PRELIMINARY GEOTECHNICAL INVESTIGATION AND INFILTRATION FEASIBILITY TESTING

PROPOSED 17.64-ACRE INDUSTRIAL/COMMERCIAL SITE
NEC PAINTED CANYON STREET AND EAST DAWES STREET
CITY OF PERRIS, RIVERSIDE COUNTY, CALIFORNIA
(APNS 303-100-012 AND -014)

FOR

ALABBASI CONSTRUCTION & ENGINEERING
764 RAMONA EXPRESSWAY, SUITE C
PERRIS, CALIFORNIA 92571

W.O. 8448-A-SC OCTOBER 14, 2022

GeoSoils, Inc.

Geotechnical • Geologic • Coastal • Environmental

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

October 14, 2022

W.O. 8448-A-SC

Alabbasi Construction & Engineering
764 Ramona Expressway, Suite C
Perris, California 92571

Attention: Ms. Corinne Mostad

Subject: Preliminary Geotechnical Investigation and Infiltration Feasibility Testing, Proposed 17.64-Acre Industrial/Commercial Site, NEC Painted Canyon Street and East Dawes Street, City of Perris, Riverside County, California (APNs 303-100-012 and -014)

Dear Ms. Mostad:

In accordance with your request and authorization, GeoSoils, Inc. (GSI) is presenting the results of our preliminary geotechnical investigation and infiltration feasibility testing for the proposed 17.64-acre industrial/commercial site located at the northeast corner of Painted Canyon Street and East Dawes Street in the City of Perris, Riverside County, California. The primary purpose of this study was to evaluate the onsite soils and geologic conditions and their effects on the proposed industrial/commercial development of the 17.64-acre site, from a geotechnical point of view. A secondary purpose of this study was to provide infiltration feasibility testing for proposed stormwater Best Management Practices (BMP) designs by the civil engineer of record, general earthwork and grading guidelines, and development criteria in light of proposed industrial/commercial development and site geologic conditions.

EXECUTIVE SUMMARY

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

- Based on our subsurface investigation, and published geologic mapping by Morton (2003), the site is underlain by early Pleistocene-age very old alluvial-fan deposits (Qvof). These surficial alluvial deposits are described as well-indurated reddish brown sand deposits.

- As encountered during our recent field work, the site is locally mantled by up to approximately 3 to 5 feet of undifferentiated tilled topsoil and colluvial soils. Due to the relatively low density and lack of uniformity, all near-surface colluvium is considered unsuitable for the support of settlement-sensitive improvements or additional engineered fill, and will need to be removed and recompacted. Additional discussions of remedial site grading and fill placement are provided within following sections of this report.

- Our review indicates no known active faults are crossing the site, and the site is not located within an Alquist-Priolo Earthquake Fault Zone (California Department of Conservation, California Geological Survey [CGS], 2018). In addition, the site is not located within a County of Riverside fault zone. Based on our review of the Riverside County Information Technology website (RCIT, 2022), the site is located within a zone of "low" liquefaction potential, and is characterized as being potentially susceptible to subsidence (RCIT, 2021). Further discussions of the potentials for liquefaction and subsidence are provided within following sections of this report.

- Our review of the City of Perris general plan safety element (2021) indicates the site is located within a dam inundation zone associated with the nearby Perris reservoir (Lake Perris). As such, the potential for flooding should be further evaluated by the design civil engineer for the project.

- An evaluation of storm water infiltration feasibility testing indicates a moderate to relatively low infiltration potential at the project site. Further discussions of the test procedures used, onsite USDA soil groups, general infiltration system siting requirements and limitations, along with the converted infiltration rates obtained are presented herein.

- Adverse geologic features that would preclude project feasibility (e.g., shallow regional groundwater, liquefaction, subsidence, active faulting, etc.) were not encountered.

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

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

Respectfully submitted,

GeoSoils, Inc.

TODD A. GREER
No. 2377

STEPHEN J. COOVER
GE 2057
Exp. 9/30/23

Todd A. Greer
Engineering Geologist, CEG 2377

Stephen J. Coover
Geotechnical Engineer, GE 2057

MAM/TAG/JPF/SJC/sh

Distribution: (1) Addressee (via email PDF)

TABLE OF CONTENTS

FIGURES:

ATTACHMENTS:

PRELIMINARY GEOTECHNICAL INVESTIGATION
AND INFILTRATION FEASIBILITY TESTING
PROPOSED 17.64-ACRE INDUSTRIAL/COMMERCIAL SITE
NEC PAINTED CANYON STREET AND EAST DAWES STREET
CITY OF PERRIS, RIVERSIDE COUNTY, CALIFORNIA
(APNS 303-100-012 AND -014)

SCOPE OF SERVICES

The scope of our services has included the following:

1. Review of online and in-house geologic maps and literature for the site, review of the City of Perris general plan safety element (2021), and review of aerial photographs provided by Google Earth Pro (GEP, 2022) and the United States Department of Agricultural (USDA, 1980, see Appendix A).

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

3. The advancement of 6 exploratory borings across the site for geotechnical logging and soil sample collection, and to evaluate subsurface conditions onsite. In addition, two (2) relatively shallow borings were advanced for infiltration feasibility testing. The borings were advanced on September 15, 2022 using a hollow-stem drill rig (Appendix B).

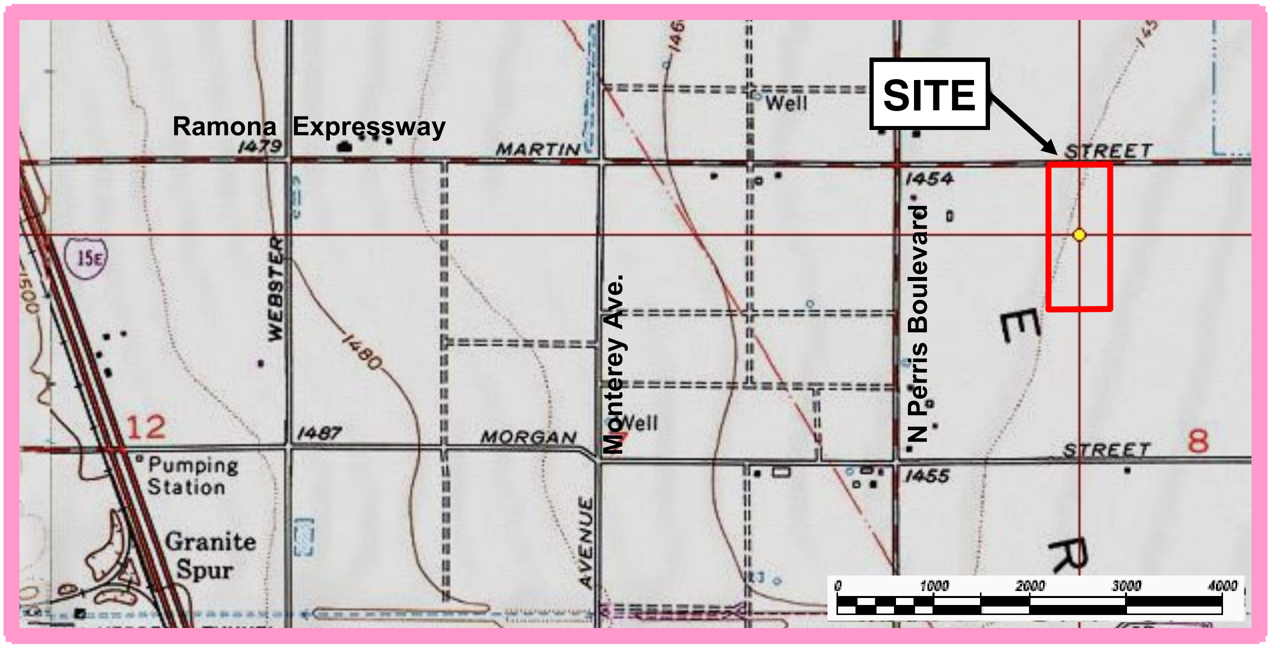
4. General areal seismicity evaluation (Appendix C).

5. Pertinent laboratory testing of representative soil samples collected during our subsurface exploration program. Testing included in-situ moisture and density, maximum density testing, expansion index, sulfate/corrosion, remolded shear, and R-value testing of the materials encountered during our field study. Results of our laboratory testing are provided in Appendix D.

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

SITE LOCATION

The subject 17.64-acre property (APNs 303-100-012 and -014) is located on the NEC of Painted Canyon Street and East Dawes Street (South of Ramona Expressway) in the City of Perris, Riverside County, California (see Figure 1, Site Location Map). Based on our review, the site is generally vacant and undeveloped. Topographically, the property consists of flat-lying terrain that varies in elevation from approximately 1,454 feet MSL (Mean Sea Level) near the northwest corner of the site to approximately 1,449 feet MSL near the middle of the property to approximately 1,451 feet MSL near the southeast corner of the site. Therefore, overall relief is on the order of 3 to 5 feet. Based on our review, the



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

Base Map: Google Maps, Copyright 2022, Map Data Copyright 2022 Google

N

GeoSoils, Inc.

W.O.

8448-A-SC

SITE LOCATION MAP

Figure 1

site is underlain by early Pleistocene-age very old alluvial-fan deposits. The site is currently covered with a moderate growth of native brush and grasses, as well as scattered deleterious materials. The site has been previously tilled for weed abatement purposes.

PROPOSED DEVELOPMENT

Based on the site plan provided by Alabbasi Construction & Engineering (ACE, 2022) it is our understanding that the proposed development of the project would consist of the construction of one (1) 300,000 sq/ft industrial warehouse structure, two (2) 4,000 to 6,000 sq/ft restaurant structures, as well as one (1) 15,000 sq/ft hotel structure within the 17.64-acre property. Development of the project would also include the installation of underground utilities, site infrastructure, and street/parking improvements. We assume that the proposed industrial/commercial structures will be one- or two-stories, and will use continuous footings and slab-on-grade floors, or mat foundations, using wood-frame, masonry block, or tilt-up type of construction. Building loads are assumed to be typical for these types of light industrial/commercial structures. Sewage disposal is to be accommodated by tying into the regional municipal disposal system. The development is also anticipated to include water quality BMP storm water systems.

FIELD STUDIES

Field studies conducted during our evaluation of the property for this investigation consisted of geologic reconnaissance mapping, the advancement of a total of 6 exploratory borings across the property for evaluation of near-surface soil and geologic conditions, sample collection, and 2 borings for infiltration feasibility testing. Field exploration was performed on September 15, 2022, with the associated infiltration feasibility testing conducted on September 16, 2022. The borings were observed and logged by a staff geologist from our firm who also collected representative soil samples for appropriate laboratory testing. The logs of the borings are presented in Appendix B. The approximate locations of the exploratory borings and infiltration feasibility test locations conducted for this study are presented on Plate 1 (Geotechnical Map), which uses the site plan provided by ACE (2022) as a base map.

GEOLOGY

Regional Geologic Setting

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

Site Geology

Based on our recent subsurface investigation, and published geologic mapping by Morton (2003), the site is underlain by early Pleistocene-age very old alluvial-fan deposits (Qvof). Based on our subsurface investigation and geologic reconnaissance mapping, the very old alluvial-fan deposits are locally mantled by up to approximately 3 to 5 feet of native tilled topsoil materials.

Site Earth Materials

As discussed above, the earth materials encountered during our subsurface investigation included tilled topsoil and early Pleistocene-age very old alluvial-fan deposits (Morton, 2003). Mappable geologic units are shown on Plate 1 (Geotechnical Map), and the units are described as follows, from youngest to oldest:

Tilled Topsoil/Colluvium (Unmapped)

As encountered during our recent field work, the site is mantled by approximately 3 to 5 feet of undifferentiated tilled topsoil and colluvial soils (colluvium). These surficial soils were observed to consist primarily of light brownish gray to pale brown silty sands. The colluvium was generally dry to damp, with a loose consistency near the surface becoming medium dense with depth, likely due to previous surficial tilling for weed abatement purposes. These soils typically have a very low expansion potential. However, locally low expansive soils cannot be precluded from occurring onsite. Due to the relatively low density and lack of uniformity, the colluvium throughout the site is deemed unsuitable for the support of new structures or additional fill placement, and will require complete removal and recompaction during rough grading. The colluvial soils may be reused for compacted fills, provided that they have been cleansed of deleterious materials (i.e., trash, debris, weeds, grasses, and concentrations of organic matter) prior to placement onsite as engineered fill.

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

As observed onsite, the very old alluvial-fan deposits generally consisted of pale brown to brown, silty, fine- to coarse-grained sands, interbedded with light yellowish brown to reddish brown clayey, fine- to medium- grained sands. The alluvial sediments varied from damp to locally saturated, and were generally medium dense to very dense with depth.

Expansion index (E.I.) testing performed on a representative sample of the very old alluvial-fan deposits indicates very low expansive soil conditions (E.I. 0-20) across the majority of the site, on a preliminary basis. However, at the conclusion of site grading, low expansive soils (E.I. 21-50) may not be precluded from occurring. The near-surface weathered very old alluvial-fan deposits (upper 1 foot) are locally dry and porous and should be ripped, moisture conditioned, and processed in-place during rough grading. The very old alluvial-fan deposits may be reused for compacted fills, provided that they have been cleansed of deleterious materials (i.e., weeds, grasses, and concentrations of organic matter), prior to placement onsite as engineered fill.

GROUNDWATER/SURFACE WATER

Groundwater was encountered in one (1) of the geotechnical borings (Boring B-1) advanced during our field investigation at a depth of 40½ feet below the ground surface (bgs). Based upon our review of the California Department of Water Resources, Water Data Library (2022), two (2) groundwater wells were located within the site vicinity and reported groundwater depths ranging between 43.6 feet (Station No. 338371N1172274W001, measured March 14, 2022), to 55.9 feet bgs (Station No. 338464N1172319W001, measured November 30, 2020). However, the possibility of localized perched groundwater within drainage areas or along the interface between compacted fills and the underlying very old alluvial-fan deposits cannot be discounted. Seepage may also occur locally (due to heavy precipitation or irrigation) in areas where thin soils overlie less permeable materials. Thus, perched groundwater conditions may occur in the future, and should be anticipated. Additionally, our review of the City of Perris, General Plan (CP, 2016), indicates the site is located within the Lake Perris dam inundation zone. The site is also located within flood Zone X to the south, a low risk, "500 Year Flood Area," and partially within Zone AE to the north, a higher risk, "100 Year Flood Area" (CP, 2016). As such, the potential for flooding and dam inundation should be further evaluated by the design civil engineer.

FAULTING AND REGIONAL SEISMICITY

Local and Regional Faults

Our review indicates that there are no known active faults crossing this site, and the site is not within an Alquist-Priolo Earthquake Fault Zone (California Geological Survey [CGS], 2018). However, the site is situated in a region subject to strong earthquakes occurring along active faults. These faults include, but are not limited to, the local San Jacinto fault systems, the Glen Ivy segment of the Elsinore Fault, and the San Andreas Fault.

According to Blake (2000a), the closest known active fault to the site is the San Jacinto Valley/Casa Loma segment of the San Jacinto Fault Zone, and is located approximately 8 miles (12.8 km) northeast of the site. The San Jacinto Valley/Casa Loma segment of the San Jacinto Fault zone has demonstrated movement in the Holocene Epoch (i.e., last 11,700 years), and therefore, is considered active and is located within an Alquist-Priolo Earthquake Fault Zone (CGS, 2018). Cao, et al. (2003) indicates that the San Jacinto Valley/Casa Loma segment of the San Jacinto Fault zone is an “A” fault and is capable of producing a maximum magnitude (M_w) 6.9 earthquake. The possibility of ground acceleration, or shaking at the site, may be considered as approximately similar to the Southern California region as a whole.

Seismicity

The acceleration-attenuation relations of Bozorgnia, Campbell, and Niazi (1999), have been incorporated into EQFAULT (Blake, 2000a). For this study, peak horizontal ground accelerations anticipated at the site were determined based on the mean plus 1 - sigma attenuation curves developed by those authors. The EQFAULT computer program performs deterministic seismic hazard analyses using digitized California faults as earthquake sources. The program estimates the closest distance between each fault and a given site. If a fault is found to be within a user-selected radius, the program estimates peak horizontal ground acceleration that may occur at the site from an upper bound ("maximum credible") earthquake on that fault. Site acceleration (g) is computed by user-selected acceleration-attenuation relations that are contained in EQFAULT. Based on the EQFAULT program, peak horizontal ground accelerations (deterministic acceleration values) from an upper bound event at the site may be on the order of 0.4203g.

Historical Site Acceleration

Historical site seismicity was evaluated with the acceleration-attenuation relations of Bozorgnia, Campbell, and Niazi (1999) and the computer program EQSEARCH (Blake, 2000b). This program was used to perform a search of historical earthquake records for magnitude 5.0 to 9.0 seismic events within a 100 km radius, between the years 1800 to May 8, 2021. Based on the selected acceleration-attenuation relation, a peak horizontal ground acceleration has been estimated, which may have affected the site during the specific seismic events in the past. Based on the available data and attenuation relationship used, the estimated maximum (peak) site acceleration during the period of 1800 to May 8, 2021, was 0.411g. In addition, a seismic recurrence curve is also estimated/generated from the historical data (see Appendix C).

Seismic Design Parameters

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

2019 CBC SEISMIC DESIGN PARAMETERS

PARAMETER	SITE-SPECIFIC DESIGN VALUE PER ASCE 7-16	2019 CBC or REFERENCE
Risk Category$^{(1)}$	I, II, or III	Table 1604.5
Site Class	D	Section 1613.2.2/Chap. 20 ASCE 7-16 (p. 203-204)
Spectral Response - (0.2 sec), S_s	0.882 g	Section 1613.2.1 Figure 1613.2.1(1)
Spectral Response - (1 sec), S_1	0.666 g	Section 1613.2.1 Figure 1613.2.1(2)
Site Coefficient, F_a	1.0$^{(2)}$	Table 1613.2.3(1)
Site Coefficient, F_v	2.5$^{(3)}$ (Section 21.3)	Table 1613.2.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec), S_{MS}	1.321 g$^{(4)}$ (Section 21.4)	Section 1613.2.3 (Eqn 16-36)
Maximum Considered Earthquake Spectral Response Acceleration (1 sec),S_{M1}	1.067 g$^{(5)}$ (Section 21.4)	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	0.881 g$^{(6)}$	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.712 g$^{(7)}$ (Section 21.4)	Section 1613.2.4 (Eqn 16-39)
PGA_M - Probabilistic Vertical Ground Acceleration may be assumed as about 50% of these values.	0.586 g	ASCE 7-16 (Eqn 11.8.1)
Seismic Design Category	D$^{(8)}$ (Section 11.6)	Section 1613.2.5/ASCE 7-16 (p. 85: Table 11.6-1 or 11.6-2)

1. Risk Category to be confirmed by the Project Architect or Structural Engineer.
2. Per Table 11.4-1 of ASCE 7-16
3. Per Section 21.3 of ASCE 7-16, if $S_1 \geq 0.2$ then F_v is taken as 2.5.
4. Per Section 21.4 of ASCE 7-16, $S_{MS} = (1.5)(S_{DS}) = (1.5)(0.881\text{ g}) = 1.321\text{ g}$
5. Per Section 21.4 of ASCE 7-16, $S_{M1} = (1.5)(S_{D1}) = (1.5)(0.712\text{ g}) = 1.067\text{ g}$
6. Per Section 21.4 of ASCE 7-16, S_{DS} shall be taken as 90 percent of the maximum spectral acceleration (S_a) obtained from the site-specific spectrum at any period within the range from 0.2 to 5 seconds, inclusive.
7. Per Section 21.4 of ASCE 7-16, S_{D1} shall be taken as the maximum value of the product TS_a obtained from the site-specific spectrum from the period within the range of 1 to 5 seconds, inclusive.
8. Per Tables 11.6-1 and 11.6-2 of ASCE 7-16, Mapped S_1 (0.583 g) $\leq$ 0.75 . Thus, the seismic design category is “D”.

GENERAL SEISMIC PARAMETERS	
PARAMETER	**VALUE**
Distance to Seismic Source (San Jacinto Valley Fault)	8.0 mi (12.8 km)(1)
Upper Bound Earthquake (San Jacinto Valley Fault)	M_W = 6.9(2)
(1) - Blake (2000a) (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 or ground failure will not occur in the event of a large earthquake. The primary goal of seismic design is to protect life, not to eliminate all damage, since such design may be economically prohibitive. Cumulative effects of seismic events are not addressed in the 2019 CBC (CBSC, 2019a) and regular maintenance and repair following locally significant seismic events (i.e., M_w5.5) will likely be necessary, as is the case in all of Southern California.

In the event of a maximum probable or credible earthquake occurring on any of the nearby major faults, strong ground shaking would occur in the subject site's general area. Potential damage to any structure(s) would likely be greatest from the vibrations and impelling force caused by the inertia of a structure's mass. This potential would be no greater than that for other existing structures and improvements in the immediate vicinity.

SECONDARY SEISMIC HAZARDS

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

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

A review of the Riverside County Information Technology (RCIT, 2022), or “Map My County v10,” indicates that the site is not located within a County of Riverside fault zone. However, based on our review, the site is located within a zone of “low” liquefaction potential, and is characterized as being potentially susceptible to subsidence (RCIT, 2022). However, our general liquefaction screening evaluation (pursuant to Special Publication 117 [CGS, 2008

SP117]) indicates that the potential for liquefaction and associated adverse effects within the site is considered low, based on the medium dense to very dense very old alluvial-fan deposits which underlie the site at shallow depths, the materials induration (cementation), and anticipated removal of near-surface potentially compressible soils during site grading activities.

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

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

MASS WASTING/LANDSLIDE SUSCEPTIBILITY

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

LABORATORY TESTING

Classification

Soils were classified visually according to the Unified Soils Classification System (Sowers and Sowers (1979). The soil classifications are shown on the Boring Logs presented in Appendix B. The Laboratory Test Results are discussed below and presented in Appendix D.

Moisture-Density

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

Laboratory Standard

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

SOIL TYPE	BORING AND DEPTH (ft.)	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
Silty SAND, Yellowish Brown	B-2 @ 0-5	129.6	9.0

Expansion Potential

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

SOIL TYPE	LOCATION & DEPTH (FT)	EXPANSION INDEX (E.I.)	EXPANSION POTENTIAL
Silty SAND, Yellowish Brown	B-2 @ 0-5	15	Very Low

Soluble Sulfates/Corrosion

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

LOCATION AND DEPTH (FT.)	SOLUBLE SULFATES (PERCENTAGE BY WEIGHT)	CHLORIDE (PPM)	pH	RESISTIVITY (OHMS-CM)
B-2 @ 0-5	< 0.003	11	8.0	3,500

For preliminary planning purposes, based upon the soluble sulfate test results obtained, and the latest edition of the 2019 CBC (CBSC, 2019a), the soluble sulfate content is considered Class "S0" per the ACI 318-14 (0.00 to 0.10 soil percentage by weight is considered Class "S0"). As such, sulfate-resistant concrete is currently not anticipated. Based on the results of the resistivity and pH testing, the onsite soils are generally considered moderately alkaline (a pH of 7.9 to 8.4 is considered moderately alkaline), and are considered moderately corrosive to ferrous metals in a saturated state (2,000 to 10,000 ohm-cm is considered moderately corrosive). Chlorides are generally low.

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

Direct Shear Tests

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

SAMPLE LOCATION AND DEPTH (FT)	PRIMARY		RESIDUAL	
	COHESION (PSF)	FRICTION ANGLE (DEGREES)	COHESION (PSF)	FRICTION ANGLE (DEGREES)
B-2 @ 0-5	16	36	7	35

Resistance Value

Resistance value, or R-Value testing, was performed on representative soil samples in accordance with CalTrans Test Method 301, and yielded a test result of R=19. The results of R-Value testing are presented in Appendix D.

