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All Peoples Church
c/o Hamann Companies
1000 Pioneer Way
El Cajon, CA 92020

January 20, 2020
P/W 1805-05
Report No. 1805-05-B-3

Attention: **Mrs. Linda Richardson**

Subject: **Updated Preliminary Geotechnical Investigation and Design Recommendations, Proposed Church Facility, APN 463-010-1000, San Diego, California 90212**

Gentlemen:

In accordance with your request, presented herein are the results of Advanced Geotechnical Solutions, Inc.'s (AGS) updated preliminary geotechnical investigation and design recommendations for the proposed church development northeast of the intersection of College Avenue and Interstate 8, in the City of San Diego, California. It is our understanding that the site will be graded to support a church, a parking structure and associated improvements.

AGS appreciates the opportunity to provide you with geotechnical consulting services and professional opinions. If you have questions regarding this report, please contact the undersigned at (619) 850-3980.

Respectfully Submitted,
Advanced Geotechnical Solutions, Inc.

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ATTACHMENTS:

UPDATED PRELIMINARY GEOTECHNICAL INVESTIGATION PROPOSED CHURCH AND PARKING STRUCTURES COLLEGE AVENUE AND INTERSTATE 8 SAN DIEGO, CALIFORNIA

1.0 INTRODUCTION

1.1. Purpose and Background

This study is aimed at providing geologic and geotechnical information and recommendations for the development of the proposed church and parking structure. This report has been prepared in a manner consistent with City of San Diego geotechnical report guidelines and the current standard of practice.

1.2. Scope of Work

The scope of our preliminary geotechnical investigation consisted of the following tasks:

- Review readily available geologic maps, literature, aerial photographs, and previous geotechnical studies (Appendix A);
- Compile previous subsurface data (Appendix B) and laboratory test results (Appendix C);
- Prepare a geotechnical/geologic map depicting exploratory locations, approximate distribution of geologic units onsite, and proposed improvements (Plate 1);
- Prepare geologic cross-sections A-A’ and B-B’ depicting underlying geology, existing and proposed conditions (Plate 2);
- Evaluate groundwater conditions and potential effects on construction;
- Analyze and discuss excavation characteristics (i.e. rippability) of onsite materials, earthwork recommendations, unsuitable soil removals, and compaction criteria for use of on-site earth materials as compacted fill for the proposed development;
- Provide seismic design parameters in accordance with 2019 California Building Code;
- Provide foundation design recommendations based upon anticipated site geotechnical conditions.
- Prepare preliminary foundation and retaining wall design parameters and recommendations;
- Evaluate the impacts of the proposed improvements and excavations on adjacent improvements; and,
- Summarize this data in a report suitable for design, bidding and regulatory review..

2.0 REPORT LIMITATIONS

The conclusions and recommendations in this report are based on the data developed during our previous investigation at the site and a review of readily available geologic and geotechnical information. The materials immediately adjacent to, or beneath those observed in the exploratory excavations may have different characteristics and no representations are made as to the quality or extent of materials not observed. The recommendations presented herein are specific to the development as reflected on the current grading plan. Modifications to the design or development plans could necessitate revisions to these recommendations.

3.0 SITE LOCATION AND DESCRIPTION

The site consists of an approximately 9.2-acre L-shaped parcel located northeast of the intersection of College Avenue and Interstate 8 West, in the City of San Diego, California (see Figure 1, Site Location Map). The site is currently vacant, supporting a light growth of seasonal grasses, shrubs, and small trees. Access to the site is via northbound College Avenue. The site topography generally slopes down toward the southwest. Approximate elevations range from 450 feet above mean sea level (msl) at the northerly limits to 356 feet msl at the southwest corner of the site. Ascending slopes up to approximately 25 feet in height are present along the westerly/northwesterly property boundary adjacent to College Avenue. Existing slopes descend to a minor drainage basin at the southwesterly corner of the site.

4.0 PREVIOUS DEVELOPMENT

As part of our preliminary investigation several historic aerial photos and topographic maps of the project area were reviewed by representatives of AGS. Based on our review it was determined that the site was previously graded to its current configuration. This grading was likely accomplished in multiple phases. The first phase of grading appears to have occurred in the late 1950's to early 1960's during construction of the residential development superjacent to the east, College Avenue to the west, Interstate 8 (previously Highway 80) and associated College Avenue off ramp to the south and southwest.

