REVISED GEOTECHNICAL UPDATE
AND STORM WATER INFILTRATION STUDY
ASSISTED LIVING FACILITY
13860 EL CAMINO REAL
CITY OF SAN DIEGO, CALIFORNIA
APN 304-650-37-00

FOR

PMB LLC
3394 CARMEL MOUNTAIN ROAD, SUITE 200
SAN DIEGO, CALIFORNIA 92121

W.O. 7971-A-SC **SEPTEMBER 17, 2020**
REVISED APRIL 8, 2021

GeoSoils, Inc.

Geotechnical • Geologic • Coastal • Environmental

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September 17, 2020
Revised April 8, 2021

W.O. 7971-A-SC

PMB LLC
3394 Carmel Mountain Road, Suite 200
San Diego, California 92121

Attention: Mr. Nolan Weinberg

Subject: Revised Geotechnical Update And Storm Water Infiltration Study Assisted Living Facility 13860 El Camino Real, City Of San Diego, California APN 304-650-37-00

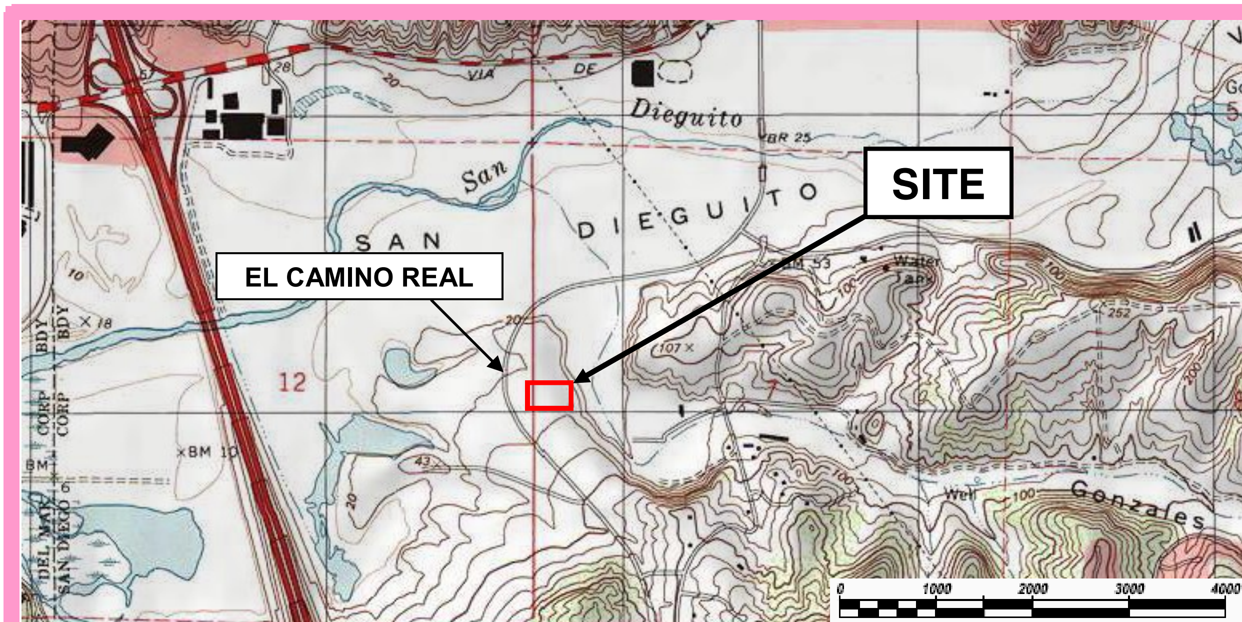
Dear Mr. Weinberg:

In accordance with your request and authorization, GeoSoils, Inc. (GSI) has prepared the following update of our previous geotechnical work (GSI, 2011), with respect to the governing Building Code (2019 edition of the California Building Code [2019 CBC], California Building Standards Commission [CBSC], 2019a]) for this project. We note that the grading plans for the Assisted Living Facility have not been completed to date. GSI's scope of services included a review of the referenced report (see Appendix A), desktop infiltration study (Appendix C), engineering and geologic analyses, and preparation of this update report. This report is to be used as a supplement to the previous GSI preliminary investigation report (GSI, 2011).

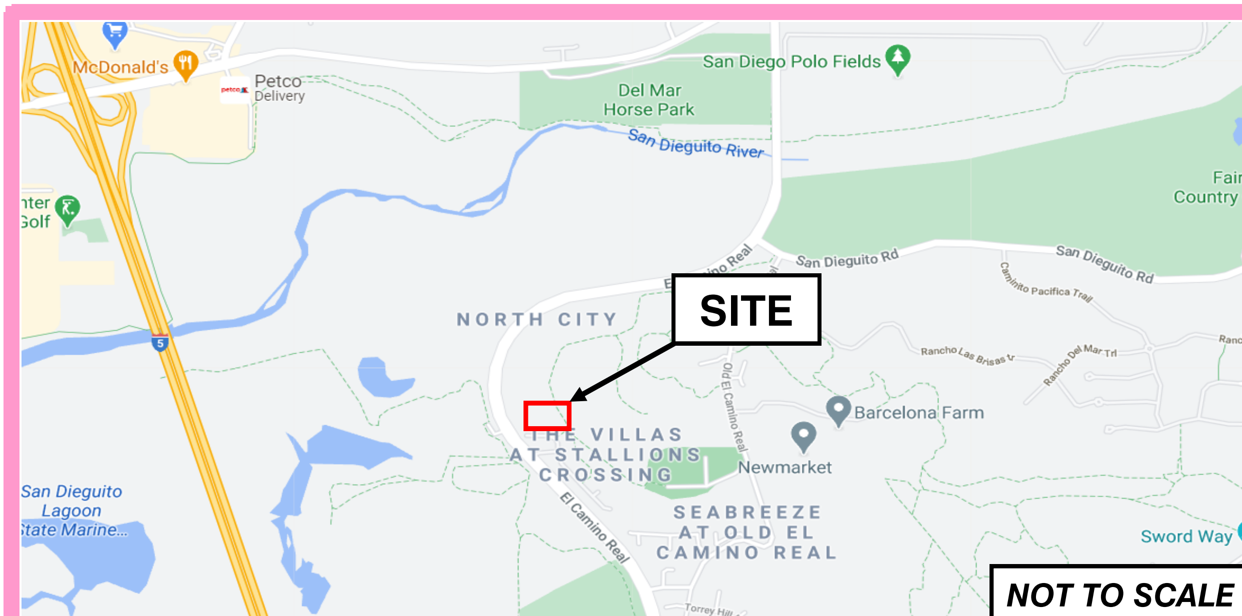
Unless specifically superseded herein, the conclusions and recommendations provided in GSI (2011) remain valid and applicable. The additional conclusions and recommendations presented herein should be appropriately incorporated into project design and construction.

SITE DESCRIPTION/PROPOSED DEVELOPMENT

The roughly 4-acre trapezoid-shaped property consists of essentially vacant land, located at 13860 El Camino Real, City of San Diego, San Diego County, California. (see Figure 1, Site Location Map), and is the southern portion of a larger, 17 acre parcel that includes the property immediately to the north, where construction of the St. John Garabed Armenian Church Facility is currently underway. The site is bounded by existing residential development on the south, a church facility on the west, the aforementioned church facility currently under construction to the north, and relatively undeveloped open space to the east. Topographically, the majority of the site consists of a very gently northward sloping



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



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SITE LOCATION MAP

Figure 1

area, referred to in this, and previous reports as a "mesa." Along the northeastern and eastern edges of the mesa, a natural slope was observed to descend eastward into alluviated areas located beyond the project area. The slope averaged approximately 40 feet in height, at gradients on the order of 2:1 to 3:1 (h:v) along the eastern edge of the mesa. Elevations within the project area vary from approximately 47 to 60 feet mean sea level ([MSL] south to north) within the mesa area, and are on the order of 18 to 21 feet mean sea level MSL within the alluviated areas of the site located beyond the planned improvement area. Surface drainage (sheet flow) generally appears to be directed offsite to the north and northwest.

It is our understanding that the planned development will be limited to the relatively flat-lying to very gently sloping mesa area of the site, while the existing east facing slope, descending from the east side of the "mesa" area and the alluviated area beyond the base of this slope will remain undisturbed and/or natural. Development will include site preparation for the construction of a 105-unit assisted living facility with a library, fitness area, kitchen, café, dining room, spa, salon, locker room, therapy room, offices, garden areas, parking/driveway areas, and associated landscape improvements. Typical cut and fill grading techniques are anticipated to be used to create the building pad. Based on current topography, cuts and fills on the order of 1 to 14 feet (or less) are estimated for the currently planned building area.

It is our understanding that the building proposed is a three-story structure, with slab-on-grade/continuous footings, utilizing wood-frame construction. Building loads are assumed to be typical for this type of construction. Sewage disposal is anticipated to be accommodated by tying into the regional system. The need for import soils is not anticipated at this time.

PREVIOUS WORK

A preliminary geotechnical evaluation (report) including the subject site was prepared by Geocon, Inc. ([Geocon], 2008). That evaluation included the excavation of seven (7) exploratory borings, of which two ([2] Borings B-3 & B-4) are located within the project boundary, as well as associated laboratory testing of samples collected. A geotechnical report, including findings, conclusions, and recommendations for a previous development concept, for the site was issued on July 17, 2008 (Geocon, 2008). An update geotechnical investigation, including the subject site, was prepared by GSI (2011) and included additional subsurface exploration (test pits), laboratory testing, and engineering analysis. This update included a review of readily available geologic literature for the site, including the previous geotechnical report for the project, geologic site reconnaissance, additional subsurface exploration, sampling, and mapping, an evaluation of site seismicity and seismic hazards, appropriate laboratory testing of representative soil samples, engineering and geologic evaluation of data collected, and report preparation. It should be noted that at that time, the subject site was proposed to consist of a sheet graded pad within the central and eastern portion of the site, with the western portion contour graded for drainage.

In 2012, GSI performed a review of the existing mesa and slope conditions regarding previous grading and improvements at the subject site (GSI, 2012), that encompassed a larger overall project to the north, northeast, and east of the mesa. While not completed specifically for the subject site, a storm water infiltration study was completed by GSI for the site immediately adjacent to the subject site (GSI, 2017) and characterized infiltration conditions for BMP desgn.

SITE EXPLORATION

Site exploration completed in preparation of this study consisted of completing three (3) hand auger borings and geologic reconnaissance mapping, performed on September 2, 2020. The approximate location of the hand auger borings are presented on the Geotechnical Map (see Plate 1) included in this report. A GSI engineering geologist observed the hand auger boring excavations, and collected representative samples of materials encountered for visual examination and subsequent laboratory testing.

Soils encountered in the hand auger borings were classified in general accordance with the Unified Soil Classification System (U.S.C.S.), as described in Appendix B. Logs of the hand auger borings (this study), as well as the logs of borings completed in preparation of Geocon report (2008), and a test pit completed in preparation of GSI (2011), are presented in Appendix B. The locations of all subsurface explorations completed onsite are depicted on Plate 1.

SITE GEOLOGIC UNITS

General

Geologic units encountered during our subsurface investigation and site reconnaissance included undocumented fill and Quaternary-age very old paralic deposits. A review of GSI (2011), and Geocon (2008) indicate that surficial deposits of colluvium (topsoil) older and Eocene-age sedimentary bedrock also occur either as thin surficial, or near surface deposits (colluvium), or at depth (bedrock). The earth materials encountered are generally described below from the youngest to oldest.

Undocumented Artificial Fill (Map Symbol - afu)

Existing, undocumented fill was observed within two (2) general areas of the site. The first area includes the westernmost two-thirds of the site, and appear to be associated with construction of the church site to the north, as the subject site was periodically used to stockpile soil. Where observed, existing fills in this area appear to consist of dry, silty to clayey sand, and appear to form a thin veneer, ranging from ±0.3 to 1 foot in thickness, from the eastern portion of the lot to the west end of the proposed construction, respectively (see Plate 1). The second area includes a thin veneer of surficial fills that appear to have been pushed over the existing, east facing slope. These fills appear to have been placed as push fills over the existing slope resulting from previous agricultural work onsite and do not appear to be located in the vicinity of the planned improvements construction. Undocumented fills are considered potentially compressible in their existing state and therefore should be removed and recompacted, if settlement-sensitive improvements and/or planned fills are proposed within their influence.