PERCOLATION/INFILTRATION TESTING

In general accordance with guidelines of the Riverside County Flood Control (RCFC, 2011) Design Handbook for Low Impact Development Best Management Practices, and errata (RCFC, 2016), two (2) percolation/infiltration tests were conducted within the proposed water quality BMP locations onsite (see Geotechnical Map, Plate 1), area, as provided by the Client. The percolation testing was conducted at a depth of approximately 5 feet at each test location. The percolation/infiltration testing was performed to further evaluate site conditions with respect to the proposed water quality BMP systems that will retain and filter onsite storm water. The percolation testing was performed in general conformance with the RCFC (2011 and 2016) and CASQA (2003) design handbooks for such testing. The field percolation testing and geologic logging were performed by a staff geologist from our firm. Logs of the borings advanced for this study are included in Appendix B. The field percolation data sheets from our study are presented in Appendix E. Procedures for testing are outlined briefly below:

Percolation Test Procedures

Test Borings:

1. Diameter - 8 inches.
2. After the removal of loose materials, 2 inches of gravel was placed on the bottom of each test boring.
3. A perforated pipe was then installed at each test location to facilitate accurate field measurements and prevent caving during the pre-soak period and testing periods.

Pre-Soaking: After the installation of the perforated pipes, the boring was filled with clear water to a depth of approximately 25 inches. The pre-soak period for the percolation tests continued overnight, as all the water did not seep away while the tester was present.

Sandy Soil Test: During the sandy soil test period, two (2) consecutive measurements were conducted at each test location at intervals of approximately 25 minutes. More than 6 inches of water seeped away during each of the two (2) measurements at test location P-1, therefore sandy soils testing began at that location. Less than 6 inches of water seeped away during each of the two (2) measurements at test location P-2, therefore, non-sandy soils testing methods began at that location.

Testing:	After required pre-soak period and sandy soil test periods, percolation testing measurements were made. A column of clear water was re-established at each of the test locations. The drop in water level was measured from a fixed reference point, refilling after each test measurement. For test location P-1, a series of test measurements were taken for an additional hour, at time intervals of approximately 10 minutes. For test location P-2, a series of test measurements were taken for an additional six hours, at time intervals of approximately 30 minutes.
Accuracy:	All test measurements were read to the nearest ¼-inch.
Test Results:	Calculations from our field testing indicate percolation rates of 6.67 minutes/inch at test location P-1 and 7.06 minutes/inch at test location P-2. Per the RCFC (2011) guidelines, the percolation rates obtained were then converted to infiltration rates using the "Porchet Method," to be used by the design engineer for appropriate sizing of the water quality BMP system. The converted infiltration rates obtained varied between 0.62 inches/hour at test location P-1, and 0.70 inches/hour at test location P-2, with an average of 0.66 inches/hour. Typically, the lowest infiltration rate obtained is applied to the design. The converted infiltration rates, along with the formulas used are presented on Figure 2.

USDA Site Soil Groups, Soil Units, Ksat Values

Our review of the United States Department of Agriculture (USDA, 2022) Web Soil Survey (https://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm), indicates two (2) major soil units underlie the project site. The Domino silt loam (Dv), is distributed throughout the north- northeast quarter of the site. The Exeter sandy loam(EpA) is distributed throughout the southwestern three quarters of the site, and is in the vicinity of both the potential locations of the proposed water quality BMP systems. Based on our review USDA (1971), the Domino silt loam belongs to Hydrologic Soil Group (HSG) "C," with the capacity of the most limiting soil layer to transmit water classified as "low." The "K_{sat}" value (i.e., hydraulic conductivity or infiltration rate) for the soil type onsite was evaluated by the USDA to range from 0.63 to 2.00 inches per hour. The more extensive Exeter sandy loam also belongs to HSG "C," with the capacity of the most limiting soil layer to transmit water also classified as "very low." The "K_{sat}" value (i.e., hydraulic conductivity or infiltration rate) for the soil type onsite was also evaluated by the USDA to range from 0.63 to 2.00 inches per hour. The results of site specific infiltration testing (this study) are within the general data sets presented in the USDA soil web survey.

Percolation Rate to Infiltration Rate Conversion

* Infiltration Rate (I_t) = $\frac{\Delta H \, \pi r^2 \; 60}{\Delta t(\pi r^2 + 2\pi r H_{avg})}$ = $\frac{\Delta H \; 60 \; r}{\Delta t(r+2H_{avg})}$

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

		Δt	**Init Level**	**Fnl Level**	ΔH	H_{avg}	I_t	
Infiltration Test Numbers	P-1 @ 5.5 ft.	10	28	26 1/2	1 1/2	27 1/4	0.62	Low = 0.70 Average = 0.66 **DIR = 0.62
	P-2 @ 5 ft.	30	24 1/2	20 1/4	4 1/4	22 3/8	0.70	

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

** DIR = Design Infiltration Rate

W.O. 8448-A-SC
Figure 2

Infiltration Basin Siting Requirements

Our review of the general infiltration basin siting requirements and limitations (CASQA, 2003), indicates sites characterized as belonging to Hydrologic Soil Group "A," "B," and "C" may be suitable for infiltration, requiring a minimum soil infiltration rate of 0.5 inches/hour (CASQA, 2003). Based on our review of historic regional groundwater levels and recent onsite subsurface investigation, a minimum 10-foot vertical separation from the bottom of the BMP system to the top of historic high groundwater levels should be maintained, provided shallow (i.e., ≤ 5 feet) BMP systems are used.

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

Onsite Storm Water Quality Best Management Practice (BMP) Systems

It is our understanding that infiltration-runoff retention systems (OIRRS) are planned for Best Management Practices (BMP's) or Low Impact Development (LID) principles for the project. Therefore, certain guidelines must be followed in the planning, design, and construction of such systems. Such systems, if improperly designed or implemented without consideration of the geotechnical aspects of site conditions, can contribute to flooding, saturation of bearing materials beneath site improvements, slope instability, and possible concentration and contribution of pollutants into the groundwater, storm drain system, or utility trench systems.

A key factor in these systems is the infiltration rate (often referred to as the percolation rate) which can be ascribed to, or determined for, the earth materials within which these systems are installed. Additionally, the infiltration rate of the designed system (which may include gravel, sand, mulch/topsoil, or other amendments, etc.) will need to be considered. The project infiltration testing is very site specific, any changes to the location of the proposed OIRRS or estimated size of the OIRRS, may require additional infiltration testing. Locally, relatively impermeable formations include: clayey surficial soils, igneous and metamorphic bedrock, as well as future fine grained fill soils.

Some of the methods which are used for onsite infiltration include percolation basins, dry wells, bio-swale/bio-retention, permeable pavers/pavement, infiltration trenches, filter boxes and subsurface infiltration galleries/chambers. Some of these systems are constructed using native and import soils, perforated piping, and filter fabrics while others employ structural components such as stormwater infiltration chambers and filters/separators. Every site will have characteristics which should lend themselves to one or more of these methods, but not every site is suitable for OIRRS. In practice, OIRRS are usually initially designed by the project design civil engineer. Selection of methods should include (but should not be limited to) review by licensed professionals including the geotechnical engineer, hydrogeologist, engineering geologist, project civil engineer, landscape architect, and environmental professional. Applicable governing agency requirements should be reviewed and included in design considerations.

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

1. It is not good engineering practice to allow water to saturate soils, especially near slopes or improvements; however, the controlling agency/authority is now requiring this for OIRRS purposes on many projects.

2. Impermeable liners used in conjunction with basins should consist of a 30-mil polyvinyl chloride (PVC) membrane that is covered by a minimum of 12 inches of clean soil, free from rocks and debris, at a maximum inclination of 3:1 (h:v), and meets the following minimum specifications:

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

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

4. 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.

5. Infiltrations systems should not be installed within 8 feet of building foundations utility trenches, and walls/retaining walls, or a 1:1 (horizontal to vertical [h:v]) slope (down and away) from the bottom elements of these improvements. Alternatively, deepened foundations or pile/pier supported improvements may be used.

6. Infiltrations systems should not be installed adjacent to pavement or hardscape improvements. Alternatively, deepened/thickened edges and curbs or impermeable liners may be used in areas adjoining the OIRRS.

7. As with any OIRRS, localized ponding and groundwater seepage should be anticipated. The potential for seepage or perched groundwater to occur after site development should be disclosed to all interested/affected parties.

8. Installation of infiltrations systems should avoid expansive soils (Expansion Index [E.I.] $\geq$ 51) or soils with a relatively high plasticity index (P.I. > 20).

9. Infiltration systems should not be installed where the vertical separation of the groundwater level is less than 10 feet from the base of the system.

10. Where permeable pavements are planned as part of the system, the site Traffic Index (T.I.) Should be less than 25,000 Average Daily Traffic (ADT), as recommended in Allen, et al. (2011).

11. Infiltration systems should be designed using a suitable factor of safety (FOS) to account for uncertainties in the known infiltration rates (as generally required by the controlling authorities), and reduction in performance over time.

12. As with any OIRRS, proper care will need to provided. Best management practices should be followed at all times, especially during inclement weather. Provisions for the management of any siltation, debris within the OIRRS, or overgrown vegetation (including root systems) should be considered. An appropriate inspection schedule will need to adopted and provided to all interested/affected parties.

13. Any designed system will require regular and periodic maintenance, which may include rehabilitation or complete replacement of the filter media (e.g., sand, gravel, filter fabrics, topsoils, mulch, etc.) or other components used in construction, so that the design life exceeds 15 years. Due to the potential for piping and adverse seepage conditions, a burrowing rodent control program should also be implemented onsite.

14. Newly established vegetation/landscaping (including phreatophytes) may have root systems that will influence the performance of the OIRRS or nearby LID systems.

15. The potential for surface flooding, in the case of system blockage, should be evaluated by the design engineer.

16. Any proposed utility backfill materials (i.e., inlet/outlet piping or other subsurface utilities) located within or near the proposed area of the OIRRS may become saturated. This is due to the potential for piping, water migration, or seepage along the utility trench line backfill. If utility trenches cross or are proposed near the OIRRS, cut-off walls or other water barriers will need to be installed to mitigate the potential for piping and excess water entering the utility backfill materials. Planned or existing utilities may also be subject to piping of fines into open-graded gravel backfill or pipe bedding layers unless separated from overlying or adjoining OIRRS by geotextiles or slurry backfill. Slurry backfill is recommended.

17. The use of OIRRS above existing utilities that might degrade/corrode with the introduction of water/seepage should be avoided.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

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

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

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

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

EARTHWORK CONSTRUCTION RECOMMENDATIONS

General

All earthwork should conform to the guidelines presented in the 2019 CBC (CBSC, 2019a), the requirements of the City of Perris and County of Riverside, and the General Earthwork and Grading Guidelines presented in Appendix F, except where specifically superceded in the text of this report. Prior to earthwork, a GSI representative should be present at the preconstruction meeting to provide additional earthwork guidelines, if needed, and review the rough grading and earthwork schedules. This office should be notified in advance of any fill placement, supplemental regrading of the site, or backfilling underground utility trenches and retaining walls after rough earthwork has been completed. This includes grading for driveway approaches, driveways, and exterior hardscape improvements.

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

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

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

3. In general, and based upon the available data to date, regional groundwater does not appear to be a factor in site development or underground utility installation. However, seepage may be encountered along fill/native contacts or throughout the site along with seasonal perched water. Seepage and a transient perched water table can also develop along, or near, the contact between near surface fills and the underlying native soil, most likely after heavy rains, due to irrigation practices, BMP systems, or other factors not evident at the time of our review. This may occur after development. Although generally not anticipated, the need for localized subdrainage systems for the control of seepage and perched water may be necessary.

4. Based upon the proposed development planned and our field exploration, the very old alluvial-fan deposits throughout the site should be readily rippable with conventional earthwork equipment, in good working order.

5. Due to the non-cohesive and locally dry nature of some of the onsite materials, caving and sloughing should be anticipated in all subsurface excavations and trenching. Therefore, current local and state/federal safety ordinances for subsurface trenching should be enforced.

6. General earthwork, grading guidelines, and preliminary criteria are provided at the end of this report as Appendix F. Specific recommendations are provided below.

Demolition/Grubbing

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

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

3. Cavities or loose soils remaining after demolition and site clearance should be cleaned out, observed by the soils engineer, processed, and replaced with fill that has been moisture conditioned to <u>at least</u> optimum moisture content and compacted to at least 90 percent of the laboratory standard (ASTM D 1557), if not removed by proposed cuts.

Treatment of Existing Ground

1. All surficial tilled topsoil materials and low density near surface very old alluvial fan deposits (upper 3 to 5 feet, with an average of approximately 4 feet) should be removed to competent very old alluvial-fan deposits (i.e., greater than or equal to 85 percent compaction, or greater than or equal to 105 pcf for in-place native materials), if not removed by proposed excavation within areas proposed for settlement-sensitive improvements. For preliminary planning purposes, removal depths are estimated to be approximately 3 to 5 feet across the site, with the potential for localized deeper removals. However, a minimum of 2 feet of compacted fill should underlie proposed building foundations. Actual depths of removals will be evaluated in the field during grading by the geotechnical consultant.

2. After the above removals, the upper 6 inches of the exposed subsoils should be scarified, brought to <u>at least</u> optimum moisture content, and recompacted to a minimum relative compaction of 90 percent of the laboratory standard.

3. The existing site soils may be reused as compacted fill <u>provided</u> that any significant concentrations of vegetation and miscellaneous trash/debris are removed prior to or during fill placement.

4. Localized deeper removal may be necessary due to localized undocumented artificial fills or dry porous materials. The project geotechnical consultant/geologist should observe all removal areas during the grading.

Fill Placement

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

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

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

Transition and Overexcavation Areas

Although generally not anticipated based on the flay-lying nature of the project site, in order to reduce the potential for possible differential settlements between cut and fill materials or materials of differing engineering properties, the entire cut portion of cut/fill transitions should be overexcavated to a minimum depth of 3 feet below finish grade, or a maximum ratio of fill thickness of 3:1 (maximum to minimum), and replaced with compacted fill. In addition, building pads located entirely in cut areas, if any, should be overexcavated and capped with at least 3 feet of fill, or 2 feet below the bottom of proposed footings, whichever is greater.

Subdrains

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

Preliminary Earthwork Factors

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

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

Tilled Topsoil	15% to 25% shrinkage
Very Old Alluvial Fan Deposits	5% to 8% shrinkage

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

PRELIMINARY RECOMMENDATIONS - FOUNDATIONS

General

Preliminary recommendations for foundation design and construction are provided in the following sections. These preliminary recommendations have been developed from our understanding of the currently planned site development, site observations, subsurface exploration, laboratory testing, and engineering analyses. Foundation design should be re-evaluated at the conclusion of site grading/remedial earthwork for the as-graded soil conditions. Although not anticipated, revisions to these recommendations may be necessary.

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

Expansive/Corrosive Soils

The laboratory testing conducted for this study indicates that the onsite soils do not meet the criteria of detrimentally expansive soils as defined in Section 1803.5.3 of the 2019 CBC. With recommended site grading, the overall expansive character of site soils is anticipated to be very low expansive.

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

PRELIMINARY FOUNDATION DESIGN FOR INDUSTRIAL/COMMERCIAL STRUCTURES

We anticipate average and maximum static column loads of 50 and 150 kips, respectively for the proposed industrial/commercial structures. Maximum wall loads are anticipated to be on the order of 2 to 5 kips per lineal foot. Static differential settlement is estimated at 1 inch in 50 feet, and seismic differential settlement is estimated at ½ inch in 50 feet. Based on the above, we have considered the following:

- Conventional spread/continuous footings for very low expansive soils, with design to accommodate the differential settlement provided herein.

The preliminary foundation design and construction recommendations provided herein are based on laboratory testing and engineering analysis of onsite earth materials by GSI. Recommendations for footings/foundation systems and associated design parameters are provided in the following sections. The foundation systems may be used to support the proposed industrial/commercial structures, provided they are founded in competent bearing materials. As discussed previously, conventional spread/continuous footings may be used; however, they will need to consider static and seismic settlement. Mat-type foundation systems may also be used. The site structural engineer should be informed of this to aid in preliminary foundation designs. The proposed foundation systems should also be designed and constructed in accordance with other applicable guidelines contained in the 2019 CBC (CBSC, 2019a).

Isolated Spread and Continuous Footings

1. Based on the anticipated foundation loads and preliminary design information, it is our opinion that the proposed structure(s) can favorably be supported on recompacted fill soils. Building loads may be supported on continuous or isolated spread footings (typically 18 to 30 inches below planned 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 = 4 FEET)	ALLOWABLE BEARING CAPACITY FOR CONTINUOUS WALL FOOTINGS (MINIMUM WIDTH = 2 FEET)
18	1.5 ksf	1.5 ksf
24	2.0 ksf	2.0 ksf
30	2.5 ksf	2.5 ksf

The above values are for dead plus live loads and may be increased by one-third for short-term wind or seismic loads. Where column or wall spacings are less than twice the width of the footing, some reduction in bearing capacity may be necessary to compensate for the effects of group action. Reinforcement should be designed in accordance with local codes and structural considerations.

The recommended allowable bearing capacity is generally based on maximum total and differential settlements indicated herein 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 exceed those indicated herein. With increased footing depth/width ratios, differential settlement should be less, provided a minimum fill cap is maintained beneath all footings. GSI should review foundation plans and evaluate foundation specific load patterns.

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

3. Passive earth pressure may be computed as an equivalent fluid having a density of 250 pounds per cubic foot (pcf) with a maximum lateral earth pressure of 2,500 pounds per square foot (psf).

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

5. Due to the anticipated granular nature of the site fill, i.e., native soils, as well as the potential for seismic loading, all footings should maintain a minimum 7-foot horizontal distance from the base of the footing to any adjacent descending slope, and minimally comply with the guidelines depicted on Figure 1808.7.1 of the 2019 CBC (CBSC, 2019a).

Preliminary Construction Recommendations for Industrial/Commercial Structures

The following foundation construction recommendations are presented as a minimum criteria from a soils engineering viewpoint. The expansion potential of onsite soils is considered very low. Accordingly, the following preliminary foundation construction recommendations are for soils in the upper 7 feet from finish grade, which may have a very low expansion potential. For foundation design, the project's design-structural engineer or architect, may exceed the geotechnical consultants recommendations and should take precedence over the following minimum requirements.

1. Conventional continuous footings should be founded at a minimum depth of 18 to 30 inches (depending on the allowable bearing value from the previous section) below the lowest adjacent ground surface for typical industrial/commercial building loads. Interior footings may be founded at a minimum depth of 18 to 30 inches below the lowest adjacent ground surface. The entire foundation should be supported by at least 2 feet of compacted fill. Footings should have a minimum width of 24 inches. All continuous footings should be reinforced with a minimum of four No. 5 reinforcing bars, two at the top and two No. 5 reinforcing bars at the bottom.

2. Isolated exterior pier and column footings should be constructed 24 inches square by 24 inches deep, and tied to the main foundation in at least one direction with a grade beam. Isolated footing reinforcement should be designed by the project structural engineer.

3. A grade beam, reinforced as above and at least 18 inches deep (minimum of 12 inches square), should be provided across garages, large, or wide entrances. The base of the reinforced grade beam should be at the same elevation as the adjoining footings.

4. Concrete slabs should be constructed and underlain with a vapor retarder and slab underlayment as indicated in the “Soil Moisture Transmission Considerations” section of this report.

5. A minimum slab thickness of 5 inches is recommended, and the slab subgrade should be free of loose and uncompacted material prior to placing concrete. The design engineer should determine the actual thickness of concrete slabs based upon proposed loading and use.

6. Concrete slabs should be reinforced with No. 3 reinforcement bars (per the CBC 2019), placed on 18-inch centers, in two horizontally perpendicular directions (i.e., long axis and short axis).

7. 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.

8. Specific presaturation is not required for very low expansive soil conditions; however, the moisture content of the subgrade soils should be equal to, or greater than, optimum moisture to a depth of 18 to 30 inches below the adjacent ground grade in the slab areas for very low expansive soils, or the depth of the foundation. This should be evaluated by the geotechnical consultant within 72 hours of the vapor retarder and steel reenforcement placement.

9. Foundations near the top of slope should be deepened to conform to the latest edition of the 2019 CBC (CBSC, 2019) and provide a minimum of 7 feet horizontal distance from the slope face. Rigid block wall designs, located along the top of slope, should be reviewed by a geotechnical consultant.

PRELIMINARY FLOOR SLAB DESIGN RECOMMENDATIONS - ISOLATED SPREAD AND CONTINUOUS FOOTING FOUNDATION SYSTEMS

General

Concrete slab-on-grade floor construction is anticipated. The following are presented as minimum design parameters for the slab, but they are in no way intended to supercede design by the structural engineer. Design parameters do not account for concentrated loads (e.g., fork lifts, heavy rack loads, other machinery, etc.) or the use of freezers or heating boxes. These recommendations are meant as minimums. The project architect or structural engineer should review and verify that the minimum recommendations presented herein are considered adequate with respect to anticipated uses.

Light Load Floor Slabs

The slabs in areas that will receive relatively light live loads (i.e., office space, less than 50 psf) should be a minimum of 5 inches thick and be reinforced with No. 3 reinforcing bar on 18-inch centers in two horizontally perpendicular directions. Reinforcing should be properly supported to ensure placement near the vertical midpoint of the slab. “Hooking” of the reinforcement is not considered an acceptable method of positioning the steel.

The project structural engineer should consider the use of transverse and longitudinal control joints to help control slab cracking due to concrete shrinkage or expansion. Two of the best ways to control this movement are: 1) add a sufficient amount of reinforcing steel to increase the tensile strength of the slab; and 2) provide an adequate amount of control or expansion joints to accommodate anticipated concrete shrinkage and expansion. Transverse and longitudinal crack control joints should be spaced no more than 12 feet on center and constructed to a minimum depth of T/4, where “T” equals the slab thickness in inches.

Heavy Load Floor Slabs

The project structural engineer should design the slabs in areas subject to high loads (machinery, forklifts, storage racks, etc.). The Modulus of subgrade reaction (k_S-value) may be used in the design of the floor slab supporting heavy truck traffic, fork lifts, machine foundations, and heavy storage areas. A k_S-value of 100 pounds per square inch per inch (pci) would be prudent to use for preliminary slab design. An R-value test or plate load test may be used to verify the k_S-value on near-surface fill soils.

Concrete slabs should be at least 6 inches thick and reinforced with No. 5 reinforcing bars placed 12 inches on center in two horizontally perpendicular directions. Selection of slab thickness compatibility with anticipated loads should be provided by the structural engineer.

Transverse and longitudinal crack control joints should be spaced no more than 14 feet on center and constructed to a minimum depth of T/4, where "T" equals the slab thickness in inches. The use of expansion joints in the slab should be considered. Spacing of expansion or crack control joints should be modified based on the footprint of the area to be heavily loaded.

Slab Subgrade Preparation

Subgrade material should be compacted to a minimum of 90 percent of the maximum laboratory dry density (ASTM D 1557). Prior to placement of concrete, the subgrade soils should be moisture conditioned to 18 to 30 inches below grade (depending on the footing depth used) to at least the soils' optimum moisture content, for very low to low expansive soils.

After moisture conditioning, the subgrade soils should be evaluated by a field representative of GSI prior to vapor retarder placement, and prior to and within 72 hours of concrete placement. Alternative methods, including sealing the subgrade surface with select sand/base and periodic moisture conditioning, may also be considered, as long as the minimum recommended soil moisture content is achieved. In summary:

EXPANSION INDEX	PAD SOIL MOISTURE	CONSTRUCTION METHOD	SOIL MOISTURE RETENTION
Very Low (0-20)	Upper 18 to 30 inches of pad at or above soil optimum moisture	Wetting or reprocessing	Periodically wet or cover with plastic after trenching. Evaluation 72 hours prior to placement of concrete.

PRELIMINARY MAT FOUNDATION DESIGN RECOMMENDATIONS

Mat Foundations

In lieu of using a conventional foundation system, the Client may consider a mat foundation which uses steel bar reinforcement. The structural engineer may supercede the following recommendations based on the planned building loads and use. Wire Reinforcement Institute (WRI, 2016) methodologies for design may be used.

