present elevated sulfate exposure to concrete ("Exposure Class S1" per Table 19.3.1.1 of American Concrete Institute (ACI) 318-14); and have slightly elevated concentrations of soluble chlorides. It should be noted that GSI does not consult in the field of corrosion engineering. Therefore, additional comments and recommendations may be obtained from a qualified corrosion engineer based on the level of corrosion protection required for the project, as determined by the Project Architect, Civil Engineer, and/or Structural Engineer.

7. It should be noted that the 2019 CBC (CBSC, 2019) indicates that removals of unsuitable soils be performed across all areas to be graded, under the purview of the grading permit, and not just within the influence of the proposed facility 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. In general, any planned settlement-sensitive improvement located above a 1:1 (h:v) projection up from the bottom, outboard edges of the remedial grading excavations at the property boundaries would be affected by perimeter conditions. On a preliminary basis, any planned settlement-sensitive improvements located within approximately 2½ and 3 feet from the property boundaries would require deepened foundations or additional reinforcement by means of ground improvement or specific structural design. Otherwise, these improvements may be subject to distress and a reduced service life.

8. Building foundations will need to be designed to accommodate the potential static and seismic deformations indicated herein.

9. A moderate to high amount of difficulty was encountered when excavating the test pits with a rubber tire backhoe, and the seismic line survey SL-1 shows that weathered bedrock/bedrock has average velocities exceeding non-trenchable and non-rippable cut-offs. Thus, on a preliminary basis, GSI anticipates productive ripping with a Caterpillar D-9R bulldozer (or equivalent) and productive trenching with an equivalent excavator to the planned excavation depths. This assumes excavation equipment in good working order. It should be noted that non-productive excavation into the granitic bedrock, even with heavy excavation equipment, may be encountered at approximately 3 to 5 feet B.E.G. Thus, all excavation equipment should be appropriately sized and powered for the required excavation task. Localized deep excavations (i.e., 3 to 5 feet B.E.G.) could require the use of rock breaking equipment such as hoe rams.

10. Planned and remedial excavations into the onsite earth material could generate oversized rock constituents that could require special handling and placement in engineered fills. On a preliminary basis, oversized rock constituents greater than 12 inches and 6 inches should not be included in fills placed within the building pad areas and underground utility trench backfills, respectively. Please note that underground utility providers may have stricter requirements regarding oversized materials in trench backfills.

11. Existing site improvements that are to remain in service and underlain by left-in-place undocumented fill will retain the potential to undergo settlement-related deformations and associated distress. In addition, given their age, these existing improvements may require increased maintenance and repairs, and perhaps replacement in time frames that are much shorter than the currently proposed improvements. This should be disclosed to all interested/affected parties.

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

13. General Earthwork and Grading Guidelines are provided at the end of this report as Appendix G. Specific recommendations are provided in the following section.

EARTHWORK CONSTRUCTION RECOMMENDATIONS

General

All grading should conform to the guidelines presented within Appendix Chapter J of the 2019 CBC (CBSC, 2019), the requirements of the County of San Diego, and the Grading Guidelines presented in Appendix G, except where specifically superceded in the text of this report. Prior to grading, a GSI representative should be present at the preconstruction meeting to provide additional grading guidelines, if needed, and review the earthwork schedule.

During earthwork construction, all site preparation and the general grading procedures of the contractor should be observed and the fill selectively tested by a representative(s) of GSI. If unusual or unexpected conditions are exposed in the field, they should be reviewed by this office and, if warranted, modified and/or additional recommendations will be offered. All applicable requirements of local and national construction and general industry safety orders, the Occupational Safety and Health Act (California Code Of Regulations CAL-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.

Consideration of Expansive Soils

Current laboratory testing indicates that the onsite soils exhibit an expansion index (E.I.) value of <21 (very low expansive). As such, the following foundation construction recommendations are intended to support planned improvements underlain by at least 7 feet of non-detrimentally expansive soils (i.e., E.I.<21 and PI <15). Although not anticipated based on the available data, should foundations be underlain by expansive soils, 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.

Demolition/Grubbing

1. Vegetation and any miscellaneous debris should be removed from the areas of proposed grading, including the removal of larger trees and their root systems.

2. Demolition of any existing improvements should be performed after an environmental review of the existing construction/building materials. All existing building materials, including foundations, should be removed from the site. If necessary, concrete generated from demolition work may be reused in engineered fills, provided that it is certified not to contain lead-based paints, all steel reinforcing bars are removed, and the concrete fragments are reduced to 6 inches or less in dimension. Existing asphaltic concrete fragments generated from site demolition should not be reused in engineered fills placed within the building pad. However, they may be reused in proposed parking lot fill areas provided that the fragments are fully cured and 2 inches or less in dimension, per Cal-Green requirements.

3. Any existing subsurface structures, underground utilities and vaults, ground rods, sumps, wells, etc., uncovered during the recommended removal should be observed by GSI so that appropriate remedial recommendations can be provided. Existing underground utility services that will be abandoned should be plugged at the property boundary in accordance with the utility provider's requirements.

4. Cavities or loose soils remaining after demolition and site clearance should be cleaned out and observed by the geotechnical consultant. The cavities should be replaced with fill materials that have been moisture conditioned to at least optimum moisture content and compacted to at least 92 percent of the laboratory standard (per ASTM D 1557) or backfilled with a 2- to 3-sack sand-cement slurry.

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

Treatment of Existing Ground

Remedial Excavations (Removals)

1. All colluvium, undocumented artificial fill (if encountered), and any highly weathered or loose disturbed zones of granitic bedrock (if encountered) should be removed to expose dense, relatively unweathered granitic bedrock. Based on the available subsurface data, removals depths on the order of 2½ to 3 feet should be anticipated. However, deeper removals cannot be precluded and variable removal depths should be considered in preliminary construction cost estimating. Some of the removals will be completed by virtue of the planned excavations. The removed soils may be reused as engineered fill, provided that the soil is cleaned of any deleterious or organic materials, rock constituents with sizes in excess of 12 inches in dimension, concrete fragments containing reinforcing steel or with sizes greater than 6 inches, or uncured asphaltic concrete fragments with sizes greater than 2 inches. As noted previously, acceptable asphaltic concrete fragments should only be reused below proposed parking lot areas. Removals should be completed throughout the area of proposed development and should extend at least 5 feet beyond the limits of any settlement-sensitive improvement or below a 1:1 (h:v) plane projected down from the bottom, outboard edge of any settlement-sensitive improvement (whichever is greater).

2. The bottoms of removal excavations should be observed by GSI, prior to scarification and engineered fill placement.

3. Subsequent to the completion of removals, the exposed bottom should be scarified to a depth of at least 8 inches, brought to <u>at least</u> optimum moisture content, and recompacted to a minimum relative compaction of 92 percent of the laboratory standard (ASTM D 1557), prior to any fill placement.

4. Localized deeper removals may be necessary due to buried underground utility trenches, septic systems, etc., that may be present.

Overexcavation

In order to mitigate any planned cut/fill transitions (i.e., engineered fill/bedrock contacts) or transitions due to significant heterogeneity of the granitic bedrock within the proposed building pad, and to provide uniform foundation support for the proposed facility building, the granitic bedrock within the building pad should be overexcavated (undercut) to the greater of either 3 feet below pad grade or 2 feet below the lowest foundation element, and replaced with compacted fill. In addition, the maximum to minimum fill thickness beneath the building foundations should not exceed a ratio of 3:1 (maximum:minimum), and should be sloped to drain to street areas or an approved drainage facility. GSI recommends that the overexcavation be completed to a minimum distance of 5 feet beyond the perimeter footings for the proposed facility building. Where deeper overexcavation is necessary to maintain the recommended 3:1 maximum to minimum fill ratio, the overexcavation should extend to a distance of "D" beyond the perimeter building footings, where "D" equals the overexcavation depth. The overexcavation bottoms should be observed by GSI prior to scarification and the installation of the replacement fill.

Subsequent to the completion of the overexcavation, the exposed bottom should be scarified to a depth of at least 8 inches, brought to at least optimum moisture content, and recompacted to a minimum relative compaction of 92 percent of the laboratory standard, prior to any fill placement.

Fill Placement

Subsequent to ground preparation, fill materials should be brought to at least optimum moisture content, placed in thin 6- to 8-inch lifts, and mechanically compacted to obtain a minimum relative compaction of 92 percent of the laboratory standard (ASTM D 1557). Fill materials should be cleansed of major vegetation and debris prior to placement.

Fill Suitability

The available subsurface data indicates that the excavated materials will generate oversized rock constituents. GSI recommends that rock constituents with sizes greater than 12 inches in dimension not be included in engineered fills, placed within the building pad area. In addition, rock constituents with sizes greater than 6 inches in dimension should not be included in backfills for underground utility trenches or retaining walls. However, underground utility providers may have stricter requirements for oversized rock constituents, included in underground utility trench backfills. Utility backfill requirements should be revisited by the design team once civil utility drawings are available. Concrete fragments, absent of reinforcing steel, may be included in engineered fills, provided that their sizes do not exceed those recommended above. Fully cured asphaltic concrete fragments may be included in engineered fills placed below proposed drive lanes and parking areas, only, provided that their sizes do not exceed 2 inches in dimension. Nesting of rocks, concrete, or asphaltic concrete fragments should not be performed. That is to say they should be incorporated into a matrix of engineered fill soil.

Any soil import should be evaluated by this office prior to importation in order to assure compatibility with the onsite site soils and the recommendations presented in this report. Import soils, if used, should be relatively sandy and very low expansive (i.e., E.I. $\leq$ 20 and P.I. $\leq$ 14). In addition, environmental reports of proposed import source sites should be provided to GSI prior to importation. At least five (5) business day will be required for document review and laboratory testing of any import submittals, prior to importation.

Temporary Slopes

On a preliminary basis, unsupported temporary slopes, exposing unsaturated colluvium and/or artificial fill (if encountered), with gross overall heights ranging between 4 and 20 feet, may be constructed in accordance with CAL-OSHA guidelines for Type B soils (i.e., 1:1 [h:v] slope) unless Stable Rock is present. If Stable Rock is encountered, temporary slopes may be constructed in accordance with CAL-OSHA guidelines for Stable Rock (i.e., vertical [90˚]) slope. Soils and building materials should not be stockpiled nor should heavy equipment be stored, or operated within "H" feet of the tops of temporary slopes, where "H" equals the temporary slope height. All temporary slopes should be observed by a licensed engineering geologist or geotechnical engineer prior to entry by an unprotected worker. Should adverse conditions be exposed, the temporary slopes may require flatter gradients and/or the use of shoring. If the recommended temporary slopes conflict with property lines or existing improvements that need to remain in service, shoring or alternating slot excavations recommendations would need to be evaluated.

Graded Slopes

The grading plans (EXE, 2021) appear to show minor graded slopes south of the proposed Building 2, and near the concrete generator pad. The proposed graded slopes are shown as being 5 feet or less in overall height, and sloped at a 2:1 [h:v]. General guidelines for graded slope construction (including basal shear keys) are provided below and in Appendix G. These guidelines may require amendments once grading plans have been provided for our review.

Fill Slopes

Graded fill slopes should be properly keyed (see Appendix G) and benched and constructed at gradients no steeper than 2:1 (h:v) without further evaluation. The fill slopes should be constructed of fill materials placed and compacted in accordance with the recommendations in the "Fill Placement" section of this report, including the slope face.

Cut Slopes (Temporary)

Graded cut slopes should be constructed at gradients no steeper than 2:1 (h:v) without further evaluation. All graded cut slopes should be mapped during grading to evaluate the presence of adverse geologic structures (daylighted joint and/or fracture planes), highly weathered bedrock, or other geologic conditions that could affect cut slope stability. Should adverse conditions be exposed, mitigation measures would be recommended. Such mitigation measures may include, but not necessarily be limited to: inclining cut slopes to flatter gradients or stabilization fills.

Earthwork Balance (Shrinkage/Bulking)

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

Undifferentiated Artificial Fill/Quaternary Colluvium . . . . . . 5% to 10% shrinkage
Bedrock
75% Earth/25% Rock (weathered bedrock) . . . . 8% shrinkage
50% Earth/50% Rock . . . . 5% shrinkage
25% Earth/75% Rock (unweathered bedrock) . . . . 12% bulking

It should be noted that the above factors are estimates only, based on preliminary data. Existing colluvium (or fill, if encountered) may achieve higher shrinkage if organics or clay content is higher than anticipated, or if compaction averages more than 92 percent of the laboratory standard (ASTM D 1557). 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.

Excavation Observation and Monitoring (All Excavations)

When excavations are made adjacent to an existing improvement (i.e., underground utilities, walls, roads, buildings, parking lots, etc.) there is a risk of some damage even if a well designed system of excavation is planned and executed. We recommend, therefore, that a systematic program of observations be made before, during, and after construction to determine the effects (if any) of construction on existing improvements.

We believe that this is necessary for two reasons: First, if excessive movements (i.e., more than ½ inch) are detected early enough, remedial measures can be taken which could possibly prevent serious damage to existing improvements. Second, the responsibility for damage to the existing improvement can be determined more equitably if the cause and extent of the damage can be determined more precisely.

Monitoring should include the measurement of any horizontal and vertical movements of the existing structures/improvements. Locations and type of the monitoring devices should be selected prior to the start of construction. The program of monitoring should be agreed upon between the project team, the site surveyor and the Geotechnical Engineer-of-Record, prior to excavation.

Reference points should be made on existing walls, buildings, and other settlement-sensitive improvements within "H" of the top of an excavation/cut, where "H" equals the depth of the excavation/cut. These points should be placed as low as possible on the improvements adjacent to the excavation/cut. Exact locations may be dictated by critical points, such as bearing walls or columns for buildings; and surface points on site walls, pavements, or curbs near the top of the excavation.

For a survey monitoring system, an accuracy of a least 0.01 foot should be required. Reference points should be installed and read initially prior to excavation. The readings should continue until all construction below ground has been completed and the permanent backfill has been brought to final grade.

The frequency of readings will depend upon the results of previous readings and the rate of construction. Weekly readings could be assumed throughout the duration of construction with daily readings during rapid excavation near the bottom of the excavation. The reading should be plotted by the Surveyor and then reviewed by the Geotechnical Engineer. In addition to the monitoring system, it would be prudent for the Geotechnical Engineer and the Contractor to make a complete inspection of the existing structures both before and after construction. The inspection should be directed toward detecting any signs of damage, particularly those caused by settlement. Pre-construction notes should be made and photographs or video recordings should be taken where necessary.

Observation

It is recommended that all excavations be observed by the Geologist and/or Geotechnical Engineer. Any fill which is placed should be approved, tested, and verified if used for engineered purposes. Should the observation reveal any unforseen hazard, the Geologist or Geotechnical Engineer will recommend treatment. Please inform GSI at least 24 hours prior to any required site observation.

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 proposed 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, configuration, or loading conditions of the proposed facility building 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.

Based on the very low expansion potential of the onsite soils, preliminary recommendations for conventional-type foundation and slab-on-grade floor systems are included herein. As previously mentioned, final recommendations for foundation design and construction would be provided at the conclusion of grading, and would be based on the as-graded soil conditions and the expansion potential of soils near pad grade. These assumptions should be revisited once the final plans and structural design are made available to this office.

Preliminary Foundation Design - Conventional Foundation Systems

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

2. A conventional foundation and slab-on-grade floor system may be used to support the proposed facility building, provided that soils within the upper 7 feet of pad grade exhibit an E.I. $\leq$ 20 and a P.I. $\leq$ 14. This will require further evaluation either during or following grading.
3. Based on the anticipated foundation loads and laboratory test results, it is our opinion that the proposed facility building can be favorably supported by shallow foundations on at least 3 feet of recompacted fill soils overlying the dense, relatively unweathered granitic bedrock. Building loads may be supported on continuous or isolated spread footings (typically 12 to 18 inches below pad grades) designed in accordance with the following recommendations:

ALLOWABLE BEARING VALUES FOR FOOTINGS*		
DEPTH BELOW LOWEST ADJACENT FINISHED GRADE (INCHES)	ALLOWABLE BEARING CAPACITY FOR INTERIOR SPREAD FOOTINGS (MINIMUM WIDTH = 2 FEET)	ALLOWABLE BEARING CAPACITY FOR CONTINUOUS WALL FOOTINGS (MINIMUM WIDTH = 1 FOOT)
12	3.5 ksf	3.5 ksf
18	4.0 ksf	4.0 ksf

* For the allowable bearing capacity indicated in the above table, the factor of safety used in our analysis in $\geq$ 3. Therefore, the Factor-of-Safety (FOS) with regard to the bearing on the foundation will not be less than 3 (this uses an Ω_0 of at least 3.0). GSI will revisit the structural loads and foundation plans for bearing pressures, when these drawings and calculations are available.

4. 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 group action. Reinforcement should be designed in accordance with local codes and structural considerations.

5. The recommended allowable bearing capacity is generally based on maximum total and differential settlements indicated in this report for building areas. Actual settlement can be estimated on the basis that settlement is roughly proportional to

the net contact bearing pressure. The majority of the settlement should occur during construction. Since settlement is a function of footing size and contact bearing pressure, some differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists. However, for most cases, static differential settlements are considered unlikely to those previously indicated. With increased footing depth/width ratios, differential settlement should be less, provided a minimum of 3 feet of compacted fill overlying dense, relatively unweathered granitic bedrock is maintained beneath all footings. GSI should review foundation plans and should evaluate foundation-specific load patterns.

6. A passive earth pressure may be computed as an equivalent fluid having a density of 250 pcf, with a maximum lateral earth pressure of 2,500 psf for compacted low expansive fill, and 350 pcf and a maximum lateral pressure of 3,500 psf for foundation elements embedded in granitic bedrock.

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

8. 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.

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

10. 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.

11. Subgrade soils that meet the criteria of detrimentally expansive soils as defined in Section 1803.5.3 of the 2019 CBC should use specialized foundations. In addition to the design/construction recommendations presented herein, foundation systems constructed within the influence of detrimentally expansive soils (i.e., E.I. > 20 and PI $\geq$ 15) will require specific design to resist expansive soil effects per Sections 1808.6.1 or 1808.6.2 of the 2019 CBC, and should be reviewed by the project structural engineer.

12. Foundations should be design for the anticipated settlement values in the "Preliminary Settlement Evaluation" section of this report.

PRELIMINARY FOUNDATION CONSTRUCTION RECOMMENDATIONS

Current laboratory testing indicates that onsite soils do not 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. The following foundation construction recommendations are intended to support planned improvements underlain by at least 7 feet of non-detrimentally expansive soils (i.e., E.I.<21 and P.I. <15). Should foundations be underlain by expansive soils at depths of less than 7 feet, 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 engineered fill at a minimum depths of 12 inches to 18 inches below the lowest adjacent grade, and a minimum width of 12 or 15 inches, for the planned, one- or two-story floor load structure, respectively. Isolated, exterior column and panel pads, or wall footings, should be at least 24 inches, square, and founded at a minimum depth of 24 inches into properly engineered fill or bedrock. All footings 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 of pad footings 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 one direction. The grade beam should be at least 12 inches square in cross section, and should be provided with a minimum of one No. 4 reinforcing bar at the top, and one No. 4 reinforcing bar at the bottom of the grade beam. The base of the reinforced grade beam should be at the same elevation as the adjoining footings.

3. A grade beam, reinforced as previously recommended and at least 12 inches square, should be provided across large (garage) entrances. The base of the reinforced grade beam should be at the same elevation as the adjoining footings.

4. A minimum concrete slab-on-grade thickness of 5 inches is recommended. Recommendations for floor slab underlayment are presented in a later section of this report.

5. Concrete slabs should be reinforced with a minimum of No. 3 reinforcement bars placed at 18-inch on centers, in two horizontally perpendicular directions (i.e., long axis and short axis).

6. 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.

7. 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.

8. Soils generated from footing excavations to be used onsite should be compacted to a minimum relative compaction of 92 percent of the laboratory standard (per 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. Loose, excess soils should be removed from the building pad prior to final preparation of the slab subgrade. The slab subgrade should be evaluated by GSI prior to placement of the slab underlayment section.

9. Reinforced concrete mix design should conform to "Exposure Class S1, W0, and C1" in Table 19.3.1.1 of ACI 318R.

SOIL MOISTURE TRANSMISSION CONSIDERATIONS

GSI has evaluated the potential for vapor or water transmission through the concrete floor slabs, 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 manufacturer's 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, 2021). 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/developer expectations and repairs/replacement). It should also be noted that moisture transmission will occur in new slab-on-grade floors as a result of chemical reactions taking place within the curing concrete. Moisture 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 moisture transmission rates without significant distress, the following alternatives are provided:

- The thickness of concrete slab-on-grade floors may be increased to improve performance and reduce vapor/moisture transmission (Kinare, 2005).