Colluvium (Topsoil) (not Mapped)

Surficial deposits of colluvium (Topsoil per Geocon, 2008) were encountered in preparation of Geocon (2008) and GSI (2011). These deposits were not noted at the selected exploration sites during this study as they were likely removed, redistributed, or otherwise disturbed during earthwork associated with the church site to the north. While not encountered during this study, these deposits likely occur elsewhere across the planned improvement area.

As encountered in preparation of Geocon (2008) and GSI (2011) colluvial soils consist of a surficial, or near surface layer varying from a silty to clayey fine sand to a silty sand with clay. Where observed (Geocon, 2008; GSI, 2011), these soils were typically dark brown, dry to moist, loose and porous. Colluvium is considered potentially compressible in its existing state and therefore should be removed and recompacted, if settlement-sensitive improvements and/or planned fills are proposed within their influence.

Very Old Paralic Deposits (Map Symbol - Qvop)

Quaternary-age very old paralic deposits were encountered beneath surficial deposits of fill. Where observed, these deposits consist of predominately silty sand. These sediments are typically dark gray to reddish brown, dry, and very dense. Weathered very old paralic deposits are considered potentially compressible in their existing state, and therefore should be removed and recompacted if settlement-sensitive improvements and/or planned fills are proposed within its' influence. *Unweathered* very old paralic deposits are considered suitable for the support of settlement-sensitive improvements and/or planned fill in their existing state. Bedding structure was observed to be approximately sub-horizontal.

GROUNDWATER

Regional groundwater was encountered in preparation of Geocon (2008) within alluvial soils located offsite to the east and northeast (offsite) at an approximate elevation of 7 feet MSL, or about 36 feet below the lowest surface grade onsite. Water was not encountered during our investigation, nor within previous borings (Geocon, 2008) completed within, or adjacent to, the area planned for development, and should not significantly affect site development. It should be noted that planned development is generally limited to areas of the site underlain with relatively dense terrace/paralic deposits.

Perched groundwater may occur in the fill or along zones of contrasting permeabilities (i.e., along fill lifts, bedrock joints/fractures, and/or bedding) due to migration from adjacent drainage areas, and during or after periods of above normal or heavy precipitation or irrigation. Thus, perched groundwater conditions may occur in the future, after development, and should be anticipated. Groundwater observations reflect site conditions at the time of this report and do not preclude changes in local groundwater conditions in the future. The potential for perched groundwater conditions should be disclosed to any interested or potentially affected parties. The performance of the site is, to a large degree, dependent on the proper control of irrigation, as discussed. As such, more rigorous slab design is necessary for any new slab-on-grade floor (State of California, 2011). Recommendations for reducing the amount of water and/or water vapor through slab-on-grade floors are provided in the "Soil Moisture Considerations" sections of this report.

GEOLOGIC HAZARDS

Landslide Susceptibility

According to regional landslide susceptibility mapping by Tan and Giffen (1995), the site is located within landslide susceptibility Area 3-1, which is characterized as being "generally susceptible" to landsliding. However, given the site's relative location to ascending or descending slopes, its gentle relief, the absence of adverse geologic structure, and the generally dense nature of the underlying formational sediments, the potential for landslides to affect the proposed site development is considered low.

Faulting

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 (CGS, 2018). However, the site is situated in an area of active faulting. These include, but are not limited to: the San Andreas fault; the San Jacinto fault; the Elsinore fault; the Coronado Bank fault zone; and the Newport-Inglewood - Rose Canyon fault zone (NIRCFZ). location of these, and other major faults relative to the site, are indicated in Appendix C (California Fault Map). The possibility of ground acceleration, or shaking at the site, may be considered as approximately similar to the Southern California region as a whole. Major active fault zones that may have a significant affect on the site, should they experience activity, are listed in Appendix C (modified from Blake, 2000a).

Other Seismic/Fault Related Hazards

The following list includes other seismic related hazards that have been considered during our evaluation of the site, and during our review of GSI (2011) and Geocon (2008). The hazards listed are considered negligible and/or completely mitigated as a result of site location, soil characteristics, and typical site development procedures:

- Dynamic Settlement
- Liquefaction
- Surface Fault Rupture
- Ground Lurching or Shallow Ground Rupture
- Seiche

City Seismic Safety Study

Based on our review of City of San Diego (2008), the site does not appear to be underlain by active, or potentially active, faults. The City has evaluated the planned improvement area of the site as belonging within "Geologic Hazard Category 52, gently sloping to steep terrain, favorable geologic structure, low risk."

LABORATORY TESTING

Laboratory tests were performed on representative samples of site earth materials in order to evaluate their physical characteristics. The results of our evaluation are summarized as follows:

Classification

Soils were classified with respect to the Unified Soil Classification System (USCS) in general accordance with ASTM D 2487 and ASTM D 2488.

Expansion Index

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

SAMPLE LOCATION AND DEPTH (ft)	EXPANSION INDEX	EXPANSION POTENTIAL
HA-2 @ 2 (This Study)	16	Very Low
TP-3 @ 4 (GSI, 2011)	17	Very Low
B-3 @ 0-2 (Geocon, 2008)	75	Medium

Maximum Density Testing

The laboratory maximum dry density and optimum moisture content for the soil type encountered during the recent investigation was evaluated in general accordance with test method ASTM D 1557. The following table presents the results:

SOIL TYPE	MAXIMUM DENSITY (PCF)	MOISTURE CONTENT (PERCENT)
A - Dark Brown, Clayey Sand (HA-2 @2')	126.4	9.5

Direct Shear Tests (Remolded)

Strain-controlled remolded shear tests (displacement ≤0.005 inches per minute), were performed on a prepared sample in the formational material (bedrock) in general accordance with the ASTM D 3080 test method. The results of shear testing are summarized in the following table.

The shear testing results are shown below.

SAMPLE LOCATION AND DEPTH (ft)	WET UNIT WEIGHT (PCF)	PRIMARY		RESIDUAL	
		COHESION (PSF)	FRICTION ANGLE (DEGREES)	COHESION (PSF)	FRICTION ANGLE (DEGREES)
HA-2 @ 2 (remolded)	138.4	146	30.3	98	30.8

Particle-Size Analysis

A grain size evaluation was performed in preparation of Geocon (2008) on a selected soil sample obtained from Boring B3. The grain-size distribution curve for this sample indicates textural distribution consisting of about 52 percent sand and 48 percent fines (silt and clay).

Corrosivity Testing

Corrosivity testing, performed on a representative sample of onsite soil in preparation of GSI (2001) indicates a pH of 7.7 (which is considered relatively neutral, to slightly alkaline), a soluble sulfate content of 0.081 percent by weight (which is considered “S0” per Table 19.3.2.1 of ACI 318-14, a chloride content of 110 parts per million (ppm), and a saturated resistivity of 490 ohm-cm (which is considered corrosive to ferrous metals). Reinforced concrete mix design for foundations, slab-on-grade floors, and pavements should minimally 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. It should be noted that GSI does not consult in the field of corrosion engineering. The client and project architect should agree on the level of corrosion protection required for the project and seek consultation from a qualified corrosion consultant as warranted. Conformation testing is recommended upon the completion of rough grading.

SEISMIC DESIGN

General

It is important to keep in perspective that in the event of an upper bound (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 than from those induced by the hazards listed above. This potential would be no greater than that for other existing structures and improvements in the immediate vicinity.

Seismic Shaking Parameters

The following table summarizes the reevaluated site-specific design criteria obtained from the 2019 CBC, 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, 2020) has now been utilized to aid in design (https://seismicmaps.org). A seismic “site class C” was assigned to this site based on average blow count data obtained from Geocon (2008). The short spectral response utilizes a period of 0.2 seconds.

2019 CBC SEISMIC DESIGN PARAMETERS		
PARAMETER	VALUE	2019 CBC OR REFERENCE
Risk Category	I, II, III	Table 1604.5
Site Class	C	Section 1613.2.2/Chap. 20 ASCE 7-16 (p. 203-204)
Spectral Response - (0.2 sec), S_s	1.098 g	Section 1613.2.1 Figure 1613.2.1(1)
Spectral Response - (1 sec), S_1	0.392 g	Section 1613.2.1 Figure 1613.2.1(2)
Site Coefficient, F_a	1.2	Table 1613.2.3(1)
Site Coefficient, F_v	1.5	Table 1613.2.3(2)

2019 CBC SEISMIC DESIGN PARAMETERS		
PARAMETER	**VALUE**	**2019 CBC OR REFERENCE**
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec), S_{MS}	1.318 g	Section 1613.2.3 (Eqn 16-36)
Maximum Considered Earthquake Spectral Response Acceleration (1 sec), S_{M1}	0.588	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	0.879 g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.392	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	Section 1613.2.5/ASCE 7-16 (p. 85: Table 11.6-1 or 11.6-2)

GENERAL SEISMIC PARAMETERS	
PARAMETER	**VALUE**
Distance to Seismic Source (B fault)[1]	4.2 mi (6.8 km)[2]
Upper Bound Earthquake (Rose Canyon Fault)	$M_W = 7.2$ [1]
[1] - Cao, et al. (2003) [2] - Blake (2000)	

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. A summary of the seismic data is included in Appendix C.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on our current and previous field exploration, current and previous laboratory testing, and geotechnical engineering analysis, it is our opinion that the site appears suitable for the proposed development from a geotechnical engineering and geologic viewpoint. Unless specifically superceded in the following sections, the conclusions and recommendations presented in GSI (2011) remain valid and applicable.

SITE EARTHWORK

General

All grading should conform to the guidelines presented in the 2019 CBC (CBSC, 2019a), the City, and as recommended herein. When Code references are not in agreement, the more stringent code should be followed. During earthwork construction, all site preparation and the general grading procedures of the contractor should be observed and the fill selectively tested by a representative(s) of GSI. If unusual or unexpected conditions are exposed in the field, they should be reviewed by this office and, if warranted, modified and/or additional recommendations will be offered. All applicable requirements of local and national construction and general industry safety orders, the Occupational Safety and Health Act (OSHA), and the Construction Safety Act should be met. It is the onsite general contractor'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.

Demolition/Grubbing

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

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

3. Cavities or loose soils remaining after demolition and site clearance should be cleaned out and observed by the soil engineer. The cavities should be replaced with fill materials that have been moisture conditioned to *at least* optimum moisture content and compacted to at least 90 percent of the laboratory standard.

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

Treatment of Existing Ground/Remedial Earthwork

Removals

Due to the relatively loose/soft condition of the near surface undocumented fills, colluvium, and highly weathered paralic deposits (if encountered), these materials should be removed and recompacted in areas proposed for settlement-sensitive improvements or areas to receive compacted fill. Removal depths across the site are anticipated to be on the order of about 1 to 6 feet across a majority of the site, with deeper removals anticipated near the northern project boundary.

Removed fill soils may be reused as fill, provided that the soil is cleansed of any deleterious material, moisture conditioned, and compacted to a minimum 90 percent relative compaction per ASTM D 1557. Removals should be completed throughout the site, and minimally at least 5 feet beyond the limits of any settlement-sensitive improvement (including plan fill) area, or to a lateral distance equal to the depth of the removal beneath the improvement, whichever is greater.

Subsequent to the above removals, the exposed bottom(s) should be scarified to a depth of at least 8 inches, brought to *at least* optimum moisture content, and recompacted to a minimum relative compaction of 90 percent of the laboratory standard, prior to any fill placement.

Overexcavation

In order to provide for the uniform support of the building(s), the cut portion of any plan transition (i.e., cut/fill) should be overexcavated to provide a minimum 4-foot thick layer (cap) of compacted fill beneath the building(s), or two (2) feet beneath building foundations, whichever is deeper. Where the total thickness of plan fill plus remedial earthwork (i.e., removals) is less than the minimum fill cap thickness, that portion of the pad(s) shall also be undercut to provide the recommended minimum fill thickness.