Mat Foundation Design

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

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

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

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

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

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

Confirmation Testing for Final Foundation Design

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

SOIL MOISTURE TRANSMISSION CONSIDERATIONS

GSI has evaluated the potential for vapor or water transmission through the concrete floor slab, in light of typical floor coverings and improvements. Please note that slab moisture emission rates range from about 2 to 27 lbs/24 hours/1,000 square feet from a typical slab (Kanare, 2005), while floor covering manufacturers generally recommend about 3 lbs/24 hours as an upper limit. The recommendations in this section are not intended to preclude the transmission of water or vapor through the foundation or slabs. Foundation systems and slabs shall not allow water or water vapor to enter into the structure so as to cause damage to another building component or to limit the installation of the type of flooring materials typically used for the particular application (State of California, 2022). These recommendations may be exceeded or supplemented by a water "proofing" specialist, project architect, or structural consultant. Thus, the client will need to evaluate the following in light of a cost versus benefit analysis (owner expectations and repairs/replacement), along with disclosure to all interested/affected parties.

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

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

- Concrete slab-on-grade floors should be thicker.

- Concrete slab underlayment should consist of a 15-mil vapor retarder, or equivalent, with all laps sealed per the 2019 CBC and the manufacturer's recommendation. The vapor retarder should comply with the ASTM E 1745 - Class A criteria, and be installed in accordance with ACI 302.1R-04 and ASTM E 1643.

- The 15-mil vapor retarder (ASTM E 1745 - Class A) shall be installed per the recommendations of the manufacturer, including <u>all</u> penetrations (i.e., pipe, ducting, rebar, etc.).

- Concrete slabs, including garages, should be underlain by 2 inches of clean sand (S.E. $\geq$ 30) above a 15-mil vapor retarder (ASTM E 1745 - Class A, per Engineering Bulletin 119 [Kanare, 2005]). The vapor retarder should in-turn, be underlain by

2 inches of sand (S.E. $\geq$ 30) placed directly on the prepared, moisture conditioned, subgrade. The vapor retarder should be sealed to provide a continuous retarder under the entire slab and should be installed per the recommendations of the manufacturer, including all penetrations (i.e., pipe, ducting, rebar, etc.). The manufacturer should provide instructions for lap sealing, including minimum width of lap, method of sealing, and either supply or specify suitable products for lap sealing (ASTM E 1745), and per Code.

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

- Additional concrete mix design recommendations should be provided by the structural consultant or waterproofing specialist. Concrete finishing and workability should be addressed by the structural consultant and a waterproofing specialist.

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

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

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

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

PRELIMINARY WALL DESIGN PARAMETERS

General

Recommendations for the design and construction of conventional masonry retaining walls are provided below. Recommendations for specialty walls (i.e., crib, earthstone, mechanically stabilized earth [MSE], gravity, etc.) can be provided upon request, and would be based on site specific conditions.

Conventional Retaining Walls

The design parameters provided below assume that either very low expansive soils (typically Class 2 permeable filter material or Class 3 aggregate base) or native onsite materials with an expansion index up to 50 are used to backfill any retaining wall. The type of backfill (i.e., select or native), should be specified by the wall designer, and clearly shown on the plans. Building walls, below grade, should be water-proofed. Waterproofing should also be provided for site retaining walls in order to reduce the potential for efflorescence staining.

Preliminary Retaining Wall Foundation Design

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

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

Minimum Footing Width - 24 inches.

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

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

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

Backfill Soil Density - Soil densities ranging between 125 pcf and 135 pcf may be used in the design of retaining wall foundations. This assumes an average engineered fill compaction of at least 90 percent of the laboratory standard (ASTM D 1557).

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

Restrained Walls

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

Cantilevered Walls

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

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

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

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

Seismic Surcharge

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

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

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

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

Retaining Wall Backfill and Drainage

Positive drainage must be provided behind all retaining walls in the form of gravel wrapped in geofabric and outlets. A backdrain system is considered necessary for retaining walls that are 2 feet or greater in height. Details 1, 2, and 3, present the backdrainage options discussed below. Backdrains should consist of a 4-inch diameter perforated PVC or ABS pipe encased in either Class 2 permeable filter material or ¾-inch to 1½-inch gravel wrapped in approved filter fabric (Mirafi 140 or equivalent). For select backfill, the filter material should extend a minimum of 1 horizontal foot behind the base of the walls and upward at least 1 foot. For native backfill that has up to E.I. = 20, continuous Class 2 permeable drain materials should be used behind the wall. This material should be laterally continuous behind the wall, and it should be constructed in accordance with the enclosed Detail 1 (Typical Retaining Wall Backfill and Drainage Detail). For limited access and confined areas, (panel) drainage behind the wall may be constructed in accordance with Detail 2 (Retaining Wall Backfill and Subdrain Detail Geotextile Drain). Materials with an expansion index (E.I.) potential of greater than 50 should not be used as backfill for retaining walls. Retaining wall backfill materials should be moisture conditioned and mixed to achieve the soil's optimum moisture content, placed in relatively thin lifts (6 to 10 inches), and compacted to at least 90 percent relative compaction. For more onerous expansive situations, backfill and drainage behind the retaining wall should conform with Detail 3 (Retaining Wall And Subdrain Detail Clean Sand Backfill).

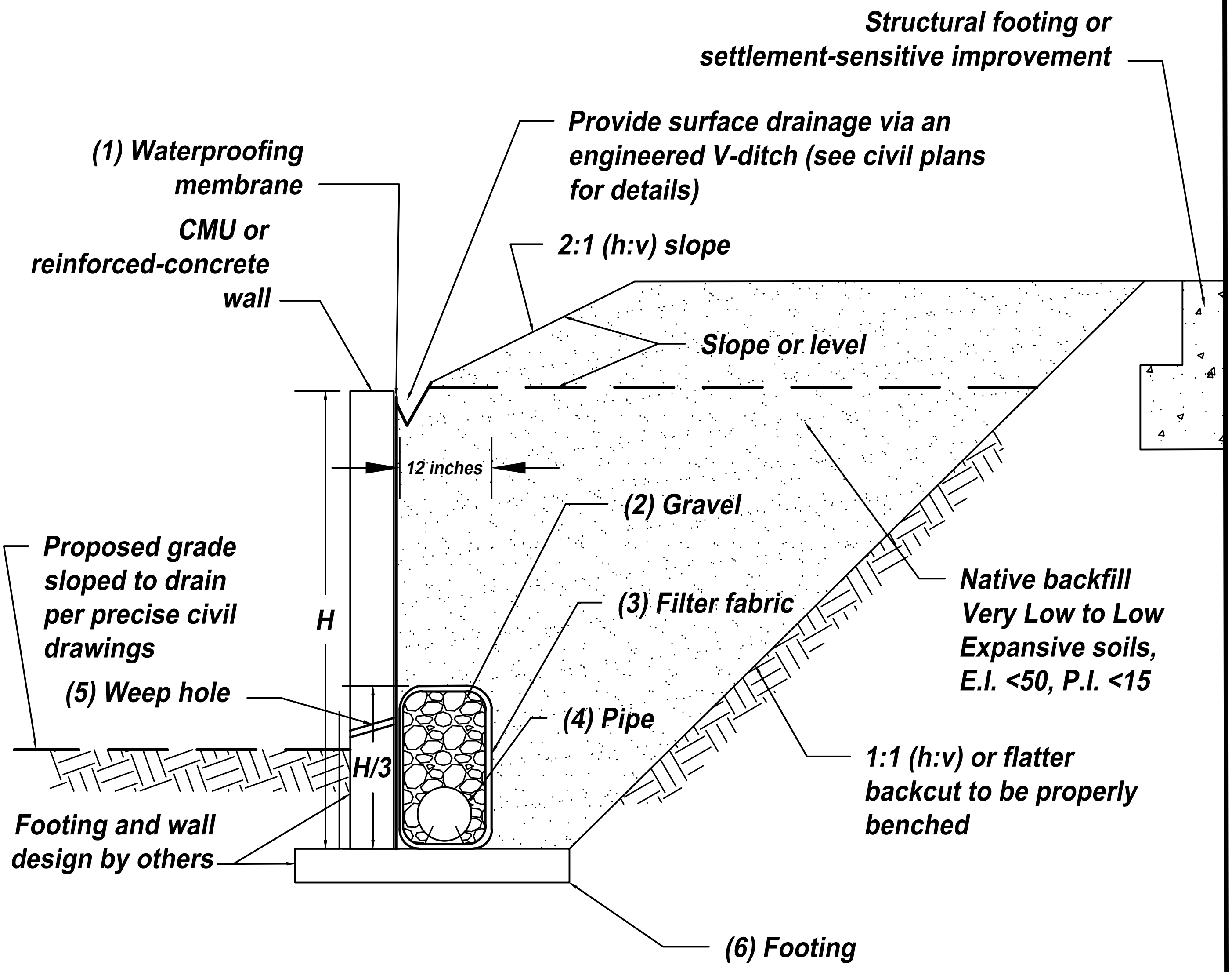
Outlets should consist of a 4-inch diameter solid PVC or ABS pipe spaced no greater than 100 feet apart, with a minimum of two outlets, one on each end. The use of weep holes, only, in walls higher than 2 feet, is not recommended. The surface of the backfill should be sealed by pavement or the top 18 inches compacted with native soil (E.I. ≤ 50). Proper surface drainage should also be provided. For additional mitigation, consideration should be given to applying a water-proof membrane to the back of all retaining structures. The use of a waterstop should be considered for all concrete and masonry joints.

Wall/Retaining Wall Footing Transitions

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

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.



(1) Waterproofing membrane.

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

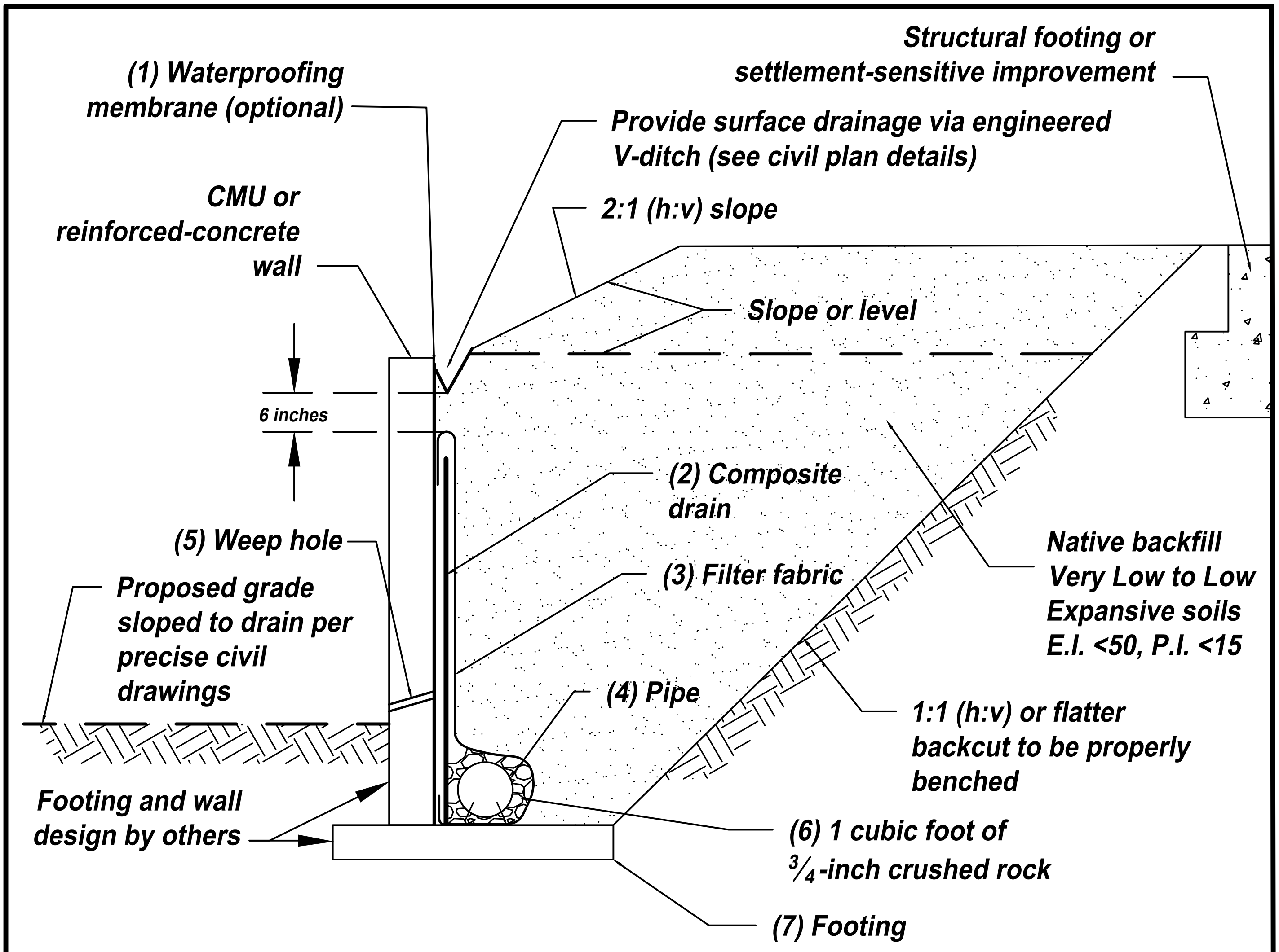
(3) Filter fabric: Mirafi 140N or approved equivalent.

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

(5) Weep holes: For CMU walls, Omit grout every other block, at or slightly above finished surface. For reinforced concrete walls, minimum 2-inch diameter weep holesspaced at 20 foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

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

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.

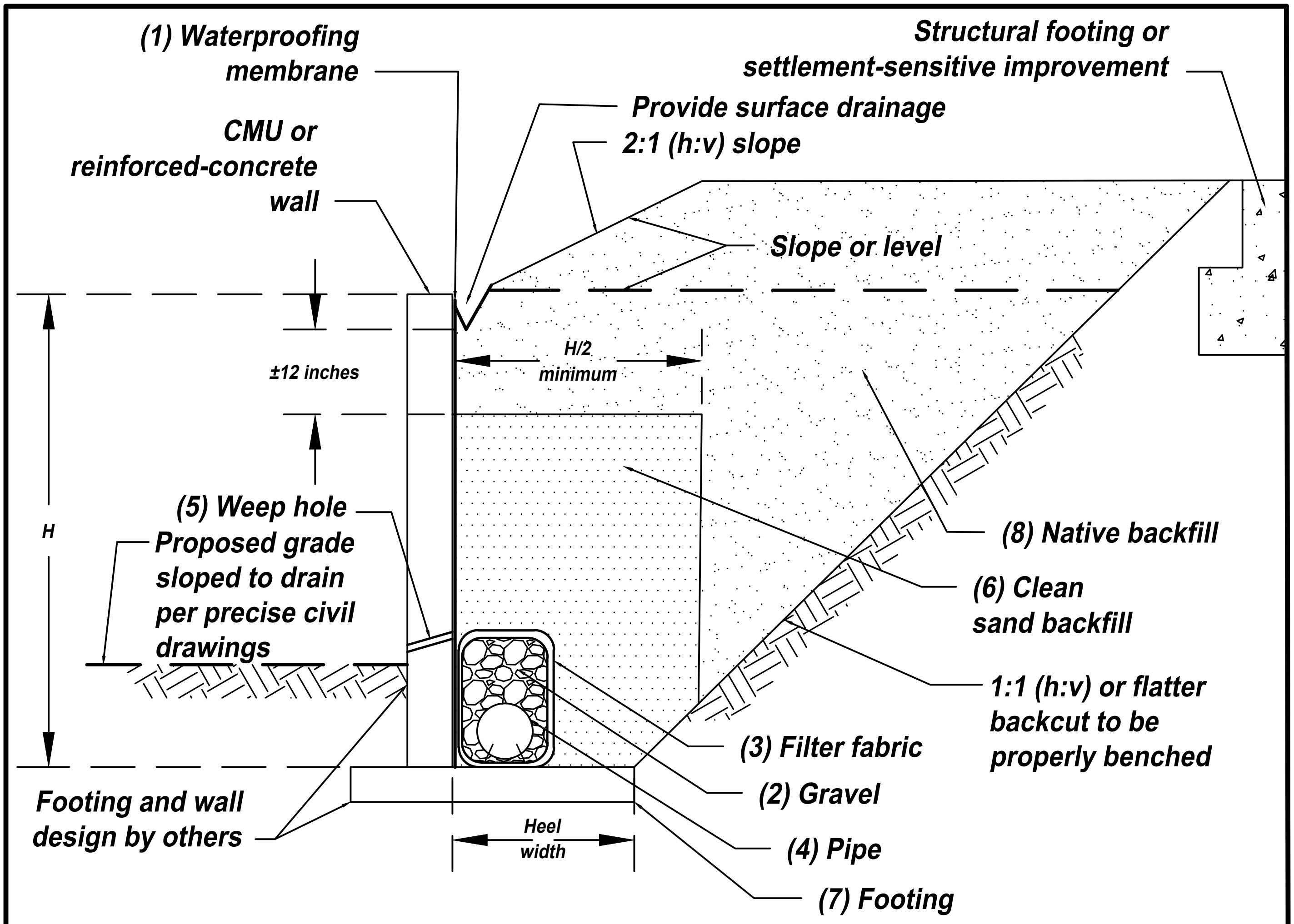
(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

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

GSI 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: For CMU walls, Omit grout every other block, at or slightly above finished surface. For reinforced concrete walls, minimum 2-inch diameter weep holesspaced at 20 foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

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

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

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

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

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

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

PRELIMINARY PAVEMENT DESIGN AND CONSTRUCTION

General

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

Asphaltic Concrete (AC) Pavements

The preliminary design for Asphaltic Concrete (AC), and Portland Cement Concrete Pavement (PCCP) was evaluated based on a R-value of 19, and the use of concrete shoulders (curb or gutter) at the edge of PCC pavement. GSI does not recommend the use of an ADTT value of less than 25 for any pavement section, unless the ADTT significantly less than 25 is certified by a civil engineer specializing in traffic engineering.

Pavement Design

The preliminary pavement sections presented in the following table are based on the preliminary R-value test results obtained, the minimum paving thickness provided by the County of Riverside (Standard Details Nos. 103, 111 and 114) for a local street, a collector (road), an industrial collector street, and the guidelines presented in the latest revision to the California Department of Transportation (Caltrans, 2020) "Highway Design Manual" seventh edition. It is our understanding that the traffic index (TI) value for a local street is 5.5, and the minimum pavement section required by the County of Riverside 3.0 inches of AC (asphaltic concrete) on 6.0 inches of Class 2 aggregate base. Based on the R-value of 19 obtained (i.e., R=19), the minimum asphalt concrete and base thickness are presented below. Applicable sections of City and/or County ordinances should be followed during design of public roads, fire access lanes, etc.

Asphaltic Concrete Pavement (ACP)

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

STREET CLASSIFICATION	TRAFFIC INDEX (T.I.)[1]	STANDARD PAVEMENT DESIGNS		
		R-VALUE	AC INCHES	CLASS 2 BASE ROCK[2] INCHES
Local Street	5.5	19	3.0*	9.0
Collector (Road)	7.0	19	4.0*	12.0
Industrial Collector Street	8.0	19	4.7*	14.0

[1] T.I.s provided in the County of Riverside Standard Details (Standard Detail No. 114).
[2] Denotes standard Caltrans Class 2 aggregate base rock ® ≥ 78, SE ≥ 25).
* Denotes County of Riverside minimum asphaltic concrete or aggregate base.

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

Portland Concrete Cement (PCC) Pavement

The preliminary design for Portland Cement Concrete Pavement (PCCP) was evaluated using a subgrade R-value of 19, a modulus of rupture (MR) of 420 and 500 psi. GSI does not recommend the use of an ADTT value of less than 25 for any pavement section, unless the ADTT significantly less than 25 is certified by a civil engineer specializing in traffic engineering. The preliminary PCCP sections are provided in the following table:

PORTLAND CONCRETE CEMENT PAVEMENTS (PCCP)*

TRAFFIC AREAS	CONCRETE TYPE	PCCP THICKNESS (INCHES)	TRAFFIC AREAS	CONCRETE TYPE	PCCP THICKNESS (INCHES)
Light Vehicles	520-C-2500	6.0	Dumpster Aprons (Trash Service)	520-C-2500	8.0
	560-C-3250	5.0		560-C-3250	7.0

NOTE: All PCCP is designed as un-reinforced and bearing directly on compacted subgrade. However, a 4-inch thick leveling course of compacted aggregate base, or crushed rock may be considered to improve performance. All PCCP should be properly detailed (jointing, etc.) per the industry standard. Pavements may be additionally reinforced with #4 reinforcing bars, placed 18 inches on center, each way, for improved performance.
* To be re-evaluated based on exposed field conditions and R-values obtained following rough grading.

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

Weakened Plane Joints

Transverse and longitudinal weakened plane joints may be constructed per Greenbook Standard Specifications (2021), Section 302-6.5, or the structural/civil engineer. Transverse weakened plane joints should be spaced no farther than 15 feet apart and no closer than 5 feet. Longitudinal weakened plane joints should be spaced no farther than 20 feet apart, but not less than 5 feet.

Expansion Joints and Contact Joints

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

Slab Reinforcement

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

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

Concrete/Pervious Pavers

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

PAVEMENT GRADING RECOMMENDATIONS

General

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

Subgrade

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

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

Aggregate Base Rock

Compaction tests are required for the recommended base section. Minimum relative compaction required will be 95 percent of the laboratory maximum density as evaluated by ASTM Test Designation D 1557. Base aggregate should be in accordance to the Caltrans Class 2 base rock (minimum R-value=78).

Drainage

Positive drainage should be provided for all surface water to drain towards the area swale, curb and gutter, catch basin, or to an approved drainage channel. Positive site drainage should be maintained at all times. Water should not be allowed to pond or seep into the ground. If planters or landscaping are adjacent to paved areas, measures should be taken to minimize the potential for water to enter the pavement section, such as thickened edges, enclosed planters, etc.

Additional Considerations

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

DRIVEWAYS, CONCRETE APRONS, FLATWORK, AND OTHER IMPROVEMENTS

Based on the very low expansive soil materials on the site, the following recommendations are presented for all exterior flatwork:

1. The subgrade area for sidewalk slabs should be compacted to achieve a minimum 90 percent relative compaction, the subgrade area for access drive slabs and concrete aprons should be compacted to achieve a minimum 95 percent relative compaction, and then be presoaked to 2 to 3 percentage points above (or 125 percent of) the soils' optimum moisture content, to a depth of 18 inches below subgrade elevation. If very low expansive soils are present, only optimum moisture content, or greater, is required and specific presoaking is not warranted. The moisture content of the subgrade should be proof tested within 72 hours prior to concrete placement.

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

3. Exterior sidewalk slabs should be a minimum of 4 inches thick. Access drive slabs should be a minimum of 5 inches thick. Slabs and approaches should additionally have a thickened edge (12 inches) adjacent to all landscape areas, to help impede infiltration of landscape water under the slab. Trash disposal (dumpster) area aprons should be a minimum of 6 inches thick and meet minimum City standards, as necessary.

4. Curbs next to slopes should have a thickened edge similar to drives and approaches.

5. The use of transverse and longitudinal control joints are recommended to help control slab cracking due to concrete shrinkage or thermal expansion. Two ways

to mitigate such cracking are: a) add a sufficient amount of reinforcing steel, increasing tensile strength of the slab; and, b) provide an adequate amount of control or expansion joints to accommodate anticipated concrete shrinkage and thermal expansion.

In order to reduce the potential for unsightly cracks, slabs should be reinforced at mid-height with a minimum of No. 3 bars placed at 18 inches on center, in each direction. If subgrade soils within the top 7 feet from finish grade are very low expansive soils (i.e., E.I. ≤ 20), then 6x6-W1.4xW1.4 welded-wire mesh may be substituted for the rebar, provided the reinforcement is placed on chairs, at slab mid-height. The exterior slabs should be scored or saw cut, 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. Presoaking, as indicated earlier, is recommended for slab subsoils.