- 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 the latest editions of 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 slab-on-grade floors should 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. Testing may be performed on the excavated granitic bedrock materials during the planned excavations to evaluate if these materials are suitable for slab underlayment sand.

  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.

- For building pads underlain by very low expansive soil conditions, the vapor retarder should 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.

- Concrete with a maximum water to cement ratio of 0.5 should be used in foundation and slab-on-grade floor construction.

- 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 flooring contractor 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 waterproofing consultant, 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.

RETAINING WALL DESIGN PARAMETERS

Conventional Retaining Walls

The design parameters provided below assume that either non expansive soils (typically Class 2 permeable filter material or Class 3 aggregate base) or native onsite materials (up to and including an E.I. of 20) are used to backfill any retaining walls. 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. To reduce the potential for site retaining walls to experience efflorescence staining, they may also be water-proofed. The foundation system for the proposed retaining walls should be designed in accordance with the recommendations presented in this and preceding sections of this report, as appropriate. Recommendations for specialty walls (i.e., crib, earthstone, geogrid, etc.) can be provided upon request, and would be based on site specific conditions.

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, relatively unweathered granitic bedrock. This pressure may be increased by one-third for short-term wind and/or seismic loads.

Passive Earth Pressure - A passive earth pressure of 250 pcf with a maximum lateral 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 very low to medium expansive engineered fill.

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

Backfill Soil Density - Soil densities ranging between 120 pcf and 130 pcf may be used in the design of retaining wall foundations. This assumes a mantle or relatively thin fill with an average engineered fill compaction of at least 92 percent of the laboratory standard (ASTM D 1557) overlying dense, relatively unweathered granitic bedrock.

Any retaining wall footing near the perimeter of the site will likely need to be deepened into suitable bedrock 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 San Diego Regional Standard Design (SDRSD). However, owing to the low equivalent fluid pressures used in the design of SDRSD retaining walls, select backfill materials would be required. 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. For heavy-axle (HS20) truck loads, a traffic surcharge of 300 psf/ft should be applied in the upper 5 feet of the backfill. 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	48
2 to 1	55	65

[1] Level backfill behind a retaining wall is defined as compacted earth materials, properly drained, without a slope for a distance of 2H behind the wall, where H is the height of the wall.
[2] SE $\geq$ 30, P.I. < 15, E.I. < 21, and $\leq$ 10% passing No. 200 sieve.
[3] E.I. = 0 to 50, SE $\geq$ 20, P.I. < 25, and $\leq$ 20% passing No. 200 sieve (may not be sufficiently available onsite).
NOTE: The use of clay as wall backfill is prohibited.

Seismic Surcharge

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

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

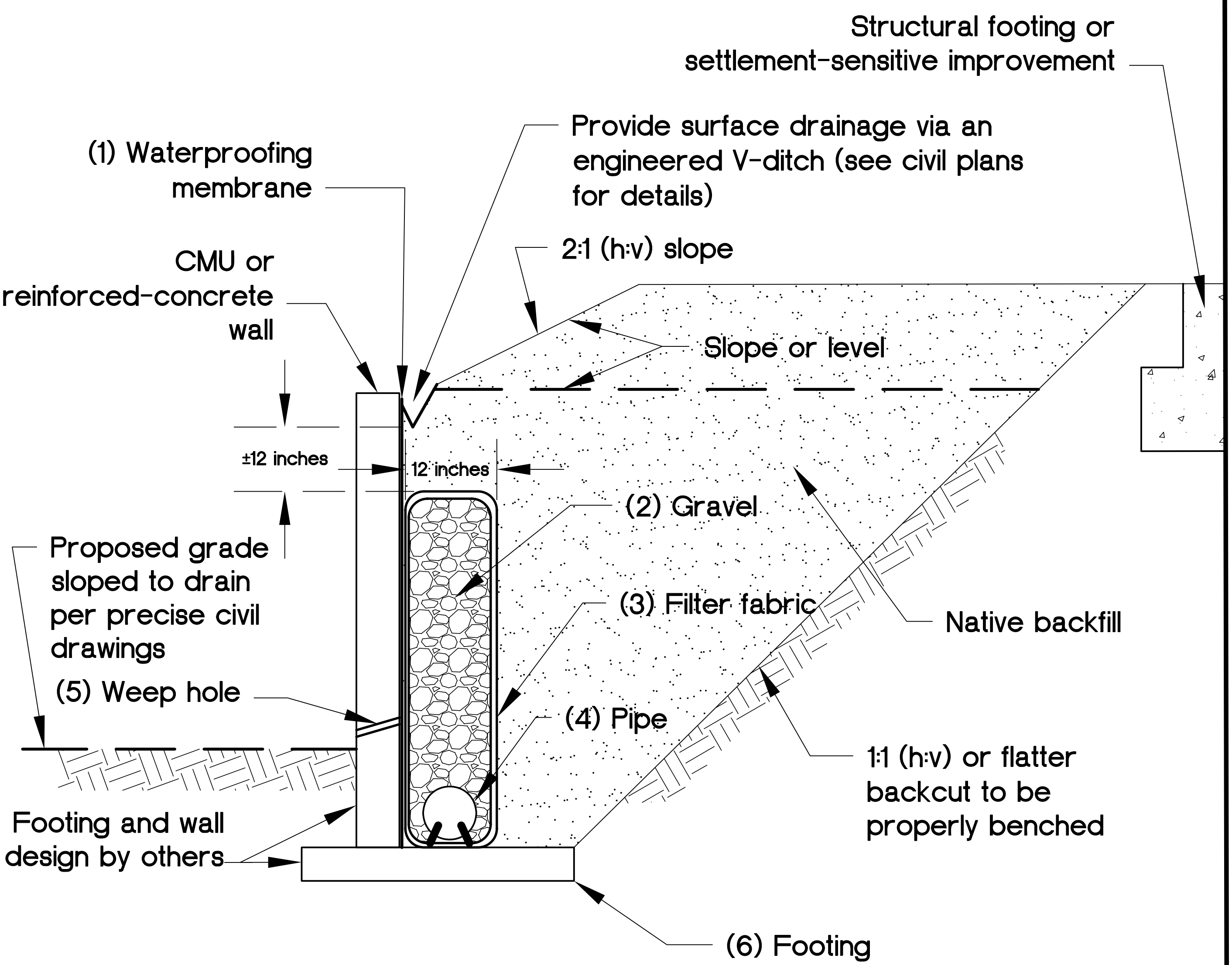
$$P_h = \tfrac{3}{8} \bullet a_h \bullet \gamma_t H$$

Where:	P_h	=	Seismic increment
	a_h	=	Probabilistic horizontal site acceleration with a percentage of "g"
	γ_t	=	total unit weight (110 to 120 pcf for site soils @ 90% relative compaction)
	H	=	Height of the wall from the bottom of the footing or point of supporting 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. Materials with an E.I. greater than 20 should not be used as backfill for retaining walls. For more onerous expansive situations, backfill and drainage behind the retaining wall should conform with Detail 3 (Retaining Wall And Subdrain Detail Clean Sand Backfill). Retaining wall backfill should be moisture conditioned and mixed to at least optimum moisture content and be compacted to at least 92 percent of the laboratory standard (ASTM D 1557). GSI should perform observation and field density testing during retaining wall 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



(1) Waterproofing membrane.

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

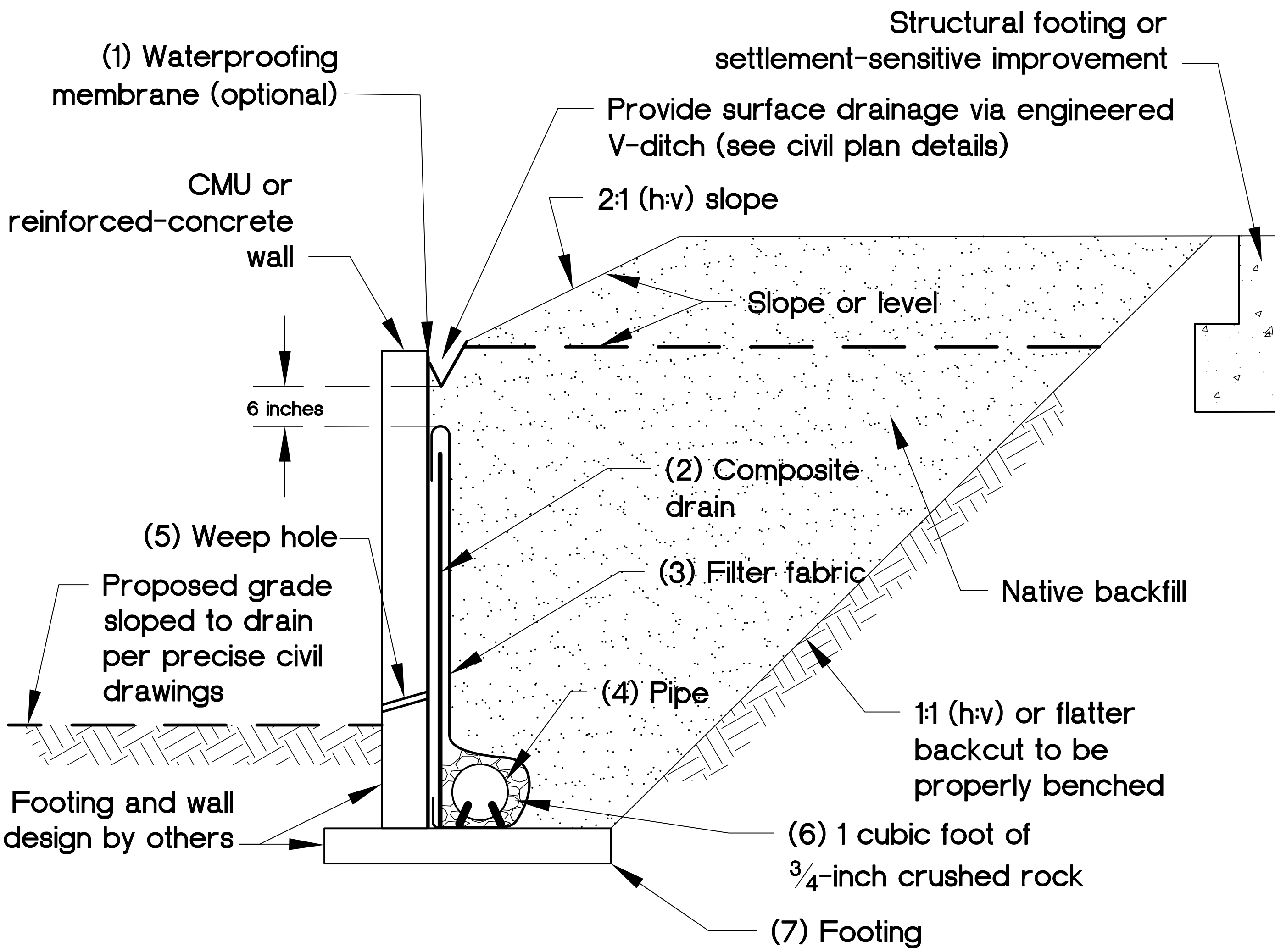
(3) Filter fabric: Mirafi 140N or approved equivalent.

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

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

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

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



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

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

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

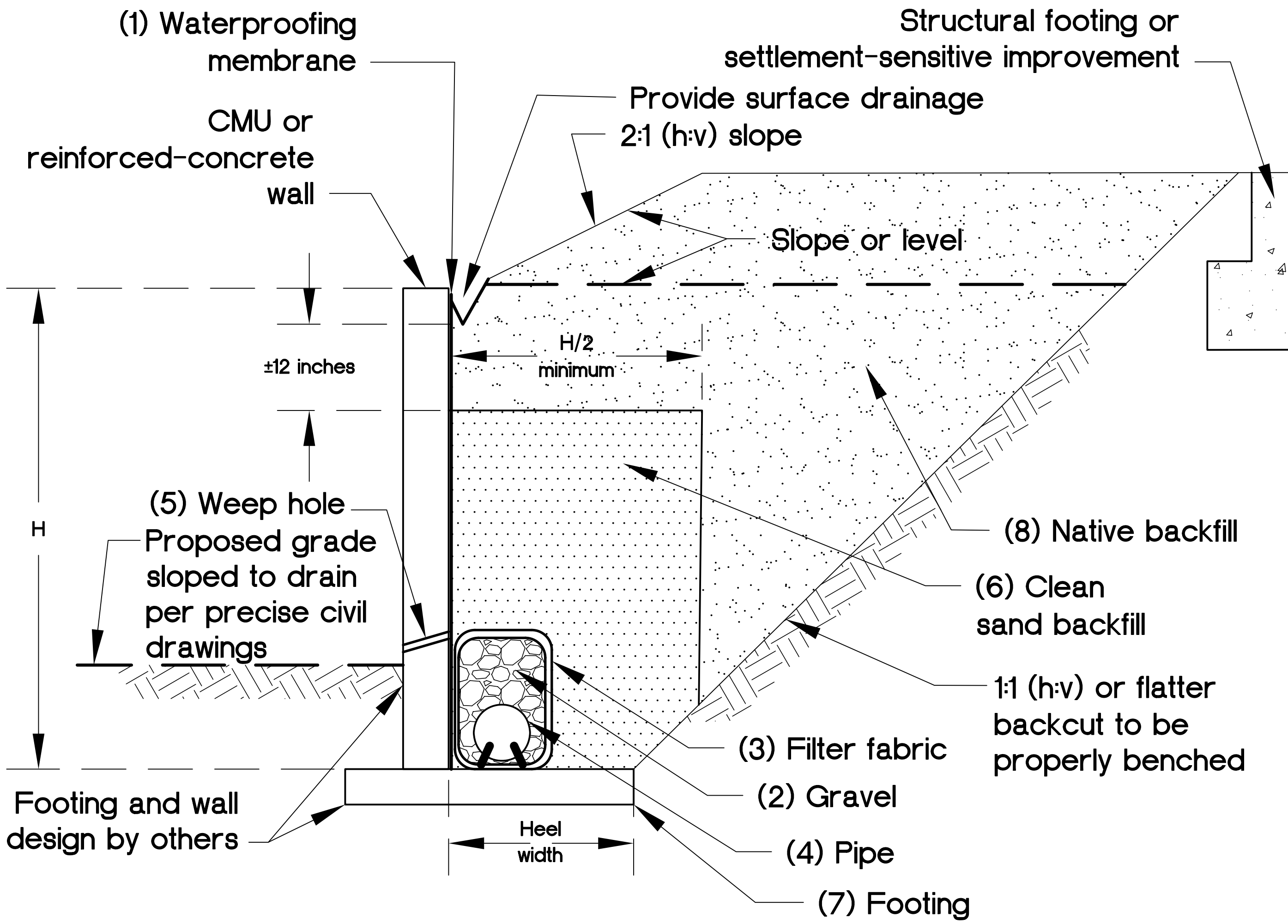
(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

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

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

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

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



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

(2) Gravel: Clean, crushed, $^{3}/_{4}$ to $1^{1}/_{2}$ inch.

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

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

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

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

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

(8) Native backfill: If E.I. <21 and S.E. ≥ 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

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, FLATWORK, AND OTHER IMPROVEMENTS

The soil materials on site may be expansive. The effects of expansive soils are cumulative, and typically occur over the lifetime of any improvements. On relatively level areas, when the soils are allowed to dry, the desiccation and swelling process tends to cause heaving and distress to flatwork and other improvements. The resulting potential for distress to improvements may be reduced, but not totally eliminated. To that end, it is recommended that the owner/developer should notify any interested/affected parties of this long-term potential for distress. To reduce the likelihood of distress, the following recommendations are presented for all exterior flatwork:

1. Concrete slabs should be founded entirely on properly compacted fill. The subgrade area for concrete slabs should be compacted to achieve a minimum 92 percent relative compaction, and then be presoaked to 2 to 3 percentage points above (or 125 percent of) the soils' optimum moisture content, to a depth of 18 inches below subgrade elevation. This moisture content should be maintained

in the subgrade soils during concrete placement to promote uniform curing of the concrete and minimize the development of unsightly shrinkage cracks. If very low expansive soils are present, only optimum moisture content, or greater, is required and specific presoaking is not warranted. The moisture content of the subgrade should be proof tested within 72 hours prior to pouring concrete.

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

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

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

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

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

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

8. Planters and walls should not be tied to the building.

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

10. 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.

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

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

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

14. Shrinkage cracks could become excessive if proper finishing and curing practices are not followed. Finishing and curing practices should be performed per the Portland Cement Association 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.

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

ONSITE PRELIMINARY PAVEMENT DESIGN/CONSTRUCTION

Structural Section

Traffic Indices (TI) were assumed as 5.0 for the onsite driveway/parking, and should be reviewed by the Project Civil Engineer for comment, and any revisions, as necessary. An assumed R-Value of 35 has been issued for the material onsite, as R-value testing of onsite soils was not performed for this study. The recommended pavement sections for both asphaltic concrete (A.C.) pavement over aggregate base (A.B.), and Portland cement concrete pavement (PCCP), are provided in the following tables. Final pavement design should be based on subgrade R-values obtained from testing at the conclusion of grading and underground utility trench backfill.

ASPHALTIC CONCRETE (AC) PAVEMENTS				
APPROXIMATE TRAFFIC AREA	**TRAFFIC INDEX(1)**	**SUBGRADE R-VALUE(2)**	**A.C. THICKNESS (INCHES)**	**A.B. THICKNESS(3) (INCHES)**
Driveway/Parking Stalls	5.0	35	3.0 4.0	4.5 3.0
Loop Road/Driveway	5.0	35	3.0 4.0	4.5 3.0
(1) The T.I. is an estimation based on the intended use. The T.I. should be reviewed for comment by the Project Civil Engineer. Trash disposal areas, entry areas, fire vehicle access may require special design detailing. (2) Estimated R-Value based on site soils (3) Denotes Class 2 Aggregate Base R >78, SE >25)				

PORTLAND CONCRETE CEMENT PAVEMENTS (PCCP)					
TRAFFIC AREAS	**CONCRETE TYPE**	**PCCP THICKNESS (inches)**	**TRAFFIC AREAS**	**CONCRETE TYPE**	**PCCP THICKNESS (inches)**
Light Vehicles	520-C-2500	7.0	Heavy Truck Traffic	520-C-2500	9.0
	560-C-3250	6.0		560-C-3250	8.0
NOTE: All PCCP is designed as un-reinforced and bearing directly on compacted subgrade. However, a 6-inch thick leveling course of compacted aggregate base may be considered where pavement subgrade is uneven due to the presence of rock. All PCCP should be properly detailed (jointing, etc.) per the industry standard. Pavements may be additionally reinforced with #4 reinforcing bars, placed 12 inches on center, each way, for improved performance. Trash truck loading pads (aprons) shall adhere to the County's minimum thickness.					

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

The recommended pavement sections provided above are intended as a minimum guideline. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. If the ADT (average daily traffic) or ADTT (average daily truck traffic) increases beyond that intended, as reflected by the TI used for design, increased maintenance and repair could be required for the pavement section. Consideration should be given to the increased potential for distress from overuse of paved street areas by heavy equipment and/or construction related heavy traffic (e.g., concrete trucks, loaded supply trucks, etc.), particularly when the final section is not in place (i.e., topcoat). Best management construction practices should be followed at all times, especially during inclement weather.

ONSITE STORM WATER TREATMENT AND HYDROMODIFICATION MANAGEMENT

United States Department of Agriculture/Natural Resources Conservation Service Study

A review of soil survey mapping by the United States Department of Agriculture/Natural Resources Conservation Service ([USDA], 1973; [USDA/NRCS], 2021) indicates that the onsite soils predominately consist of the Vista coarse sandy loam (VsE), 15 to 30 percent slopes. However, on the northern half of the property (offsite), where three (3) infiltration basins are proposed for adjacent construction, the USDA/NRCS (2021) indicate that the onsite soils consist of the Placentia sandy loam (PfC), thick surface, 2 to 9 percent slopes. The USDA/NRCS (2021) further indicate that the onsite soils are classified as belonging to Hydrologic Soil Group (H.S.G.) “B,” while the offsite soils surrounding the proposed infiltration basins are classified as belonging to H.S.G. “D”, which generally limits the use of infiltration-based stormwater management systems (County of San Diego, 2014). It should be noted that storm water management of the site will be handled by the three (3) offsite infiltration basins to the north, and as such, infiltration feasibility will only take into account the PfC H.S.G. “D” soils that the basins are located within.