Overexcavation should be minimally completed to at least 5 feet beyond the building(s) footprint (including any exterior isolated footing, etc.). Where the maximum fill thickness within a given pad area exceeds 12 feet (not anticipated), the cut portion, or portion of the pad with thinner fill, shall be undercut to maintain a maximum to minimum fill ratio of not more than 3:1 (maximum to minimum) completed below a 1:1 projection down and away from the edge of any settlement-sensitive improvements and/or limits of proposed fill, per the requirements of the 2019 CBC (CBSC, 2019a).

Subsequent to the above overexcavation, the exposed bottom(s) should be scarified to a depth of at least 8 inches, brought to *at least* optimum moisture content, and recompacted to a minimum relative compaction of 90 percent of the laboratory standard, prior to any fill placement.

Expansive Soils and Mitigation

Current laboratory testing indicates expansive soil conditions ranging from very low (expansion index [E.I.] range of 0-20), to medium expansive (50 < E.I. < 90) present onsite where tested. As such, some site soil meets the criteria of expansive soil as defined in Section 1803.5.2 of the 2016 CBC. Foundation systems constructed within the influence of expansive soils (i.e., E.I. > 20 and P.I. ≥ 15) will require specific design to resist expansive soil effects per Sections 1808.6.1 or 1808.6.2 of the 2019 CBC, and should be reviewed by the project structural engineer, unless mitigated in the field during site grading.

Based on our site work, expansive soils appear to be associated with surficial and near surface deposits of colluvium, and highly weathered paralic deposits. In order to mitigate the potential effects of expansive soil, the expansive soils may be: 1) blended with less expansive site soil to reduce the overall expansion potential, 2) placed beyond (outside) the building footprint, or 3) placed in areas no closer than 7 feet vertically from finish pad grade.

Fill Placement

Subsequent to ground preparation, fill materials should be brought to *at least* optimum moisture content, placed in thin 6- to 8-inch lifts, and mechanically compacted to obtain a minimum relative compaction of 90 percent of the laboratory standard. Fill materials should be cleansed of major vegetation and debris prior to placement.

Fill Suitability

Onsite soils appear to vary from silty to clayey sands, and oversize material (12-inch plus) is not anticipated in any significant quantity. Existing site soils appear to vary from very low to medium expansive (expansion index [EI] range of 0 to 90). Any soil import should be evaluated by this office prior to importing in order to assure compatibility with the onsite site soils and the recommendations presented in this report. Import soils, if used, should be relatively sandy and very low expansive (i.e., E.I. less than 20).

Shrinkage/Bulking

Based on our experience, a preliminary value of 8 to 15 percent shrinkage for artificial fill, and highly weathered formation may be considered. Shallow cuts in formation may result in nominal shrinkage (ranging to ±5 percent).

Perimeter Conditions

It should be noted, that the 2019 CBC (CBSC, 2019a) indicates that removals of unsuitable soils be performed across all areas under the purview of the grading permit, not just within the influence of the proposed buildings. Relatively deep removals may also necessitate a special zone of consideration, on perimeter/confining areas.

Any proposed improvement or future homeowner improvements such as walls, swimming pools, house additions, etc. that are located above a 1:1 (h:v) projection up from the outermost limit of the remedial grading excavations will require deepened foundations that extend below this plane. Other site improvements, such as pavements, constructed above the aforementioned plane would retain some potential for settlement and associated distress, which may require increased maintenance/repair or replacement. This potential should be disclosed to all interested/affected parties should remedial grading excavations be constrained by property lines.

Graded Slope Construction

Based on site grades and the planned construction, graded fill and cut slope are anticipated to be on the order of 10 feet or less in height and are considered stable, assuming proper construction and maintenance.

Existing Slopes

The existing east-facing slope, located within the eastern portion of the site is located beyond the limits of planned improvements. While this slope appears to have performed adequately to date, a formal analysis of stability was not included in the scope of this study. This slope presently supports a growth of existing vegetation and irrigation is not recommended.

Temporary Slopes

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

Fill Sub-Drainage

Based on site grades and the planned construction, subdrainage is not anticipated, but may not be entirely precluded.

PRELIMINARY RECOMMENDATIONS - FOUNDATIONS

General

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

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

The foundation design recommendations, included herein, are based on anticipated column loads of 5 to 50 kips, respectively. Maximum wall loads are anticipated to be on the order of 1.5-3 kips per linear foot. The slabs-on-grade are anticipated to have typical car and/or light loads on the order of 50 to 200 psf. It is unknown if equipment and elevator pit areas will be included in the design. GSI does not anticipate high vibratory equipment loads on the floor slabs. GSI also does not anticipate highly sensitive electrical equipment mounted on the floor slab.

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

Expansive and Corrosive Soils

Current laboratory testing indicates that the onsite soils range from very low expansive (E.I. <21) to medium expansive (E.I. range of 51 to 90). As such, some site soils appear to meet the criteria of detrimentally expansive soils as defined in Section 1803.5.2 of the 2019 CBC (CBSC, 2019a). With adequate blending and placement of expansive sill soils, the overall expansive character of site soil is anticipated to exhibit an expansion index of E.I. 21, or an effective plasticity Index (PI) of 15, or less, within the upper 15 feet of the underlying soil column.

Previous testing completed in preparation of GSI (2011) indicates that site soils present a potentially negligible sulfate exposure (exposure class S0 per Table 19.3.2.1 of ACI 318-14) to concrete. However, reinforced concrete mix design for foundations, slab-on-grade floors, and pavements should also conform to "Exposure Class C1" in Table 19.3.2.1 of ACI 318-14, as concrete would likely be exposed to moisture. A chloride content of 110 parts per million (ppm), which is considered relatively non-corrosive per ACI (2014a) and Caltrans (2003), and a saturated resistivity of 490 ohm-cm (which is considered corrosive to ferrous metals) were also evaluated. While it is our understanding that typical structural (f'c ≥ 3,000) concrete cover is generally sufficient mitigation for such conditions, GSI recommends consultation with a corrosion consultant. Corrosion test results evaluated during this study (including GSI, 2011) are in general agreement with those included in Geocon (2008) regarding soluble sulfates.

Concrete mix design should be designed to comply. Exposure classes S0, W0, and C1, per ACI 318-14, should be followed. GSI does not practice in the field of corrosion engineering. Accordingly, consultation from a qualified corrosion engineer may obtained based on the level of corrosion protection requirements by the project architect and structural engineer. Upon completion of grading, laboratory testing should be performed of site materials for corrosion to concrete and corrosion to steel. Additional guidance may be obtained from a qualified corrosion engineer at that time. It is assumed by the project architect that all steel will evaluate the need for epoxy-coated, or other, corrosion protection.

Foundation Design

General:

1. The foundation systems should be designed and constructed in accordance with guidelines presented in the 2019 CBC (CBSC, 2019a). All foundations should be embedded entirely into newly compacted or mitigated fill (90 percent of ASTM D 1557).

2. An allowable bearing value of 2,000 pounds per square foot (psf) may be used for design of footings that maintain a minimum width of 15 inches and a minimum depth of 24 inches, and founded in compacted fill. This value may be increased by 20 percent for each additional 12 inches in depth to a maximum value of 2,500 psf. In addition, this value may be increased by one-third when considering short duration wind or seismic loads. Isolated pad footings should have a minimum dimension of at least 24 inches square and minimum depth of 24 inches. Where not confined by slabs, isolated footings shall be connected in two directions back to the main portion of the foundation with grade beams.

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 psf. Lateral passive pressures for shallow foundations within 2019 CBC setback zones should be reduced following a review by the geotechnical engineer unless proper setback can be established.

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

5. For the evaluation of total lateral resistance on the foundation and combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third. For effect of shrink-swell soils on hillside foundations, the geotechnical consultant should review foundation designs when available. The addition of creep loads on top-of-slope or mid-slope foundations should be considered.

Settlement:

For preliminary design purposes, foundations bearing into dense, engineered fill overlying formational soil, should be designed to minimally accommodate a static and dynamic total settlement of 2 inches and a differential settlement of 1 inch in 40 feet, respectively (angular distortion of 1/480). As grading plans become available, and based on the as-built configuration of the site, this value should be revisited. These static and dynamic (seismic) settlement estimates do not include periodic shrink/swell of expansive soils, or top-of-slope deformations.

Conventional Foundation Construction

The following foundation construction recommendations are presented as a minimum criteria from a soils engineering viewpoint. Recommendations by the project's design/structural engineer or architect, which may exceed the soils engineer's recommendations, should take precedence over the following minimum requirements.

1. Continuous footings should be founded at a minimum depth of 24 inches below the lowest adjacent ground surface bearing on very low expansive soils, for the planned three-story floor loads, respectively. All footings should be reinforced with a minimum of two No. 5 reinforcing bars at the top and two No. 5 reinforcing bars at the bottom (four bars total). Reinforcement of Isolated footings should be provided by the structural engineer. The depth of embedment is measured from the lowest adjacent grade, and does not include slab underlayment or the landscape zone.

2. A grade beam, reinforced as above, and at least 12 inches square, should be provided across any large entrance (garage, etc.). The base of the reinforced grade beam should be at the same elevation as the adjoining footings.

3. Concrete slabs (including garage, if applicable) should be a minimum of 5 inches.

4. Concrete slabs, including large building entrance areas, should be minimally reinforced with No. 4 reinforcement bars placed on 18-inch centers, in two horizontally perpendicular directions (i.e., long axis and short axis). All slab reinforcement should be supported to ensure proper mid-slab height positioning during placement of the concrete. "Hooking" of reinforcement is not an acceptable method of positioning.

5. The slab and footing subgrade should be free of loose and uncompacted material prior to placing concrete.

6. Soils generated from footing excavations to be used onsite should be compacted to a minimum relative compaction 90 percent of the laboratory standard (ASTM D 1557), whether it is to be placed inside the foundation perimeter or in the

yard/right-of-way areas. This material must not alter positive drainage patterns that direct drainage away from the structural areas and toward the street.

7. Footings should maintain a horizontal distance, X, between any adjacent descending slope face and the bottom outer edge of the footing. The horizontal distance, X, may be calculated by using X = H/3, where "H" is the height of the slope. X should not be less than 7 feet, nor need not be greater than 40 feet. X may be maintained by deepening the footings. Setbacks should minimally conform to Section 1808.7.2, and 1808.7.3 of the 2019 CBC (CBSC, 2019a) guidelines as applicable, unless specifically superceded herein.

SOIL MOISTURE TRANSMISSION CONSIDERATIONS FOR FLOOR SLABS

GSI has evaluated the potential for vapor or water transmission through the concrete floor slab, in light of typical floor coverings and improvements. Please note that slab moisture emission rates range from about 2 to 27 lbs/24 hours/1,000 square feet from a typical slab (Kanare, 2005), while floor covering manufacturers generally recommend about 3 lbs/24 hours as an upper limit. The recommendations in this section are not intended to preclude the transmission of water or vapor through the foundation or slabs. Foundation systems and slabs shall not allow water or water vapor to enter into the structure so as to cause damage to another building component or to limit the installation of the type of flooring materials typically used for the particular application (State of California, 2020). These recommendations may be exceeded or supplemented by a water "proofing" specialist, project architect, or structural consultant. Thus, the client will need to evaluate the following in light of a cost versus benefit analysis (owner expectations and repairs/replacement), along with disclosure to all interested/affected parties. It should also be noted that vapor transmission will occur in new slab-on-grade floors as a result of chemical reactions taking place within the curing concrete. Vapor transmission through concrete floor slabs as a result of concrete curing has the potential to adversely affect sensitive floor coverings depending on the thickness of the concrete floor slab and the duration of time between the placement of concrete, and the floor covering. It is possible that a slab moisture sealant may be needed prior to the placement of sensitive floor coverings if a thick slab-on-grade floor is used and the time frame between concrete and floor covering placement is relatively short.

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

- Concrete slabs should be increased in thickness from a minimum recommended thickness of 5 inches for a conventional slab (for non-expansive conditions)

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

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

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

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

- The vapor retarder shall be underlain by 2 inches clean of sand (sand equivalent [S.E.] $\geq$ 30) placed directly on the prepared, moisture conditioned, subgrade and should be sealed to provide a continuous retarder under the entire slab, as discussed above.