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

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

8. Planters and walls (sound walls or retaining walls) should not be tied to the structure.

9. Overhang structures should be supported on the slabs, or structurally designed with continuous footings tied in at least one direction for very low expansive soils.

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

11. Utilities may be enclosed within a closed utilidor (vault) or designed with flexible connections to accommodate differential settlement and thermal expansion conditions.

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

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

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

DEVELOPMENT CRITERIA

Slope Maintenance and Planting

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

Drainage

Adequate lot surface drainage is a very important factor in reducing the likelihood of adverse performance of foundations, hardscape, and slopes. Surface drainage should be sufficient to prevent ponding of water anywhere on a lot, and especially near structures and tops of slopes. Lot surface drainage should be carefully taken into consideration during fine grading, landscaping, and building construction. Therefore, care should be taken that future landscaping or construction activities do not create adverse drainage conditions.

Positive site drainage within lots and common areas should be provided and maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond or seep into the ground. In general, the area within 5 feet around a structure should slope away from the structure. We recommend that unpaved lawn and landscape areas have a minimum gradient of 1 percent sloping away from structures, and whenever possible, should be above adjacent paved areas. Consideration should be given to avoiding construction of planters adjacent to structures (buildings, retaining walls, etc.). Pad drainage should be directed toward the street or other approved area(s). Although not a geotechnical requirement, roof gutters, downspouts, or other appropriate, means may be used to control roof drainage. Downspouts, or drainage devices, should outlet a minimum of 5 feet from structures or into a subsurface drainage system. Areas of seepage may develop due to irrigation or heavy rainfall, and should be anticipated. Minimizing irrigation will lessen this potential. If areas of seepage develop, recommendations for minimizing this effect could be provided upon request.

Erosion Control

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

Landscape Maintenance

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

Subsurface and Surface Water

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

Site Improvements

If any additional improvements (e.g., trash enclosures, walls, etc.) are planned for the site, recommendations concerning the geological or geotechnical aspects of design and construction of said improvements could be provided upon request. This office should be notified in advance of any fill placement, grading of the site, or trench backfilling after rough grading has been completed. This includes any grading, utility trench and retaining wall backfills, flatwork, etc.

Tile Flooring

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

Additional Grading

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

Footing Trench Excavation

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

Trenching/Temporary Construction Backcuts

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

Utility Trench Backfill

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

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

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

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

SUMMARY OF RECOMMENDATIONS REGARDING GEOTECHNICAL OBSERVATION AND TESTING

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

- During grading/recertification.
- During excavation.
- During placement of subdrains, toe drains, or other subdrainage devices, prior to placing fill or backfill.
- After excavation of building footings, retaining wall footings, and free standing walls footings, prior to the placement of reinforcing steel or concrete.
- Prior to pouring any slabs or flatwork, after presoaking/presaturation of building pads and other flatwork subgrade, before the placement of concrete, reinforcing steel, capillary break (i.e., sand, pea-gravel, etc.), or vapor barriers (i.e., Stego Wrap, Husky Guard, etc.).
- During retaining wall subdrain installation, prior to backfill placement.
- During placement of backfill for area drain, interior plumbing, utility line trenches, and retaining wall backfill.
- During slope construction/repair.
- When any unusual soil conditions are encountered during any construction operations, after to the issuance of this report.
- When any developer or owner improvements, such as flatwork, walls, etc., are constructed, prior to construction.
- A report of geotechnical observation and testing should be provided at the conclusion of each of the above stages, in order to provide concise and clear documentation of site work, or to comply with code requirements.
- GSI should review project sales documents to owners/owners associations for geotechnical aspects, including irrigation practices, the conditions outlined above, etc., prior to any sales. At that stage, GSI will provide owners maintenance guidelines which should be incorporated into such documents.

OTHER DESIGN PROFESSIONALS/CONSULTANTS

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

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

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

PLAN REVIEW

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

LIMITATIONS

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

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

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

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

Alabbasi Construction & Engineering, 2022, General plot plan, 1 sheet, scale 1"=40'-0", undated.

Allen, V., Connerton, A., and Carlson, C., 2011, Introduction to Infiltration Best Management Practices (BMP), Contech Construction Products, Inc., Professional Development Series, dated December.

American Concrete Institute, 2015, Guide to concrete floor and slab construction (ACI 318-15): reported by ACI Committee 302, dated June.

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

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

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

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

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

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

_____, 2010, Minimum design loads for buildings and other structures, ASCE Standard ASCE/SEI 7-10.

Blake, T.F., 2000a, EQFAULT, A computer program for the estimation of peak horizontal acceleration from 3-D fault sources; Windows 95/98 version, updated to September, 2004.

_____, 2000b, EQSEARCH, A computer program for the estimation of peak horizontal acceleration from California historical earthquake catalogs; Updated through August, 2018.

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

Bryant, W.A., and Hart, E.W., 2007, Fault-rupture hazard zones in California, Alquist-Priolo earthquake fault zoning act with index to earthquake fault zones maps; California Geological Survey, Special Publication 42, interim revision.

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

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

California Code Of Regulations, 2011, CAL-OSHA State of California Construction and Safety Orders, dated February.

California Department of Conservation, California Geological Survey (CGS), 2018, Earthquake fault zones, a guide for government agencies, property owners/developers, and geoscience practitioners for assessing fault rupture hazards in California: California Geological Survey Special Publication 42 (revised 2018), 93 p.

_____, 2008, Guidelines for evaluating and mitigating seismic hazards in California: California Geological Survey Special Publication 117A (revised 2008), 102 p.

California Department of Transportation (Caltrans), 2020, Highway design manual, seventh edition, https://dot.ca.gov/programs/design/manual-highway-design-manual-hdm, dated July 1.

_____, 2010, Caltrans, standard specifications, dated May.

California Department of Water Resources, 2022, Water Data Library, interactive website, (http://wdl.water.ca.gov/waterdatalibrary/), last modified May 29.

California Office of Statewide Health Planning and Development (OSHPD), 2022, Seismic design maps, https://seismicmaps.org/.

California Stormwater Quality Association (CASQA), 2003, Stormwater best management practice handbook, new development and redevelopment, dated January.

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

City of Perris, 2021, General plan safety element, approved January 25, 2022.

County of Riverside Transportation and Land Management Agency, 2000, Technical guidelines for review of geotechnical and geologic reports.

CTL Thompson, 2005, Controlling moisture-related problems associated with basement slabs-on-grade in new residential construction.

Federal Emergency Management Agency, 2008, National flood hazard mapping "Map Center," www.fema.gov/hazard/flood/index.shtm

Google Earth Pro - v7.3.4, 2022, by Google, LLC, https://www.google.com/earth/download/gep/agree.html, [accessed September 14, 2022].

Jennings, C.W., and Bryant, W.A., 2010, Fault activity map of California, scale 1:750,000, California Geological Survey, Geologic Data Map No. 6.

Kanare, Howard, M., 2005, Concrete Floors and Moisture, Engineering Bulletin 119, Portland Cement Association, pp. 35-42.

Morton, D.M., 2003, Preliminary geologic map of the Perris 7.5' quadrangle, Riverside County, California, U.S. Geological Survey Open-File Report 03-270, version 1.0.

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

Public Works Standards, Inc., 2021, "Greenbook" standard specifications for public works construction, 2021 edition.

Riverside County Flood Control and Water Conservation District, 2016, Errata for September, 2011 design handbook for low impact development best management practices, posted June 30.

_____, 2011, Design handbook for low impact development best management practices, dated September.

Riverside County Information Technology (RCIT), Graphic Information Services (GIS), 2022, Map my County, v10, website: https://gis1.countyofriverside.us/Html5Viewer/index.html?viewer=MMC_Public.

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

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

State of California, 2022, Civil Code, Sections 895 et seq.

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

United States Department of Agriculture (USDA), Natural Resources Conservation Services, 2021, Web soil survey, version 3.3.2, (website: https://websoilsurvey.sc.egov. usda.gov/App/HomePage.htm).

____, 1980, Aerial photographs, region 615120, flight line 1380, photo numbers 80, and 81, scale 1"=3,333 feet, dated October 30.

____, 1971, Soil Survey Western Riverside Area, California, issued November.

Wire Reinforcement Institute, Inc., 2016, Manual of standard practice, structural welded wire reinforcement, dated December.

APPENDIX B

BORING 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.

File:Mgr: c;\SoilClassif.wpd

PLATE B-1

GeoSoils, Inc.

BORING LOG

PROJECT: ALABASSI - PERRIS COMMERCIAL

W.O. 8448-A-SC *BORING* B-1 *SHEET* 1 *OF* 2

DATE EXCAVATED 9-15-22 *LOGGED BY:* MAM *APPROX. ELEV.:* 1449'

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

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
0				SM				**TILLED TOPSOIL/COLLUVIUM:** @ 0', SILTY SAND, light brownish gray to pale brown, dry to damp, medium dense; fine to coarse SAND, organics at surface.
		⌶	47	SM	112.3	7.0	39.0	@ 2.5', SILTY SAND, brown, damp, dense; fine to coarse SAND, contains calcium carbonate.
				SM				**QUATERNARY VERY OLD ALLUVIAL FAN DEPOSITS (Qvof):** @ 3', SILTY SAND, brown, damp, dense; fine to coarse SAND.
5		⌶	61	SM	122.9	5.4	41.3	@ 5, SILTY SAND, brown, damp, very dense; fine to coarse SAND.
		⌶	23	SM	108.3	9.6	48.3	@ 7.5', SILTY SAND, pale brown, damp, medium dense; fine to medium SAND.
10		⌶	31	SM	114.0	5.5	32.3	@ 10', As per 7.5', yellowish brown, damp.
15		▼	15	SC-SM		12.3		@ 15', SILTY CLAYEY SAND, pale brown, wet, medium dense.
								@ 17', Rig chatter, operator added water.
20		⌶	26	SC	113.6	16.0	93.1	@ 20', CLAYEY SAND, light yellowish brown, wet, medium dense; fine to medium SAND.
25		▼	21	SC		18.0		@ 25', As per 20'.
30		⌶	36	SC	123.1	12.0	93.6	@ 30', As per 25', reddish brown, wet.

▼ *Standard Penetration Test*

⌶ *Undisturbed, Ring Sample*

▼ *Groundwater*

Seepage

GeoSoils, Inc.

BORING LOG

PROJECT: ALABASSI - PERRIS COMMERCIAL

W.O. 8448-A-SC *BORING* B-1 *SHEET* 2 *OF* 2

DATE EXCAVATED 9-15-22 *LOGGED BY:* MAM *APPROX. ELEV.:* 1449'

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

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
35		▼	24	SC		13.2		@ 35', As per 30'; water added by operator.
40 ▼		⌶	50/5	SC	129.3	9.2	87.4	@ 40', As per 35', wet, very dense; fine to coarse SAND.
45		▼	28	SC		16.2		@ 45', As per 40', wet, medium dense.
50		⌶	50/4.5	SC	121.9	11.7	87.1	@ 50', As per 40'.
								@ 51.5', SILTY SAND, reddish brown, wet, very dense; fine to coarse SAND, driller terminated boring at target depth, very difficult drilling.
55								
60								
65								

Total Depth = 51.5'.
Groundwater Encountered at 40.5'.
No Caving.
Backfilled 9-15-22.

▼ *Standard Penetration Test*
⌶ *Undisturbed, Ring Sample*
▼ *Groundwater*
♀ *Seepage*

GeoSoils, Inc.

BORING LOG

PROJECT: ALABASSI - PERRIS COMMERCIAL

W.O. 8448-A-SC *BORING* B-2 *SHEET* 1 *OF* 1

DATE EXCAVATED 9-15-22 *LOGGED BY:* MAM *APPROX. ELEV.:* 1449'

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

	Sample							
Depth (ft.)	Bulk	Undisturbed	Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	**Material Description**
0	■			SM				**TILLED TOPSOIL/COLLUVIUM:** @ 0', SILTY SAND, light brownish gray to pale brown, dry to damp, medium dense; fine to coarse SAND, organics at surface.
5		⌶	85	SM	116.8	4.5	28.5	**QUARTERNARY VERY OLD ALLUVIAL FAN DEPOSITS (Qvof):** @ 5', SILTY SAND, brown, damp, very dense; fine to coarse SAND.
10		▼	17	SM		6.4		@ 10', As per 5'; more fine SAND, medium dense.
15		⌶	29	SC-SM	102.3	23.0	99.4	@ 15', SILTY CLAYEY SAND, brown, saturated, medium dense; fine to medium SAND.
20		▼	17	SM		17.0		@ 20', SILTY SAND, grayish brown, wet, medium dense; fine to medium SAND.
25								
30								

Total Depth - 21.5'
No Groundwater Encountered.
No Caving.
Backfilled 9-15-22.

▼ *Standard Penetration Test*
⌶ *Undisturbed, Ring Sample*

▼ *Groundwater*
♀ *Seepage*

GeoSoils, Inc.

BORING LOG

PROJECT: ALABASSI - PERRIS COMMERCIAL

W.O. 8448-A-SC *BORING* B-3 *SHEET* 1 *OF* 1

DATE EXCAVATED 9-15-22 *LOGGED BY:* MAM *APPROX. ELEV.:* 1450'

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

Depth (ft.)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	**Material Description**
	Bulk	Undisturbed	Blows/Ft.					
0				SM				**TILLED TOPSOIL/COLLUVIUM:** @ 0', SILTY SAND, light brownish gray to pale brown, dry to damp, medium dense; fine to coarse SAND, organics at surface.
5		⌶	25	SM	87.9	17.1	51.3	**QUATERNARY VERY OLD ALLUVIAL FAN DEPOSITS (Qvof):** @ 5', SILTY SAND, brown to yellowish brown, moist, medium dense; fine to coarse SAND, trace calcium carbonate inclusions.
10		▼	15	SM		14.9		@ 10', As per 5'; no calcium carbonate.
15		⌶	48	SM	120.3	10.3	72.8	@ 15', As per 10'.
20		▼	19	SC-SM		12.0		@ 20', SILTY CLAYEY SAND, yellowish brown, moist, medium dense; fine to medium and trace coarse SAND.
								Total Depth = 21.5' No Groundwater Encountered. No Caving. Backfilled 9-15-22.
25								
30								

▼ *Standard Penetration Test*

⌶ *Undisturbed, Ring Sample*

▼ *Groundwater*

Seepage

GeoSoils, Inc.

PLATE B-5

GeoSoils, Inc.

BORING LOG

PROJECT: ALABASSI - PERRIS COMMERCIAL

W.O. 8448-A-SC *BORING* B-4 *SHEET* 1 *OF* 1

DATE EXCAVATED 9-15-22 *LOGGED BY:* MAM *APPROX. ELEV.:* 1451'

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

Depth (ft.)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
	Bulk	Undisturbed	Blows/Ft.					
0				SM				**TILLED TOPSOIL/COLLUVIUM:** @ 0', SILTY SAND, pale brown, dry, medium dense; fine to coarse SAND, organics at surface.
								QUARTERNARY VERY OLD ALLUVIAL FAN DEPOSITS (Qvof): @ 3', SILTY SAND, brown, damp, medium dense.
5		▼	45	SM		10.4		@ 5', SILTY SAND, brown, moist, dense; fine to coarse SAND.
10		⌶	30	SM	117.0	10.3	65.9	@ 10', As per 5', medium dense.
15		▼	14	SC		22.0		@ 15', CLAYEY SAND, brown, moist to wet, medium dense; fine to medium SAND, trace coarse SAND. @ 17', Rig chatter.
20		⌶	39	SC	109.4	16.0	80.5	@ 20', As per 15', wet, dense.
25								Total Depth - 21.5' No Groundwater Encountered. No Caving. Backfilled 9-15-22.
30								

▼ *Standard Penetration Test*
⌶ *Undisturbed, Ring Sample*
▽ *Groundwater*
Seepage

GeoSoils, Inc.

GeoSoils, Inc.

BORING LOG

PROJECT: ALABASSI - PERRIS COMMERCIAL

W.O. 8448-A-SC *BORING* B-5 *SHEET* 1 *OF* 1

DATE EXCAVATED 9-15-22 *LOGGED BY:* MAM *APPROX. ELEV.:* 1451'

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

Depth (ft.)	Sample Bulk	Sample Undisturbed	Sample Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
0				SM				**TILLED TOPSOIL/COLLUVIUM:** @ 0', SILTY SAND, pale brown, dry, loose; fine to coarse SAND, organics at surface.
				SM				**QUARTERNARY VERY OLD ALLUVIAL FAN DEPOSITS (Qvof):** @ 3', As per 0', medium to light brown; no organics.
5		⌶	78	SM	127.9	4.3	38.7	@ 5', SILTY SAND, pale brown to brown, damp, very dense; fine to coarse SAND.
10		▼	6	SC-SM		25.1		@ 10', SILTY CLAYEY SAND, brownish yellow to pale brown, wet, loose; fine to medium SAND.
15		⌶	45	SC-SM	122.8	10.8	82.2	@ 15', As per 10', brownish yellow, dense.
20		▼	16	SC-SM		11.9		@ 20', As per 15', medium dense.
								Total Depth - 21.5' No Groundwater Encountered. No Caving. Backfilled 9-15-22.
25								
30								

▼ *Standard Penetration Test*

⌶ *Undisturbed, Ring Sample*

▽ *Groundwater*

♀ *Seepage*

GeoSoils, Inc.

PLATE B-7

GeoSoils, Inc.

BORING LOG

PROJECT: ALABASSI - PERRIS COMMERCIAL

W.O. 8448-A-SC *BORING* B-6 *SHEET* 1 *OF* 1

DATE EXCAVATED 9-15-22 *LOGGED BY:* MAM *APPROX. ELEV.:* 1451'

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

Depth (ft.)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
	Bulk	Undisturbed	Blows/Ft.					
0				SM				**TILLED TOPSOIL/COLLUVIUM:** @ 0', SILTY SAND, pale brown, dry, medium dense; fine to coarse SAND, organics at surface.
				SM				**QUARTERNARY VERY OLD ALLUVIAL FAN DEPOSITS (Qvof):** @ 3', SILTY SAND, brown, damp, loose; fine to coarse SAND.
5		▼	10	SM		8.7		@ 5', SILTY SAND, pale brown to brown, damp, loose; fine to coarse SAND.
10		⌶	31	SC-SM	103.4	20.4	90.5	@ 10', SILTY CLAYEY SAND, brownish yellow to pale brown, wet, medium dense; fine to medium SAND.
15		▼	15	SC-SM		15.0		@ 15', As per 10'.
20		⌶	53	SP-SM	116.1	6.0	36.2	@ 20', Poorly graded SAND with SILT, very pale brown, damp, dense; fine to coarse SAND.
								Total Depth - 21.5' No Groundwater Encountered. No Caving. Backfilled 9-15-22.
25								
30								

▼ *Standard Penetration Test*

⌶ *Undisturbed, Ring Sample*

▽ *Groundwater*

♀ *Seepage*

GeoSoils, Inc.

PLATE B-8

GeoSoils, Inc.

BORING LOG

PROJECT: ALABASSI - PERRIS COMMERCIAL

W.O. 8448-A-SC BORING P-1 SHEET 1 OF 1

DATE EXCAVATED 9-15-22 LOGGED BY: MAM APPROX. ELEV.: 1450'

SAMPLE METHOD: 8" HSA

Depth (ft.)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
	Bulk	Undisturbed	Blows/Ft.					
0				SM				**TILLED TOPSOIL/COLLUVIUM:** @ 0', SILTY SAND, light brownish gray to pale brown, dry to damp, medium dense to loose; fine to coarse SAND. @ 4', As per 0', brown, medium dense.
5								Total Depth = 5' No Groundwater Encountered. No Caving. Gravel and Pipe Placed. Presoaked @ 11:34 AM. Backfilled 9-16-22.
10								
15								
20								
25								
30								

▼ *Standard Penetration Test*

⌶ *Undisturbed, Ring Sample*

▼ *Groundwater*

♀ *Seepage*

GeoSoils, Inc.

PLATE B-9

GeoSoils, Inc.

BORING LOG

PROJECT: ALABASSI - PERRIS COMMERCIAL

W.O. 8448-A-SC *BORING* P-2 *SHEET* 1 *OF* 1

DATE EXCAVATED 9-15-22 *LOGGED BY:* MAM *APPROX. ELEV.:* 1449'

SAMPLE METHOD: 8" HSA

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
0				SM				**TILLED TOPSOIL/COLLUVIUM:** @ 0', SILTY SAND, pale brown, dry, medium dense; fine to coarse SAND.
5								Total Depth = 5' No Groundwater Encountered. No Caving. Gravel and Pipe Placed. Presoaked @ 12:16 PM. Backfilled 9-16-22
10								
15								
20								
25								
30								

◪ *Standard Penetration Test*
⌶ *Undisturbed, Ring Sample*
▽ *Groundwater*
♀ *Seepage*

GeoSoils, Inc.