Infiltration Feasibility

A review of USDA (1973) and USDA/NRCS (2021) indicates that the capacity of the most limiting layer to transmit water (Ksat) within the Placentia sandy loam soil, is very low to moderately low (0.00 to 0.06 inches per hour [in/hr]). GSI points out that absent further evaluation, Table B.2-3 of County of San Diego (2020) allows for a default design infiltration rate of 0.025 inches/hour to be used when HSG “D” soils are present. This design infiltration rate is generally below the recommended feasibility threshold of 0.52 inches per hour per the Environmental Protection Agency (Clar, et al., 2004), and 0.50 inches per hour per the County (2020) for full infiltration. In general, the permeability of the underlying granitic bedrock can be expected to decrease with depth; thereby, promoting the development of perched groundwater, which can migrate laterally and adversely affect the proposed onsite improvements as well as existing onsite and offsite improvements. In addition, perched groundwater originating from stormwater infiltration could negatively affect the stability of onsite slopes and building pads. Lastly, some of the onsite earth materials may exhibit low expansion potentials locally. Thus, laterally migrating of perched groundwater from stormwater infiltration has the potential to exacerbate the shrink/swell effects of potentially expansive, onsite earth materials, and possibly lead to distress of improvements constructed within the influence of expansive soils.

Based on our review and engineering analysis, and in accordance with the County of San Diego BMP Design Manual (2020), the site generally appears unsuitable for stormwater infiltration from a geotechnical viewpoint. As such, a “no infiltration” BMP design is recommended. Furthermore, any basin constructed entirely of compacted fill would have similar Hydrologic Soil Group "D" properties, and recommendation for a "no infiltration" BMP design ([EPA], Clar, et al., 2004). For hydromodification structures located within 10 feet of a structure or settlement-sensitive improvement, stormwater treatment and hydromodification management should be designed for no infiltration. The civil designer should also take into account that any infiltrated storm water would likely perch upon the underlying granitic bedrock and migrate laterally, potentially adversely impacting improvements on adjoining properties, including underground utility trenches, and potentially activating expansive soils. An additional discussion of infiltration feasibility is presented in Appendix F, which contains considerations for geotechnical analysis of infiltration restrictions (Table D.1-1) and elements for determination of design infiltration rates (Table D.2-1), provided by the County (2020).

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 Strength at Break (ASTM D882): 73 (lb/in-width, min); Elongation (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): 15 (lb/in, min); Seam Peel Strength (ASTM D882) 2.6 (kN/m, min).

  The impermeable liner should extend upward to a few inches above the calculated high water (Q_{100}) elevation in order to reduce the potential for the collected water to pass underneath the liner.

- Subdrains used in conjunction with bioretention basins 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.

- Trenches for outlet pipes that exit permanent stormwater BMPs should be backfilled with a 2-sack sand-cement slurry for a horizontal distance of 5 feet outside the BMP.

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

DEVELOPMENT CRITERIA

Drainage

Adequate pad surface drainage is a very important factor in reducing the likelihood of adverse performance of foundations, hardscape, and slopes. Surface drainage should be sufficient to prevent ponding of water anywhere on a pad, and especially near structures. Pad surface drainage should be carefully taken into consideration during fine grading, landscaping, and building construction. Therefore, care should be taken that future landscaping or construction activities do not create adverse drainage conditions. Positive site drainage within pad and common areas should be provided and maintained at all times. Water should be directed away from foundations and not allowed to pond and/or seep into the ground. In general, the area within 5 feet around a structure should slope away from the structure. We recommend that unpaved lawn and landscape areas have a minimum gradient of 1 percent sloping away from structures, and whenever possible, should be above adjacent paved areas. Reversal of surface grades due to site settlement over time is possible and should be considered in the design of drainage and grades. Consideration should be given to avoiding construction of planters adjacent to structures (buildings, pools, spas, etc.). Pad drainage should be directed toward the street or other approved area(s). Although not a geotechnical requirement, roof gutters, 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

Onsite earth materials have a moderate to high erosion potential. Consideration should be given to providing hay bales and silt fences for the temporary control of surface water, from a geotechnical viewpoint.

Landscape Maintenance

Only the amount of irrigation necessary to sustain plant life should be provided. Over-watering the landscape areas will adversely affect proposed site improvements. We would recommend that any proposed open-bottom planters adjacent to proposed structures be eliminated for a minimum distance of 10 feet. As an alternative, closed-bottom type planters could be utilized. An outlet placed in the bottom of the planter, could be installed to direct drainage away from structures or any exterior concrete flatwork. If planters are constructed adjacent to structures, the sides and bottom of the planter should be provided with a moisture barrier to prevent penetration of irrigation water into the subgrade. Provisions should be made to drain the excess irrigation water from the planters without saturating the subgrade below or adjacent to the planters. 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 92 percent minimum relative compaction.

Gutters and Downspouts

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

Subsurface and Surface Water

Subsurface and surface water, may be encountered during construction or grading, but are generally 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. GSI recommends that all existing subdrains be located and evaluated for operations. 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., therapy 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 the owners, any owners association, and/or other interested parties. This office should be notified in advance of any fill placement, grading of the site, or trench backfilling after rough grading has been completed. This includes any grading, utility trench and retaining wall backfills, flatwork, etc.

Tile Flooring

Tile flooring can crack, reflecting cracks in the concrete slab below the tile, although small cracks in a 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 92 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 all subsurface excavations and trenching. Shoring or excavating the trench walls/backcuts 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. Additional comments are contained in Appendix G.

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 92 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 92 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 and shoring.
- During placement of subdrains or other subdrainage devices, prior to placing fill and/or backfill.
- After excavation of building footings, piers, retaining wall footings, and free standing walls footings, prior to the placement of reinforcing steel or concrete.
- Prior to pouring any slabs or flatwork, after presoaking/presaturation of building pads and other flatwork subgrade, before the placement of concrete, reinforcing steel, capillary break (i.e., sand, pea-gravel, etc.), or vapor barriers (i.e., visqueen, etc.).
- During retaining wall subdrain installation, prior to backfill placement.
- During placement of backfill for area drain, interior plumbing, utility line trenches, and retaining wall backfill.
- When any unusual soil conditions are encountered during any construction operations, subsequent to the issuance of this report.
- When any developer or owner improvements, such as flatwork, 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 entirely 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.

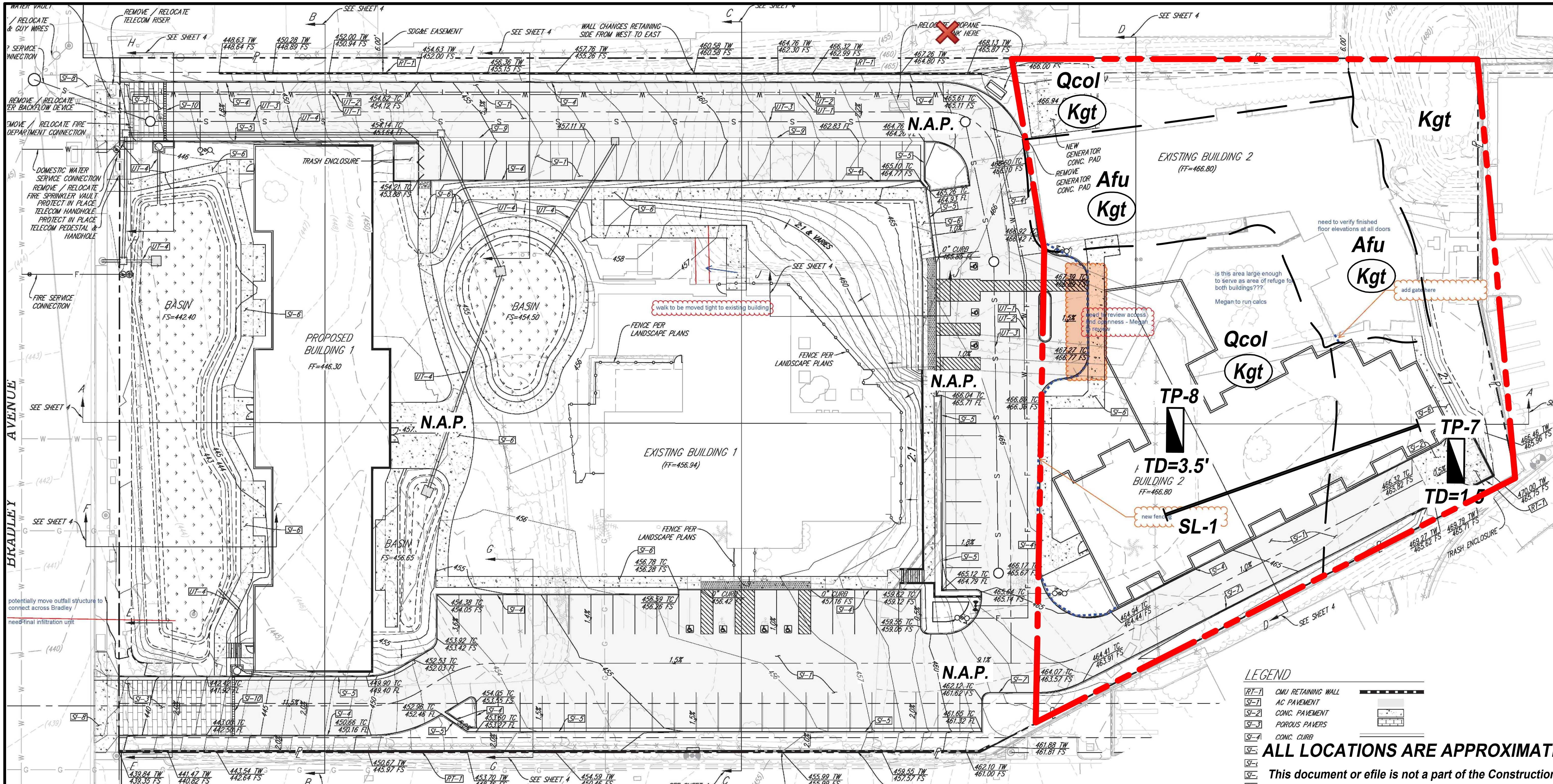
PLAN REVIEW

Final project plans (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.



GSI LEGEND

Afu	ARTIFICIAL FILL – UNDOCUMENTED	TP-8 TD=8'	APPROXIMATE LOCATION OF EXPLORATORY TEST PIT WITH TOTAL DEPTH IN FEET
Qcol	QUATERNARY COLLUVIUM, CIRCLED WHERE BURIED	SL-2	APPROXIMATE LOCATION OF SEISMIC SURVEY LINE
Kgt	CRETACEOUS TONALITE, CIRCLED WHERE BURIED		PROPOSED PLANNING AREA
?	APPROXIMATE LOCATION OF GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN	N.A.P.	NOT A PART OF THIS STUDY

N

GRAPHIC SCALE

40 0 20 40 80

1" = 40'

GeoSoils, Inc.

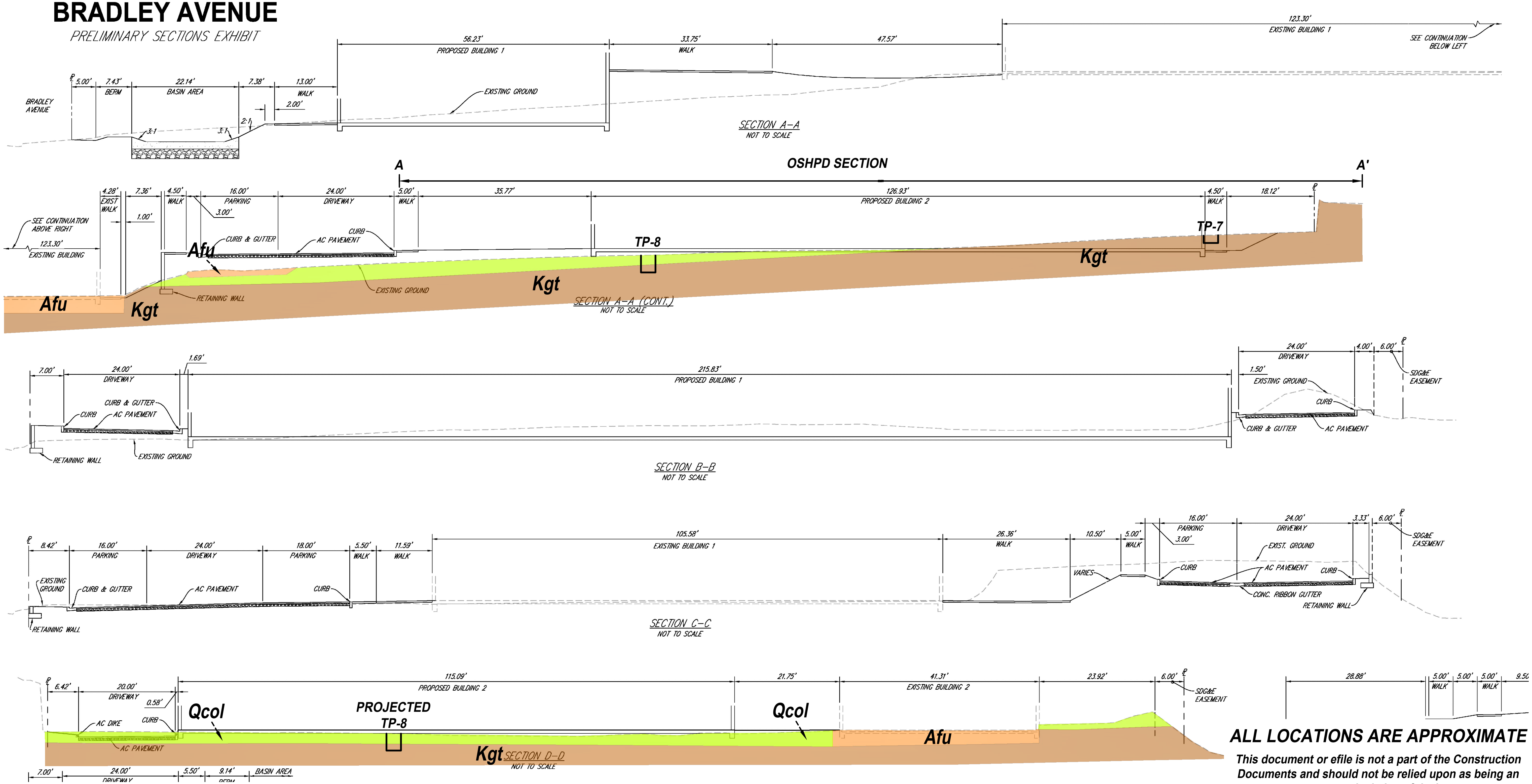
GEOTECHNICAL MAP

Plate 1

W.O.	DATE:	SCALE:
8096-A-SC	07/21	1" = 40'

BRADLEY AVENUE

PRELIMINARY SECTIONS EXHIBIT



ALL LOCATIONS ARE APPROXIMATE

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

GSI LEGEND

Afu	—	ARTIFICIAL FILL – UNDOCUMENTED
Qcol	—	QUATERNARY COLLUVIUM
Kgt	—	GRANITIC BEDROCK – TONALITE

GRAPHIC SCALE

20 0 10 20 40

1" = 20'

EXCEL ENGINEERING
LAND PLANNING • ENGINEERING • SURVEYING
440 STATE PLACE, ESCONDIDO, CA 92029
PH (760)745-8118 FX (760)745-1890

GeoSoils, Inc.

GEOLOGIC CROSS SECTIONS

Plate 2

W.O. 8096-A-SC | DATE: 07/21 | SCALE: 1"=20'

<u>APPENDIX A</u>

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APPENDIX A

REFERENCES

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APPENDIX B

TEST PIT LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM

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

CONSISTENCY OR RELATIVE DENSITY

CRITERIA

Standard Penetration Test

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

Standard Penetration Test

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

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

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

MOISTURE CONDITIONS

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

MATERIAL QUANTITY

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

OTHER SYMBOLS

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

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

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

GeoSoils, Inc.

W.O. 8096-A-SC
ARCO Construction
675 E. Bradley Ave., El Cajon
Logged By: MJS
4/28/2021

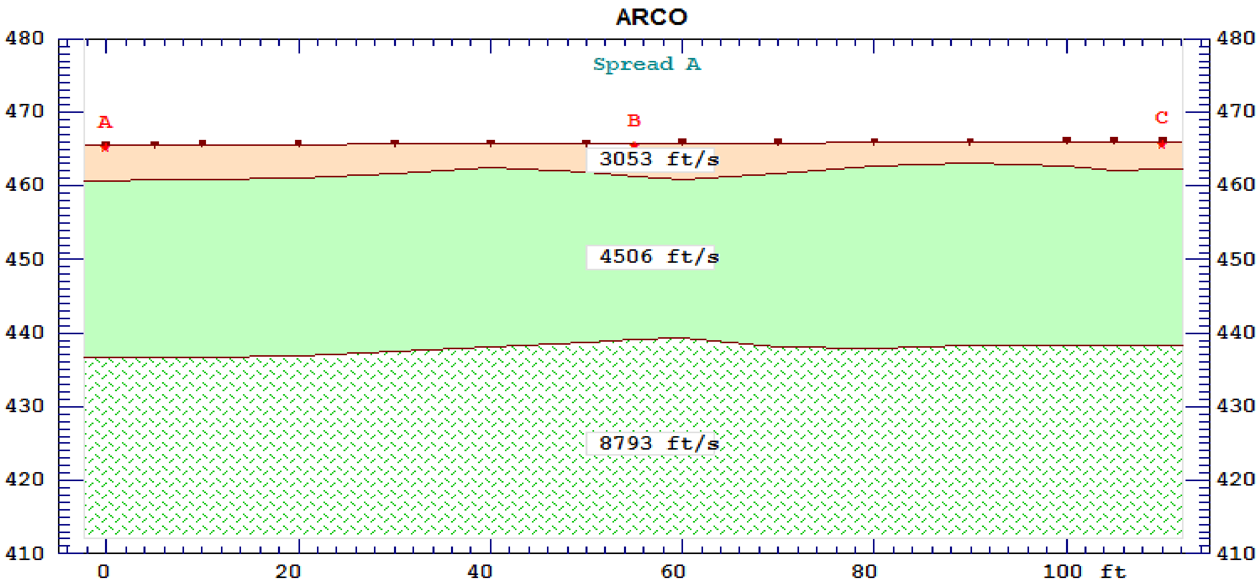
LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-7	470' MSL	0-1½	SM	Bulk @ 0-1½			**WEATHERED GRANITICS:** Disintegrates to SILTY SAND, yellowish brown to olive brown, dry to damp, very dense.
							Practical Refusal @ 1½' Total Depth = 1½' No Groundwater Encountered Backfilled 4/28/2021
TP-8	465' MSL	0-2½	SC				**COLLUVIUM:** CLAYEY SAND, moderate to dark reddish brown, dry to damp, medium dense to dense.
		2½-3½	SM-GM				**WEATHERED GRANITICS:** Disintegrates to SILTY GRAVELLY SAND (DG), dark yellowish brown, dry, very dense.
							Total Depth = 3½' No Groundwater Encountered Backfilled 4/28/2021

APPENDIX C

SEISMIC LINE SURVEY DATA

Seismic Line
SL-1
W.O. 8096-A-SC



North South

APPENDIX D

SEISMICITY DATA

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

DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 8096-A-SC

DATE: 05-27-2021

JOB NAME: Bradley Ave OSHPD

CALCULATION NAME: Test Run Analysis

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

SITE COORDINATES:
   SITE LATITUDE: 32.8177
   SITE LONGITUDE: 116.9523

SEARCH RADIUS: 62.4 mi

ATTENUATION RELATION: 13) Bozorgnia Campbell Niazi (1999) Hor.-Hard Rock-Cor.
   UNCERTAINTY (M=Median, S=Sigma): S   Number of Sigmas: 1.0
   DISTANCE MEASURE: cdist
   SCOND: 1
   Basement Depth: .10 km   Campbell SSR: 0   Campbell SHR: 1
   COMPUTE PEAK HORIZONTAL ACCELERATION