- Concrete should have a maximum water/cement ratio of 0.50. This does not supercede Table 19.3.2.1 of Chapter 4 of the ACI (2014) for corrosion or other corrosive requirements. Additional concrete mix design recommendations should be provided by the structural consultant and/or waterproofing specialist. Concrete finishing and workablity should be addressed by the structural consultant and a waterproofing specialist.

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

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

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

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

OTHER SITE IMPROVEMENTS

Preliminary recommendations for other site improvements, such as retaining walls, pavements, flatwork, top of slope fences/walls, and general development criteria (i.e., drainage, landscaping, etc.) are presented in GSI (2011).

STORM WATER INFILTRATION RATE EVALUATION AND DISCUSSION

USDA Study

A review of the United States Department of Agriculture database ([USDA]; 1973, 2019) indicates infiltration rates, between 0.00-0.06 inches per hour for the Las Flores loamy fine sand (5 to 7 percent slope, eroded) mapped on the site. The USDA study further indicates that site soils are classified as belonging to Hydrologic Soil Group D, which appears primarily due to a relatively shallow "depth to restrictive feature" estimated at more than "80 inches." The infiltration rate of the site immediately north of the subject site yielded an average rate of 0.028 inches per hour GSI (2017).

Infiltration Feasibility

Infiltration feasibility for this site was evaluated. An evaluation of the soils infiltration characteristics and potential impact on site development was performed for this evaluation, using a "desk top" analysis. Based on our review, including; adjacent slopes, existing (or proposed) utility backfill, and/or existing moisture-sensitive improvements, such as pavements, and utility trench backfill, foundations, retaining walls, and below grade building walls, would likely be adversely affected by soil infiltration, including offsite improvements, causing settlement and distress.

In general accordance with the City BMP Manual (City, 2018), the “categorization of infiltration feasibility condition based on geotechnical conditions” was evaluated. A review of Work Sheet C.4-1, presented in Appendix D of this report categorizes this site as a no infiltration site and should be considered in BMP design.

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

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

- Impermeable liners used in conjunction with bioretention basins should consist of a 30-mil polyvinyl chloride (PVC) membrane that is covered by a minimum of 12 inches of clean soil, free from rocks and debris, with a maximum 4:1 (h:v) slope inclination, or flatter, and meets the following minimum specifications:

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

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

- Utility backfill within BMP areas should consist of a two-sack mix of slurry.

OTHER DESIGN PROFESSIONALS/CONSULTANTS

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

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

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

PLAN REVIEW

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

LIMITATIONS

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

Inasmuch as our study is based upon our review, engineering analyses, and laboratory data, the conclusions and recommendations presented herein are professional opinions. These opinions have been derived in accordance with current standards of practice, and no warranty is express or implied. Standards of practice are subject to change with time. This report has been prepared for the purpose of providing soil design parameters derived from testing of a soil sample received at our laboratory, and does not represent an evaluation of the overall stability, suitability, or performance of the property for the proposed development. 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.

The opportunity to be of service is greatly appreciated. If you have any questions concerning this report, or if we may be of further assistance, please do not hesitate to contact any of the undersigned.

Respectfully submitted,

GeoSoils, Inc.

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Attachments: Figure 1 - Site Location Map
Appendix A - References
Appendix B - Hand Auger Boring Logs
Appendix C - Seismic Data
Appendix D - Infiltration Worksheet C.4-1
Plate 1 - Geotechnical Map

Distribution: (3) Addressee, (2 hard copies via USPS and email with PDF)

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

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

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

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

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

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

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

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

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

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

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

_____, 2000b, EQSEARCH, A computer program for the estimation of peak horizontal acceleration from California historical earthquake catalogs; Updated through June, 2006, Windows 95/98 version.

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

Building News, 1995, CAL-OSHA, State of California, Construction Safety Orders, Title 8, Chapter 4, Subchapter 4, amended October 1.

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.

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

Cao, T., Bryant, W.A., Rowshandel, B., Branum, D., and Willis, C.J., 2003, The revised 2002 California probalistic seismic hazard maps, dated June, http://www.conversation.ca.gov/cgs/rghm/psha/fault_parameters/pdf/documents/2002_ca_hazardmaps.pdf.

City of San Diego, 2018, Storm water standards, BMP design manual, October.

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

Geocon, 2008, Geotechnical investigation, River Park Equestrian Center, San Diego, California, Project No. 07921-42-01, dated July 17.

GeoSoils, Inc., 2018a, Addendum to the geotechnical update, St. John Garabed Armenian Church, 13901⅓ El Camino Real, City of San Diego, California, W.O. 6306-A3-SC, dated November 15.

_____, 2018b, Infiltration Feasibility Study, St. John Garabed Armenian Church, 13901⅓ El Camino Real, City of San Diego, California, W.O. 6063-A3-SC, dated July 10, 2018 (Revised July 20, 2018).

_____, 2018c, Update/Response to Cycle Issues L64-003A, St. John Garabed Church Grading, dated November 13, 2017, by the City of San Diego Development Services Department, revised April 9.

_____, 2017, Geotechnical Update and Storm Water Infiltration Evaluation, St. John Garabed Armenian Church, 13901⅓ El Camino Real, City of San Diego, California, W.O. 6306-A3-SC, dated September 15.

_____, 2013, Geotechnical update evaluation and response to City of San Diego review comments, St. John Garabed Armenian Church, 13901⅓ El Camino Real, City of San Diego, California, W.O. 6306-A2-SC, dated April 24.

_____, 2012, Review of existing mesa and slope conditions regarding previous grading and improvements, proposed St. John Garabed Armenian Church, 13901 El Camino Real, City of San Diego, California, W.O. 6306-A-SC, dated August 20.

_____, 2017, Geotechnical Update and Storm Water Infiltration Evaluation, St. John Garabed Armenian Church, 13901⅓ El Camino Real, City of San Diego, California, W.O. 6306-A3-SC, dated September 15.

_____, 2012, Review of existing mesa and slope conditions regarding previous grading and improvements, proposed St. John Garabed Armenian Church, 13901 El Camino Real, City of San Diego, California, W.O. 6306-A-SC, dated August 20.

_____, 2011, Update geotechnical investigation, St. John Garabed Armenian Church, 13901⅓ El Camino Real, City of San Diego, California, W.O. 6306-A-SC, dated December 14.

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

Kennedy, M.P., 1975, Geology of the San Diego metropolitan area, California; California Division of Mines and Geology, Bulletin 200, Section A, Western San Diego Metropolitan Area, Del Mar, La Jolla, and Point Loma, 7½ minute quadrangles (Revised 2001).

Kennedy, M.P., and Tan, SS., 2008, Geologic map of the San Diego 30' by 60' quadrangle, California, Map no. 3, scale 1:100,000,California Geologic Survey and U.S. Geologic Survey.

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

San Diego, City of, 2016, Storm Water Standards, Part 1:BMP design manual, for permanent site design, storm water treatment and hydromodification management, storm water requirements for development applications, with Appendices, dated January.

San Diego, City of, 2008, Seismic safety study, geologic hazards and faults, Grid Tile 34, 800 scale, Development Services Department, updated April 3.

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

Tan, S.S., and Kennedy, Michael P., 1996, Geologic maps of the northwestern part of San Diego County, California: California Division of Mines and Geology, Open File Report 96-02.

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

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

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

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

Van Hoorm, J.W., 1979, Determining hydraulic conductivity with the inversed auger hole and infiltrometer methods.

APPENDIX B

BORING AND TEST PIT LOGS
THIS STUDY, GSI (2011), AND GEOCON (2008)

GeoSoils, Inc.

BORING LOG

PROJECT: PMB, LLC
El Camino Real, San Diego

W.O. 7971-A-SC *BORING* HA-1 *SHEET* 1 *OF* 1

DATE EXCAVATED 9-2-20 *LOGGED BY:* TMP *APPROX. ELEV.:* 59' MSL

SAMPLE METHOD: 3½" Hand Auger

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
0				SM				**PARALIC DEPOSITS:** @ 0' SANDSTONE, reddish brown, dry, very dense.
								Practical Refusal at 1.5' No Groundwater or Caving Encounter
5								
10								
15								
20								
25								
30								

Standard Penetration Test
Undisturbed, Ring Sample
Groundwater
Seepage

GeoSoils, Inc.

BORING LOG

PROJECT: PMB, LLC
El Camino Real, San Diego

W.O. 7971-A-SC *BORING* HA-2 *SHEET* 1 *OF* 1

DATE EXCAVATED 9-2-20 *LOGGED BY:* TMP *APPROX. ELEV.:* 55' MSL

SAMPLE METHOD: 3½" Hand Auger

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
0	■			SM/SC				**FILL:** @ 0' SILTY to CLAYEY SAND, gray brown, dry, very dense.
	■			SM				**PARALIC DEPOSITS:** @ 1' SANDSTONE, red brown, dry, very dense.
								Hand Auger Terminated on Refusal at 2' No Groundwater or Caving Encountered
5								
10								
15								
20								
25								
30								

Standard Penetration Test
Undisturbed, Ring Sample
Groundwater
Seepage

GeoSoils, Inc. BORING LOG

PROJECT: PMB, LLC
El Camino Real, San Diego

W.O. 7971-A-SC *BORING* HA-3 *SHEET* 1 *OF* 1

DATE EXCAVATED 9-2-20 *LOGGED BY:* TMP *APPROX. ELEV.:* 51' MSL

SAMPLE METHOD: 3½" Hand Auger

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
0				SM/SC				**FILL:** @ 0' SILTY to CLAYEY SAND, gray/red brown, dry, very dense; occasional debris (plastic string).
				SM				**PARALIC DEPOSITS:** @ ½' SANDSTONE, dark gray/reddish brown, dry, very dense; numerous rounded red pebbles.
5								Hand Auger Terminated on Refusal @ 1' No Groundwater or Caving Encountered
10								
15								
20								
25								
30								

Standard Penetration Test
Undisturbed, Ring Sample
Groundwater
Seepage

GeoSoils, Inc.

W.O. 6306-A-SC
St. John Garabed
Logged By: RGC
November 18, 2011

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-3	56	0-1½	SM/SC				**COLLUVIUM:** SILTY SAND with CLAY, dark brown, moist, loose, porous; some organics; twine, plastic and wood debris in upper 8-12" indicate cultivation.
		1½-2½	SM				**TERRACE DEPOSITS:** SILTY SAND, brown, moist, medium dense.
		2½-4	CL/CH				CLAY, dark olive brown, moist, very stiff; randomly fractured with abundant caliche mottlings on fracture faces, caliche less abundant with depth.
		4-6	SM				SILTY SAND with CLAY, brown, moist, dense.
							Total Depth = 6' No Groundwater Encountered Backfilled 11-18-2011

PROJECT NO. 07921-42-01

BORING B 3

ELEV. (MSL.) 50 DATE COMPLETED 06-24-2008

EQUIPMENT HOLLOW STEM AUGER BY: T. REIST

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B3-1			SC/SM	**TOPSOIL** Loose, dry, dark brown, Clayey/Silty fine to medium SAND			
2					**TERRACE DEPOSITS** Very dense, damp, reddish brown, very Silty, fine to medium SAND with clay and charcoal flakes			
6	B3-2			SM		65	126.4	13.4
10	B3-3				-Becomes dense, dark reddish brown with less silt	47		
16	B3-4				-Becomes very dense, reddish brown to light brown, silty and fine grained with abundant mica	75	110.5	13.0
20	B3-5			ML	-Becomes very stiff, moist, dark gray and orange, Clayey SILT with sand	39		

Boring terminated at 21 feet
No groundwater encountered
Boring backfilled with 7 ft³ of bentonite

Figure A-3,
Log of Boring B 3, Page 1 of 1

07921-42-01.GPJ

SAMPLE SYMBOLS: ... SAMPLING UNSUCCESSFUL; ... STANDARD PENETRATION TEST; ... DRIVE SAMPLE (UNDISTURBED); ... DISTURBED OR BAG SAMPLE; ... CHUNK SAMPLE; ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. 07921-42-01