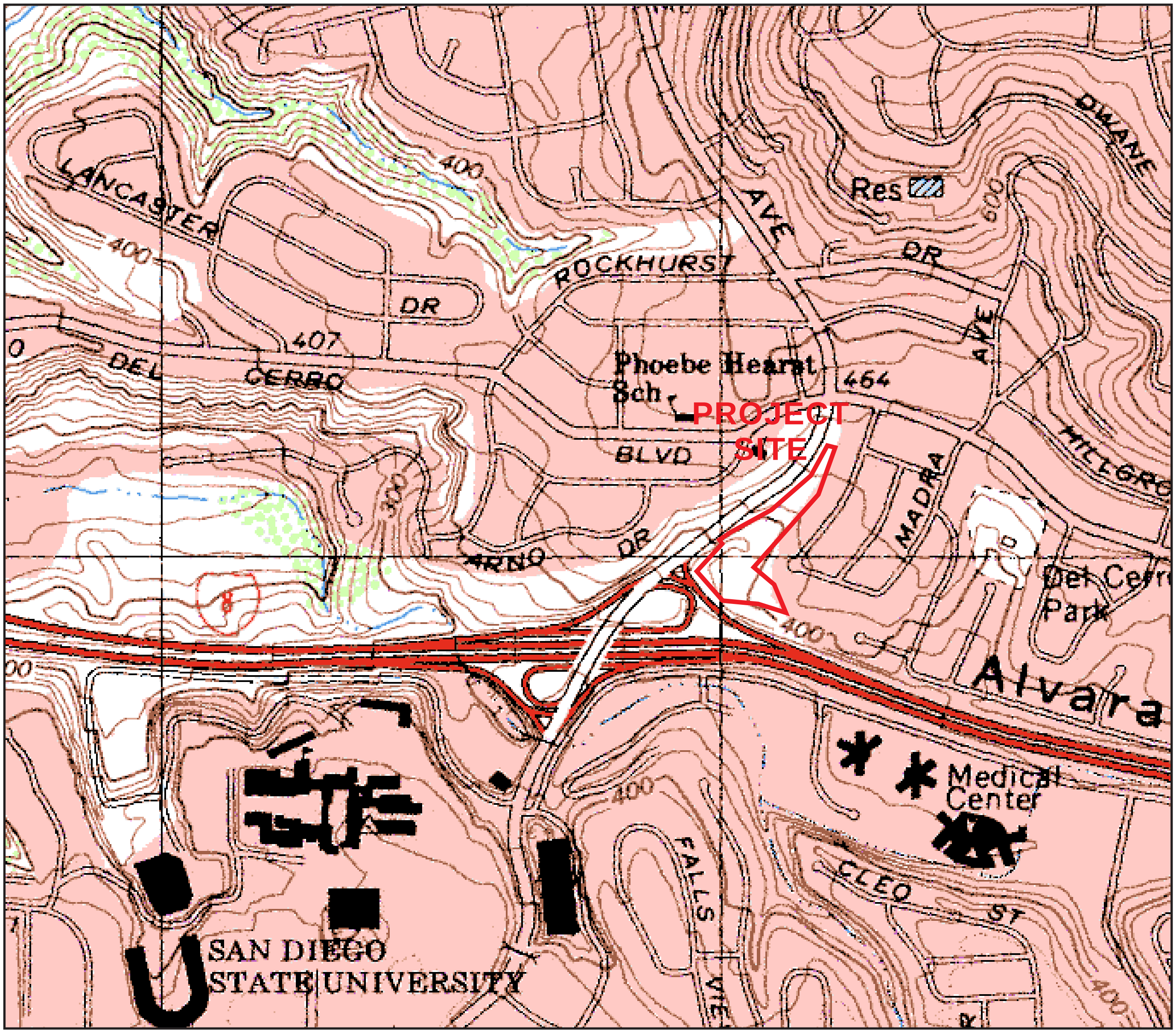
Pre-development photos show a moderate sized drainage trending southwest through the approximate central portion of the site. Minor modifications to this drainage occurred during the first phase of grading activities. Subsequently, a second phase of grading appears to have occurred in the mid- to late-1960's. During this phase, the drainage appears to have been filled and a level pad constructed in the southwest portion of the site with graded slopes descending the west and southwest. Based on our review of historic photos and topographic maps it is anticipated that fills on the order of 20 to 30 feet deep were placed in the southwesterly portion of the site. The fill materials placed during this second phase of grading may have been derived from the residential development to the southeast (Del Cerro Court).