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

MINIMUM DEPTH VALUE (km): 3.0

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	14.2(22.8)	7.2	0.272	IX
CORONADO BANK	27.0(43.4)	7.6	0.187	VIII
ELSINORE (JULIAN)	28.8(46.4)	7.1	0.122	VII
EARTHQUAKE VALLEY	33.1(53.2)	6.5	0.070	VI
ELSINORE (COYOTE MOUNTAIN)	35.8(57.6)	6.8	0.079	VII
NEWPORT-INGLEWOOD (Offshore)	36.3(58.4)	7.1	0.096	VII
ELSINORE (TEMECULA)	38.8(62.5)	6.8	0.073	VII
SAN JACINTO-COYOTE CREEK	49.7(80.0)	6.6	0.049	VI
SAN JACINTO - BORREGO	51.4(82.7)	6.6	0.047	VI
SAN JACINTO-ANZA	51.4(82.8)	7.2	0.071	VI
SUPERSTITION MTN. (San Jacinto)	61.1(98.3)	6.6	0.039	V
ELSINORE (GLEN IVY)	61.5(99.0)	6.8	0.045	VI

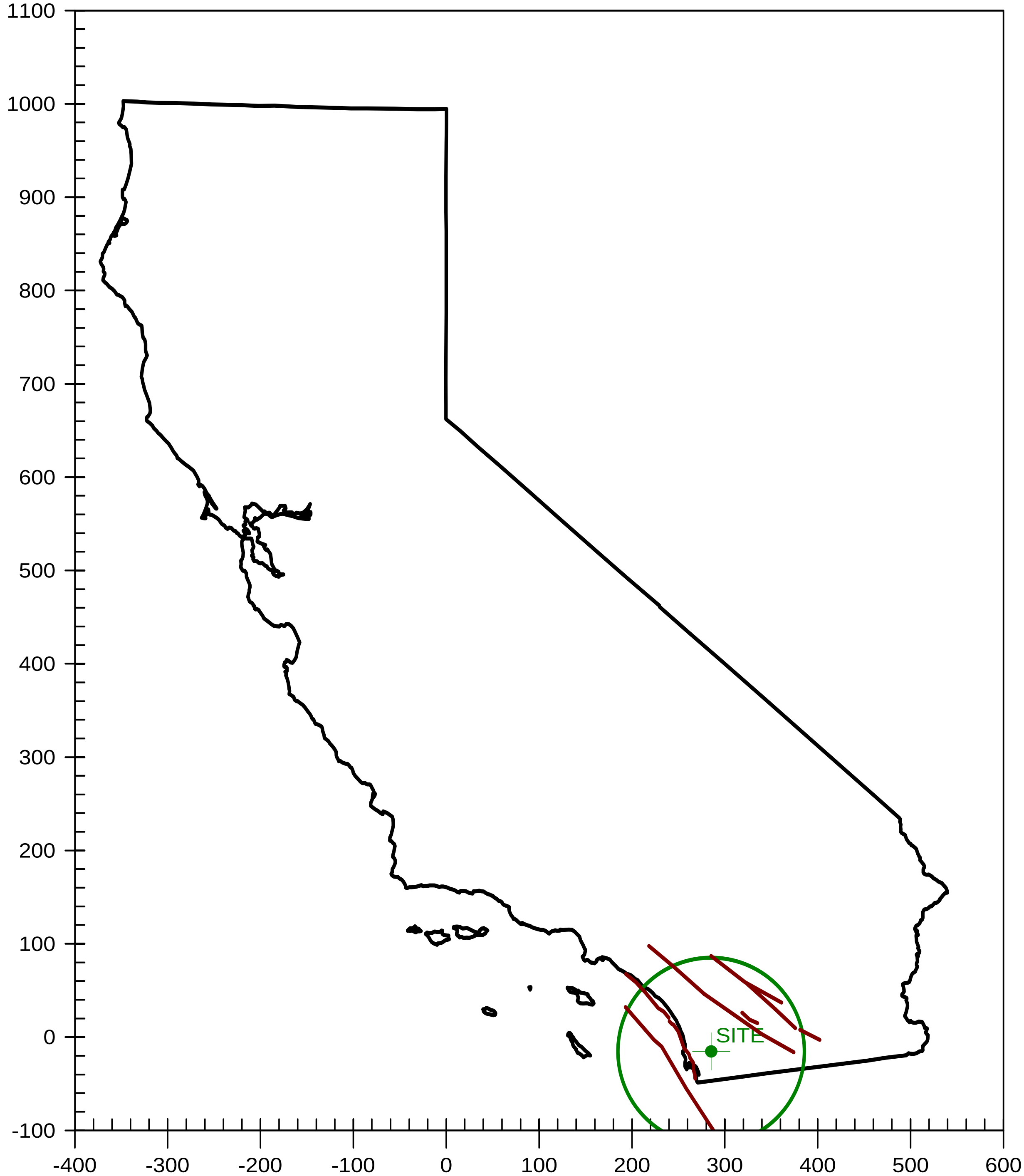
-END OF SEARCH- 12 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ROSE CANYON FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 14.2 MILES (22.8 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.2716 g

CALIFORNIA FAULT MAP

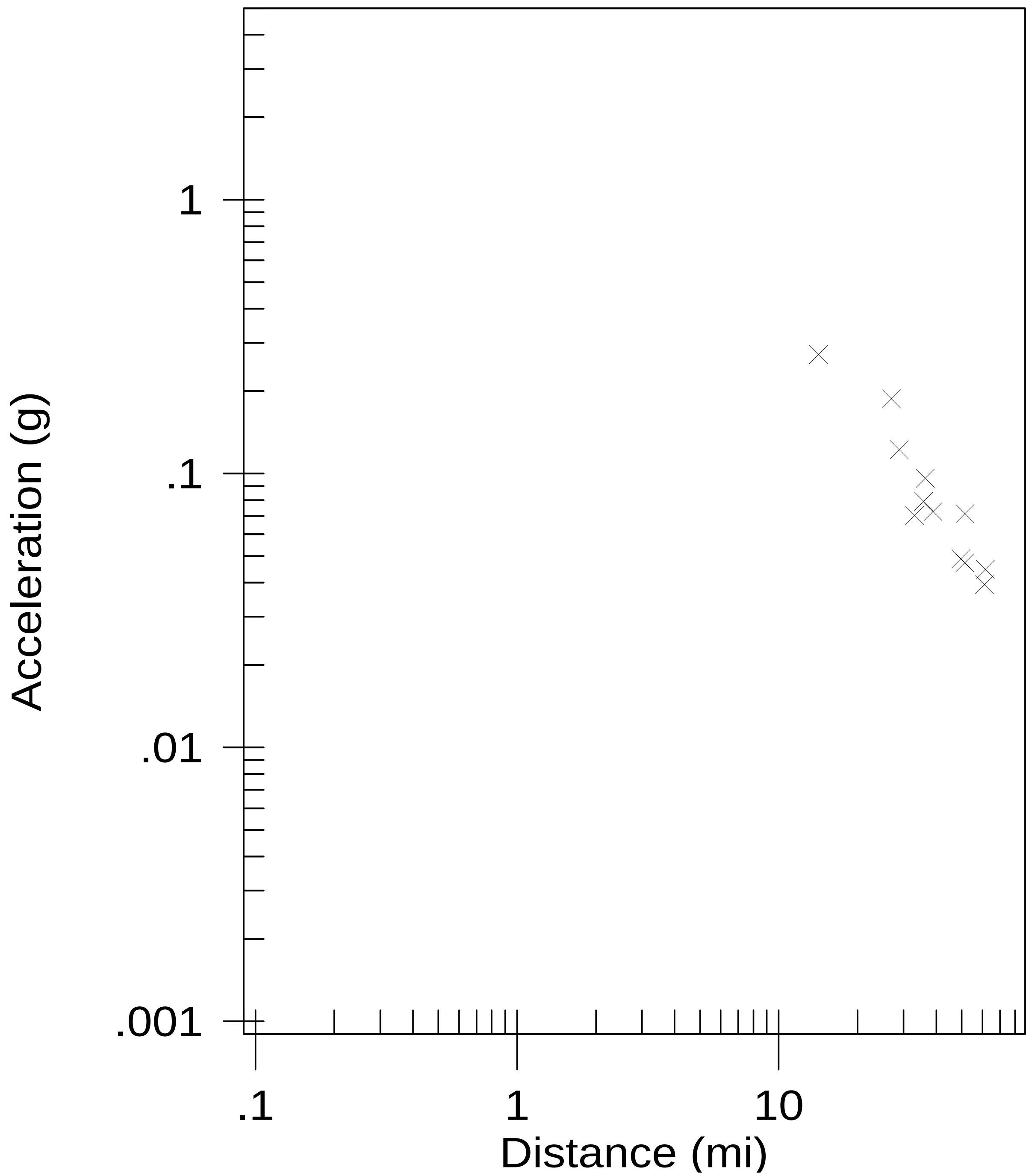
Bradley Ave OSHPD



W.O. 8096-A-SC
PLATE D-3

MAXIMUM EARTHQUAKES

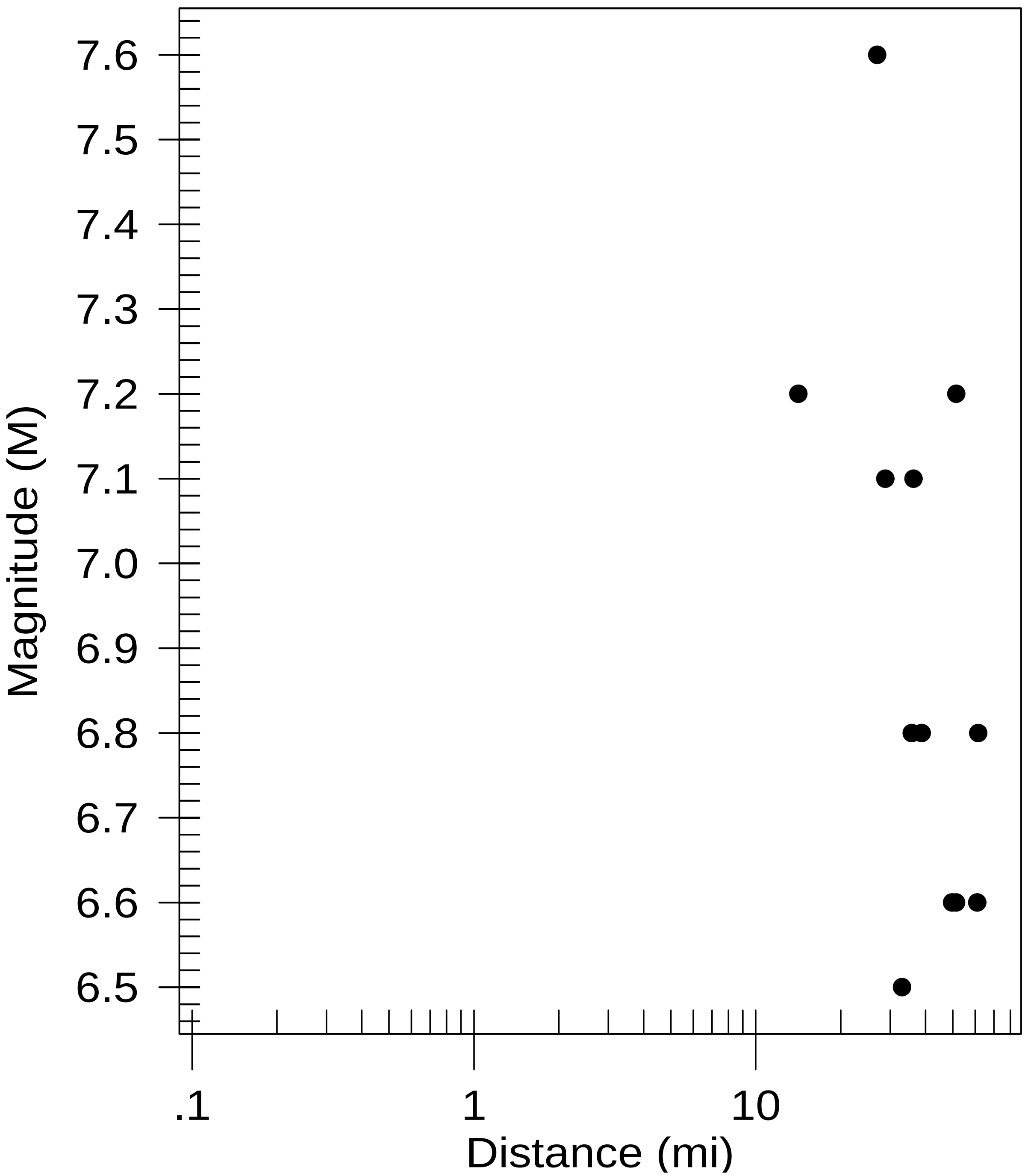
Bradley Ave OSHPD



W.O. 8096-A-SC
PLATE D-4

EARTHQUAKE MAGNITUDES & DISTANCES

Bradley Ave OSHPD



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

JOB NUMBER: 8096-A-SC

DATE: 05-27-2021

JOB NAME: Bradley Ave OSHPD

EARTHQUAKE-CATALOG-FILE NAME: C:\Program Files\EQSEARCH\ALLQUAKE 5-8-21.DAT

SITE COORDINATES:
SITE LATITUDE: 32.8177
SITE LONGITUDE: 116.9523

SEARCH DATES:
START DATE: 1800
END DATE: 2021

SEARCH RADIUS:
62.4 mi
100.4 km

ATTENUATION RELATION: 13) Bozorgnia Campbell Niazi (1999) Hor.-Hard Rock-Cor.
UNCERTAINTY (M=Median, S=Sigma): S   Number of Sigmas: 1.0
ASSUMED SOURCE TYPE: SS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
SCOND: 1 Depth Source: A
Basement Depth: .10 km   Campbell SSR: 0   Campbell SHR: 1
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]
MGI	32.8000	117.1000	05/25/1803	0 0 0.0	0.0	5.00	0.102	VII	8.7(13.9)
DMG	32.8000	116.8000	10/23/1894	23 3 0.0	0.0	5.70	0.158	VIII	8.9(14.4)
MGI	33.0000	117.0000	09/21/1856	730 0.0	0.0	5.00	0.072	VI	12.9(20.7)
T-A	32.6700	117.1700	12/00/1856	0 0 0.0	0.0	5.00	0.057	VI	16.2(26.1)
T-A	32.6700	117.1700	10/21/1862	0 0 0.0	0.0	5.00	0.057	VI	16.2(26.1)
T-A	32.6700	117.1700	05/24/1865	0 0 0.0	0.0	5.00	0.057	VI	16.2(26.1)
DMG	32.7000	117.2000	05/27/1862	20 0 0.0	0.0	5.90	0.098	VII	16.5(26.6)
DMG	33.0000	117.3000	11/22/1800	2130 0.0	0.0	6.50	0.099	VII	23.8(38.2)
DMG	33.2000	116.7000	01/01/1920	235 0.0	0.0	5.00	0.031	V	30.2(48.5)
DMG	33.0000	116.4330	06/04/1940	1035 8.3	0.0	5.10	0.030	V	32.6(52.5)
MGI	33.2000	116.6000	10/12/1920	1748 0.0	0.0	5.30	0.033	V	33.4(53.7)
DMG	32.7000	116.3000	02/24/1892	720 0.0	0.0	6.70	0.068	VI	38.7(62.3)
DMG	32.2000	116.5500	11/05/1949	43524.0	0.0	5.10	0.020	IV	48.7(78.3)
DMG	32.2000	116.5500	11/04/1949	204238.0	0.0	5.70	0.028	V	48.7(78.3)
GSG	33.4200	116.4890	07/07/2010	235333.5	14.0	5.50	0.024	V	49.5(79.6)
DMG	33.3430	116.3460	04/28/1969	232042.9	20.0	5.80	0.029	V	50.5(81.2)
T-A	32.2500	117.5000	01/13/1877	20 0 0.0	0.0	5.00	0.018	IV	50.5(81.3)
DMG	33.2000	116.2000	05/28/1892	1115 0.0	0.0	6.30	0.039	V	50.9(82.0)
GSP	33.4315	116.4427	06/10/2016	080438.7	12.3	5.19	0.019	IV	51.6(83.1)
DMG	32.0830	116.6670	11/25/1934	818 0.0	0.0	5.00	0.017	IV	53.4(85.9)
PAS	33.5010	116.5130	02/25/1980	104738.5	13.6	5.50	0.022	IV	53.6(86.2)
GSP	33.5290	116.5720	06/12/2005	154146.5	14.0	5.20	0.019	IV	53.8(86.6)
DMG	33.5000	116.5000	09/30/1916	211 0.0	0.0	5.00	0.017	IV	53.9(86.7)
GSP	33.5080	116.5140	10/31/2001	075616.6	15.0	5.10	0.018	IV	54.0(86.9)
DMG	33.1900	116.1290	04/09/1968	22859.1	11.1	6.40	0.039	V	54.2(87.2)
PAS	32.9710	117.8700	07/13/1986	1347 8.2	6.0	5.30	0.020	IV	54.2(87.3)
DMG	33.2170	116.1330	08/15/1945	175624.0	0.0	5.70	0.025	V	54.9(88.3)
DMG	33.2830	116.1830	03/23/1954	41450.0	0.0	5.10	0.017	IV	54.9(88.3)
DMG	33.2830	116.1830	03/19/1954	95429.0	0.0	6.20	0.034	V	54.9(88.3)
DMG	33.2830	116.1830	03/19/1954	95556.0	0.0	5.00	0.016	IV	54.9(88.3)
DMG	33.2830	116.1830	03/19/1954	102117.0	0.0	5.50	0.022	IV	54.9(88.3)
DMG	33.4000	116.3000	02/09/1890	12 6 0.0	0.0	6.30	0.036	V	55.1(88.7)
DMG	32.9670	116.0000	10/21/1942	162654.0	0.0	5.00	0.016	IV	56.2(90.4)
DMG	32.9670	116.0000	10/21/1942	162213.0	0.0	6.50	0.040	V	56.2(90.4)
DMG	32.9670	116.0000	10/22/1942	181326.0	0.0	5.00	0.016	IV	56.2(90.4)
DMG	32.9670	116.0000	10/21/1942	162519.0	0.0	5.00	0.016	IV	56.2(90.4)
DMG	33.1130	116.0370	04/09/1968	3 353.5	5.0	5.20	0.018	IV	56.8(91.4)
DMG	33.4080	116.2610	03/25/1937	1649 1.8	10.0	6.00	0.029	V	57.1(91.9)
DMG	32.9830	115.9830	05/23/1942	154729.0	0.0	5.00	0.016	IV	57.3(92.3)
GSG	32.7000	115.9210	06/15/2010	042658.5	5.0	5.80	0.024	IV	60.4(97.2)
DMG	33.7100	116.9250	09/23/1963	144152.6	16.5	5.00	0.014	IV	61.6(99.2)
DMG	33.2310	116.0040	05/26/1957	155933.6	15.1	5.00	0.014	IV	61.9(99.6)

****

-END OF SEARCH-   42 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 2021

LENGTH OF SEARCH TIME: 222 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 8.7 MILES (13.9 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 6.7