PLATE B-10

APPENDIX C

SEISMIC DATA

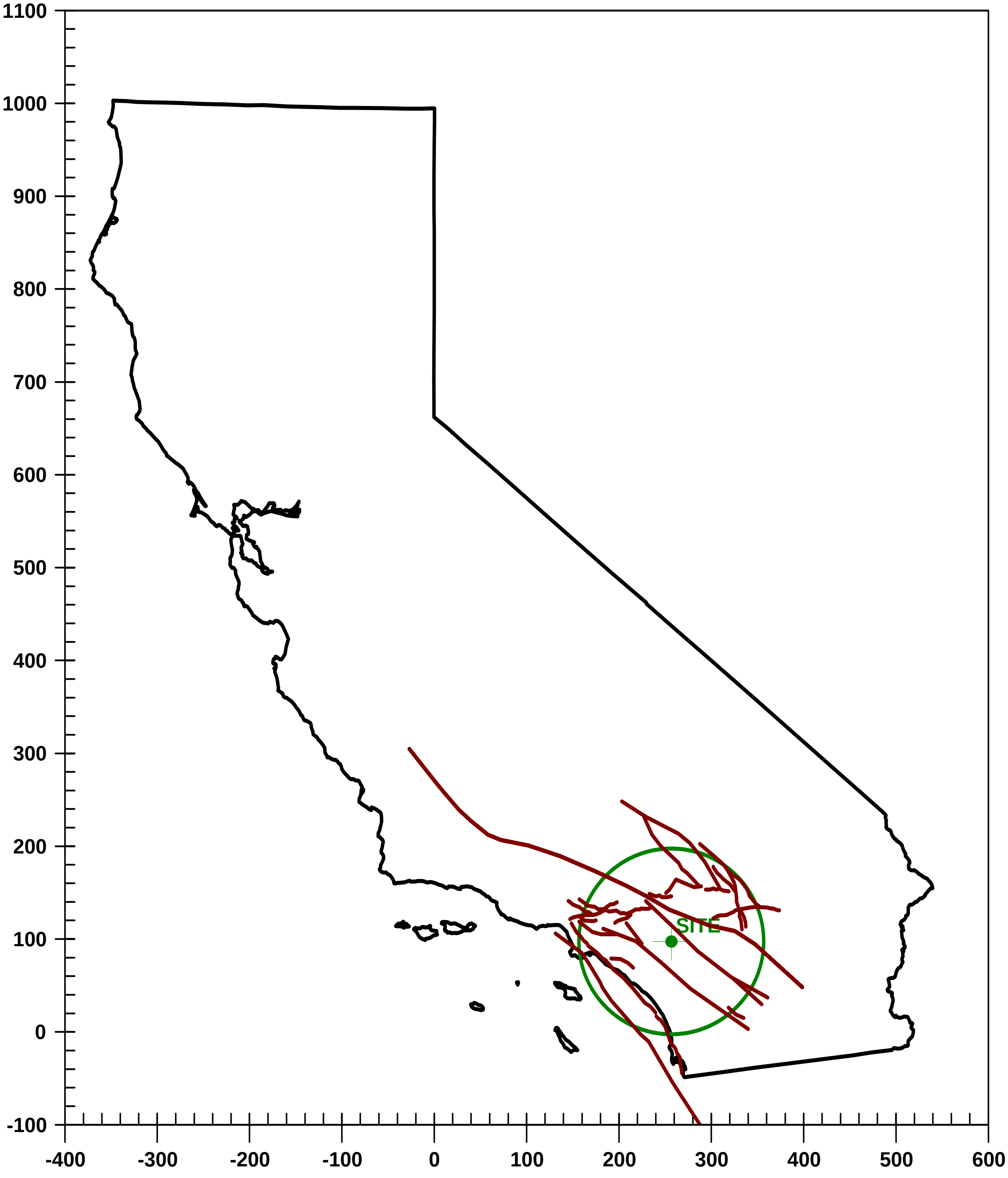
```
                    ***********************
                    *                     *
                    *    E Q F A U L T    *
                    *                     *
                    *    Version 3.00     *
                    *                     *
                    ***********************

                  DETERMINISTIC ESTIMATION OF
              PEAK ACCELERATION FROM DIGITIZED FAULTS


JOB NUMBER: 8448-A-SC
                                                     DATE: 09-30-2022

JOB NAME: Perris Commercial Site

CALCULATION NAME: Test Run Analysis

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


SITE COORDINATES:
   SITE LATITUDE:  33.8428
   SITE LONGITUDE:  117.2208

SEARCH RADIUS:   62.2  mi

ATTENUATION RELATION:  11) Bozorgnia Campbell Niazi (1999) Hor.-Pleist. Soil-Cor.
   UNCERTAINTY (M=Median, S=Sigma): S       Number of Sigmas:  1.0
   DISTANCE MEASURE:  cdist
   SCOND:   0
   Basement Depth:  5.00 km     Campbell SSR:  0     Campbell SHR:  0
   COMPUTE PEAK HORIZONTAL ACCELERATION

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


MINIMUM DEPTH VALUE (km):  3.0
























                                   ---------------
                                   EQFAULT SUMMARY
                                   ---------------



                            -----------------------------
                            DETERMINISTIC SITE PARAMETERS
                            -----------------------------
```

```
Page  1
-------------------------------------------------------------------------------
                                |              |ESTIMATED MAX. EARTHQUAKE EVENT
                                | APPROXIMATE  |-------------------------------
          ABBREVIATED           |   DISTANCE   | MAXIMUM  |   PEAK   |EST. SITE
          FAULT  NAME           |   mi   (km)  |EARTHQUAKE|   SITE   |INTENSITY
                                |              | MAG.(Mw) | ACCEL. g |MOD.MERC.
================================|==============|==========|==========|=========
SAN JACINTO-SAN J.VLY-CASA LOMA |   8.0(  12.8)|   6.9    |   0.420  |    X
SAN JACINTO-SAN BERNARDINO      |  12.2(  19.7)|   6.7    |   0.258  |   IX
ELSINORE (GLEN IVY)             |  15.3(  24.6)|   6.8    |   0.221  |   IX
ELSINORE (TEMECULA)             |  15.7(  25.3)|   6.8    |   0.215  |  VIII
SAN JACINTO-ANZA                |  18.9(  30.4)|   7.2    |   0.233  |   IX
SAN ANDREAS - SB-Coach. M-2b    |  19.4(  31.3)|   7.7    |   0.314  |   IX
SAN ANDREAS - Whole M-1a        |  19.4(  31.3)|   8.0    |   0.378  |   IX
SAN ANDREAS - San Bernardino M-1|  19.4(  31.3)|   7.5    |   0.276  |   IX
SAN ANDREAS - SB-Coach. M-1b-2  |  19.4(  31.3)|   7.7    |   0.314  |   IX
CHINO-CENTRAL AVE. (Elsinore)   |  20.8(  33.5)|   6.7    |   0.215  |  VIII
WHITTIER                        |  23.8(  38.3)|   6.8    |   0.142  |  VIII
NORTH FRONTAL FAULT ZONE (West) |  27.8(  44.7)|   7.2    |   0.224  |   IX
CUCAMONGA                       |  28.5(  45.8)|   6.9    |   0.178  |  VIII
CLEGHORN                        |  30.0(  48.2)|   6.5    |   0.092  |   VII
PINTO MOUNTAIN                  |  32.3(  52.0)|   7.2    |   0.136  |  VIII
SAN JOAQUIN HILLS               |  32.6(  52.4)|   6.6    |   0.128  |  VIII
SAN JOSE                        |  33.4(  53.7)|   6.4    |   0.108  |   VII
ELSINORE (JULIAN)               |  34.4(  55.3)|   7.1    |   0.119  |   VII
SAN ANDREAS - 1857 Rupture M-2a |  34.8(  56.0)|   7.8    |   0.193  |  VIII
SAN ANDREAS - Cho-Moj M-1b-1    |  34.8(  56.0)|   7.8    |   0.193  |  VIII
SAN ANDREAS - Mojave M-1c-3     |  34.8(  56.0)|   7.4    |   0.145  |  VIII
NORTH FRONTAL FAULT ZONE (East) |  36.0(  58.0)|   6.7    |   0.122  |   VII
SIERRA MADRE                    |  36.0(  58.0)|   7.2    |   0.172  |  VIII
PUENTE HILLS BLIND THRUST       |  37.7(  60.6)|   7.1    |   0.154  |  VIII
NEWPORT-INGLEWOOD (Offshore)    |  40.3(  64.8)|   7.1    |   0.101  |   VII
HELENDALE - S. LOCKHARDT        |  41.7(  67.1)|   7.3    |   0.112  |   VII
SAN ANDREAS - Coachella M-1c-5  |  43.5(  70.0)|   7.2    |   0.100  |   VII
NEWPORT-INGLEWOOD (L.A.Basin)   |  43.6(  70.2)|   7.1    |   0.093  |   VII
CLAMSHELL-SAWPIT                |  46.9(  75.5)|   6.5    |   0.081  |   VII
BURNT MTN.                      |  48.1(  77.4)|   6.5    |   0.056  |   VI
LENWOOD-LOCKHART-OLD WOMAN SPRGS|  48.8(  78.5)|   7.5    |   0.110  |   VII
SAN JACINTO-COYOTE CREEK        |  48.8(  78.6)|   6.6    |   0.059  |   VI
RAYMOND                         |  49.7(  80.0)|   6.5    |   0.076  |   VII
LANDERS                         |  50.7(  81.6)|   7.3    |   0.092  |   VII
ROSE CANYON                     |  51.1(  82.3)|   7.2    |   0.085  |   VII
EUREKA PEAK                     |  51.1(  82.3)|   6.4    |   0.049  |   VI
UPPER ELYSIAN PARK BLIND THRUST |  52.8(  85.0)|   6.4    |   0.067  |   VI
JOHNSON VALLEY (Northern)       |  54.6(  87.8)|   6.7    |   0.056  |   VI
PALOS VERDES                    |  55.1(  88.7)|   7.3    |   0.084  |   VII
VERDUGO                         |  56.4(  90.7)|   6.9    |   0.087  |   VII


                          -----------------------------
                          DETERMINISTIC SITE PARAMETERS
                          -----------------------------

Page  2
-------------------------------------------------------------------------------
                                |              |ESTIMATED MAX. EARTHQUAKE EVENT
                                | APPROXIMATE  |-------------------------------
          ABBREVIATED           |   DISTANCE   | MAXIMUM  |   PEAK   |EST. SITE
          FAULT  NAME           |   mi   (km)  |EARTHQUAKE|   SITE   |INTENSITY
                                |              | MAG.(Mw) | ACCEL. g |MOD.MERC.
================================|==============|==========|==========|=========
CORONADO BANK                   |  56.7(  91.3)|   7.6    |   0.101  |   VII
EARTHQUAKE VALLEY               |  58.6(  94.3)|   6.5    |   0.045  |   VI
EMERSON So. - COPPER MTN.       |  60.8(  97.9)|   7.0    |   0.061  |   VI
HOLLYWOOD                       |  61.3(  98.7)|   6.4    |   0.057  |   VI
*******************************************************************************
-END OF SEARCH-   44 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE SAN JACINTO-SAN J.VLY-CASA LOMA  FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 8.0 MILES (12.8 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.4203 g
```

W.O. 8448-A-SC
PLATE C-2

CALIFORNIA FAULT MAP

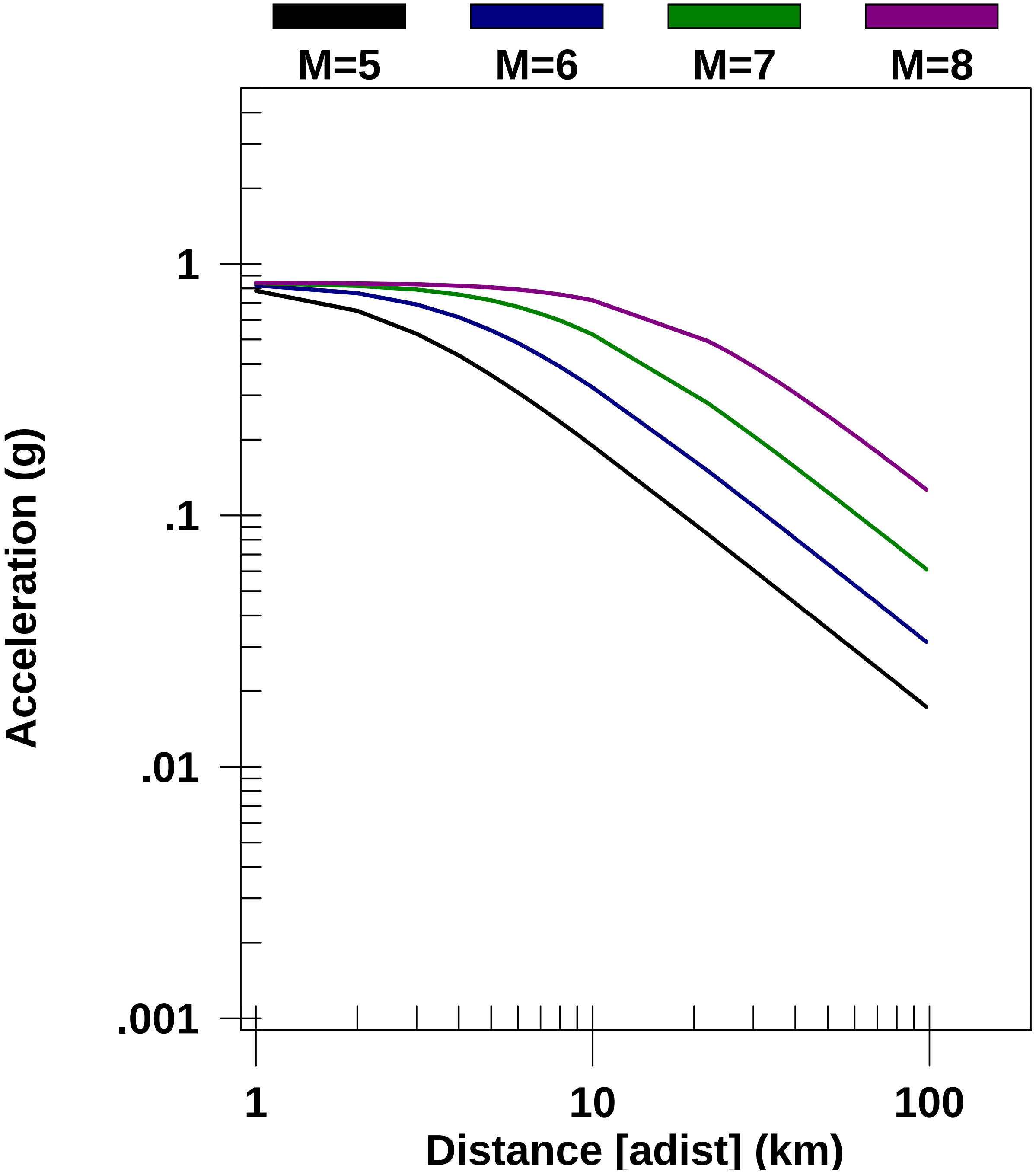
Perris Commercial Site



W.O. 8448-A-SC
PLATE C-3

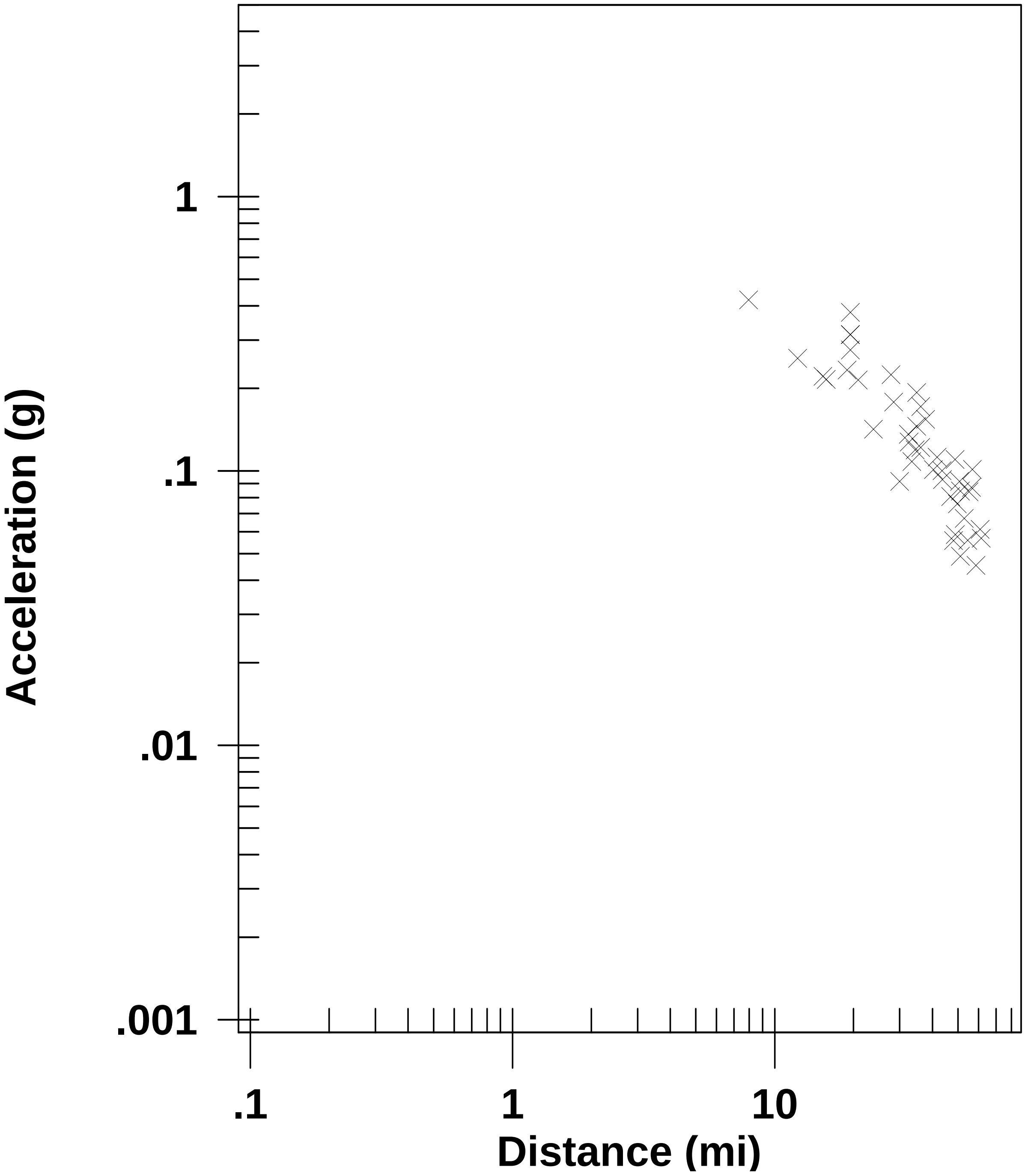
STRIKE-SLIP FAULTS

11) Bozorgnia Campbell Niazi (1999) Hor.-Pleist. Soil-Cor.



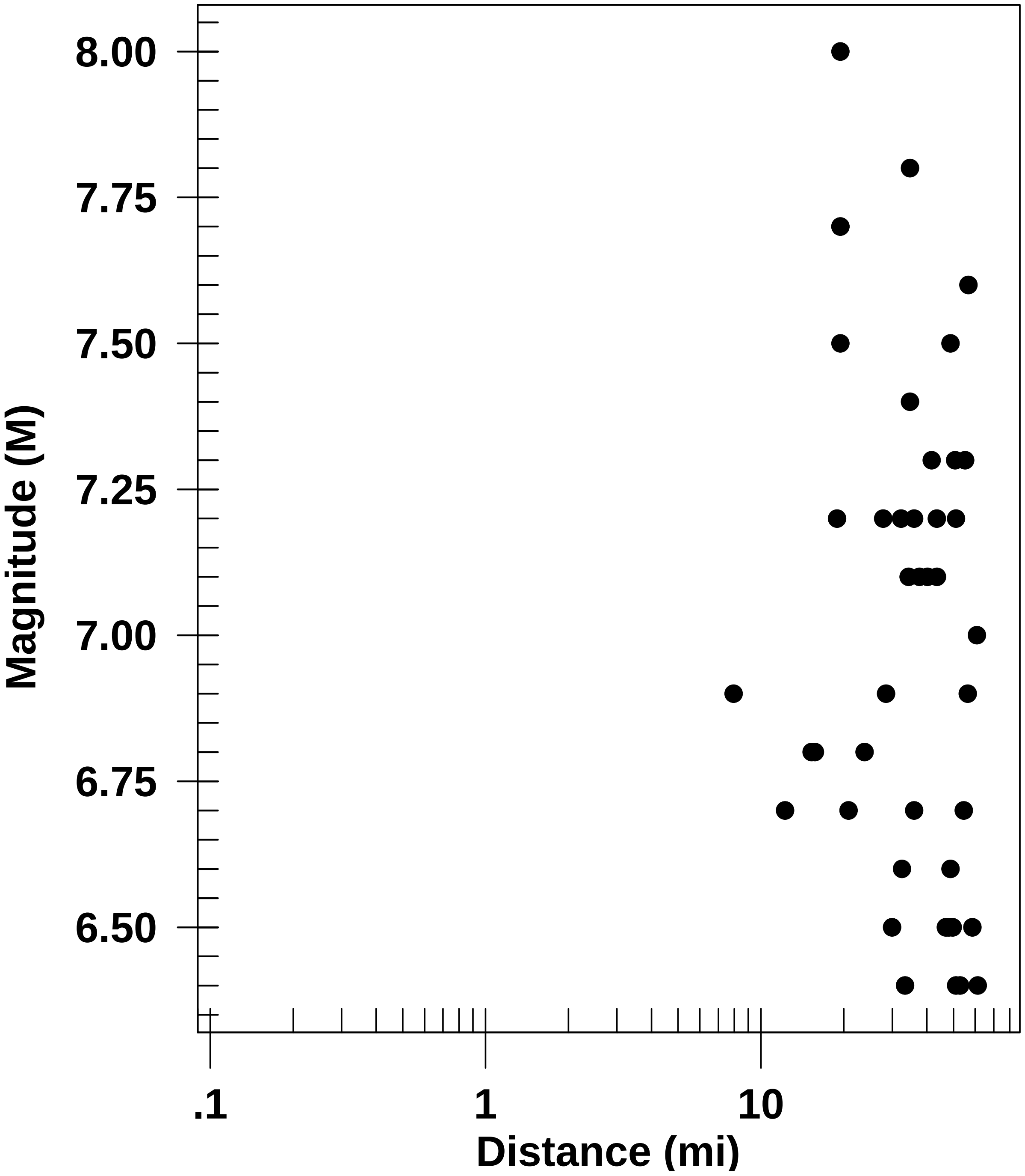
MAXIMUM EARTHQUAKES

Perris Commercial Site



EARTHQUAKE MAGNITUDES & DISTANCES

Perris Commercial Site



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

                                 ESTIMATION OF
                            PEAK ACCELERATION FROM
                        CALIFORNIA EARTHQUAKE CATALOGS


JOB NUMBER: 8448-A-SC
                                                     DATE: 09-30-2022

JOB NAME: Perris Commercial Site

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT


MAGNITUDE RANGE:
   MINIMUM MAGNITUDE:  5.00
   MAXIMUM MAGNITUDE:  9.00

SITE COORDINATES:
   SITE LATITUDE:  33.8428
   SITE LONGITUDE:  117.2208

SEARCH DATES:
           START DATE:   1800
           END DATE:   2021

SEARCH RADIUS:
           62.2 mi
           100.1 km

ATTENUATION RELATION:  11) Bozorgnia Campbell Niazi (1999) Hor.-Pleist. Soil-Cor.
   UNCERTAINTY (M=Median, S=Sigma): S       Number of Sigmas:  1.0
   ASSUMED SOURCE TYPE:  SS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
   SCOND:   0  Depth Source:  A
   Basement Depth:  5.00 km     Campbell SSR:  0     Campbell SHR:  0
   COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km):  3.0
```