BORING B 4

ELEV. (MSL.) 49 DATE COMPLETED 06-24-2008

EQUIPMENT HOLLOW STEM AUGER BY: T. REIST

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM/C	**TOPSOIL** Loose, dry to damp, dark brown, Silty/Clayey fine SAND with mulch			
2					**TERRACE DEPOSITS** Dense, damp, dark reddish brown, Silty fine to medium SAND with clay, charcoal flakes and mica			
4								
	B4-1			SM		43	120.6	15.1
6								
8								
10								
	B4-2				-Becomes less silty with clay and charcoal flakes are absent	46		

Boring terminated at 11 feet
No groundwater encountered

Figure A-4,
Log of Boring B 4, Page 1 of 1

07921-42-01.GPJ

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

APPENDIX C

SEISMIC DATA

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***********************
*                     *
*    E Q F A U L T    *
*                     *
*    Version 3.00     *
*                     *
***********************
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DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 7971-A-SC

DATE: 09-03-2020

JOB NAME: PMB LLC

CALCULATION NAME: Test Run Analysis

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

SITE COORDINATES:
   SITE LATITUDE: 32.9705
   SITE LONGITUDE: 117.2381

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: 1
   Basement Depth: 5.00 km   Campbell SSR: 0   Campbell SHR: 0
   COMPUTE PEAK HORIZONTAL ACCELERATION

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

MINIMUM DEPTH VALUE (km): 3.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
ROSE CANYON	4.2(6.8)	7.2	0.677	XI
NEWPORT-INGLEWOOD (Offshore)	17.0(27.4)	7.1	0.242	IX
CORONADO BANK	17.5(28.2)	7.6	0.323	IX
ELSINORE (JULIAN)	30.3(48.8)	7.1	0.136	VIII
ELSINORE (TEMECULA)	31.0(49.9)	6.8	0.108	VII
EARTHQUAKE VALLEY	40.8(65.7)	6.5	0.066	VI
PALOS VERDES	46.1(74.2)	7.3	0.101	VII
ELSINORE (GLEN IVY)	46.9(75.5)	6.8	0.070	VI
SAN JOAQUIN HILLS	48.7(78.4)	6.6	0.084	VII
ELSINORE (COYOTE MOUNTAIN)	50.8(81.8)	6.8	0.064	VI
SAN JACINTO-ANZA	52.9(85.2)	7.2	0.082	VII
SAN JACINTO-COYOTE CREEK	54.1(87.0)	6.6	0.053	VI
SAN JACINTO-SAN JACINTO VALLEY	56.4(90.7)	6.9	0.062	VI
NEWPORT-INGLEWOOD (L.A.Basin)	59.3(95.4)	7.1	0.067	VI
CHINO-CENTRAL AVE. (Elsinore)	61.7(99.3)	6.7	0.069	VI

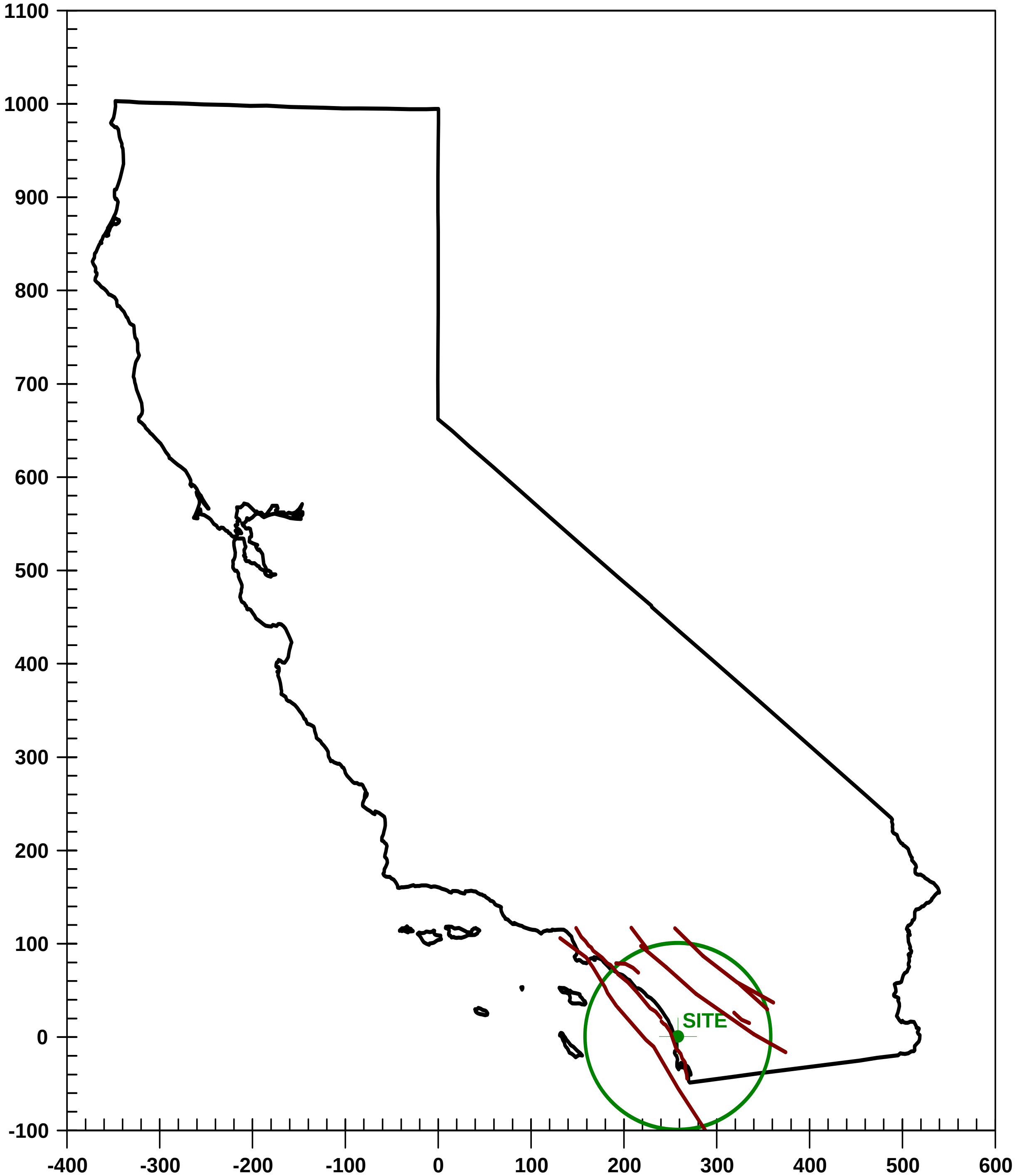
-END OF SEARCH- 15 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ROSE CANYON FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 4.2 MILES (6.8 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.6771 g