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.158 g

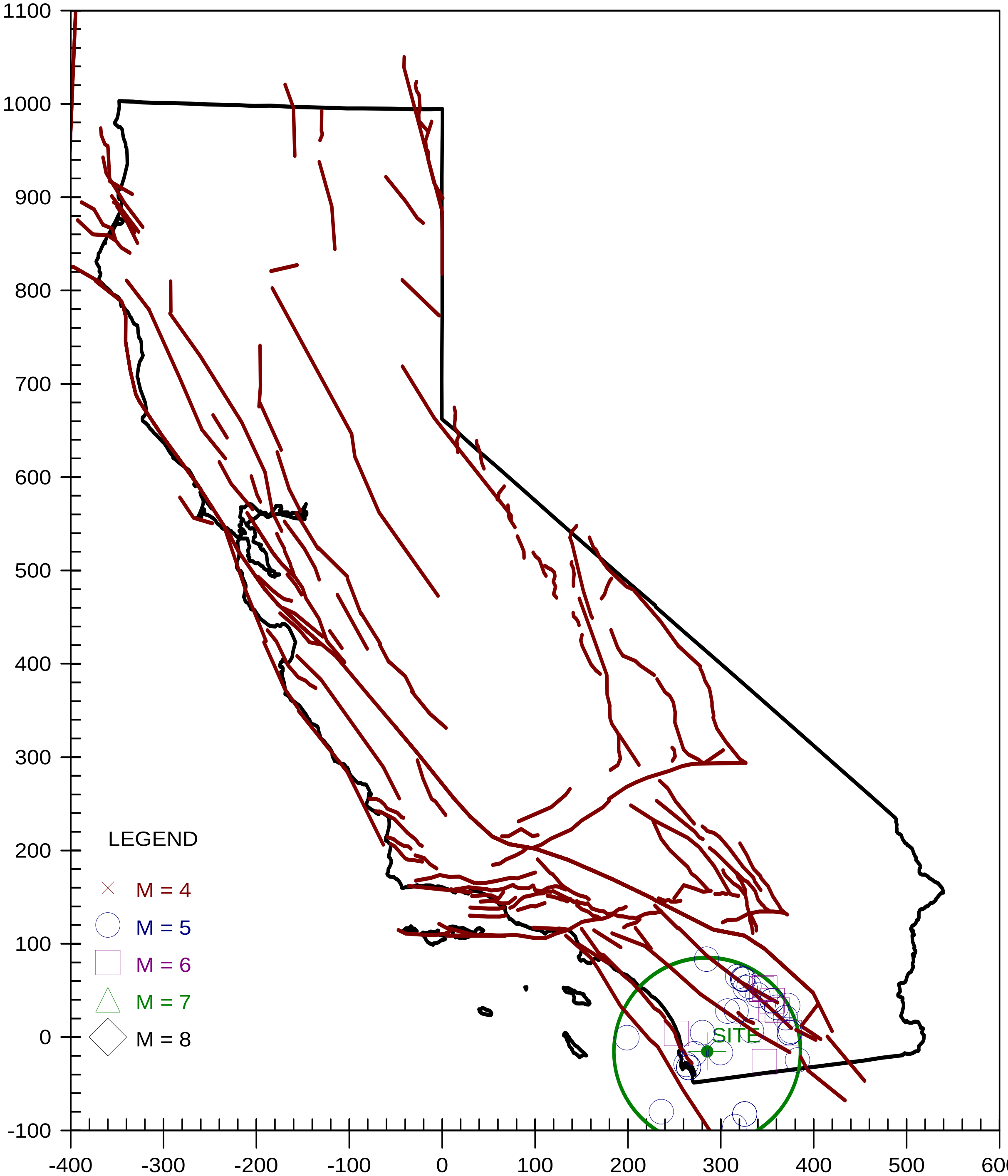
COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
a-value= 0.887
b-value= 0.367
beta-value= 0.844

TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	42	0.19005
4.5	42	0.19005
5.0	42	0.19005
5.5	17	0.07692
6.0	8	0.03620
6.5	3	0.01357

EARTHQUAKE EPICENTER MAP

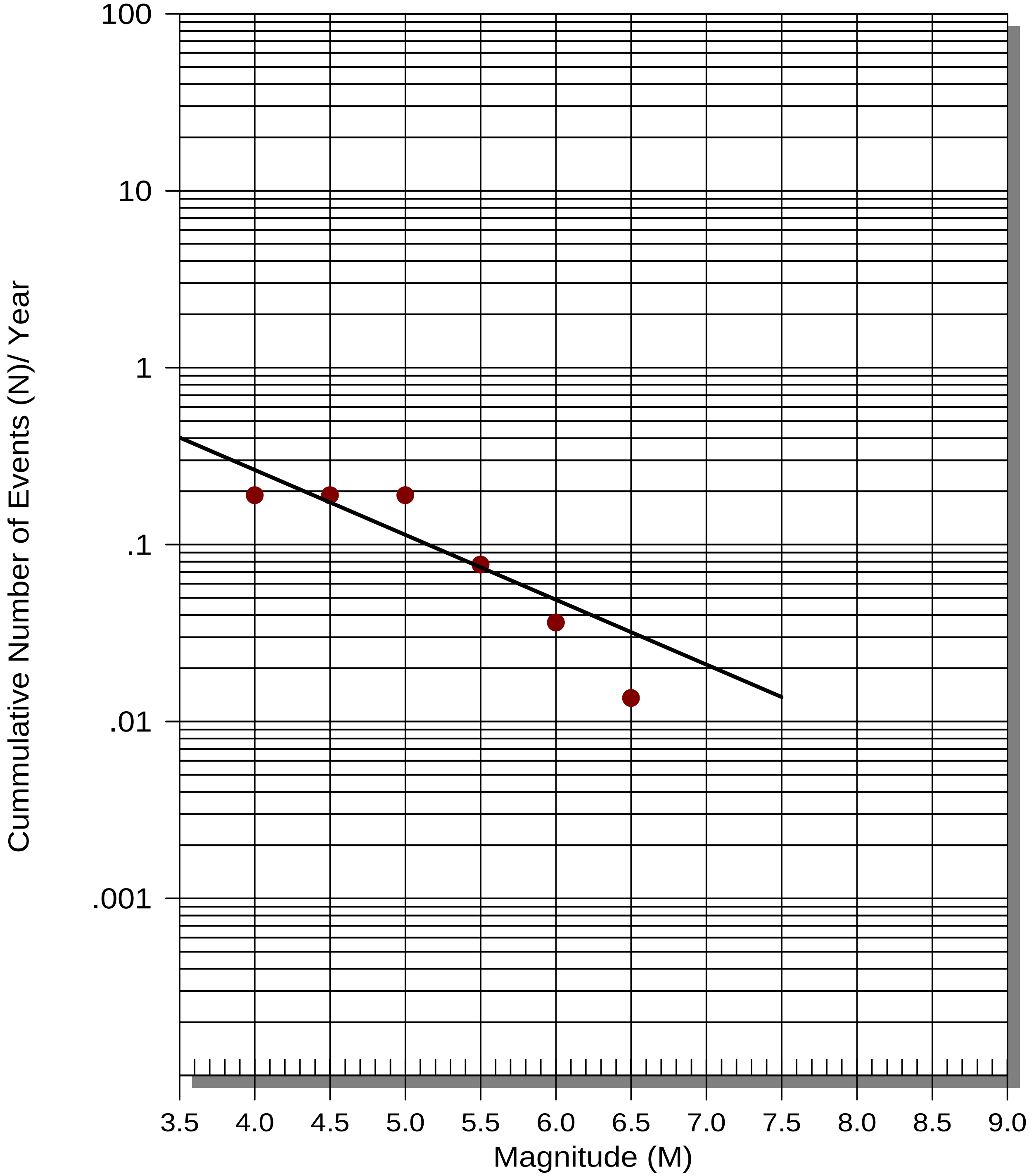
Bradley Ave OSHPD



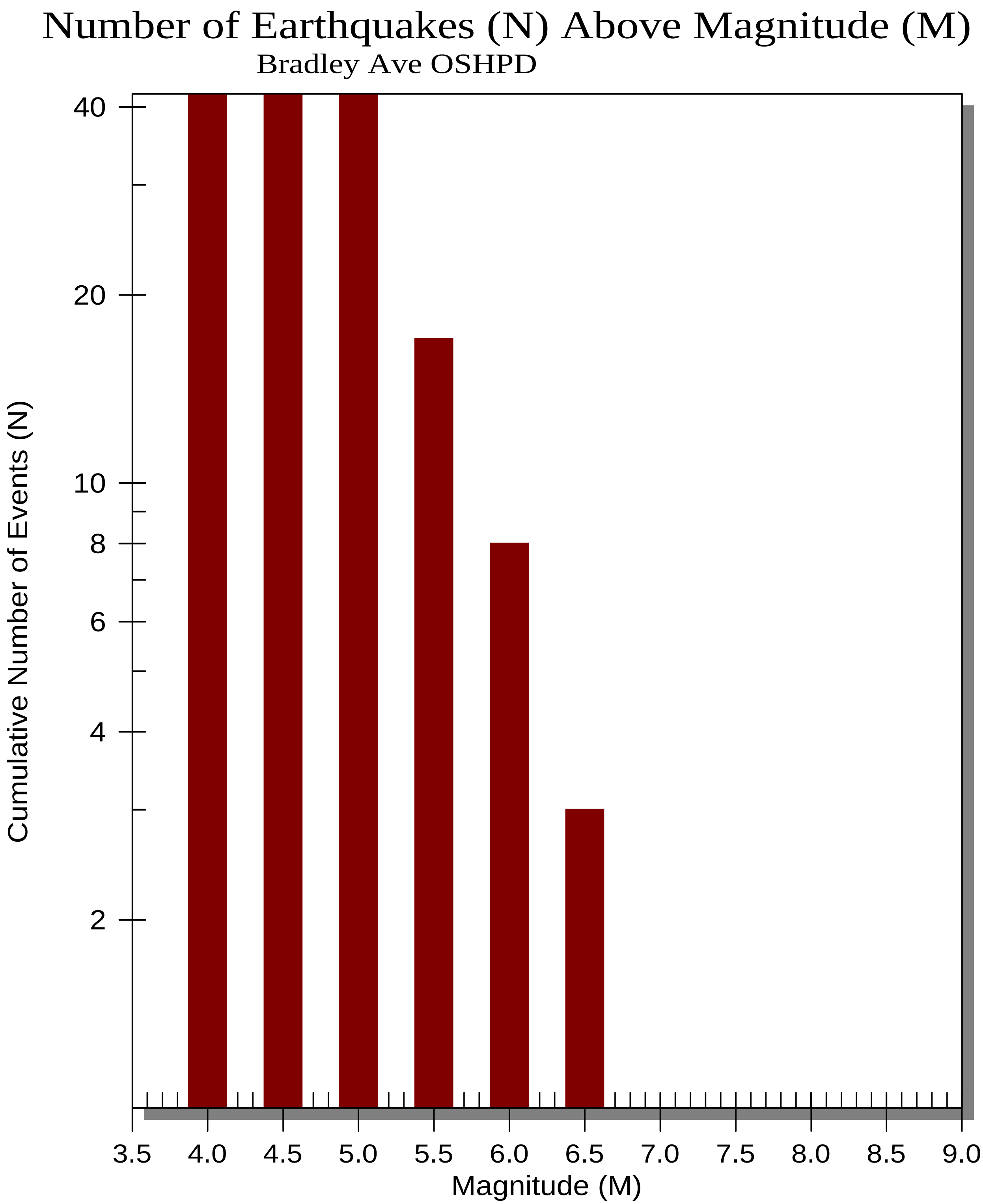
W.O. 8096-A-SC
PLATE D-9

EARTHQUAKE RECURRENCE CURVE

Bradley Ave OSHPD

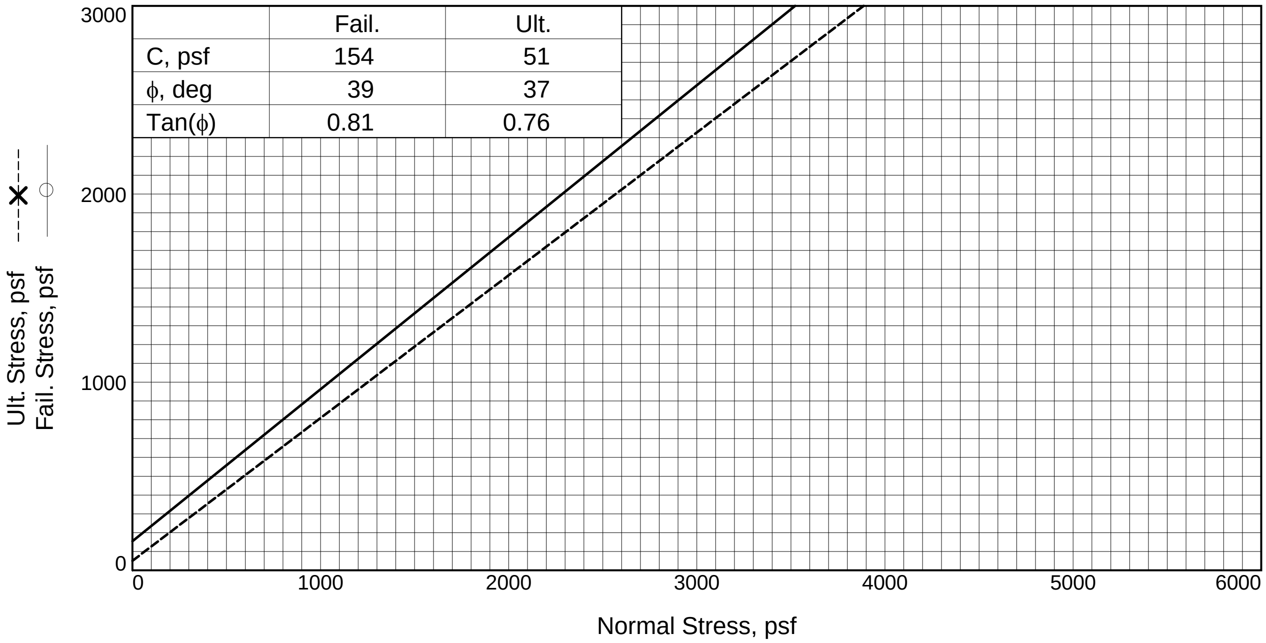


W.O. 8096-A-SC
PLATE D-10

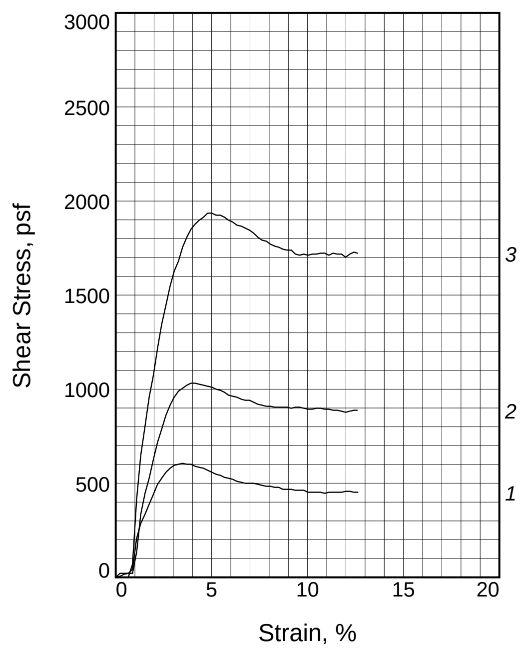

Number of Earthquakes (N) Above Magnitude (M)
Bradley Ave OSHPD
Cumulative Number of Events (N)
40
20
10
8
6
4
2
3.5
4.0
4.5
5.0
5.5
6.0
6.5
7.0
7.5
8.0
8.5
9.0
Magnitude (M)

APPENDIX E

LABORATORY DATA



	Fail.	Ult.
C, psf	154	51
ϕ, deg	39	37
Tan(ϕ)	0.81	0.76



Sample No.		1	2	3
Initial	Water Content, %	7.6	7.6	7.6
	Dry Density, pcf	117.8	118.0	118.0
	Saturation, %	49.8	50.1	50.1
	Void Ratio	0.4044	0.4025	0.4025
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	14.2	13.8	14.0
	Dry Density, pcf	118.0	118.6	118.9
	Saturation, %	93.8	92.7	95.0
	Void Ratio	0.4016	0.3955	0.3913
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	0.99
Normal Stress, psf		550	1100	2200
Fail. Stress, psf		606	1032	1935
Strain, %		3.5	3.9	4.8
Ult. Stress, psf		463	893	1717
Strain, %		9.6	10.2	9.8
Strain rate, in./min.		0.005	0.005	0.005

Sample Type: Remolded
Description: Dark Gray Silty Sand w/Silt

Specific Gravity= 2.65
Remarks:

Plate ________

Client: Arco Construction

Project: 675 East Bradley

Source of Sample: TP-7 **Depth:** 0-1.5
Sample Number: TP-7
Proj. No.: 8096-A-SC **Date Sampled:** 5-26-21

GeoSoils, Inc.

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

W.O. 8096-A-SC
PLATE E-1

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

Work Order No.: 21E5141
Client: GeoSoils, Inc.
Project No.: 8096-A-SC
Project Name: Arco Construction
Report Date: May 21, 2021

Laboratory Test(s) Results Summary

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

Sample Identification	pH (H+)	As Rec'd Resistivity (ohm-cm)	Saturated Resistivity (ohm-cm)	Sulfate Content (mg/L)	Chloride Content (mg/L)
TP-1 @ 0-2ft	7.7	1,500	690	10	130
TP-7 @ 0-1.5ft	7.4	240,000	19,000	10	50

*ND=No Detection

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

Ahmet K. Kaya, Laboratory Manager

ASTM INTERNATIONAL ORGANIZATIONAL MEMBER

APPENDIX F

STORM WATER BMP CHECKLISTS/FORMS

BASIN NAME Basin 1 - Adjacent to Bradley Avenue

Assumed Infl. Bot. = ± 442 **ft. elv, GW elv=** 433 **ft, Cut/Fill** Cut 2-4 feet

USDA Soil PfC - Placentia sandy loam, thick surface, 2 to 9 percent slopes

HSG: D, **Ksat:** Very low to moderately low (0.00 to 0.06 inches per hour)

Geologic Unit: Colluvium, Granite

Table D.1-1: Considerations for Geotechnical Analysis of Infiltration Restrictions

	Restriction Element	Is Element Applicable? (Yes/No)
Mandatory Considerations	BMP is within 100' of Contaminated Soils	No
	BMP is within 100' of Industrial Activities Lacking Source Control	No
	BMP is within 100' of Well/Groundwater Basin	No
	BMP is within 50' of Septic Tanks/Leach Fields	No
	BMP is within 10' of Structures/Tanks/Walls	No
	BMP is within 10' of Sewer Utilities	No
	BMP is within 10' of Groundwater Table	Yes
	BMP is within Hydric Soils	No
	BMP is within Highly Liquefiable Soils and has Connectivity to Structures	No
	BMP is within 1.5 Times the Height of Adjacent Steep Slopes (≥25%)	No
	County Staff has Assigned "Restricted" Infiltration Category	No
Optional Considerations	BMP is within Predominantly Type D Soil	Yes
	BMP is within 10' of Property Line	No
	BMP is within Fill Depths of ≥5' (Existing or Proposed)	No
	BMP is within 10' of Underground Utilities	Yes
	BMP is within 250' of Ephemeral Stream	No
	Other (Provide detailed geotechnical support) BMP within 10' of Bedrock/Obstruction	Yes
Result	Based on examination of the best available information, **I have not identified any restrictions above.**	**Unrestricted**
	Based on examination of the best available information, **I have identified one or more restrictions above.**	**Restricted** X

Table D.2-1: Elements for Determination of Design Infiltration Rates

Item	Value	Unit
Initial Infiltration Rate: Identify per Section D.2.1 in/hr	per NRCS Soil Survey	in/hr
Corrected Infiltration Rate: Identify per Section D.2.2 in/hr	N/A	in/hr
Safety Factor: Identify per Section D.2.3 unitless	N/A	unitless
Design Infiltration Rate: Corrected Infiltration Rate ÷ Safety Factor in/hr	0.025	in/hr

From: Appendix D- Approved Infiltration Rate Assessment Methods

Using USDA maps, if NRCS soil type present within a proposed BMP location, Default design infiltration rates (in/hr) for each NRCS soil type are: A=0.300, B=0.200, C=0.100, D=0.025, Restricted=0.000. Use of these default design infiltration rates does not require application of any correction factors or safety factors.

Table D.2-3: Determination of Safety Factor

Consideration		Assigned Weight (w)	Factor Value (v)	Product (p) $p = w \times v$
Suitability Assessment (A)	Infiltration Testing Method	0.25	Refer to Table D.2-4	
	Soil Texture Class	0.25		
	Soil Variability	0.25		
	Depth to Groundwater/Obstruction	0.25		
	Suitability Assessment Safety Factor, $S_A = \Sigma p$			
Design (B)	Pretreatment	0.50	Refer to Table D.2-4	
	Resiliency	0.25		
	Compaction	0.25		
	Design Safety Factor, $S_B = \Sigma p$			N/A
Safety Factor, $S = S_A \times S_B$ (Must be always greater than or equal to 2)				N/A

The determination of the safety factor is not required for "Desktop" methods utilizing default design infiltration rates.