```
                            -------------------------
                            EARTHQUAKE SEARCH RESULTS
                            -------------------------
```

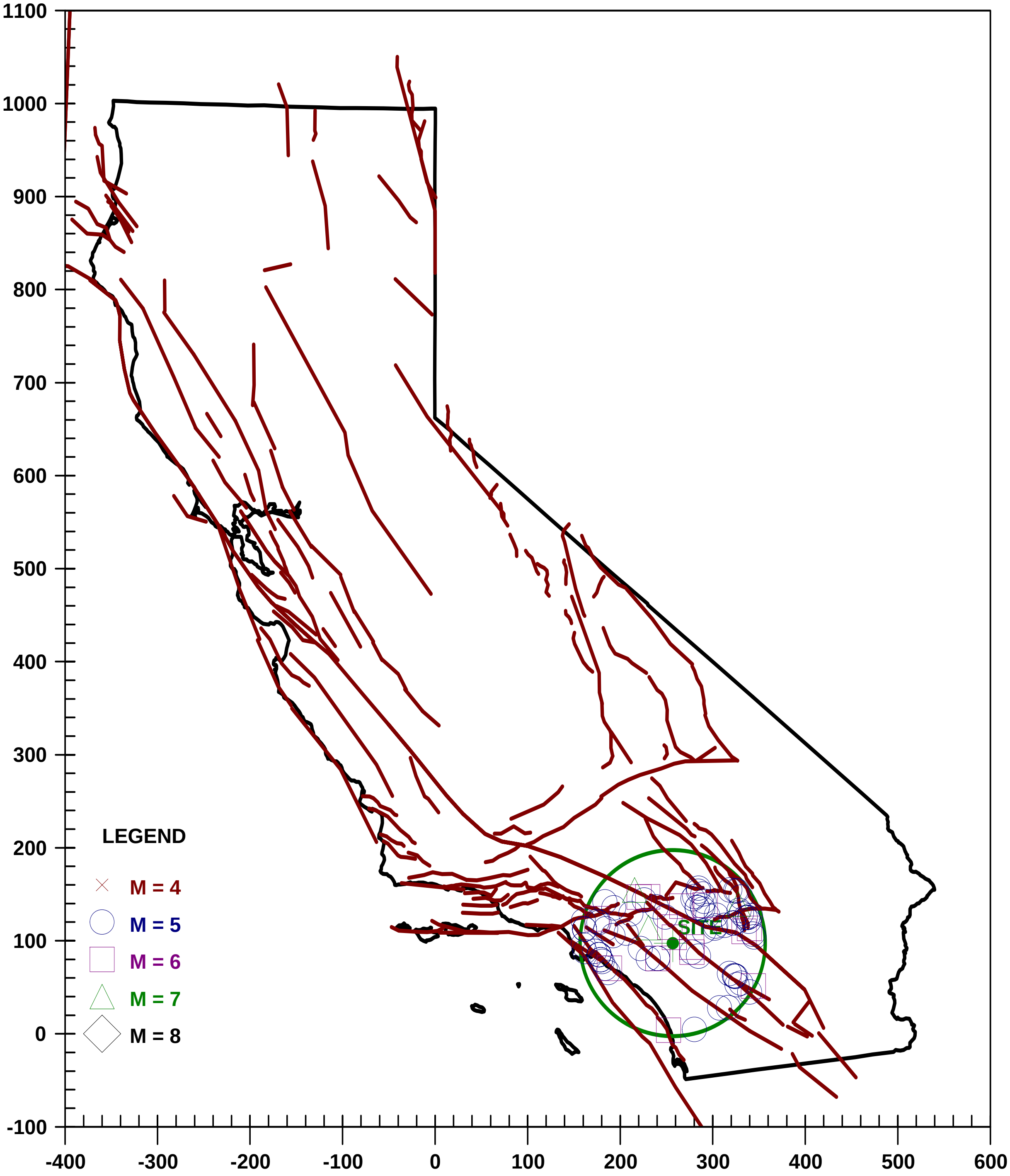
```
Page  1
-------------------------------------------------------------------------------
    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE| ACC.  | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
DMG |33.9000|117.2000|12/19/1880| 0 0 0.0|  0.0| 6.00| 0.411 |  X |   4.1(  6.6)
DMG |34.0000|117.2500|07/23/1923| 73026.0|  0.0| 6.25| 0.215 |VIII|  11.0( 17.7)
DMG |33.8000|117.0000|12/25/1899|1225 0.0|  0.0| 6.40| 0.200 |VIII|  13.0( 20.9)
DMG |33.7500|117.0000|04/21/1918|223225.0|  0.0| 6.80| 0.236 | IX |  14.2( 22.8)
DMG |33.7500|117.0000|06/06/1918|2232 0.0|  0.0| 5.00| 0.077 | VII|  14.2( 22.8)
DMG |33.7000|117.4000|05/15/1910|1547 0.0|  0.0| 6.00| 0.143 |VIII|  14.2( 22.9)
DMG |33.7000|117.4000|04/11/1910| 757 0.0|  0.0| 5.00| 0.077 | VII|  14.2( 22.9)
DMG |33.7000|117.4000|05/13/1910| 620 0.0|  0.0| 5.00| 0.077 | VII|  14.2( 22.9)
MGI |34.1000|117.3000|07/15/1905|2041 0.0|  0.0| 5.30| 0.071 | VI |  18.3( 29.5)
DMG |33.7100|116.9250|09/23/1963|144152.6| 16.5| 5.00| 0.057 | VI |  19.3( 31.0)
MGI |34.0000|117.5000|12/16/1858|10 0 0.0|  0.0| 7.00| 0.199 |VIII|  19.3( 31.1)
DMG |33.6990|117.5110|05/31/1938| 83455.4| 10.0| 5.50| 0.076 | VII|  19.4( 31.2)
MGI |33.8000|117.6000|04/22/1918|2115 0.0|  0.0| 5.00| 0.050 | VI |  21.9( 35.3)
DMG |33.9500|116.8500|09/28/1946| 719 9.0|  0.0| 5.00| 0.049 | VI |  22.5( 36.2)
DMG |34.2000|117.1000|09/20/1907| 154 0.0|  0.0| 6.00| 0.078 | VII|  25.6( 41.2)
DMG |34.2000|117.4000|07/22/1899| 046 0.0|  0.0| 5.50| 0.055 | VI |  26.7( 43.0)
DMG |34.1800|116.9200|01/16/1930| 034 3.6|  0.0| 5.10| 0.040 |  V |  29.0( 46.6)
DMG |34.1800|116.9200|01/16/1930| 02433.9|  0.0| 5.20| 0.042 | VI |  29.0( 46.6)
DMG |34.1000|116.8000|10/24/1935|1448 7.6|  0.0| 5.10| 0.038 |  V |  29.9( 48.2)
DMG |33.9760|116.7210|06/12/1944|104534.7| 10.0| 5.10| 0.038 |  V |  30.1( 48.4)
GSP |34.1630|116.8550|06/28/1992|144321.0|  6.0| 5.30| 0.042 | VI |  30.4( 49.0)
DMG |33.9940|116.7120|06/12/1944|111636.0| 10.0| 5.30| 0.042 | VI |  31.0( 49.8)
GSP |34.1950|116.8620|08/17/1992|204152.1| 11.0| 5.30| 0.041 |  V |  31.8( 51.2)
GSG |33.9530|117.7610|07/29/2008|184215.7| 14.0| 5.30| 0.040 |  V |  31.9( 51.3)
DMG |34.2670|116.9670|08/29/1943| 34513.0|  0.0| 5.50| 0.044 | VI |  32.7( 52.6)
GSN |34.2030|116.8270|06/28/1992|150530.7|  5.0| 6.70| 0.093 | VII|  33.6( 54.0)
GSP |34.1400|117.7000|02/28/1990|234336.6|  5.0| 5.20| 0.035 |  V |  34.3( 55.1)
GSP |34.2900|116.9460|02/10/2001|210505.8|  9.0| 5.10| 0.033 |  V |  34.6( 55.7)
DMG |34.2700|117.5400|09/12/1970|143053.0|  8.0| 5.40| 0.039 |  V |  34.7( 55.8)
DMG |34.1000|116.7000|02/07/1889| 520 0.0|  0.0| 5.30| 0.037 |  V |  34.7( 55.8)
GSP |34.2390|116.8370|07/09/1992|014357.6|  0.0| 5.30| 0.037 |  V |  35.1( 56.4)
DMG |34.3000|117.5000|07/22/1899|2032 0.0|  0.0| 6.50| 0.077 | VII|  35.4( 56.9)
PAS |33.9980|116.6060|07/08/1986| 92044.5| 11.7| 5.60| 0.042 | VI |  36.8( 59.2)
DMG |34.3000|117.6000|07/30/1894| 512 0.0|  0.0| 6.00| 0.051 | VI |  38.3( 61.6)
GSG |34.3100|116.8480|02/22/2003|121910.6|  1.0| 5.20| 0.031 |  V |  38.7( 62.2)
GSP |34.3400|116.9000|11/27/1992|160057.5|  1.0| 5.30| 0.033 |  V |  38.9( 62.6)
GSP |33.9325|117.9158|03/29/2014|040942.2|  5.1| 5.10| 0.028 |  V |  40.3( 64.9)
GSP |34.3690|116.8970|12/04/1992|020857.5|  3.0| 5.30| 0.031 |  V |  40.8( 65.6)
DMG |34.0170|116.5000|07/24/1947|221046.0|  0.0| 5.50| 0.033 |  V |  43.0( 69.2)
DMG |34.0170|116.5000|07/26/1947| 24941.0|  0.0| 5.10| 0.026 |  V |  43.0( 69.2)
DMG |34.0170|116.5000|07/25/1947| 61949.0|  0.0| 5.20| 0.028 |  V |  43.0( 69.2)
DMG |34.0170|116.5000|07/25/1947| 04631.0|  0.0| 5.00| 0.025 |  V |  43.0( 69.2)
GSP |33.5290|116.5720|06/12/2005|154146.5| 14.0| 5.20| 0.028 |  V |  43.1( 69.4)
DMG |34.3700|117.6500|12/08/1812|15 0 0.0|  0.0| 7.00| 0.086 | VII|  43.9( 70.6)
DMG |33.6170|117.9670|03/11/1933| 154 7.8|  0.0| 6.30| 0.052 | VI |  45.6( 73.4)
MGI |34.0000|118.0000|12/25/1903|1745 0.0|  0.0| 5.00| 0.023 | IV |  45.9( 73.9)
DMG |34.2000|117.9000|08/28/1889| 215 0.0|  0.0| 5.50| 0.031 |  V |  46.0( 74.1)
GSP |33.5080|116.5140|10/31/2001|075616.6| 15.0| 5.10| 0.024 |  V |  46.7( 75.2)
PAS |33.5010|116.5130|02/25/1980|104738.5| 13.6| 5.50| 0.030 |  V |  47.0( 75.7)
DMG |33.5750|117.9830|03/11/1933| 518 4.0|  0.0| 5.20| 0.025 |  V |  47.5( 76.5)
DMG |33.5000|116.5000|09/30/1916| 211 0.0|  0.0| 5.00| 0.022 | IV |  47.7( 76.8)
DMG |33.6170|118.0170|03/14/1933|19 150.0|  0.0| 5.10| 0.023 | IV |  48.3( 77.7)
DMG |33.9330|116.3830|12/04/1948|234317.0|  0.0| 6.50| 0.055 | VI |  48.4( 77.9)



                               -------------------------
                               EARTHQUAKE SEARCH RESULTS
                               -------------------------

Page  2
-------------------------------------------------------------------------------
    |       |        |          |  TIME  |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC) |DEPTH|QUAKE| ACC.  | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+--------+-----+-----+-------+----+------------
DMG |33.6830|118.0500|03/11/1933| 658 3.0|  0.0| 5.50| 0.029 |  V |  48.9( 78.6)
DMG |33.7000|118.0670|03/11/1933| 51022.0|  0.0| 5.10| 0.023 | IV |  49.6( 79.7)
```

```
DMG |33.7000|118.0670|03/11/1933| 85457.0|  0.0| 5.10| 0.023 | IV | 49.6( 79.7)
GSP |34.1390|116.4310|06/28/1992|123640.6| 10.0| 5.10| 0.023 | IV | 49.6( 79.9)
DMG |33.7500|118.0830|03/11/1933| 230 0.0|  0.0| 5.10| 0.023 | IV | 49.9( 80.3)
DMG |33.7500|118.0830|03/11/1933| 910 0.0|  0.0| 5.10| 0.023 | IV | 49.9( 80.3)
DMG |33.7500|118.0830|03/11/1933| 2 9 0.0|  0.0| 5.00| 0.021 | IV | 49.9( 80.3)
DMG |33.7500|118.0830|03/13/1933|131828.0|  0.0| 5.30| 0.025 |  V | 49.9( 80.3)
DMG |33.7500|118.0830|03/11/1933| 323 0.0|  0.0| 5.00| 0.021 | IV | 49.9( 80.3)
GSP |34.1080|116.4040|06/29/1992|141338.8|  9.0| 5.40| 0.027 |  V | 50.2( 80.8)
GSG |33.4200|116.4890|07/07/2010|235333.5| 14.0| 5.50| 0.028 |  V | 51.2( 82.4)
GSN |34.2010|116.4360|06/28/1992|115734.1|  1.0| 7.60| 0.112 | VII| 51.3( 82.5)
PAS |34.0610|118.0790|10/01/1987|144220.0|  9.5| 5.90| 0.035 |  V | 51.4( 82.7)
GSP |34.0640|116.3610|09/15/1992|084711.3|  9.0| 5.20| 0.023 | IV | 51.6( 83.0)
GSP |33.9610|116.3180|04/23/1992|045023.0| 12.0| 6.10| 0.039 |  V | 52.4( 84.3)
GSP |34.3410|116.5290|06/28/1992|124053.5|  6.0| 5.20| 0.023 | IV | 52.4( 84.4)
DMG |33.7830|118.1330|10/02/1933| 91017.6|  0.0| 5.40| 0.025 |  V | 52.5( 84.5)
PAS |34.0730|118.0980|10/04/1987|105938.2|  8.2| 5.30| 0.024 |  V | 52.7( 84.8)
GSP |33.4315|116.4427|06/10/2016|080438.7| 12.3| 5.19| 0.022 | IV | 53.0( 85.3)
GSP |34.0290|116.3210|08/21/1993|014638.4|  9.0| 5.00| 0.020 | IV | 53.1( 85.5)
DMG |34.0670|116.3330|05/18/1940| 55120.2|  0.0| 5.20| 0.022 | IV | 53.1( 85.5)
DMG |34.0670|116.3330|05/18/1940| 72132.7|  0.0| 5.00| 0.020 | IV | 53.1( 85.5)
GSP |34.2620|118.0020|06/28/1991|144354.5| 11.0| 5.40| 0.025 |  V | 53.2( 85.7)
MGI |34.1000|118.1000|07/11/1855| 415 0.0|  0.0| 6.30| 0.044 | VI | 53.4( 85.9)
DMG |33.2000|116.7000|01/01/1920| 235 0.0|  0.0| 5.00| 0.020 | IV | 53.6( 86.2)
GSP |33.9020|116.2840|07/24/1992|181436.2|  9.0| 5.00| 0.020 | IV | 53.9( 86.7)
GSP |33.8760|116.2670|06/29/1992|160142.8|  1.0| 5.20| 0.022 | IV | 54.7( 88.1)
GSP |34.3320|116.4620|07/01/1992|074029.9|  9.0| 5.40| 0.024 |  V | 55.0( 88.5)
DMG |34.0830|116.3000|05/18/1940| 5 358.5|  0.0| 5.40| 0.024 |  V | 55.3( 88.9)
GSP |34.2680|116.4020|06/16/1994|162427.5|  3.0| 5.00| 0.019 | IV | 55.3( 89.0)
PAS |34.3270|116.4450|03/15/1979|21 716.5|  2.5| 5.20| 0.021 | IV | 55.5( 89.4)
MGI |33.2000|116.6000|10/12/1920|1748 0.0|  0.0| 5.30| 0.022 | IV | 57.0( 91.7)
DMG |33.0000|117.3000|11/22/1800|2130 0.0|  0.0| 6.50| 0.045 | VI | 58.4( 93.9)
DMG |33.7830|118.2500|11/14/1941| 84136.3|  0.0| 5.40| 0.022 | IV | 59.2( 95.2)
MGI |33.0000|117.0000|09/21/1856| 730 0.0|  0.0| 5.00| 0.018 | IV | 59.6( 95.9)
T-A |34.0000|118.2500|01/10/1856| 0 0 0.0|  0.0| 5.00| 0.018 | IV | 60.0( 96.5)
T-A |34.0000|118.2500|03/26/1860| 0 0 0.0|  0.0| 5.00| 0.018 | IV | 60.0( 96.5)
T-A |34.0000|118.2500|09/23/1827| 0 0 0.0|  0.0| 5.00| 0.018 | IV | 60.0( 96.5)
DMG |33.8500|118.2670|03/11/1933|1425 0.0|  0.0| 5.00| 0.018 | IV | 60.0( 96.5)
DMG |33.3430|116.3460|04/28/1969|232042.9| 20.0| 5.80| 0.028 |  V | 61.0( 98.2)
DMG |33.4000|116.3000|02/09/1890|12 6 0.0|  0.0| 6.30| 0.038 |  V | 61.1( 98.4)
MGI |34.0800|118.2600|07/16/1920|18 8 0.0|  0.0| 5.00| 0.017 | IV | 61.7( 99.3)

********************************************************************************








-END OF SEARCH-   95 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH:   1800  TO  2021

LENGTH OF SEARCH TIME:   222  years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 4.1 MILES (6.6 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.6

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.411 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
  a-value=  1.344
  b-value=  0.392
  beta-value=  0.903


------------------------------------
TABLE OF MAGNITUDES AND EXCEEDANCES:
------------------------------------

  Earthquake | Number of Times | Cumulative
   Magnitude |    Exceeded     | No. / Year
  -----------+-----------------+------------
```