5.0 PROPOSED DEVELOPMENT

Based on our review of the 40-scale preliminary grading plan for All Peoples Church prepared by Pasco Laret Suiter & Associates (PLSA) dated January 20, 2020, it is our understanding that the subject site will be graded to support a nearly 37,000 square-foot church structure to the west, a two-level parking structure in the central portion of the site, paved driveways and parking areas, and several retaining walls and slopes. It is anticipated that the church will consist of a two- to three-story concrete and/or steel frame structure supported by a shallow slab-on-grade foundation system. The two-level parking garage is anticipated to be a concrete structure supported by a shallow slab-on-grade foundation system.

6.0 SUBSURFACE INVESTIGATION

As part of our previous investigation at the site, AGS excavated and logged ten (10) exploratory test pits in December 2014. The test pits were excavated with a Caterpillar 328D tracked excavator equipped with a two-foot bucket. The exploratory test pits extended to a maximum depth of 27 feet below existing grade. In addition, four (4) borehole percolation tests (P-1 through P-4) were performed evaluate the feasibility of storm water infiltration at the site. The approximate locations of the test pits, percolation test borings, interpreted geology and geologic contacts have been plotted on the 40-scale preliminary grading plan prepared by PLSA and are presented in the attached Plate 1, Geologic Map and Exploration Location Plan.



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SITE LOCATION MAP
PROPOSED ALL PEOPLES CHURCH
COLLEGE AVENUE AND INTERSTATE 8
SAN DIEGO, CALIFORNIA

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FIGURE 1

SOURCE MAP - TOPOGRAPHIC MAP OF THE
LA MESA 7.5 MINUTE QUADRANGLE,
SAN DIEGO COUNTY, CALIFORNIA

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7.0 ENGINEERING GEOLOGY

7.1. Regional Geologic and Geomorphic Setting

The subject site is situated within the Peninsular Ranges Geomorphic Province. The Peninsular Ranges province occupies the southwestern portion of California and extends southward to the southern tip of Baja California. In general, the province consists of young, steeply sloped, northwest trending mountain ranges underlain by metamorphosed Late Jurassic to Early Cretaceous-aged extrusive volcanic rock and Cretaceous-aged igneous plutonic rock of the Peninsular Ranges Batholith. The westernmost portion of the province, where the subject site is located, is predominantly underlain by younger marine and non-marine sedimentary rocks. The Peninsular Ranges' dominant structural feature is northwest-southeast trending crustal blocks bounded by active faults of the San Andreas transform system.

7.2. Site Geology

A majority of the site is mantled with pre-existing undocumented fill soils. The undocumented fill is locally underlain by young alluvium and older alluvium where a pre-development drainage was filled in. The fill and alluvial soils are underlain to maximum depths explored by Tertiary-aged Stadium Conglomerate and Cretaceous-age Santiago Peak Volcanics (see Figure 2, Regional Geologic Map). A brief description of the earth materials encountered on this site is presented in the following sections. More detailed description of these materials is provided in the test pit logs included in Appendix B.

7.2.1. Artificial Fill - Undocumented (Map Symbol afu)

The site is mantled with undocumented fill soils ranging from 2 to 22 feet in thickness. As encountered, these materials generally consist of fine to coarse grained sand and silty sand with abundant cobbles and some boulders up to 4 feet in diameter. These materials were observed to be slightly moist to very moist in a loose to medium dense condition. Buried trash and construction debris were encountered in test pit EX-9. An area of large hard rock boulders (shot rock) up to 8 feet in diameter is exposed at the surface in the central portion of the site in proximity to the proposed parking structure location.