BASIN NAME Basin 2 - Adjacent to existing building
Assumed Infl. Bot. = ±454 **ft. elv, GW elv=** 434 **ft, Cut/Fill** Cut 0.5-1.5 feet
USDA Soil PfC - Placentia sandy loam, thick surface, 2 to 9 percent slopes
HSG: D, **Ksat:** Very low to moderately low (0.00 to 0.06 inches per hour)
Geologic Unit: Colluvium, Granite

Table D.1-1: Considerations for Geotechnical Analysis of Infiltration Restrictions

	Restriction Element	Is Element Applicable? (Yes/No)
Mandatory Considerations	BMP is within 100' of Contaminated Soils	No
	BMP is within 100' of Industrial Activities Lacking Source Control	No
	BMP is within 100' of Well/Groundwater Basin	No
	BMP is within 50' of Septic Tanks/Leach Fields	No
	BMP is within 10' of Structures/Tanks/Walls	No
	BMP is within 10' of Sewer Utilities	No
	BMP is within 10' of Groundwater Table	No
	BMP is within Hydric Soils	No
	BMP is within Highly Liquefiable Soils and has Connectivity to Structures	No
	BMP is within 1.5 Times the Height of Adjacent Steep Slopes (≥25%)	No
	County Staff has Assigned "Restricted" Infiltration Category	No
Optional Considerations	BMP is within Predominantly Type D Soil	Yes
	BMP is within 10' of Property Line	No
	BMP is within Fill Depths of ≥5' (Existing or Proposed)	No
	BMP is within 10' of Underground Utilities	No
	BMP is within 250' of Ephemeral Stream	No
	Other (Provide detailed geotechnical support) BMP within 10' of Bedrock/Obstruction	Yes
Result	Based on examination of the best available information, I have not identified any restrictions above.	Unrestricted
	Based on examination of the best available information, I have identified one or more restrictions above.	Restricted X

Table D.2-1: Elements for Determination of Design Infiltration Rates

Item	Value	Unit
Initial Infiltration Rate: Identify per Section D.2.1 in/hr	per NRCS Soil Survey	in/hr
Corrected Infiltration Rate: Identify per Section D.2.2 in/hr	N/A	in/hr
Safety Factor: Identify per Section D.2.3 unitless	N/A	unitless
Design Infiltration Rate: Corrected Infiltration Rate ÷ Safety Factor in/hr	0.025	in/hr

From: Appendix D- Approved Infiltration Rate Assessment Methods

Using USDA maps, if NRCS soil type present within a proposed BMP location, Default design infiltration rates (in/hr) for each NRCS soil type are: A=0.300, B=0.200, C=0.100, D=0.025, Restricted=0.000. Use of these default design infiltration rates does not require application of any correction factors or safety factors.

Table D.2-3: Determination of Safety Factor

Consideration		Assigned Weight (w)	Factor Value (v)	Product (p) p = w x v
Suitability Assessment (A)	Infiltration Testing Method	0.25	Refer to Table D.2-4	
	Soil Texture Class	0.25		
	Soil Variability	0.25		
	Depth to Groundwater/Obstruction	0.25		
	Suitability Assessment Safety Factor, $S_A = \Sigma p$			
Design (B)	Pretreatment	0.50	Refer to Table D.2-4	
	Resiliency	0.25		
	Compaction	0.25		
	Design Safety Factor, $S_B = \Sigma p$			N/A
Safety Factor, $S = S_A \times S_B$ (Must be always greater than or equal to 2)				N/A

The determination of the safety factor is not required for "Desktop" methods utilizing default design infiltration rates.

BASIN NAME Basin 3 - Adjacent to existing parking lot

Assumed Infl. Bot. = ± 456 ft. elv, **GW elv**= 431.5 ft, **Cut/Fill** Fill 6-8 feet

USDA Soil PfC - Placentia sandy loam, thick surface, 2 to 9 percent slopes

HSG: D, **Ksat:** Very low to moderately low (0.00 to 0.06 inches per hour)

Geologic Unit: Colluvium, Granite

Table D.1-1: Considerations for Geotechnical Analysis of Infiltration Restrictions

	Restriction Element	Is Element Applicable? (Yes/No)
Mandatory Considerations	BMP is within 100' of Contaminated Soils	No
	BMP is within 100' of Industrial Activities Lacking Source Control	No
	BMP is within 100' of Well/Groundwater Basin	No
	BMP is within 50' of Septic Tanks/Leach Fields	No
	BMP is within 10' of Structures/Tanks/Walls	No
	BMP is within 10' of Sewer Utilities	No
	BMP is within 10' of Groundwater Table	No
	BMP is within Hydric Soils	No
	BMP is within Highly Liquefiable Soils and has Connectivity to Structures	No
	BMP is within 1.5 Times the Height of Adjacent Steep Slopes (≥25%)	No
	County Staff has Assigned "Restricted" Infiltration Category	No
Optional Considerations	BMP is within Predominantly Type D Soil	Yes
	BMP is within 10' of Property Line	No
	BMP is within Fill Depths of ≥5' (Existing or Proposed)	Yes
	BMP is within 10' of Underground Utilities	No
	BMP is within 250' of Ephemeral Stream	No
	Other (Provide detailed geotechnical support) BMP within 10' of Bedrock/Obstruction	Yes
Result	Based on examination of the best available information, I have not identified any restrictions above.	Unrestricted
	Based on examination of the best available information, I have identified one or more restrictions above.	Restricted X

Table D.2-1: Elements for Determination of Design Infiltration Rates

Item	Value	Unit
Initial Infiltration Rate: Identify per Section D.2.1 in/hr	per NRCS Soil Survey	in/hr
Corrected Infiltration Rate: Identify per Section D.2.2 in/hr	N/A	in/hr
Safety Factor: Identify per Section D.2.3 unitless	N/A	unitless
Design Infiltration Rate: Corrected Infiltration Rate ÷ Safety Factor in/hr	0.025	in/hr

From: Appendix D- Approved Infiltration Rate Assessment Methods

Using USDA maps, if NRCS soil type present within a proposed BMP location, Default design infiltration rates (in/hr) for each NRCS soil type are: A=0.300, B=0.200, C=0.100, D=0.025, Restricted=0.000. Use of these default design infiltration rates does not require application of any correction factors or safety factors.

Table D.2-3: Determination of Safety Factor

Consideration		Assigned Weight (w)	Factor Value (v)	Product (p) $p = w \times v$
Suitability Assessment (A)	Infiltration Testing Method	0.25	Refer to Table D.2-4	
	Soil Texture Class	0.25		
	Soil Variability	0.25		
	Depth to Groundwater/Obstruction	0.25		
	Suitability Assessment Safety Factor, $S_A = \Sigma p$			
Design (B)	Pretreatment	0.50	Refer to Table D.2-4	
	Resiliency	0.25		
	Compaction	0.25		
	Design Safety Factor, $S_B = \Sigma p$			N/A
Safety Factor, $S = S_A \times S_B$ (Must be always greater than or equal to 2)				N/A

The determination of the safety factor is not required for "Desktop" methods utilizing default design infiltration rates.

APPENDIX G

GENERAL EARTHWORK AND GRADING GUIDELINES

GENERAL EARTHWORK AND GRADING GUIDELINES

General

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

The contractor is responsible for the satisfactory completion of all earthwork in accordance with provisions of the project plans and specifications and latest adopted code. In the case of conflict, the most onerous provisions shall prevail. The project geotechnical engineer and engineering geologist (geotechnical consultant), 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 92 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.

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.

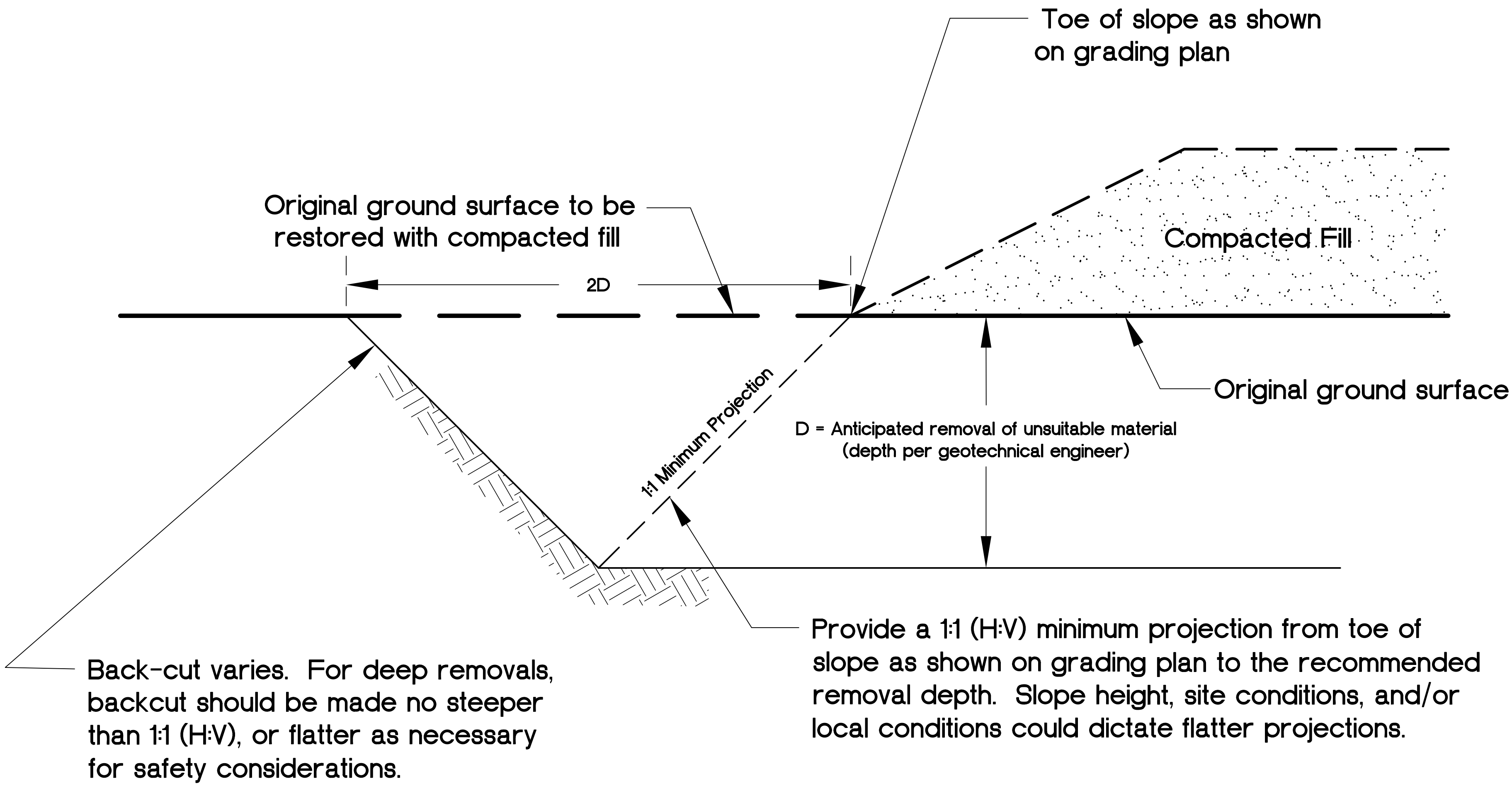
Trench and Vertical Excavation

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.

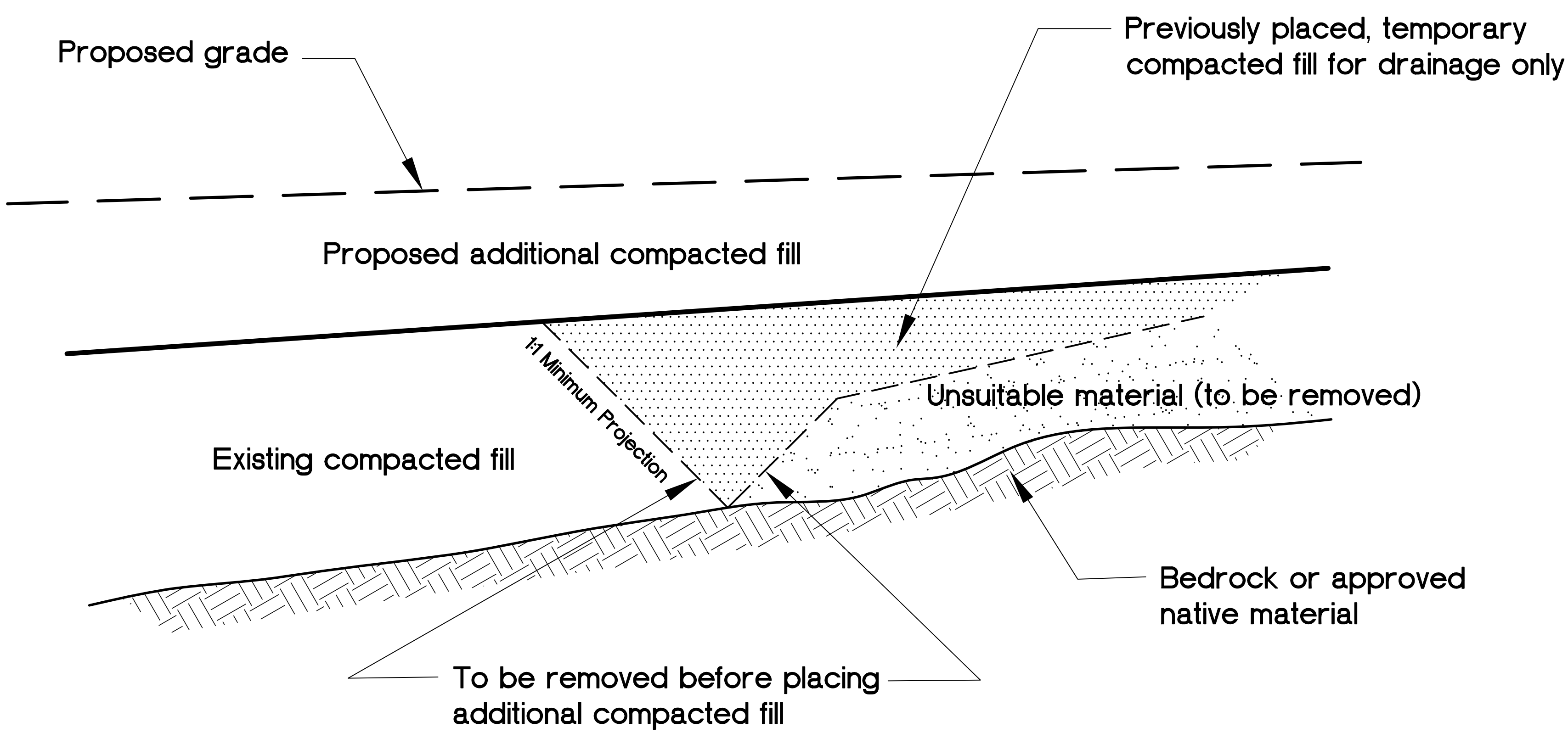
If GSI personnel become aware of anyone working beneath an unsafe trench wall or vertical excavation, we have a legal obligation to put the contractor and owner/developer on notice to immediately correct the situation. If corrective steps are not taken, GSI then has an obligation to notify Cal/OSHA and/or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



GeoSoils, Inc.

FILL SLOPE TOEING OUT ON FLAT ALLUVIATED CANYON DETAIL

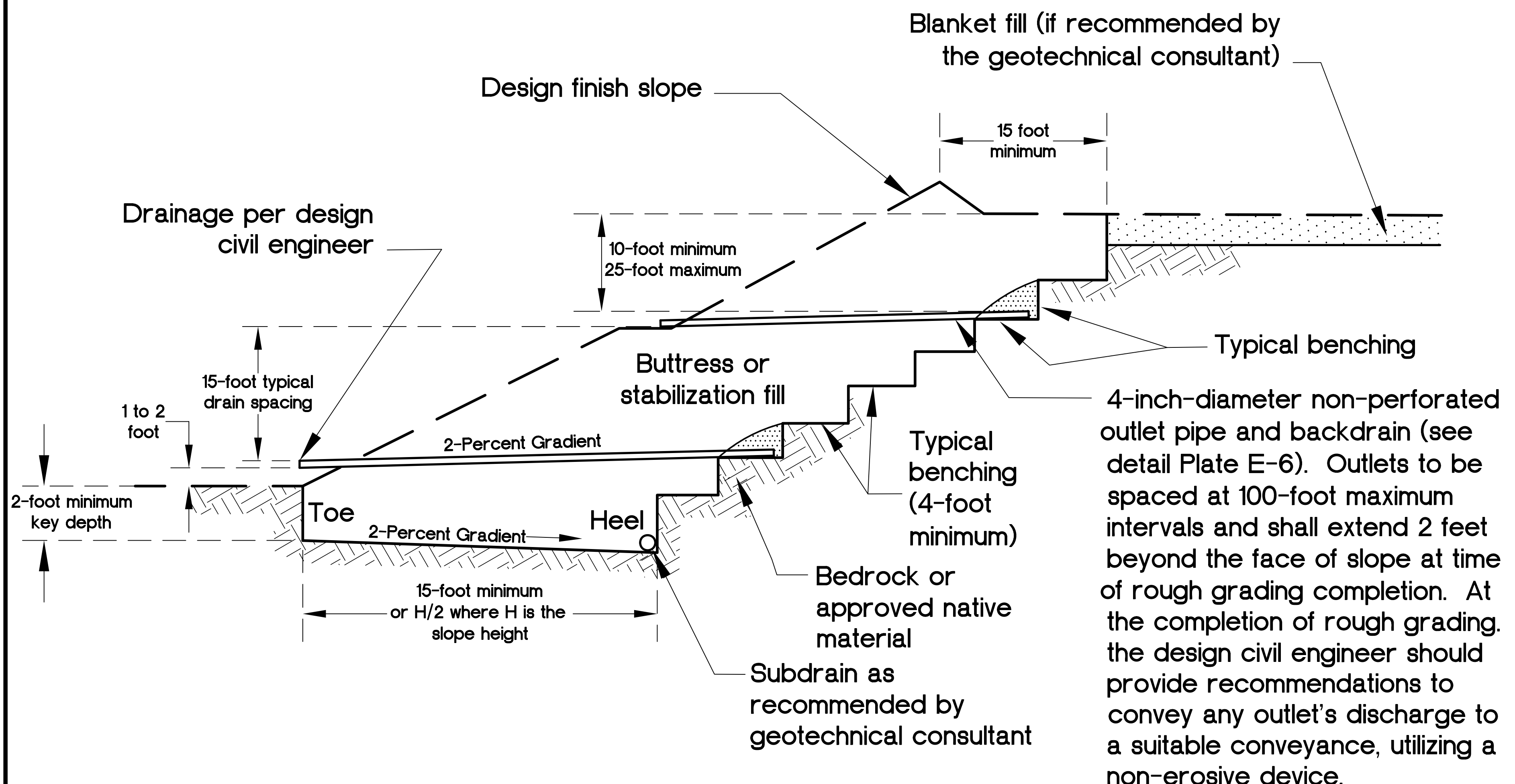
Plate G-3



GeoSoils, Inc.

REMOVAL ADJACENT TO EXISTING FILL ADJOINING CANYON FILL DETAIL

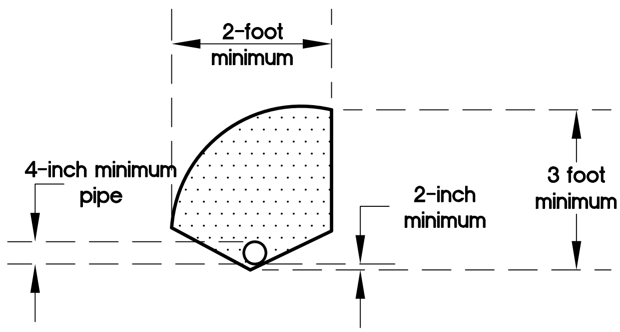
Plate G-4

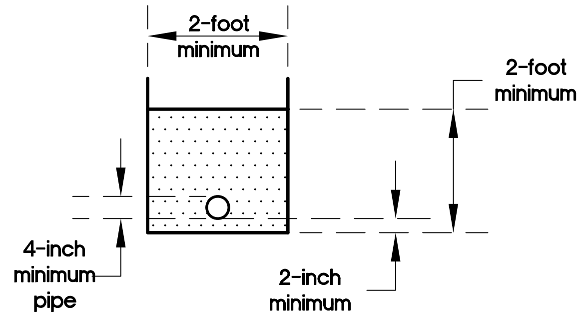


GeoSoils, Inc.