```
  4.0      |        95         |   0.42793
  4.5      |        95         |   0.42793
  5.0      |        95         |   0.42793
  5.5      |        29         |   0.13063
  6.0      |        18         |   0.08108
  6.5      |         8         |   0.03604
  7.0      |         3         |   0.01351
  7.5      |         1         |   0.00450
```

EARTHQUAKE EPICENTER MAP

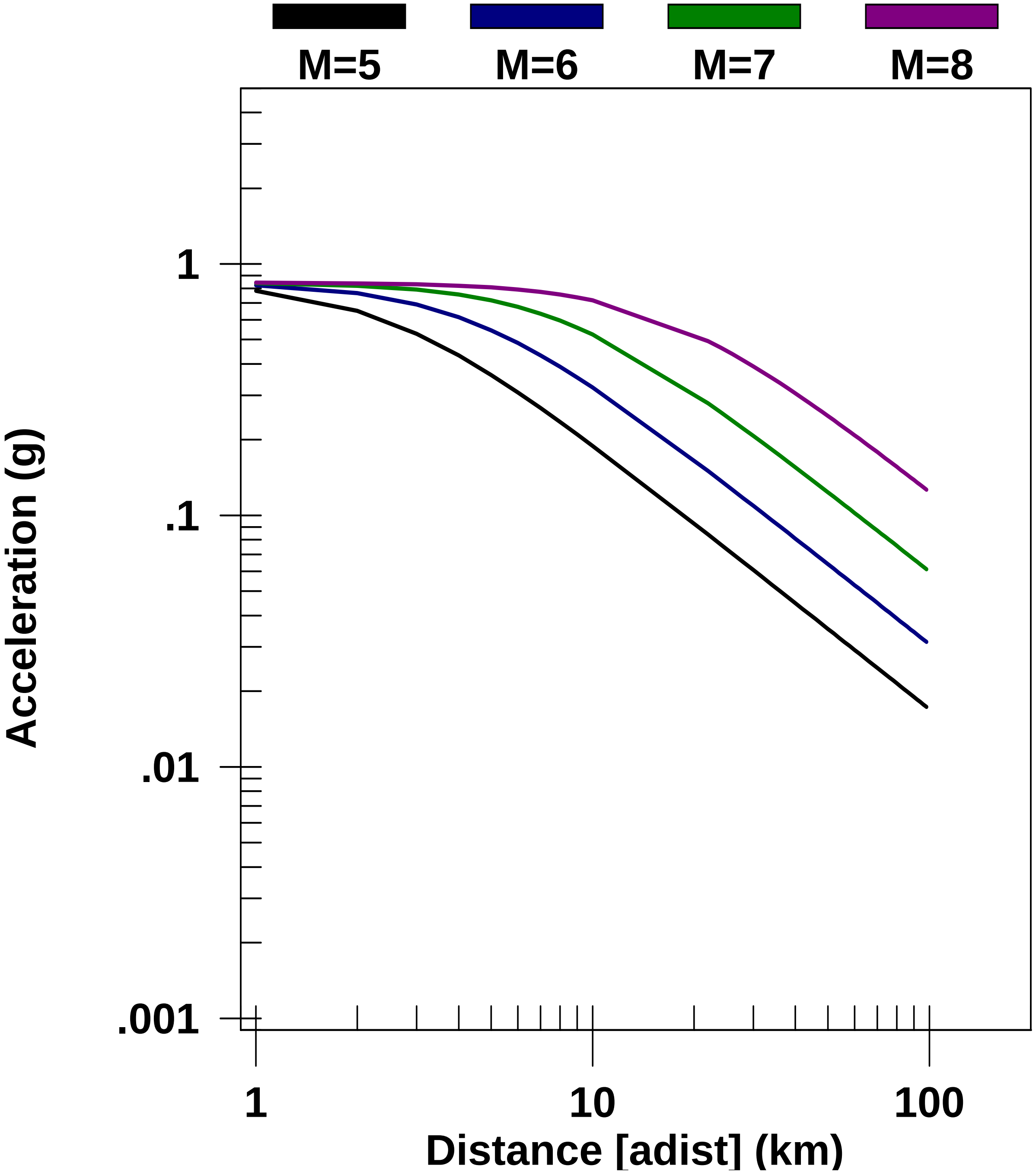
Perris Commercial Site



W.O. 8448-A-SC
PLATE C-11

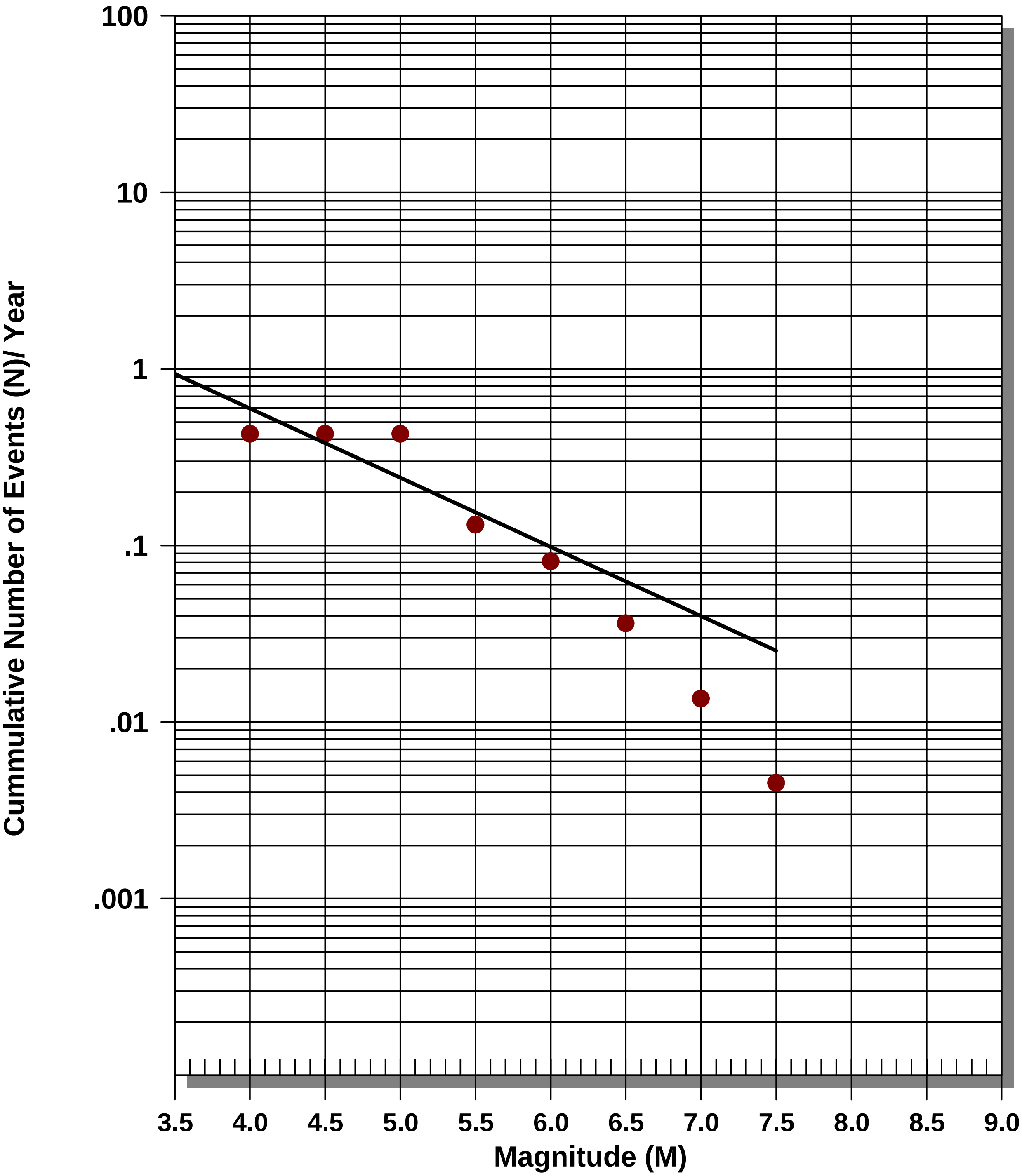
STRIKE-SLIP FAULTS

11) Bozorgnia Campbell Niazi (1999) Hor.-Pleist. Soil-Cor.



EARTHQUAKE RECURRENCE CURVE

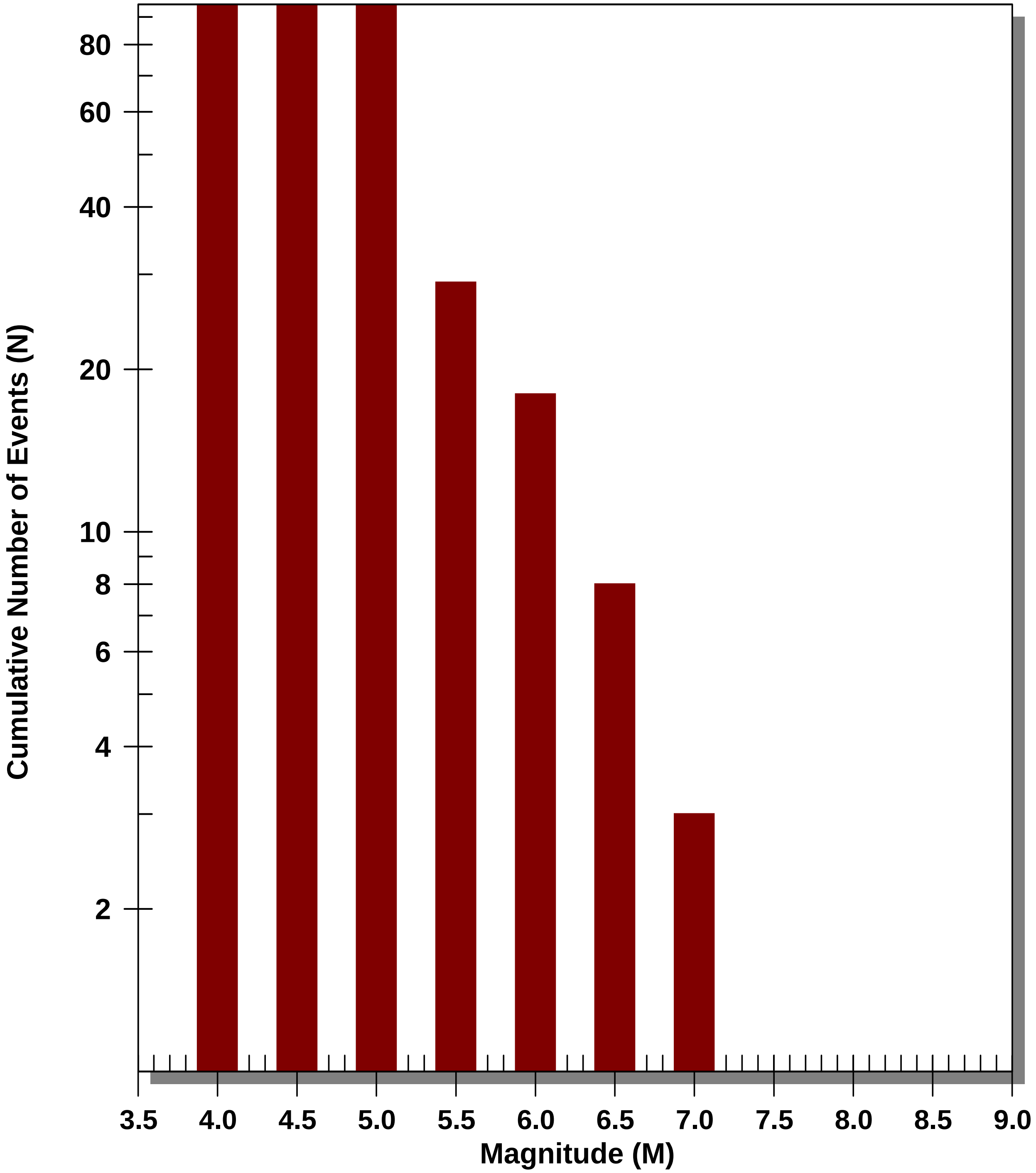
Perris Commercial Site



W.O. 8448-A-SC
PLATE C-13

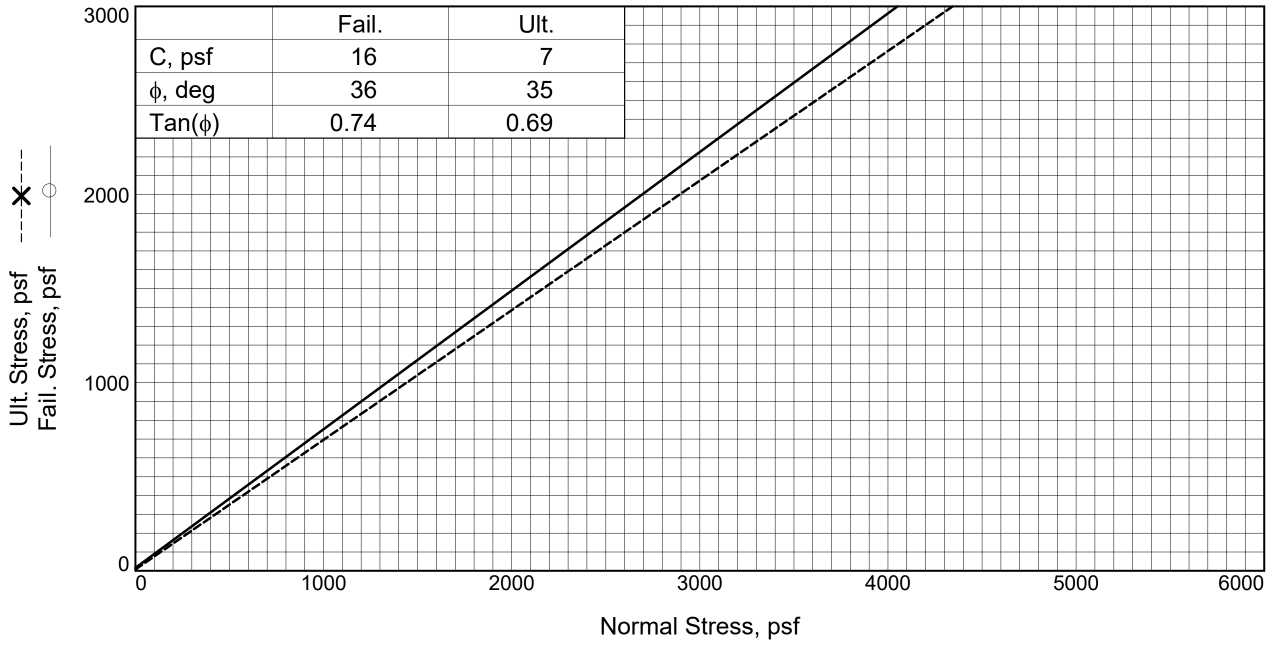
Number of Earthquakes (N) Above Magnitude (M)

Perris Commercial Site

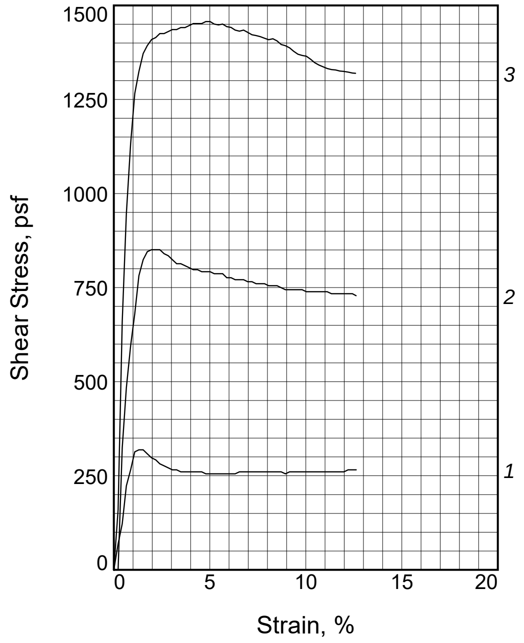


APPENDIX D

LABORATORY TEST RESULTS



	Fail.	Ult.
C, psf	16	7
ϕ, deg	36	35
Tan(ϕ)	0.74	0.69



	Sample No.	1	2	3
Initial	Water Content, %	9.0	9.0	9.0
	Dry Density, pcf	117.4	117.3	117.1
	Saturation, %	58.3	58.1	57.7
	Void Ratio	0.4092	0.4102	0.4130
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	15.0	14.7	14.9
	Dry Density, pcf	117.6	118.3	118.5
	Saturation, %	97.9	98.0	99.5
	Void Ratio	0.4064	0.3989	0.3961
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	0.99	0.99
	Normal Stress, psf	500	1000	2000
	Fail. Stress, psf	319	851	1457
	Strain, %	1.3	2.0	5.0
	Ult. Stress, psf	255	739	1332
	Strain, %	4.8	10.0	11.1
	Strain rate, in./min.	0.004	0.004	0.004

Sample Type: Remolded

Description: Yellowish Brown Silty Sand

Specific Gravity= 2.65

Remarks:

Plate ________

Client: Alabassi

Project: Commercial - Perris

Source of Sample: B-2 **Depth:** 0-5

Sample Number: B-2

Proj. No.: 8448-A-SC **Date Sampled:**

GeoSoils, Inc.

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

W.O. 8448-A-SC
PLATE D-1

TEST SPECIMEN

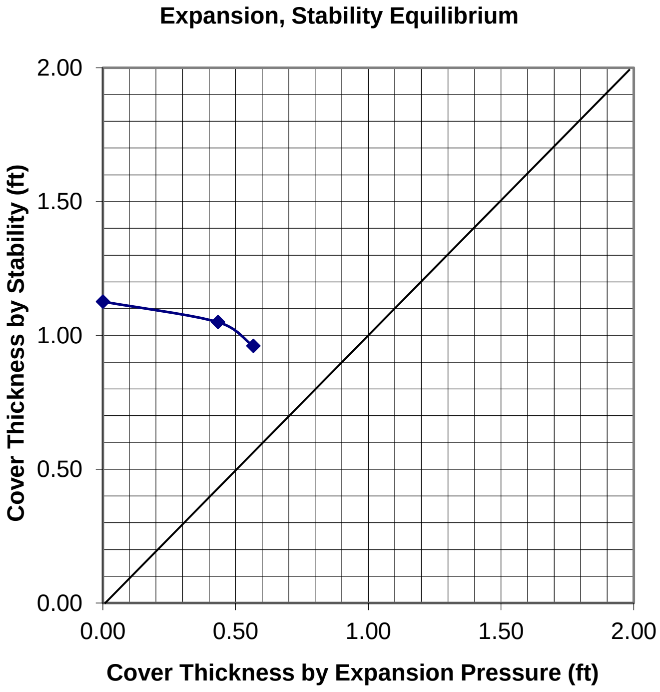
		A	B	C	D
Compactor air pressure	**PSI**	210	180	85	
Water added	**%**	3.5	4.4	6.3	
Moisture at compaction	**%**	12.2	13.1	15.0	
Height of sample	**IN**	2.54	2.6	2.65	
Dry density	**PCF**	124.1	121.0	117.0	
R-Value by exudation		25	18	12	
R-Value by exudation, corrected		25	19	12	
Exudation pressure	**PSI**	418	305	132	
Stability thickness	**FT**	0.96	1.05	1.13	
Expansion pressure thickness	**FT**	0.57	0.43	0.00	

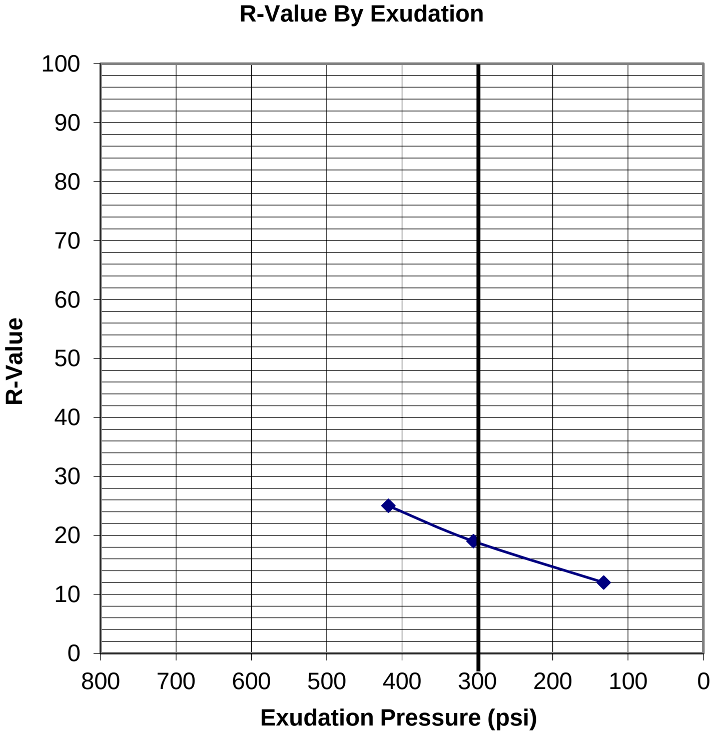
DESIGN CALCULATION DATA

Traffic index, assumed	5.0
Gravel equivalent factor, assumed	1.25
Expansion, stability equilibrium	0
R-Value by expansion	NA
R-Value by exudation	19
R-Value at equilibrium	**19**

SAMPLE INFORMATION

Sample Location: B-2, 0-5ft
Sample Description: Yellowish Brown Silty Sand
Notes: -
0% Retained on 3/4 inch sieve
Test Method: Cal-Trans Test 301





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

9/2/2010

R - VALUE TEST RESULTS

Project: Alabassi

Number: 8448-A-SC

Date: September 2022

W.O. 8448-A-SC
PLATE D-2

GeoSoils, Inc.

5741 Palmer Way, Carlsbad CA 92010
Phone (760) 438-3155

CORROSION REPORT SUMMARY

Project No: **8448-A-SC**
Project Name: **Alabassi**
Report Date: **October 4, 2022**

SAMPLE ID	pH (H+)	Minimum Resistivity (ohm/cm)	Sulfate Content (wt%)	Chloride Content (mg/kg)
B-2, 0-5ft	**8.0**	**3500**	**<0.003**	**11**

Samples testing in accordance with: pH - CTM 643, Resistivity - CTM 643
Sulfate - CTM 417, Chloride - CTM 422

Remarks: ____

APPENDIX E

FIELD PERCOLATION DATA SHEETS

Leach Line Percolation Data Sheet

Project: Abbassi – Perris Commercial	W.O. Number: 8448-A
Test Hole No.: P-1	Date Excavated: 9/15/2022
Depth of Test Hole: 5' 7"	Soil Classification: SM
Check for Sandy Soil Criteria Tested by: Madison M.	Date: 9/15/2022 Presoak: yes
Actual Percolation Tested by: Madison M.	Date: 9/16/2022

Sandy Soil Criteria Test

Trial No.	Time	Time Interval (Min.)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ in Water Level (Inches)
1	8:55 9:20	25	28.5	18.25	10.25
2	9:23 9:48	25	27.5	18.75	8.75

Use: Normal/ [✓ Sandy] (Circle One) Soil Criteria

	Time	Time Interval (min)	Total Elapsed Time (Min.)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ in Water Level (Inches)	Percolation Rate (min/inch)
First Hour	9:51 10:01	10	10	28.0	25.0	3.0	3.3$\overline{3}$
	10:02 10:12	10	20	28.0	25	3.0	3.3$\overline{3}$
Second Hour	10:13 10:23	10	30	28	25.25	2.75	3.64
	10:25 10:35	10	40	28.25	26.5	1.75	5.71
Third Hour	10:37 10:47	10	50	28	26	2.0	5.0
	10:47 10:57	10	60	28	26.5	1.5	6.67
Fourth Hour							
Fifth Hour							
Sixth Hour							

Plate No. E-1

Leach Line Percolation Data Sheet

Project: Alabassi - Perris - Commercial	W.O. Number: 8448-A-SC
Test Hole No.: P-2	Date Excavated: 9/15/2022
Depth of Test Hole: 5'2"	Soil Classification: SM
Check for Sandy Soil Criteria Tested by:	Date: 9/15/2022 Presoak: yes
Actual Percolation Tested by: Madison M.	Date: 9/16/2022

Sandy Soil Criteria Test

Trial No.	Time	Time Interval (Min.)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ in Water Level (Inches)
1	8:18 8:43	25	24.5	22.5	5.75
2	8:43 9:08	25	24.5	19.5	5.0

Use: [✓ Normal]/Sandy (Circle One) Soil Criteria

	Time	Time Interval (min)	Total Elapsed Time (Min.)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ in Water Level (Inches)	Percolation Rate (min/inch)
First Hour	9:11 9:41	30	30	25	20	5.0	6.0
	9:45 10:15	30	60	25	20	5.0	6.0
Second Hour	10:18 10:48	30	90	25	20.25	4.75	6.32
	10:51 11:21	30	120	24.5	19.75	4.75	6.32
Third Hour	11:23 11:53	30	150	24.75	20	4.75	6.32
	11:55 12:25	30	180	24.5	19.75	4.75	6.32
Fourth Hour	12:27 12:57	30	210	24.75	20	4.75	6.32
	12:59 1:29	30	240	25	20.25	4.75	6.32
Fifth Hour	1:31 2:01	30	270	25	20.5	4.5	6.67
	2:03 2:33	30	300	24.5	20	4.5	6.67
Sixth Hour	2:35 3:05	30	330	24.75	20.25	4.5	6.67
	3:07 3:37	30	360	24.5	20.25	4.25	7.06

APPENDIX F

GENERAL EARTHWORK AND GRADING GUIDELINES

GENERAL EARTHWORK AND GRADING GUIDELINES

General

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

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

EARTHWORK OBSERVATIONS AND TESTING

Geotechnical Consultant

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

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

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

Laboratory and Field Tests

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

Contractor's Responsibility

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

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

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

SITE PREPARATION

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

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