CALIFORNIA FAULT MAP

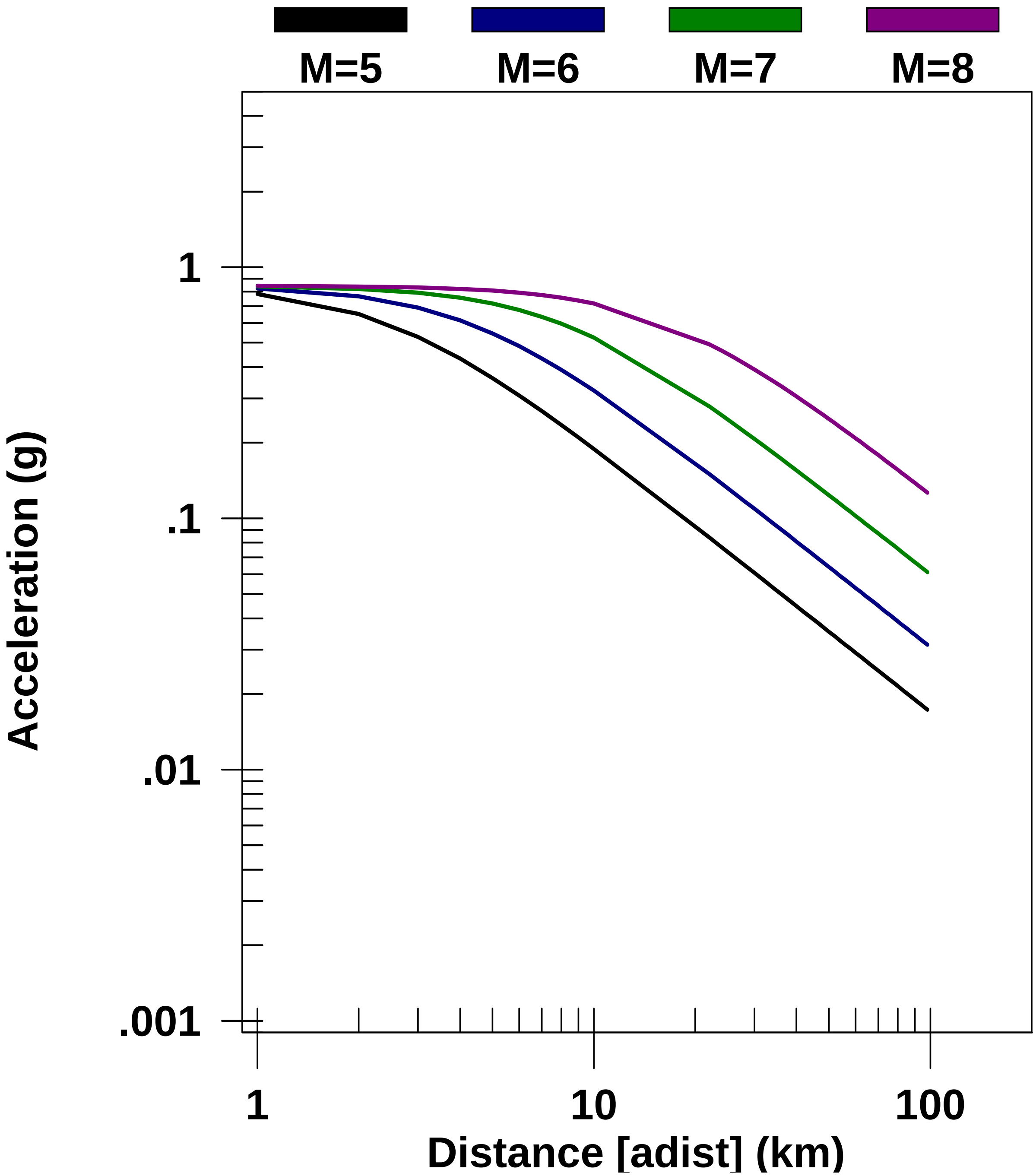
PMB LLC



W.O. 7971-A-SC
PLATE C-3

STRIKE-SLIP FAULTS

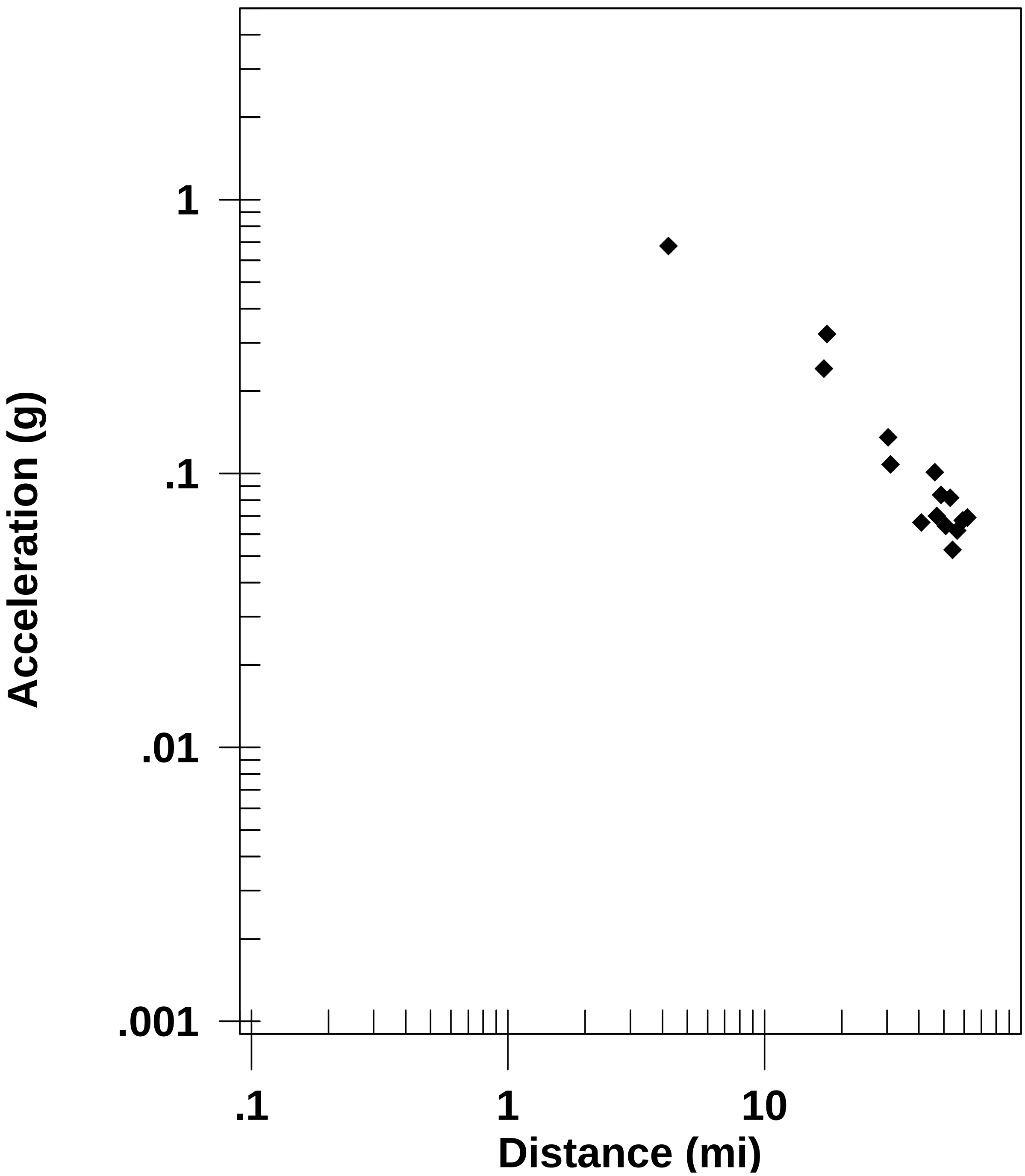
11) Bozorgnia Campbell Niazi (1999) Hor.-Pleist. Soil-Cor.



W.O. 7971-A-SC
PLATE C-4

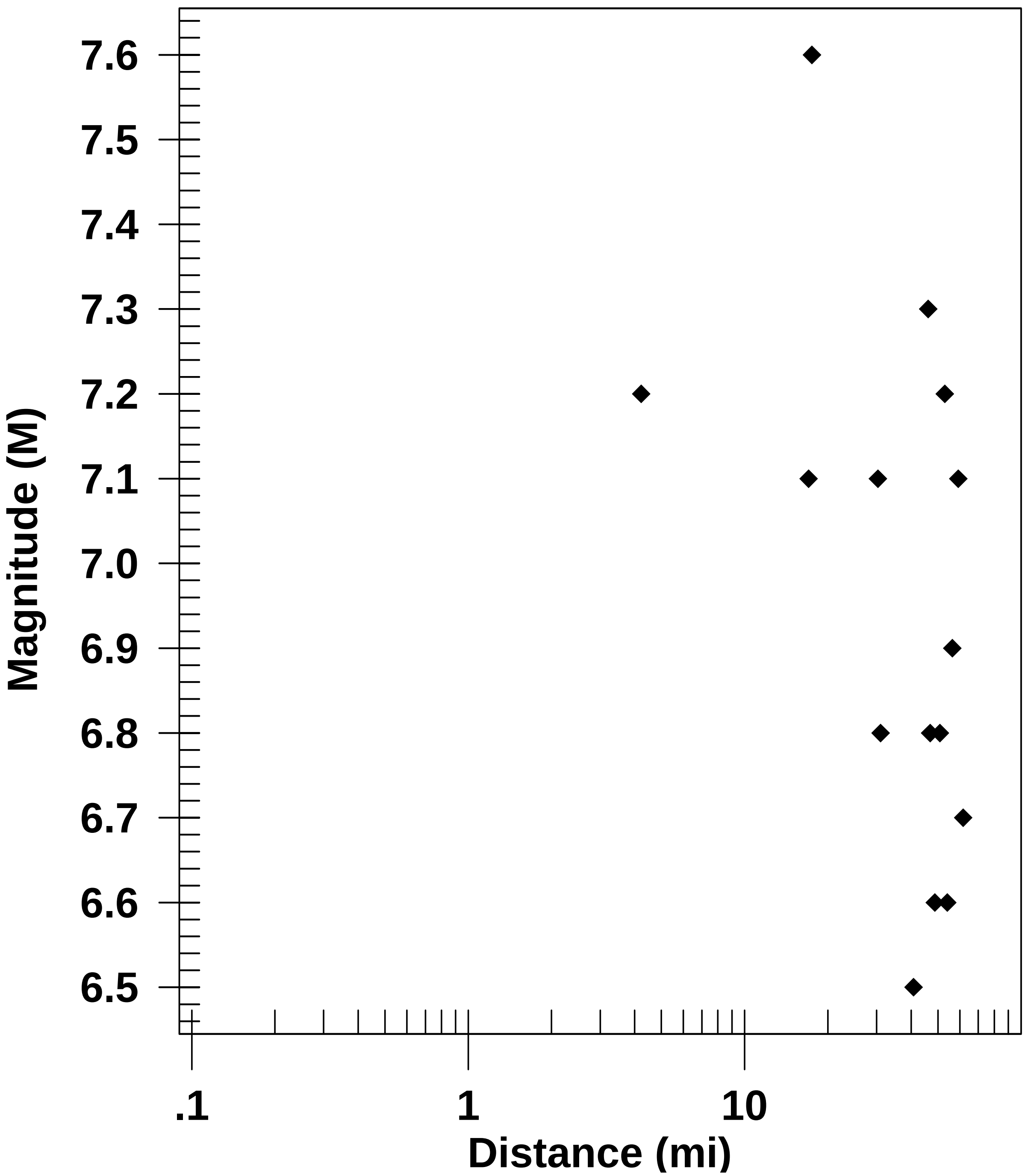
MAXIMUM EARTHQUAKES

PMB LLC



EARTHQUAKE MAGNITUDES & DISTANCES

PMB LLC



W.O. 7971-A-SC
PLATE C-6

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

JOB NUMBER: 7971-A-SC

DATE: 09-03-2020

JOB NAME: PMB LLC

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

SITE COORDINATES:
SITE LATITUDE: 32.9705
SITE LONGITUDE: 117.2381

SEARCH DATES:
START DATE: 1800
END DATE: 1999

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: 1 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

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	33.0000	117.3000	11/22/1800	2130 0.0	0.0	6.50	0.522	X	4.1(6.6)
MGI	33.0000	117.0000	09/21/1856	730 0.0	0.0	5.00	0.078	VII	13.9(22.4)
MGI	32.8000	117.1000	05/25/1803	0 0 0.0	0.0	5.00	0.077	VII	14.2(22.9)
DMG	32.7000	117.2000	05/27/1862	20 0 0.0	0.0	5.90	0.101	VII	18.8(30.3)
T-A	32.6700	117.1700	12/00/1856	0 0 0.0	0.0	5.00	0.052	VI	21.1(34.0)
T-A	32.6700	117.1700	10/21/1862	0 0 0.0	0.0	5.00	0.052	VI	21.1(34.0)
T-A	32.6700	117.1700	05/24/1865	0 0 0.0	0.0	5.00	0.052	VI	21.1(34.0)
DMG	32.8000	116.8000	10/23/1894	23 3 0.0	0.0	5.70	0.059	VI	28.0(45.0)
DMG	33.2000	116.7000	01/01/1920	235 0.0	0.0	5.00	0.031	V	34.9(56.2)
PAS	32.9710	117.8700	07/13/1986	1347 8.2	6.0	5.30	0.035	V	36.6(58.9)
MGI	33.2000	116.6000	10/12/1920	1748 0.0	0.0	5.30	0.032	V	40.2(64.6)
DMG	33.0000	116.4330	06/04/1940	1035 8.3	0.0	5.10	0.024	V	46.7(75.1)
DMG	33.7000	117.4000	05/13/1910	620 0.0	0.0	5.00	0.021	IV	51.2(82.4)
DMG	33.7000	117.4000	05/15/1910	1547 0.0	0.0	6.00	0.038	V	51.2(82.4)
DMG	33.7000	117.4000	04/11/1910	757 0.0	0.0	5.00	0.021	IV	51.2(82.4)
T-A	32.2500	117.5000	01/13/1877	20 0 0.0	0.0	5.00	0.020	IV	52.0(83.7)
DMG	33.6990	117.5110	05/31/1938	83455.4	10.0	5.50	0.027	V	52.7(84.8)
DMG	33.7100	116.9250	09/23/1963	144152.6	16.5	5.00	0.020	IV	54.2(87.1)
DMG	33.7500	117.0000	06/06/1918	2232 0.0	0.0	5.00	0.019	IV	55.5(89.4)
DMG	33.7500	117.0000	04/21/1918	223225.0	0.0	6.80	0.059	VI	55.5(89.4)
PAS	33.5010	116.5130	02/25/1980	104738.5	13.6	5.50	0.025	V	55.6(89.5)
DMG	33.5000	116.5000	09/30/1916	211 0.0	0.0	5.00	0.019	IV	56.2(90.4)
DMG	32.7000	116.3000	02/24/1892	720 0.0	0.0	6.70	0.053	VI	57.5(92.6)
DMG	33.3430	116.3460	04/28/1969	232042.9	20.0	5.80	0.029	V	57.6(92.7)
DMG	33.8000	117.0000	12/25/1899	1225 0.0	0.0	6.40	0.042	VI	58.9(94.8)
DMG	33.5750	117.9830	03/11/1933	518 4.0	0.0	5.20	0.020	IV	59.9(96.4)
MGI	33.8000	117.6000	04/22/1918	2115 0.0	0.0	5.00	0.017	IV	61.0(98.1)
DMG	33.6170	117.9670	03/11/1933	154 7.8	0.0	6.30	0.038	V	61.3(98.7)
DMG	33.4000	116.3000	02/09/1890	12 6 0.0	0.0	6.30	0.038	V	61.8(99.4)
DMG	33.2000	116.2000	05/28/1892	1115 0.0	0.0	6.30	0.037	V	62.1(99.9)