TYPICAL STABILIZATION / BUTTRESS FILL DETAIL

Plate G-5





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

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

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

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

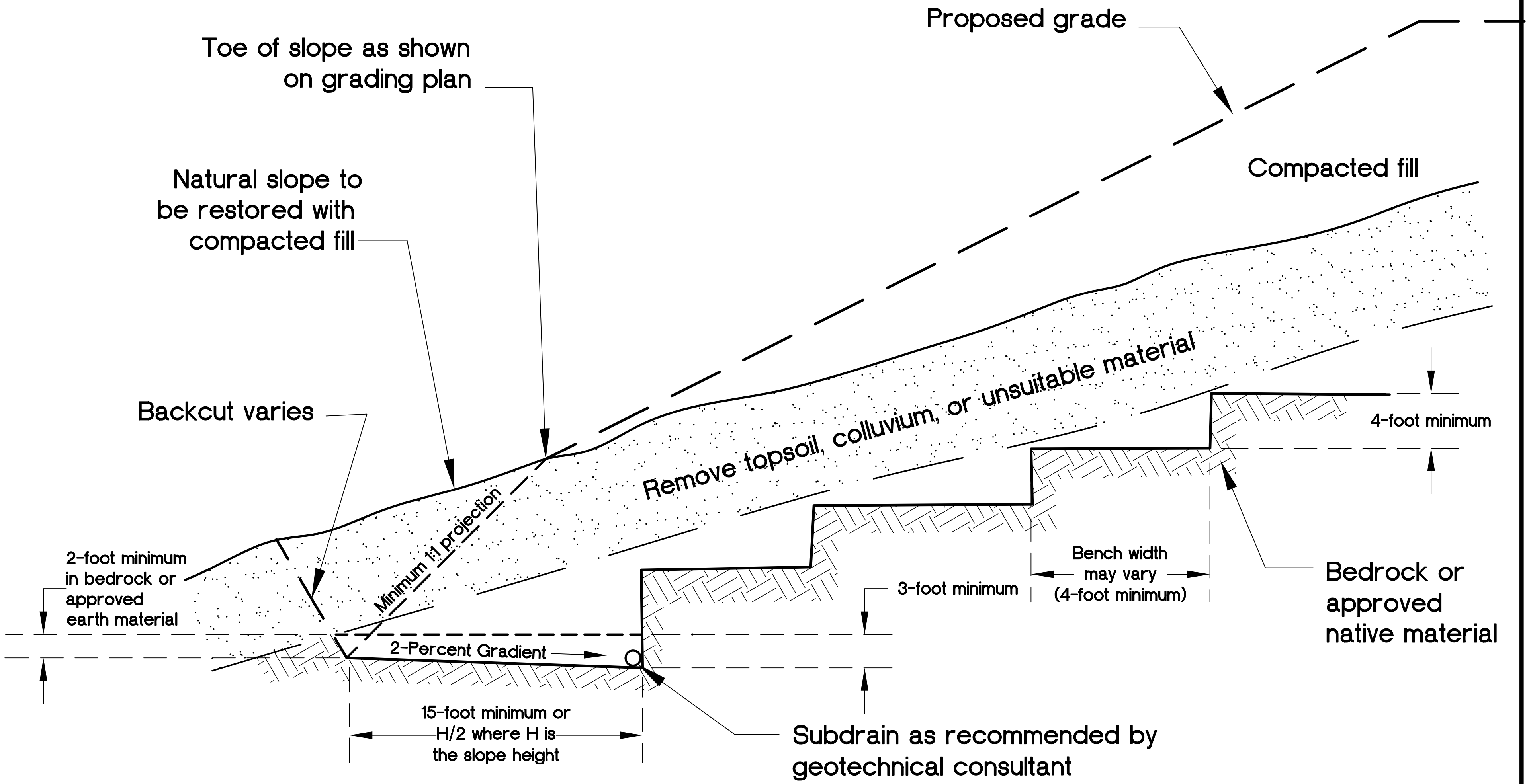
2. Backdrains and lateral drains shall be located at elevation of every bench drain. First drain located at elevation just above lower lot grade. Additional drains may be required at the discretion of the geotechnical consultant.

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

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

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

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



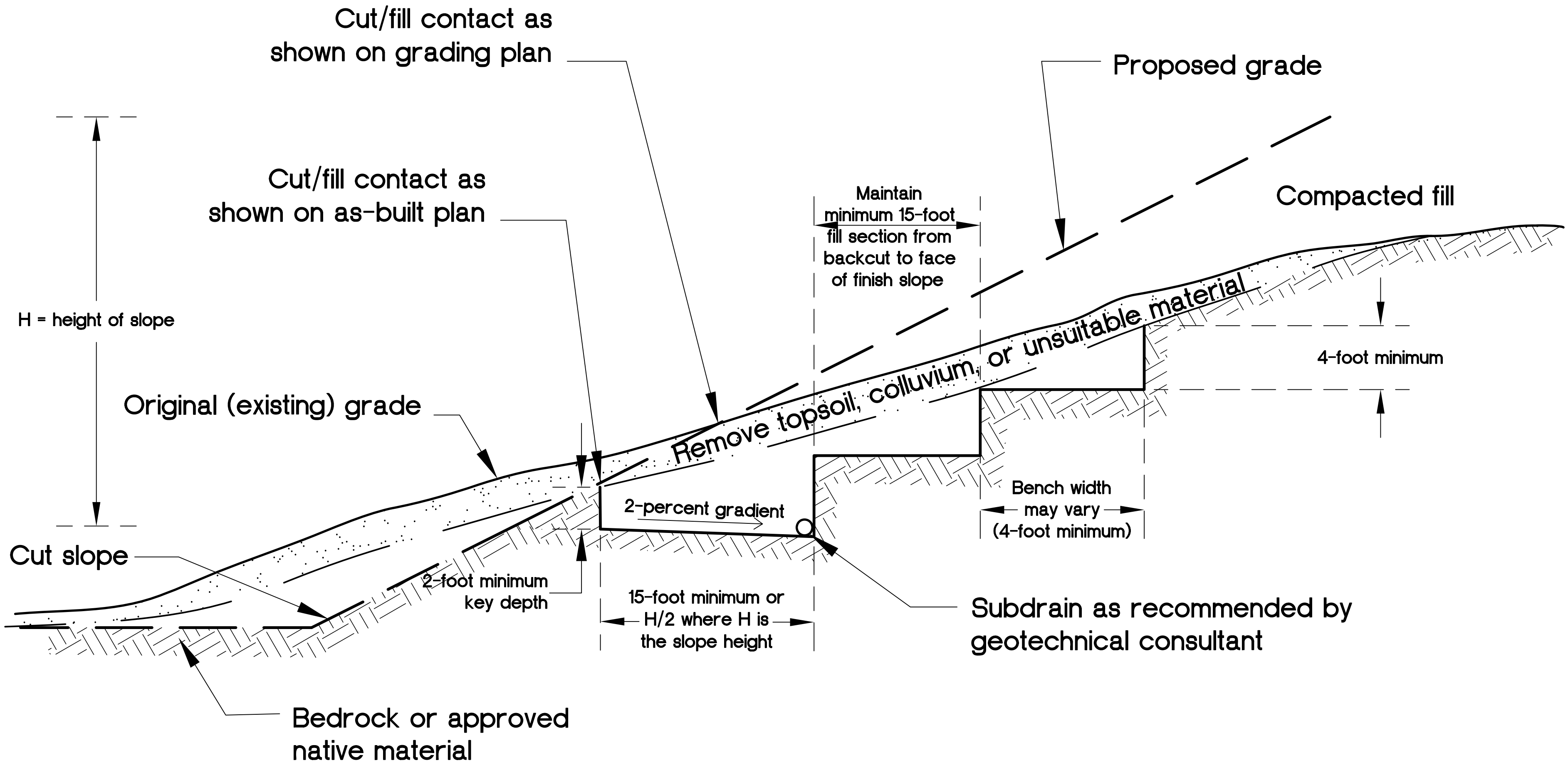
NOTES:
1. Where the natural slope approaches or exceeds the design slope ratio, special recommendations would be provided by the geotechnical consultant.

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

GeoSoils, Inc.

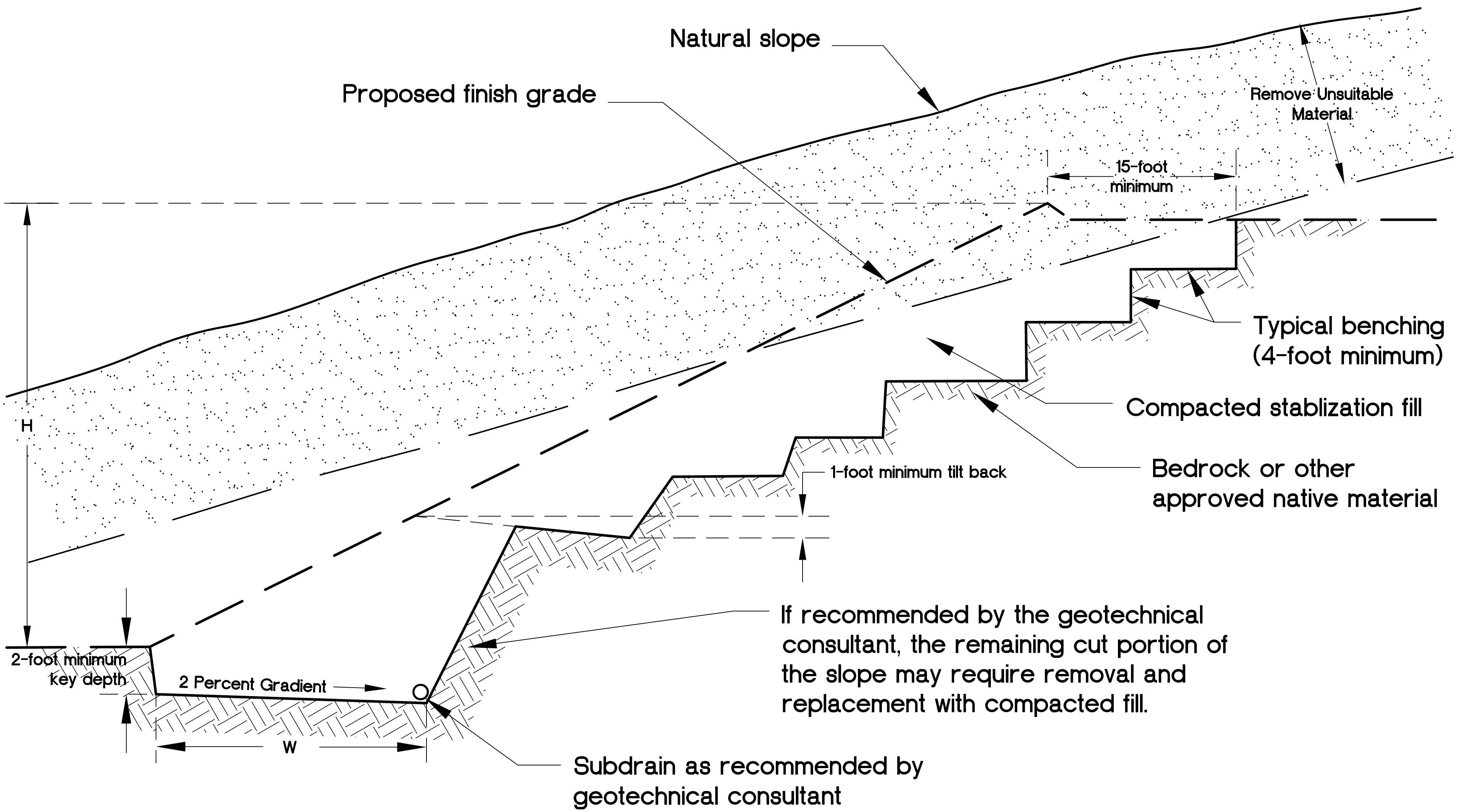
FILL OVER NATURAL (SIDEHILL FILL) DETAIL

Plate G-7



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

GeoSoils, Inc. | FILL OVER CUT DETAIL | Plate G-8

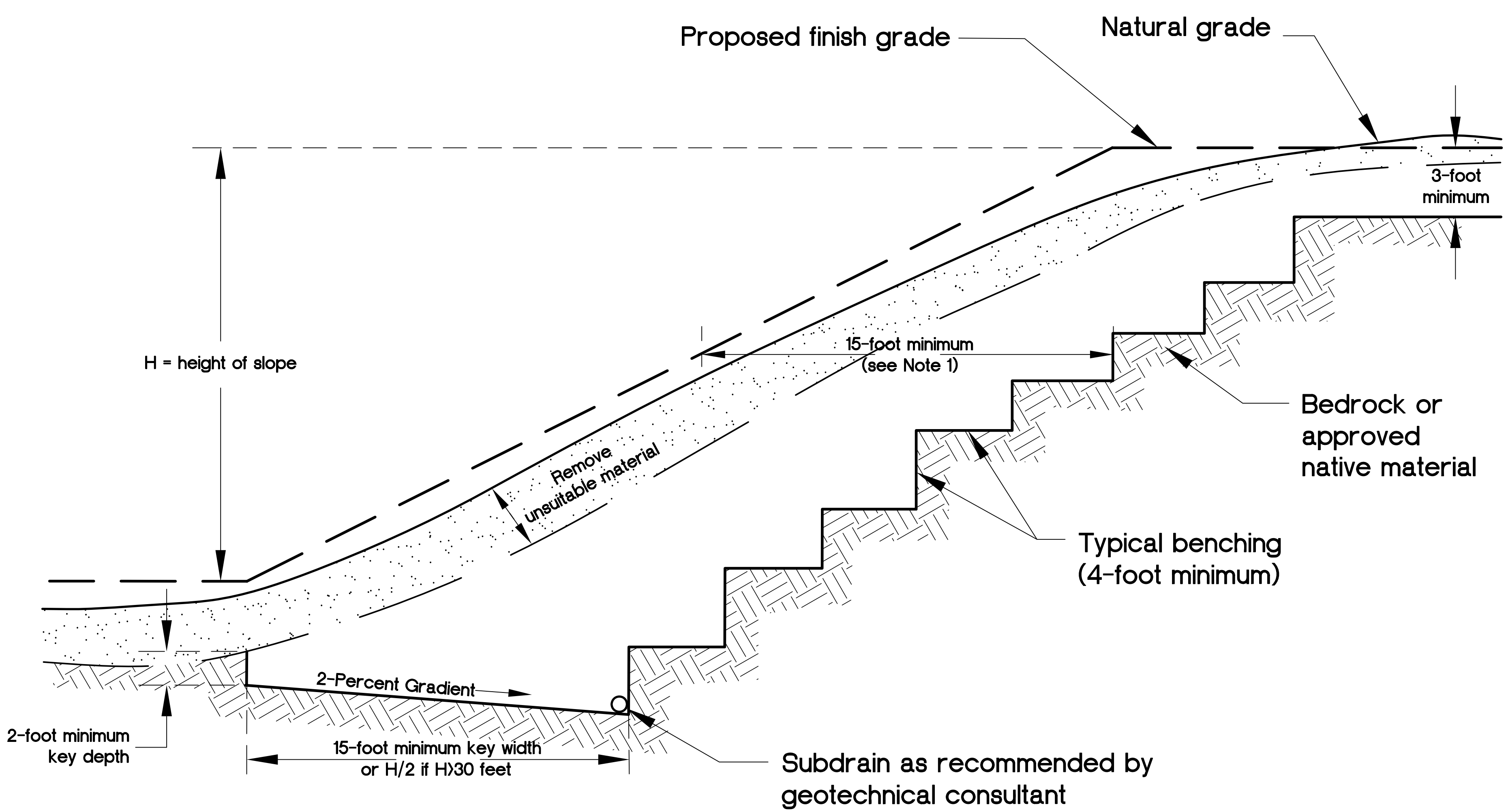


NOTES:

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

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

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



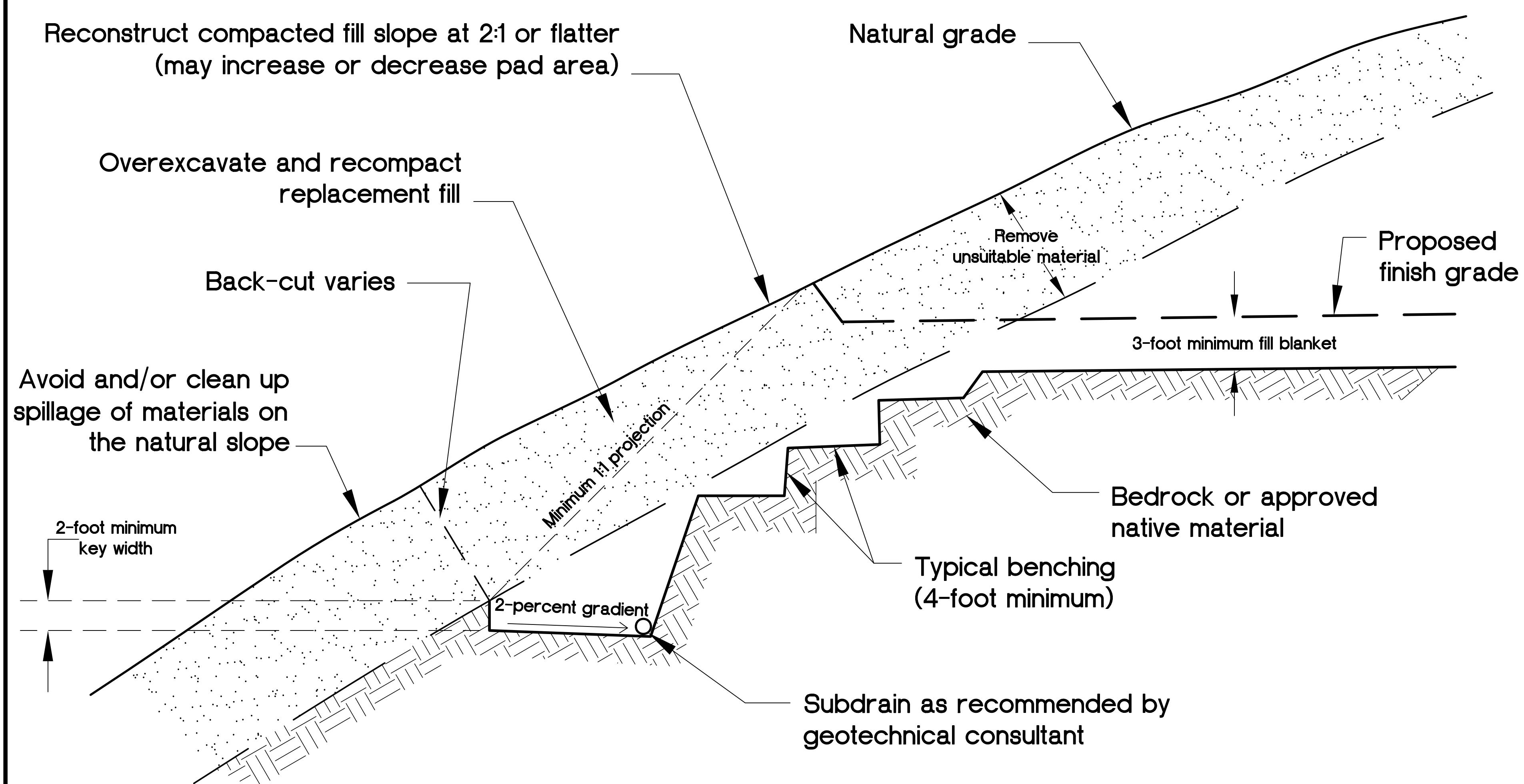
NOTES:

1. 15-foot minimum to be maintained from proposed finish slope face to backcut.