7.2.2. Young Alluvium (Qal)

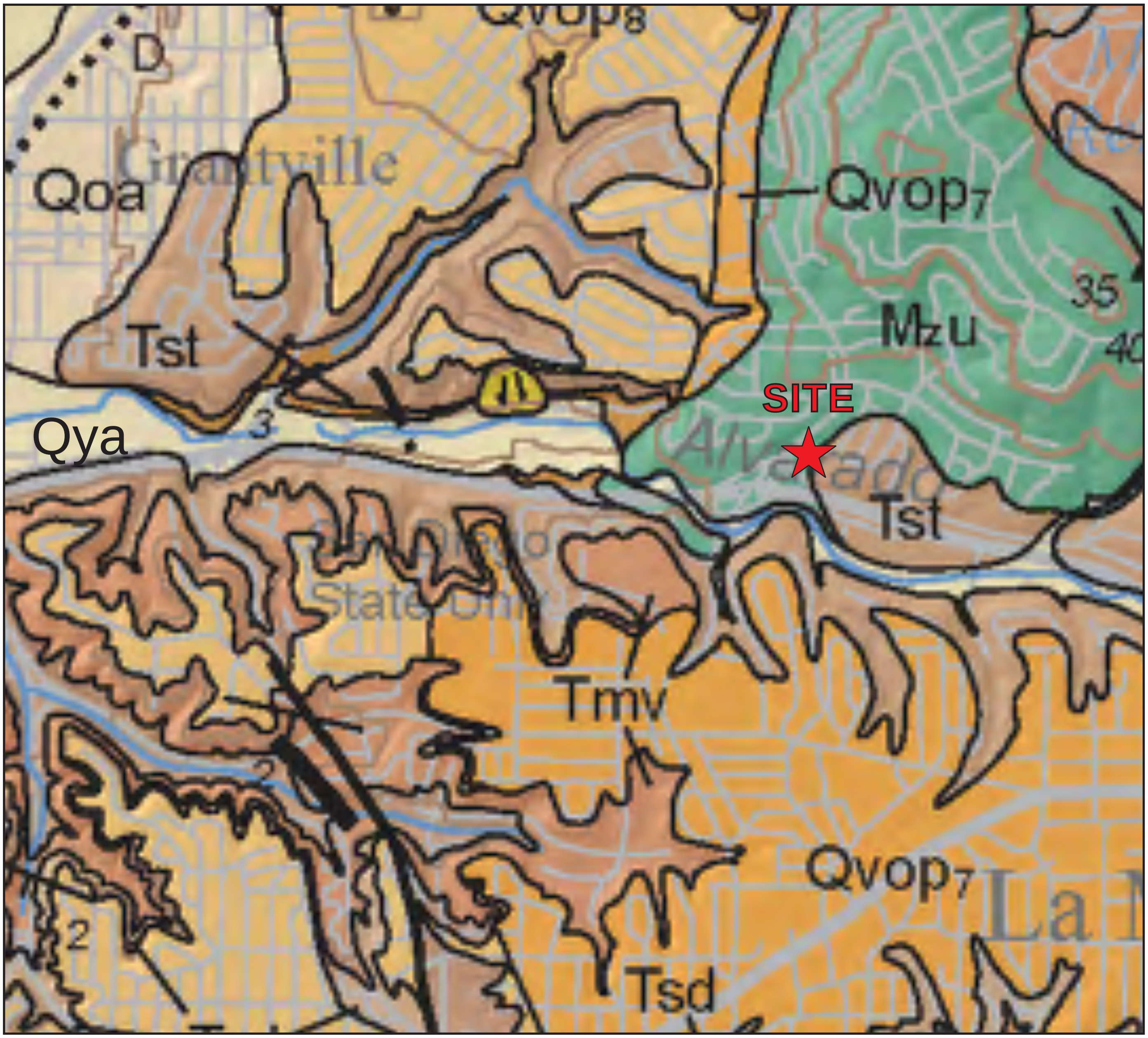
Young alluvium was encountered underlying undocumented fill in test pits EX-3 and EX-5 at 21 to 22 feet below ground surface. The alluvium encountered ranged from a few feet to as much as 4 feet thick. As encountered these materials generally consist of brown to gray, clayey silt with sand and gravel, in a very moist and firm to stiff condition.

7.2.3. Older Alluvium (Map Symbol Qoa)

Older Alluvium was encountered in test pits EX-4 through EX-7. As encountered these materials generally consist of fine-grained, yellow silty sand with silty clay lenses in a slightly moist to moist and moderately dense to dense condition.