****

-END OF SEARCH- 30 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 1999

LENGTH OF SEARCH TIME: 200 years

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

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 6.8

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.522 g

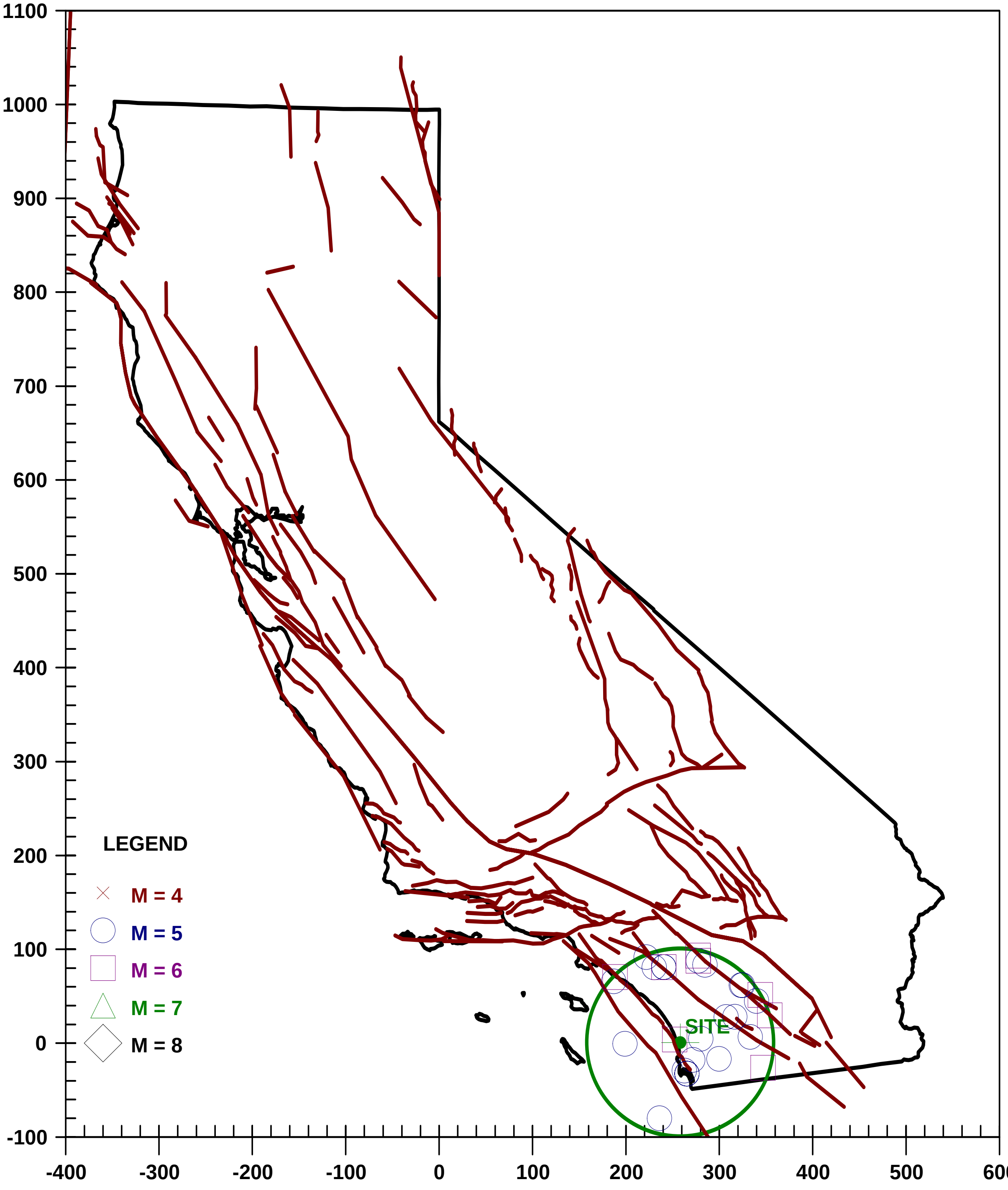
COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
a-value= 0.500
b-value= 0.302
beta-value= 0.696

TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	30	0.15075
4.5	30	0.15075
5.0	30	0.15075
5.5	13	0.06533
6.0	8	0.04020
6.5	3	0.01508

EARTHQUAKE EPICENTER MAP

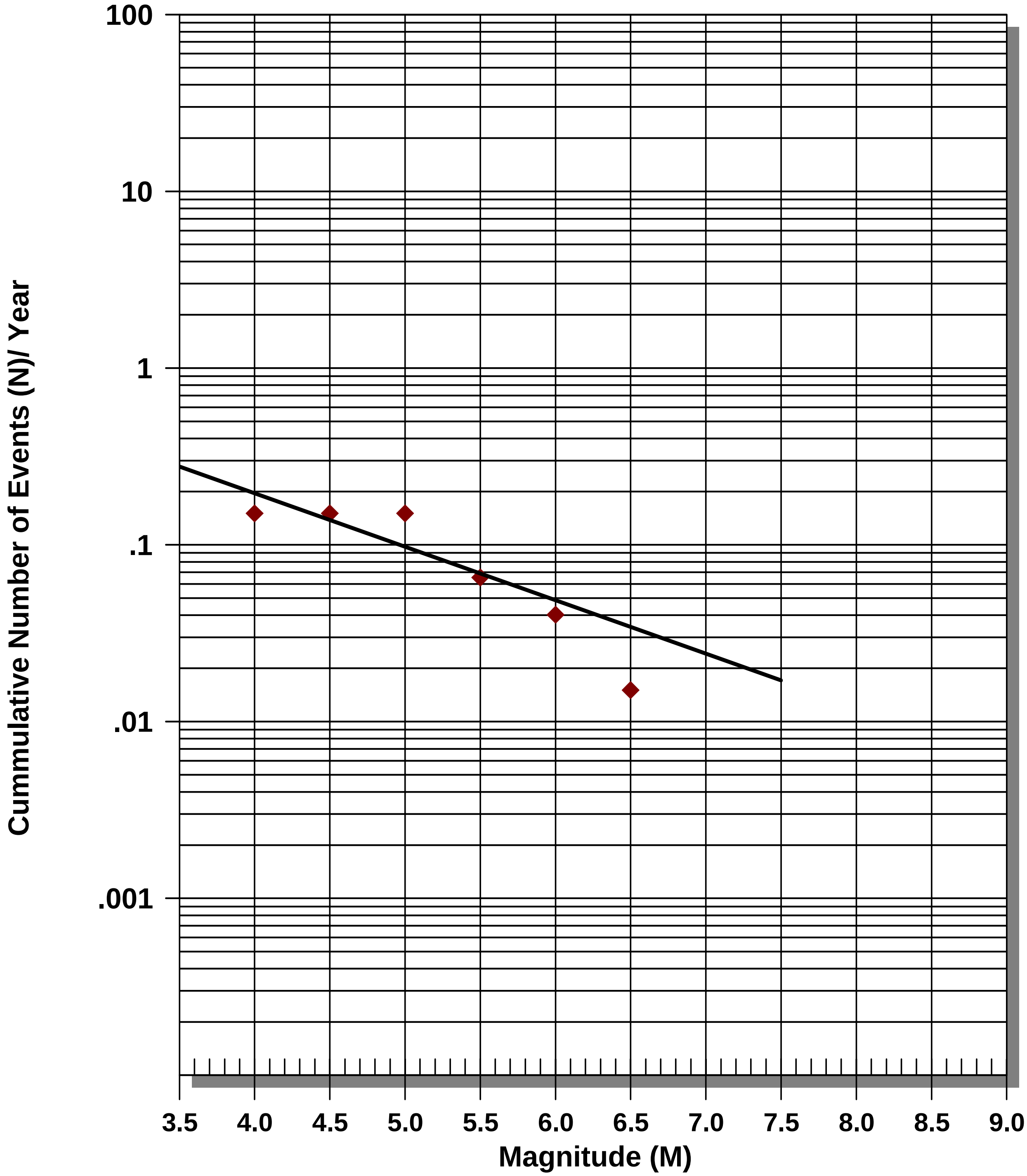
PMB LLC

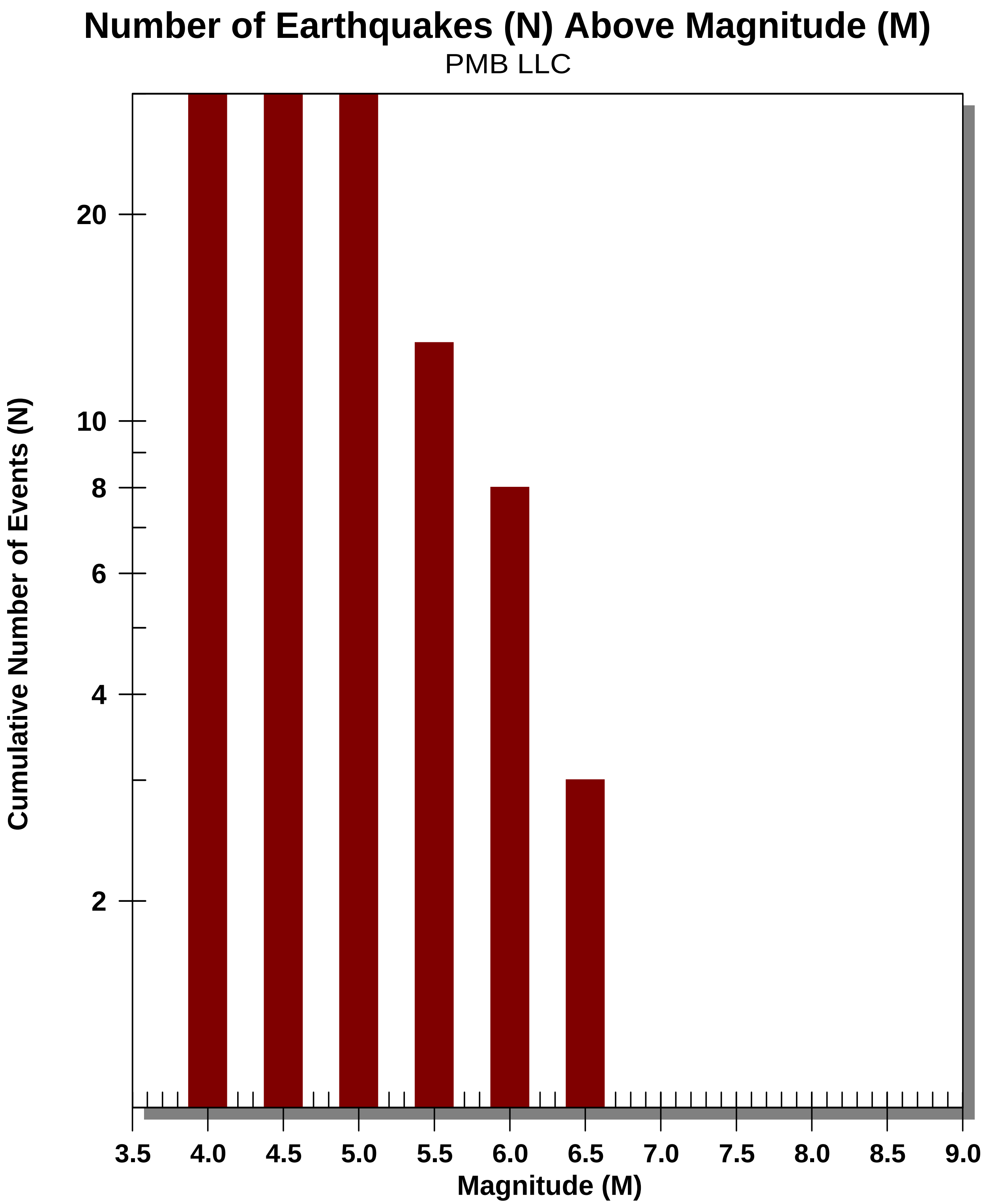


W.O. 7971-A-SC
PLATE C-10

EARTHQUAKE RECURRENCE CURVE

PMB LLC




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

APPENDIX D

INFILTRATION FEASIBILITY WORKSHEET C.4-1
PER CITY (2018)

Worksheet C.4-1: Categorization of Infiltration Feasibility Condition Based on Geotechnical Conditions[9]

Categorization of Infiltration Feasibility Condition Based on Geotechnical Conditions		Worksheet C.4-1:Form I-8A[10]
Part 1 - Full Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		**Project Phase:**
Location/limts of DMA undefined		Design Phase
Criteria 1: Infiltration Rate Screening		
1A	Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[11]? ☐Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing. ☐ No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B). ☐ No; the mapped soil types are C, D, or "urban/unclassified" and is corroborated by available site soil data. Answer "No" to Criteria 1 Result. ☒ No; the mapped soil types are C, D, or "urban/unclassified" but is not corroborated by available site soil data (continue to Step 1B).	
1B	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1? ☒ Yes; Continue to Step 1C. ☐ No; Skip to Step 1D.	
1C	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour? ☐ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result. ☒ No; full infiltration is not required. Answer "No" to Criteria 1 Result.	
1D	**Infiltration Testing Method.** Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation. ☐ Yes; continue to Step 1E. ☐ No; select an appropriate infiltration testing method.	

[9]Note that it is not required to investigate each and every criterion in the worksheet, a single "no" answer in Part 1, Part 2, Part 3, or Part 4 determines a full, partial, or no infiltration condition.
[10]This form must be completed each time there is a change to the site layout that would affect the infiltration feasibility condition. Previously completed forms shall be retained to document the evolution of the site storm water design.
[11] Available data includes site-specific sampling or observation of soil types or texture classes, such as obtained from borings or test pits necessary to support other design elements.