2. The need and disposition of drains will be evaluated by the geotechnical consultant based on field conditions.

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

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



NOTES:

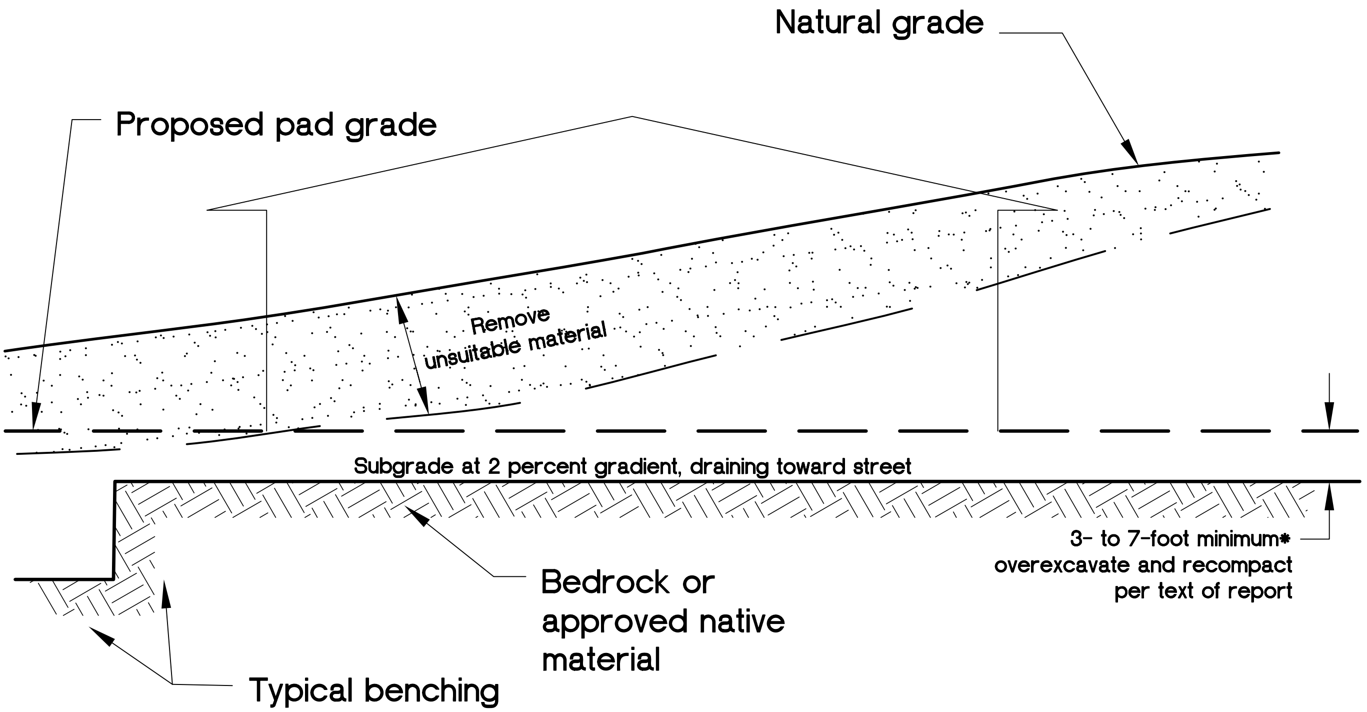
1. Subdrain and key width requirements will be evaluated based on exposed subsurface conditions and thickness of overburden.

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

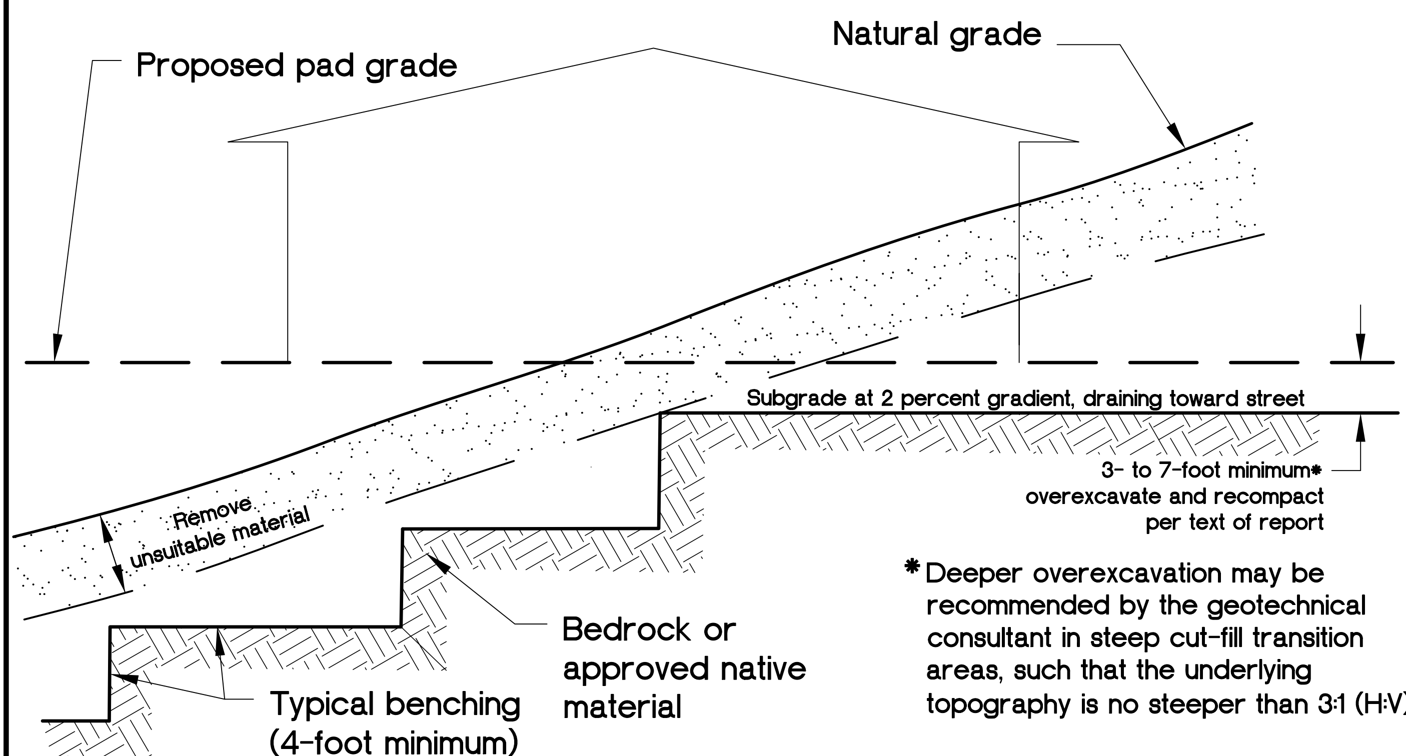
GeoSoils, Inc.

DAYLIGHT CUT LOT DETAIL

Plate G-11



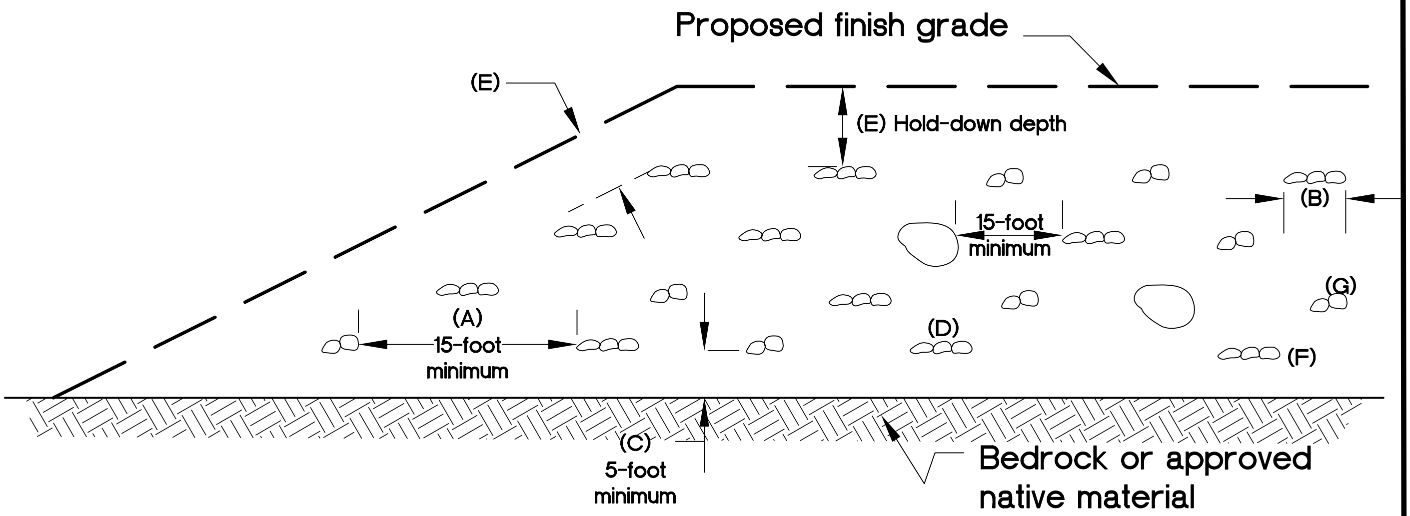
CUT LOT OR MATERIAL-TYPE TRANSITION



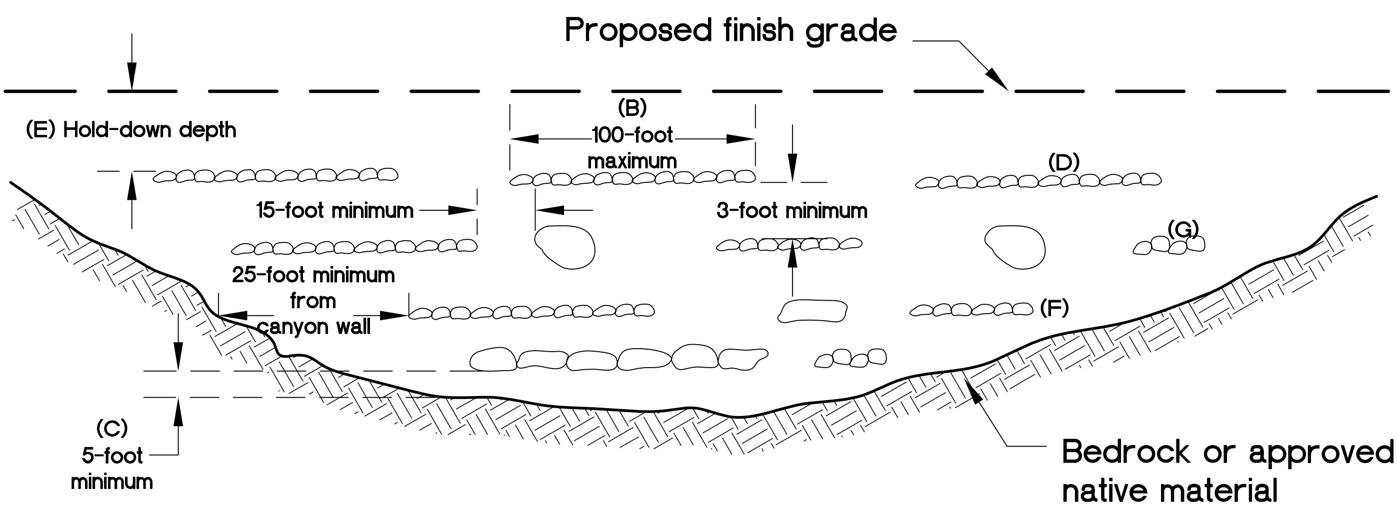
CUT-FILL LOT (DAYLIGHT TRANSITION)

GeoSoils, Inc.	TRANSITION LOT DETAILS	Plate G-12

VIEW NORMAL TO SLOPE FACE



VIEW PARALLEL TO SLOPE FACE



NOTES:

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

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

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

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

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

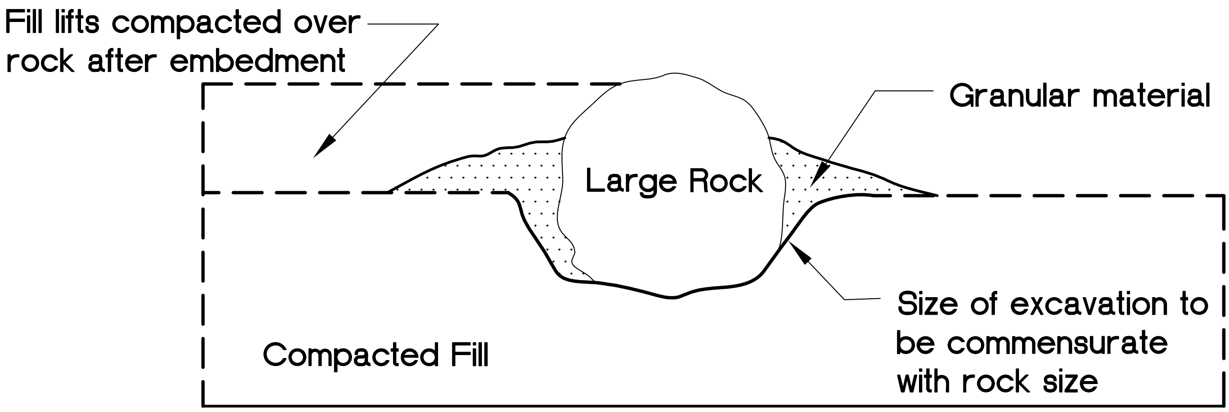
F. All fill over and around rock windrow shall be compacted to at leas
ompaction or as recommended.

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

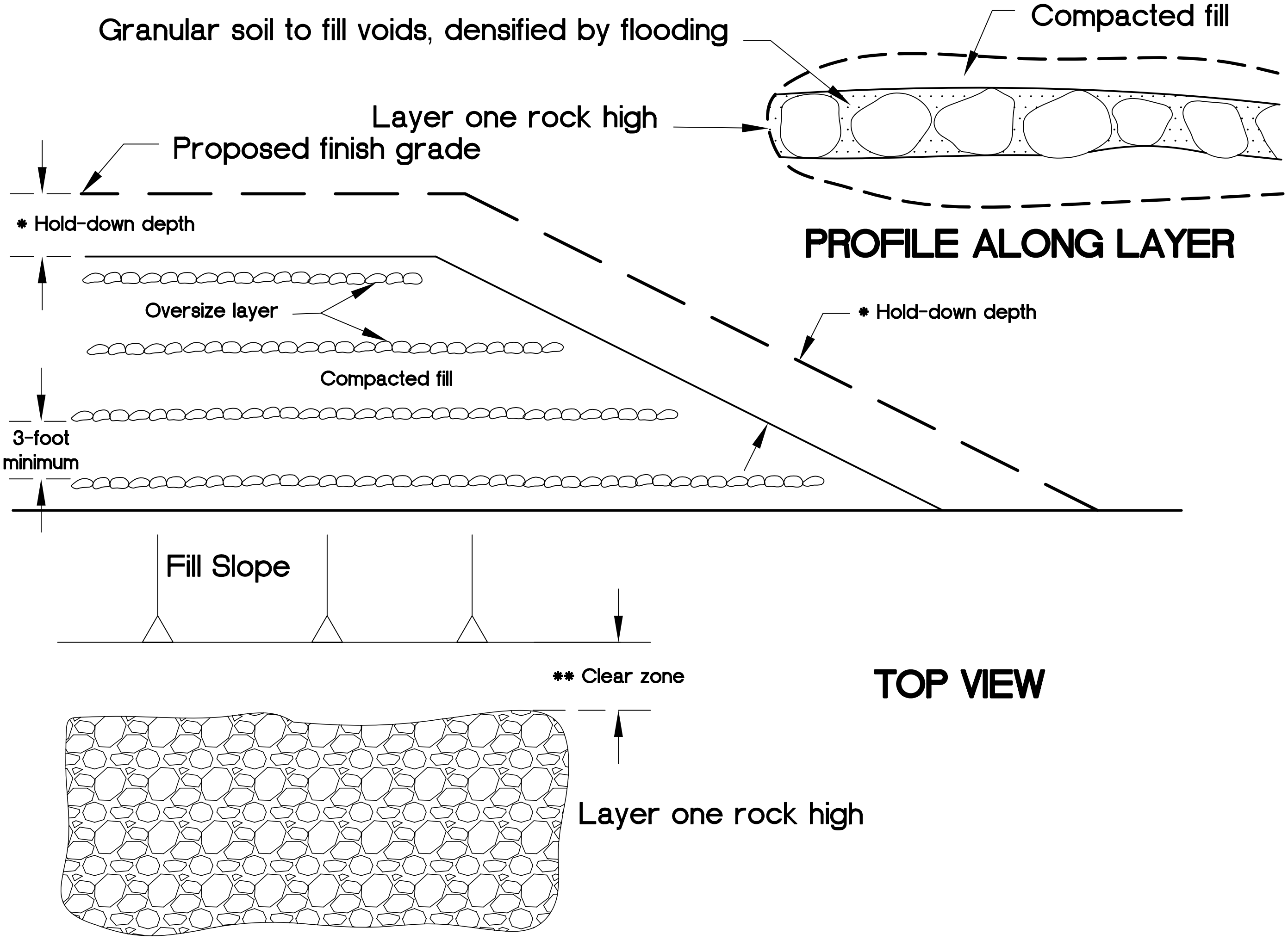
VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE
ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED

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

ROCK DISPOSAL PITS



ROCK DISPOSAL LAYERS



PROFILE ALONG LAYER

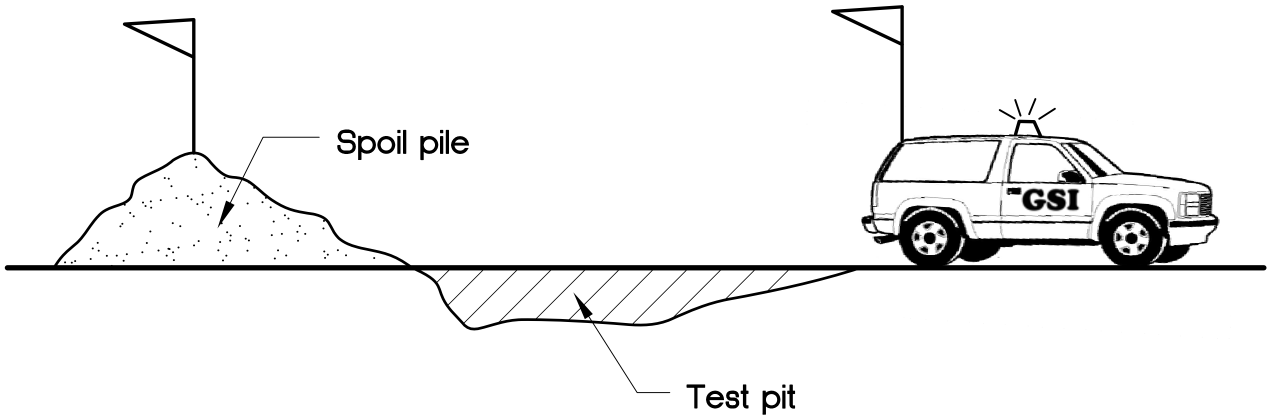
TOP VIEW

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

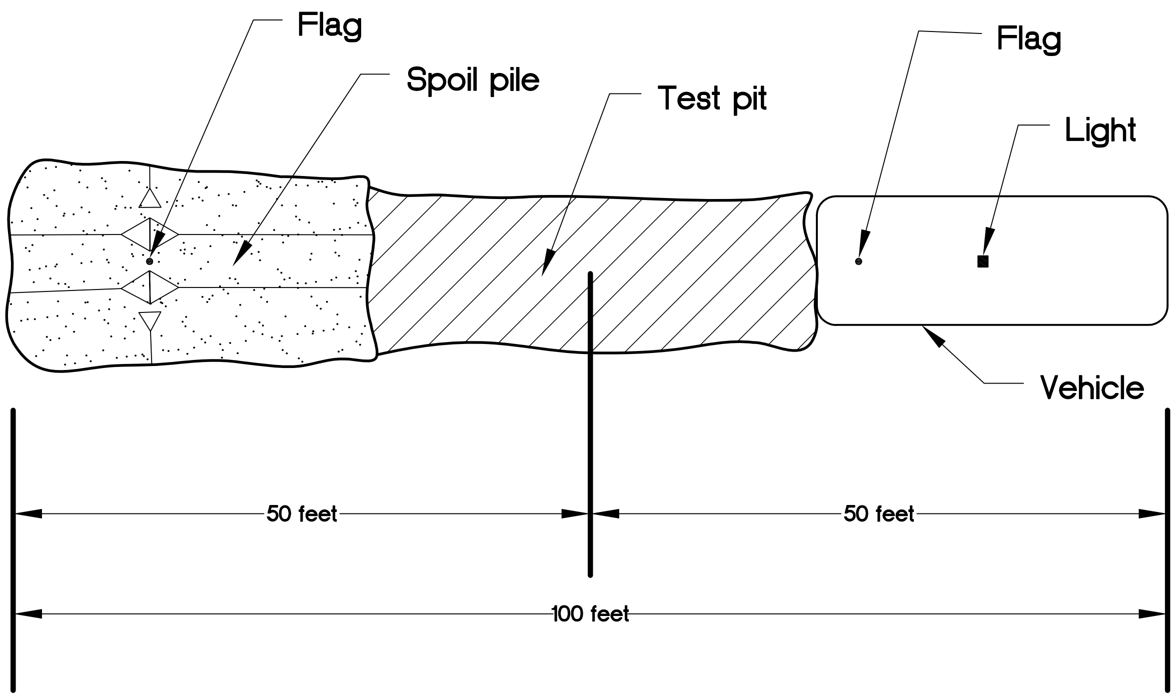
VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE
ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED IN

GeoSoils, Inc.	ROCK DISPOSAL DETAIL	Plate G-14

SIDE VIEW



TOP VIEW



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