Existing ground, which is determined to be satisfactory for support of the fills, should be scarified (ripped) to a minimum depth of 6 to 8 inches, or as directed by the geotechnical consultant. After the scarified ground is brought to optimum moisture content, or greater and mixed, the materials should be compacted as specified herein. If the scarified zone is greater than 6 to 8 inches in depth, it may be necessary to remove the excess and place the material in lifts restricted to about 6 to 8 inches in compacted thickness. Scarification, disc harrowing, or other acceptable forms of mixing should continue until the soils are broken down and free of large lumps or clods, until the working surface is reasonably uniform and free from ruts, hollows, hummocks, mounds, or other uneven features, which would inhibit compaction as described previously.

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

These materials should be free of roots, tree branches, other organic matter, or other deleterious materials. All unsuitable materials should be removed from the fill as directed by the geotechnical consultant. Soils of poor gradation, undesirable expansion potential, or substandard strength characteristics may be designated by the consultant as unsuitable and may require blending with other soils to serve as a satisfactory fill material.

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

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

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

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

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

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

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

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

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

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

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, after 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. After 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, or remedial grading of cut slopes should be performed. When fill-over-cut slopes are to be graded, unless otherwise approved, the cut portion of the slope should be observed by the geotechnical consultant prior to placement of materials for construction of the fill portion of the slope. The geotechnical consultant should observe all cut slopes, and should be notified by the contractor when excavation of cut slopes commence.

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

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

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

COMPLETION

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

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

JOB SAFETY

General

At GSI, getting the job done safely is of primary concern. The following is the company's safety considerations for use by all employees on multi-employer construction sites. On-ground personnel are at highest risk of injury, and possible fatality, on grading and construction projects. GSI recognizes that construction activities will vary on each site, and that site safety is the <u>prime</u> responsibility of the contractor; however, everyone must be safety conscious and responsible at all times. To achieve our goal of avoiding accidents, cooperation between the client, the contractor, and GSI personnel must be maintained. In an effort to minimize risks associated with geotechnical testing and observation, the following precautions are to be implemented for the safety of field personnel on grading and construction projects:

Safety Meetings:	GSI field personnel are directed to attend contractor’s regularly scheduled and documented safety meetings.
Safety Vests:	Safety vests are provided for, and are to be worn by GSI personnel, at all times, when they are working in the field.
Safety Flags:	Two safety flags are provided to GSI field technicians; one is to be affixed to the vehicle when on site, the other is to be placed atop the spoil pile on all test pits.
Flashing Lights:	All vehicles stationary in the grading area shall use rotating or flashing amber beacons, or strobe lights, on the vehicle during all field testing. While operating a vehicle in the grading area, the emergency flasher on the vehicle shall be activated.

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

Test Pits Location, Orientation, and Clearance

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

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

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

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

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

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

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

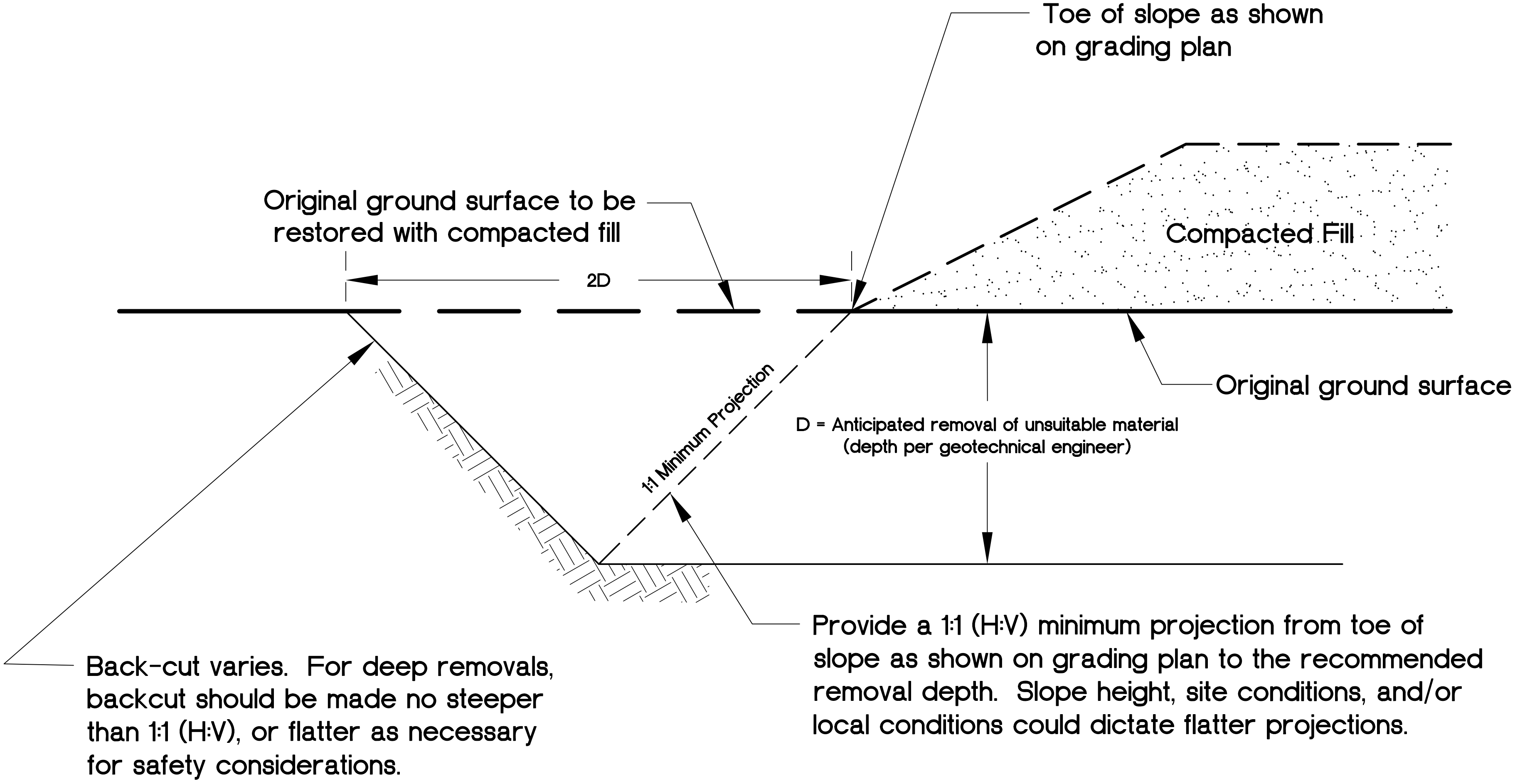
Trench and Vertical Excavation

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

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

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

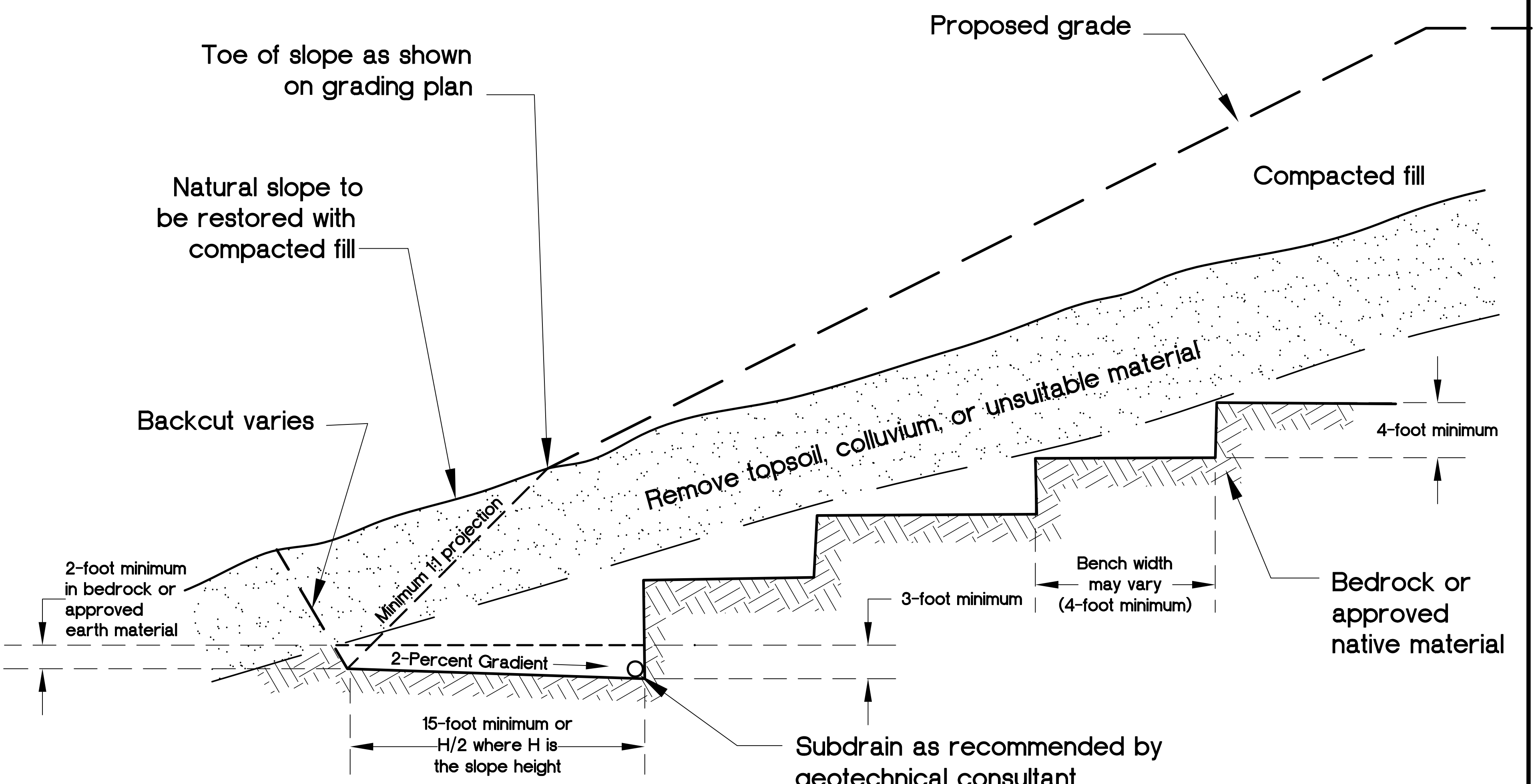
If GSI personnel become aware of anyone working beneath an unsafe trench wall or vertical excavation, we have a legal obligation to put the contractor and owner/developer on notice to immediately correct the situation. If corrective steps are not taken, GSI then has an obligation to notify Cal/OSHA or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



GeoSoils, Inc.

FILL SLOPE TOEING OUT ON FLAT ALLUVIATED CANYON DETAIL

Plate F-3



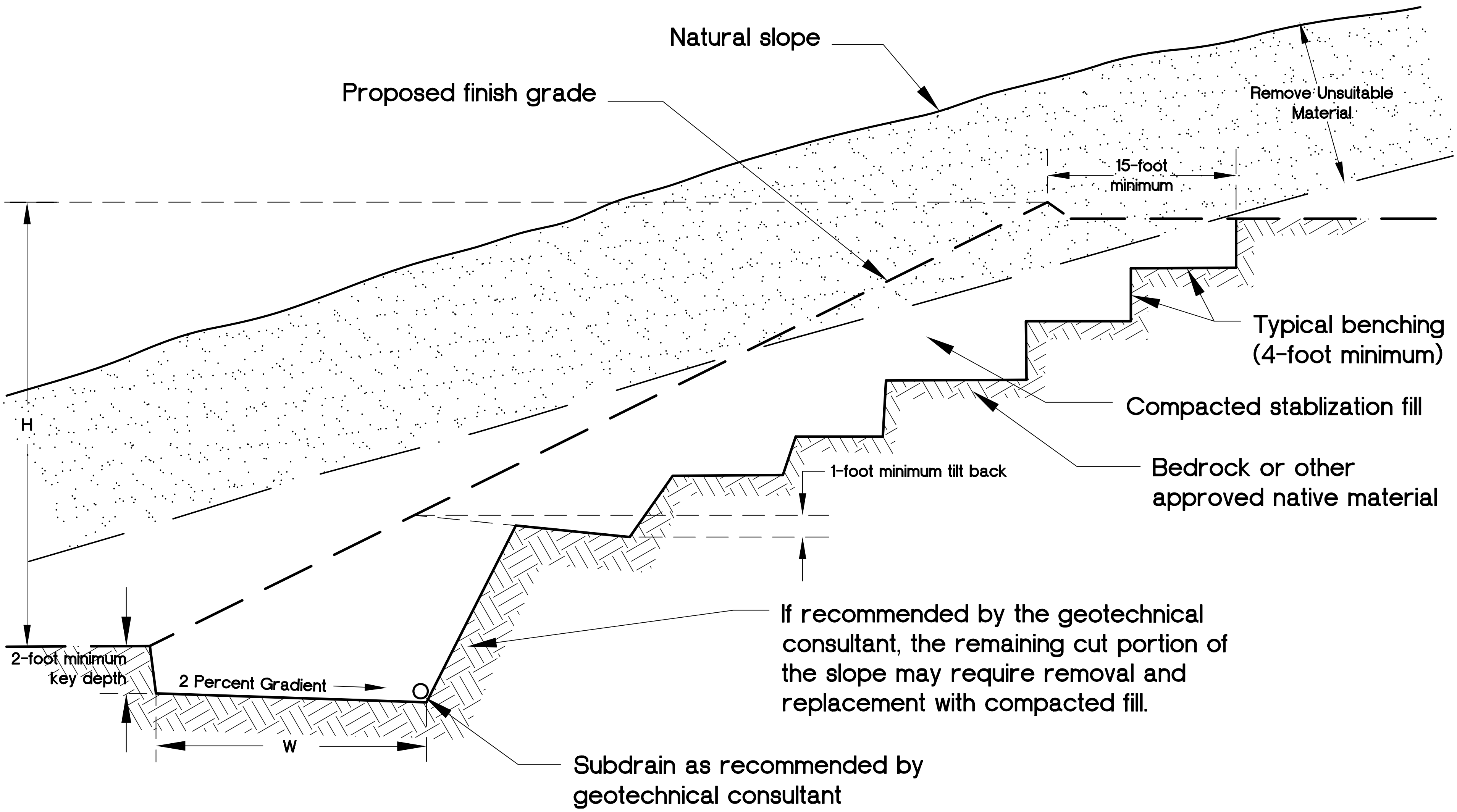
NOTES:
1. Where the natural slope approaches or exceeds the design slope ratio, special recommendations would be provided by the geotechnical consultant.

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

GeoSoils, Inc.

FILL OVER NATURAL (SIDEHILL FILL) DETAIL

Plate F-7



NOTES:

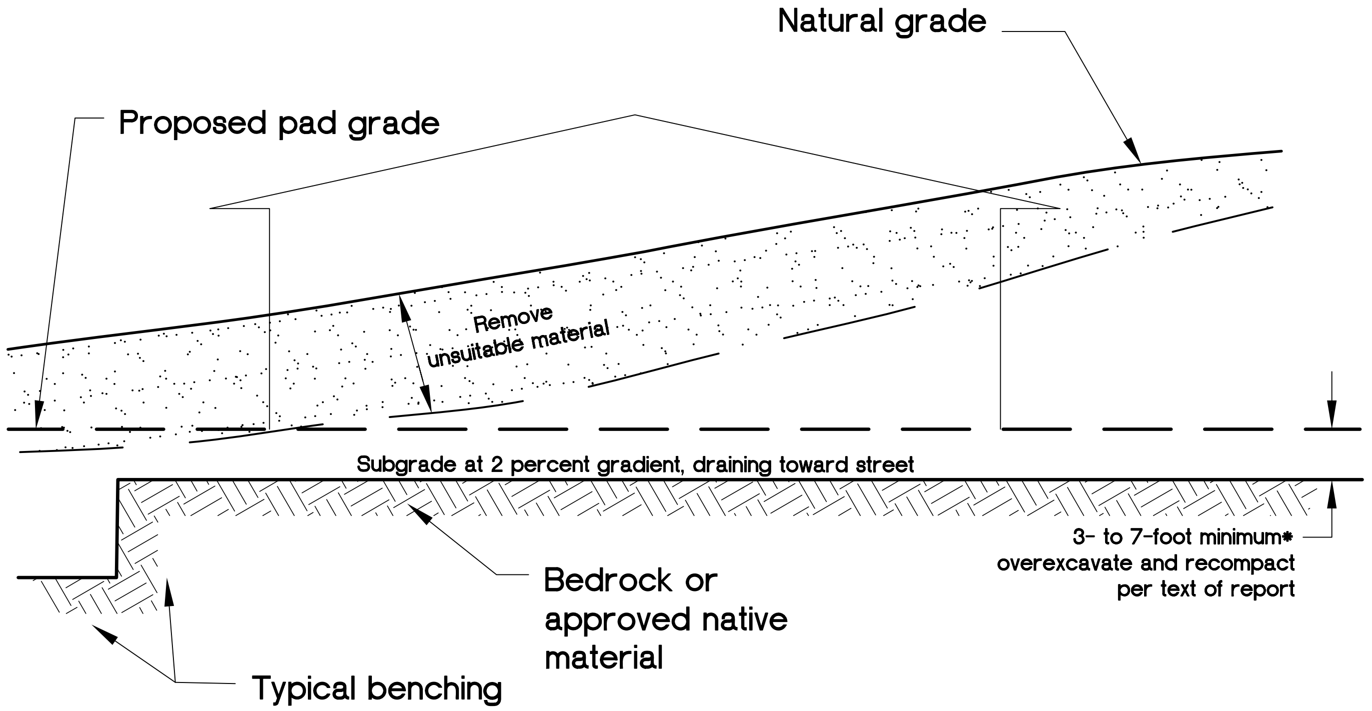
1. Subdrains may be required as specified by the geotechnical consultant.

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

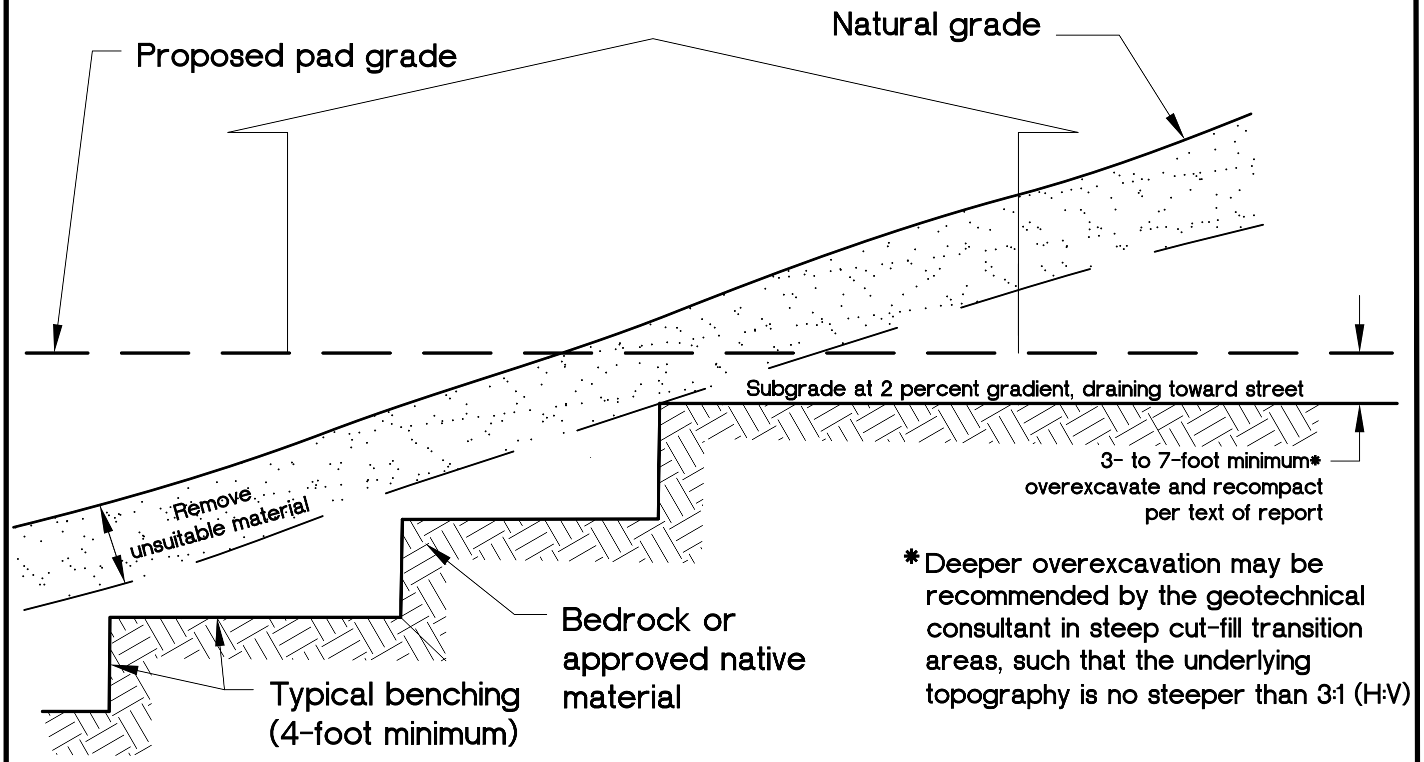
GeoSoils, Inc.

STABLIZATION FILL FOR UNSTABLE MATERIAL EXPOSED IN CUT SLOPE DETAIL

Plate F-9



CUT LOT OR MATERIAL-TYPE TRANSITION



* Deeper overexcavation may be recommended by the geotechnical consultant in steep cut-fill transition areas, such that the underlying topography is no steeper than 3:1 (H:V)

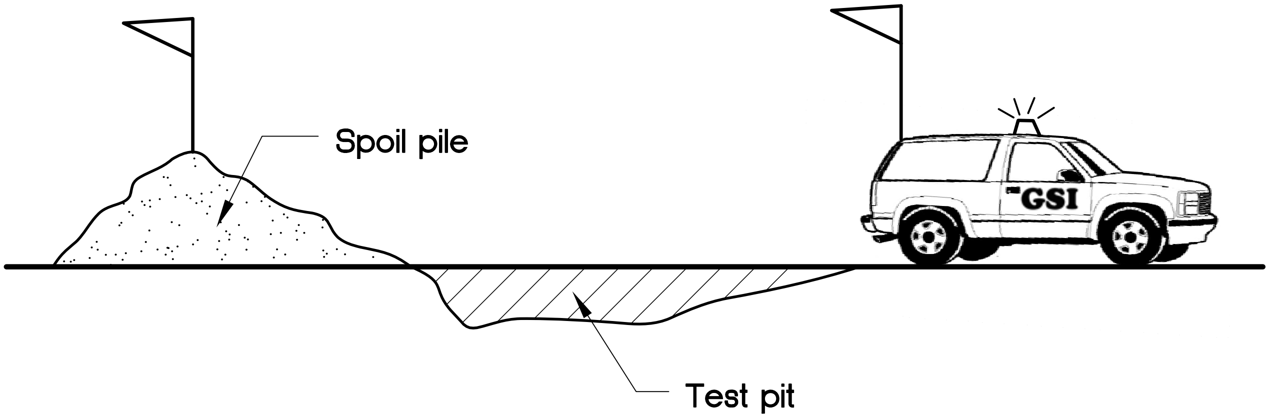
CUT-FILL LOT (DAYLIGHT TRANSITION)

GeoSoils, Inc.

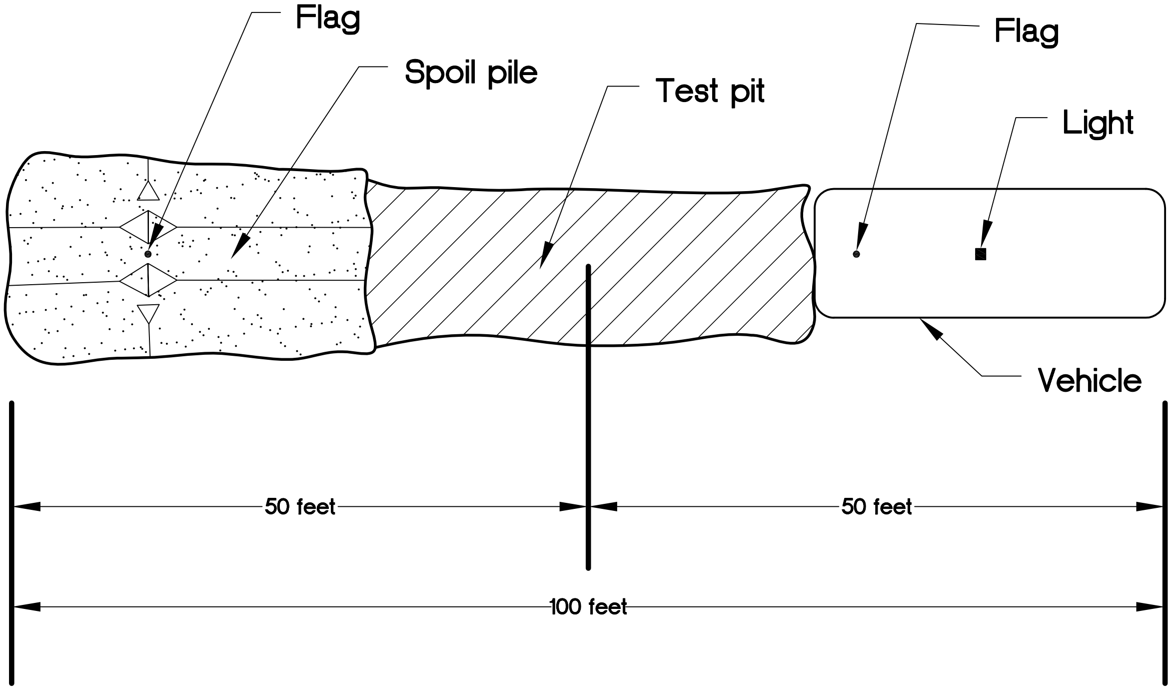
TRANSITION LOT DETAILS

Plate F-12

SIDE VIEW



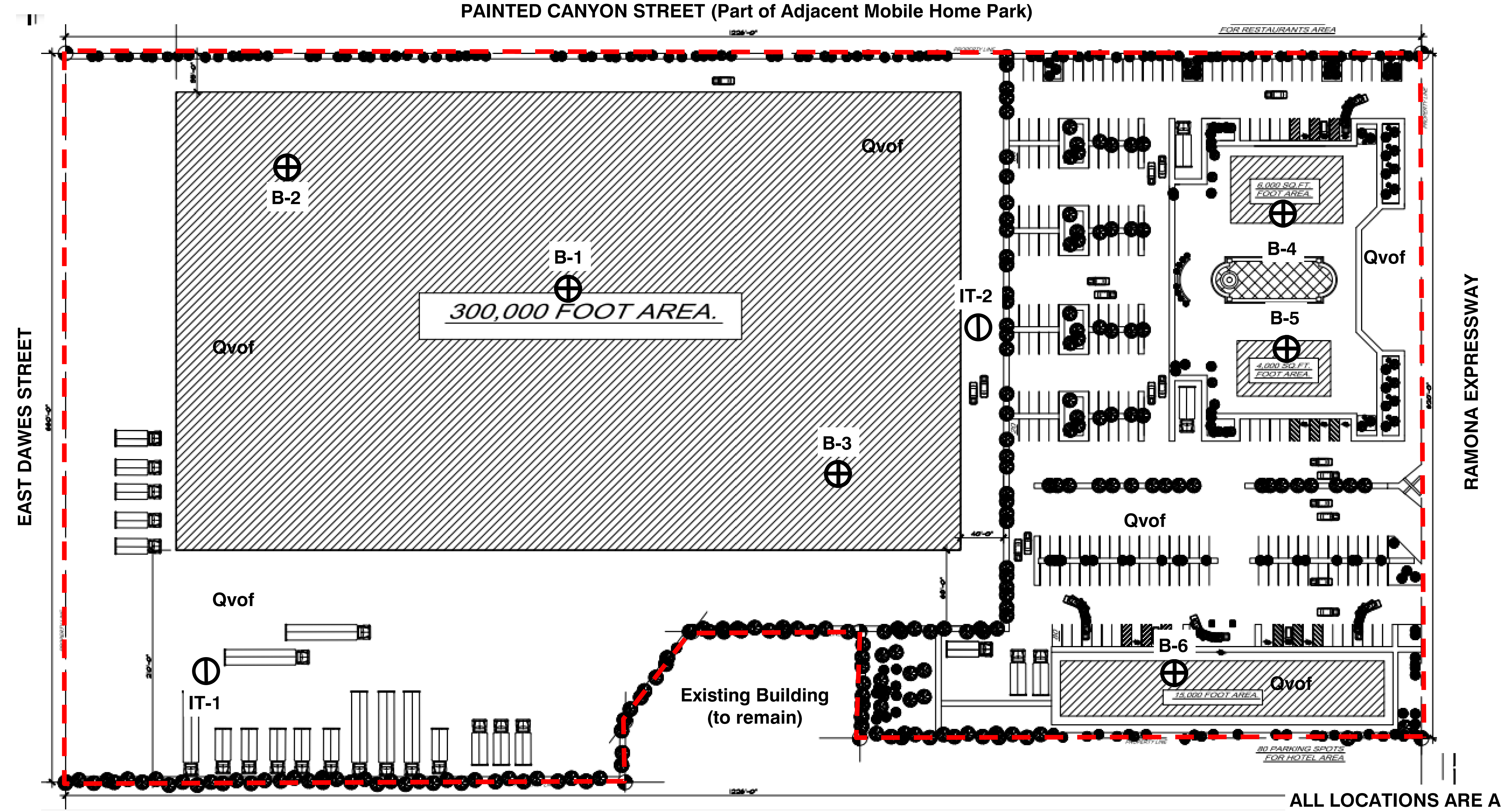
TOP VIEW



GeoSoils, Inc.

TEST PIT SAFETY DIAGRAM

Plate F-20

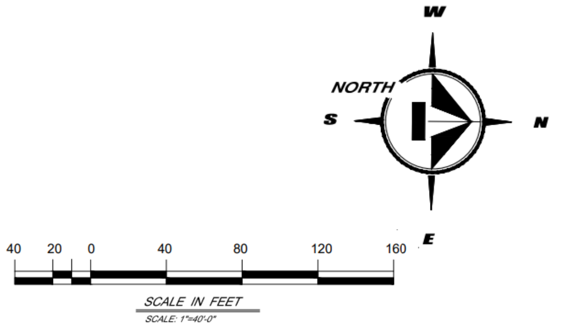


ALL LOCATIONS ARE APPROXIMATE

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

GSI LEGEND

⊕ **B-6**	Approximate Location of Exploratory Boring.
⦶ **IT-2**	Approximate Location of Percolation/ Infiltration Test Boring
Qvof	Quaternary Very Old Alluvial Fan Deposits
▬ ▬ ▬	Approximate Boundary of Property Under the Purview of this Report.



GeoSoils, Inc.

Plate 1

GEOTECHNICAL MAP

W.O. 8448-A-SC	**DATE: 10/22**	**SCALE: See Scale**