Categorization of Infiltration Feasibility Condition Based on Geotechnical Conditions	Worksheet C.4-1:Form I-8A[10]
1E	**Number of Percolation/Infiltration Tests.** Does the infiltration testing method performed satisfy the minimum number of tests specified in Table D.3-2? ☐ Yes; continue to Step 1F. ☐ No; conduct appropriate number of tests.
1F	**Factor of Safety.** Is the suitable Factor of Safety selected for full infiltration design? See guidance in D.5; Tables D.5-1 and D.5-2; and Worksheet D.5-1 (Form I-9). ☐ Yes; continue to Step 1G. ☐ No; select appropriate factor of safety.
1G	**Full Infiltration Feasibility.** Is the average measured infiltration rate divided by the Factor of Safety greater than 0.5 inches per hour? ☐ Yes; answer "Yes" to Criteria 1 Result. ☐ No; answer "No" to Criteria 1 Result.
Criteria 1 Result	Is the estimated reliable infiltration rate greater than 0.5 inches per hour within the DMA where runoff can reasonably be routed to a BMP? ☐ Yes; the DMA may feasibly support full infiltration. Continue to Criteria 2. ☒ No; full infiltration is not required. Skip to Part 1 Result. Summarize infiltration testing methods, testing locations, replicates, and results and summarize estimates of reliable infiltration rates according to procedures outlined in D.5. Documentation should be included in project geotechnical report.
Summarize infiltration testing methods, testing locations, replicates, and results and summarize estimates of reliable infiltration rates according to procedures outlined in D.5. Documentation should be included in project geotechnical report.	
Site specific infiltration testing was performed on an immediately adjacent site, see GSI (2017) and yielded an average infiltration rate of 0.28 inches per hour.	

Categorization of Infiltration Feasibility Condition Based on Geotechnical Conditions		Worksheet C.4-1:Form I-8A[10]	
Criteria 2: Geologic/Geotechnical Screening			
2A	If all questions in Step 2A are answered "Yes," continue to Step 2B. For any "No" answer in Step 2A answer "No" to Criteria 2, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
2A-1	Can the proposed full infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick below the infiltrating surface?	☐Yes	☐No
2A-2	Can the proposed full infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	☐Yes	☐No
2A-3	Can the proposed full infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	☐Yes	☐No
2B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 2B are answered "Yes," then answer "Yes" to Criteria 2 Result. If there are "No" answers continue to Step 2C.		
2B-1	**Hydroconsolidation**. Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can full infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	☐Yes	☐No
2B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can full infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	☐Yes	☐No

Categorization of Infiltration Feasibility Condition Based on Geotechnical Conditions		Worksheet C.4-1:Form I-8A[10]	
2B-3	**Liquefaction.** If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011 or most recent edition). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can full infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	☐Yes	☐No
2B-4	**Slope Stability.** If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can full infiltration BMPs be proposed within the DMA without increasing slope stability risks?	☐Yes	☐No
2B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can full infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	☐Yes	☐No
2B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can full infiltration BMPs be proposed within the DMA using established setbacks from underground utilities, structures, and/or retaining walls?	☐Yes	☐No

Categorization of Infiltration Feasibility Condition Based on Geotechnical Conditions	Worksheet C.4-1:Form I-8A[10]		
2C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 2B. Provide a discussion of geologic/geotechnical hazards that would prevent full infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for full infiltration BMPs? If the question in Step 2 is answered "Yes," then answer "Yes" to Criteria 2 Result. If the question in Step 2C is answered "No," then answer "No" to Criteria 2 Result.	☐Yes	☐No
Criteria 2 Result	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	☐Yes	☐No
Summarize findings and basis; provide references to related reports or exhibits.			

Part 1 Result - Full Infiltration Geotechnical Screening[12]	Result
If answers to both Criteria 1 and Criteria 2 are "Yes", a full infiltration design is potentially feasible based on Geotechnical conditions only. If either answer to Criteria 1 or Criteria 2 is "No", a full infiltration design is not required.	☐ Full Infiltration Condition ☒ Complete Part 2

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

Categorization of Infiltration Feasibility Condition Based on Geotechnical Conditions		Worksheet C.4-1:Form I-8A[10]
Part 2 - Partial vs. No Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		**Project Phase:**
Location/limits of DMA undefined		Design Phase
Criteria 3: Infiltration Rate Screening		
3A	**NRCS Type C, D, or "urban/unclassified":** Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper is Type C, D, or "urban/unclassified" and corroborated by available site soil data? ☐ Yes; the site is mapped as C soils and a reliable infiltration rate of 0.15 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ☒ Yes; the site is mapped as D soils or "urban/unclassified" and a reliable infiltration rate of 0.05 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ☐ No; infiltration testing is conducted (refer to Table D.3-1), continue to Step 3B.	
3B	**Infiltration Testing Result:** Is the reliable infiltration rate (i.e. average measured infiltration rate/2) greater than 0.05 in/hr. and less than or equal to 0.5 in/hr? ☐ Yes; the site may support partial infiltration. Answer "Yes" to Criteria 3 Result. ☒ No; the reliable infiltration rate (i.e. average measured rate/2) is less than 0.05 in/hr., partial infiltration is not required. Answer "No" to Criteria 3 Result.	
Criteria 3 Result	Is the estimated reliable infiltration rate (i.e., average measured infiltration rate/2) greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour at any location within each DMA where runoff can reasonably be routed to a BMP? ☐ Yes; Continue to Criteria 4. ☒ No: Skip to Part 2 Result.	
Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate).		

Categorization of Infiltration Feasibility Condition Based on Geotechnical Conditions		Worksheet C.4-1:Form I-8A[10]	
Criteria 4: Geologic/Geotechnical Screening			
4A	If all questions in Step 4A are answered "Yes," continue to Step 2B. For any "No" answer in Step 4A answer "No" to Criteria 4 Result, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
4A-1	Can the proposed partial infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick?	☐Yes	☐No
4A-2	Can the proposed partial infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	☐Yes	☐No
4A-3	Can the proposed partial infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	☐Yes	☐No
4B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1 If all questions in Step 4B are answered "Yes," then answer "Yes" to Criteria 4 Result. If there are any "No" answers continue to Step 4C.		
4B-1	**Hydroconsolidation**. Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can partial infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	☐Yes	☐No
4B-2	**Expansive Soils**. Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can partial infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	☐Yes	☐No

Categorization of Infiltration Feasibility Condition Based on Geotechnical Conditions		Worksheet C.4-1:Form I-8A[10]	
4B-3	**Liquefaction.** If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can partial infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	☐Yes	☐No
4B-4	**Slope Stability.** If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can partial infiltration BMPs be proposed within the DMA without increasing slope stability risks?	☐Yes	☐No
4B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can partial infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	☐Yes	☐No
4B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can partial infiltration BMPs be proposed within the DMA using recommended setbacks from underground utilities, structures, and/or retaining walls?	☐Yes	☐No
4C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 4B. Provide a discussion on geologic/geotechnical hazards that would prevent partial infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for partial infiltration BMPs? If the question in Step 4C is answered "Yes," then answer "Yes" to Criteria 4 Result. If the question in Step 4C is answered "No," then answer "No" to Criteria 4 Result.	☐Yes	☐No

Categorization of Infiltration Feasibility Condition Based on Geotechnical Conditions		Worksheet C.4-1:Form I-8A[10]	
Criteria 4 result	Can infiltration of greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour be allowed without increasing the risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	☐Yes	☐No
Summarize findings and basis; provide references to related reports or exhibits.			
Part 2 Result - Partial Infiltration Geotechnical Screening[13]		**Result**	
If answers to both Criteria 3 and Criteria 4 are “Yes”, a partial infiltration design is potentially feasible based on Geotechnical conditions only. If either answer to Criteria 3 or Criteria 4 is “No”, then infiltration of any volume is considered to be infeasible within the site.		☐ Partial Infiltration Condition ☒ No Infiltration Condition	

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

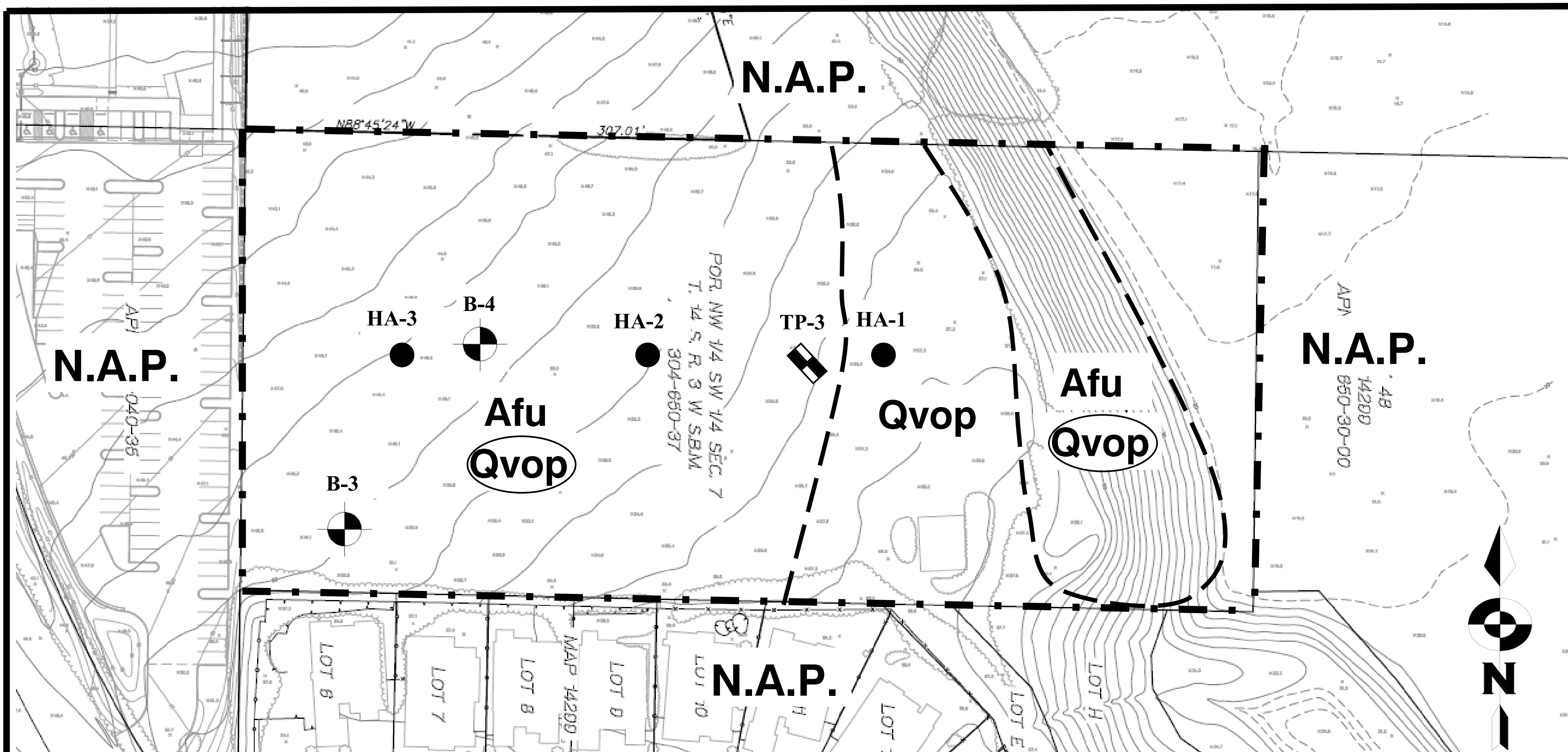


Figure Adapted from Sheet 4 of Leppart Engineering, 2011

GSI LEGEND

Afu Artificial Fill (Uncontrolled)

Qvop Quaternary Very Old Paralic Deposits (Bedrock), Circled Where Buried

Approximate Location of Geologic Contact

B-4 Approximate Location of Exploratory Boring By Geocon (2008)

TP-3 Approximate Location of Test Pit By GSI (2011)

HA-3 Approximate Location of Hand Auger Boring (This Study)

Area Under The Purview Of This Report

N.A.P. Not A Part Of This Study

50 0 50 100

GeoSoils, Inc.

PLATE 1

GEOTECHNICAL MAP

W.O. 7971-A-SC	DATE: 9/20	SEE BAR SCALE