7.2.4. Stadium Conglomerate (Map Symbol Tst)

Tertiary aged Stadium Conglomerate was encountered in test pits EX-1, EX-8 and EX-9 below undocumented fill. As encountered, these materials consist of moderately hard, cobble



REGIONAL GEOLOGIC MAP
PROPOSED ALL PEOPLES CHURCH
COLLEGE AVENUE AND INTERSTATE 8
SAN DIEGO, CALIFORNIA

Qya	Young Alluvium
Qoa	Older Alluvium
Tst	Stadium Conglomerate (Middle Eocene)
Mzu	Metamorphosed and Unmetamorphosed Volcanic and Sedimentary Rocks, Undivided (Mesozoic)

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FIGURE 2

SOURCE MAP(S):Geologic Map of the San Diego 30' x 60'Quadrangle, California, Kennedy and Tan 2008

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conglomerate, in a brownish yellow, silty sandstone matrix. Cobbles were generally on the order of 3 to 6 inches in diameter and composed of rounded volcanic 'Poway' clasts.

7.2.5. Santiago Peak Volcanics (Map Symbol Jsp)

As encountered, this unit can generally be described as moderately to slightly weathered, moderately hard to hard, metavolcanic bedrock that is reddish brown to brownish yellow on weathered surfaces, and gray on fresh surfaces.

7.3. Geologic Structure

The Stadium Conglomerate non-conformably overlies the basement rocks of the Santiago Peak Volcanics and appears to be confined to the easterly portion of the site. Based on review of historic aerial photos, the original surface contact between the Stadium Conglomerate and Santiago Peak Volcanics appears to coincide with the pre-development drainage that transected the site in a roughly northeast to southwest direction. The Stadium Conglomerate is massively bedded and is anticipated to be near horizontal to very slightly dipping to the west in line with the overall regional dip.

7.4. Groundwater

Groundwater was not encountered to the depths explored at the site. Minor seepage was observed in EX-2 at the fill and bedrock contact. No other natural groundwater condition is known to exist at the site that would impact the proposed site development. However, it should be noted that localized perched groundwater may develop at a later date, most likely at or near fill/bedrock contacts, due to fluctuations in precipitation, irrigation practices, or factors not evident at the time of our field exploration.

7.5. Faulting and Seismicity

The site is located in the tectonically active Southern California area, and will therefore likely experience shaking effects from earthquakes. The type and severity of seismic hazards affecting the site are to a large degree dependent upon the distance to the causative fault, the intensity of the seismic event, and the underlying soil characteristics. The seismic hazard may be primary, such as surface rupture and/or ground shaking, or secondary, such as liquefaction or dynamic settlement. The following is a site-specific discussion of ground motion parameters, earthquake-induced landslide hazards, settlement, and liquefaction. The purpose of this analysis is to identify potential seismic hazards and propose mitigations, if necessary, to reduce the hazard to an acceptable level of risk. The following seismic hazards discussion is guided by the California Building Code (2019), CDMG (2008), and Martin and Lew (1998).

7.5.1. Surface Fault Rupture

No known active faults have been mapped at or near the subject site. The nearest known active surface fault is the Silver Strand section of the Newport-Inglewood-Rose Canyon fault zone, located approximately 7.1 miles southwest of the site. Accordingly, the potential for fault surface rupture on the subject site is considered very low to remote. This conclusion is based on our literature and map review.

7.5.2. Seismicity

As noted, the site is within the tectonically active southern California area, and is approximately 7 miles from an active fault. Given the proximity of the site to the nearest active fault the potential exists for strong ground motion that may affect future improvements.

At this point in time, non-critical structures (commercial, residential, and industrial) are designed according to the California Building Code (2019) and the requirements of the controlling local agency.

7.5.3. Liquefaction

Given the dense nature of the formational materials underlying the site, the proposed remedial grading as recommended herein, and the lack of a shallow groundwater table at the project site, the potential for seismically induced liquefaction is considered remote.

7.5.4. Dynamic Settlement

Dynamic settlement occurs in response to an earthquake event in loose sandy earth materials. The potential of dynamic settlement at the subject site is considered to be remote due to the presence of well consolidated/indurated formational materials underlying the site and the proposed removal of loose, sandy soils as recommended herein.

7.5.5. Seismically Induced Landsliding

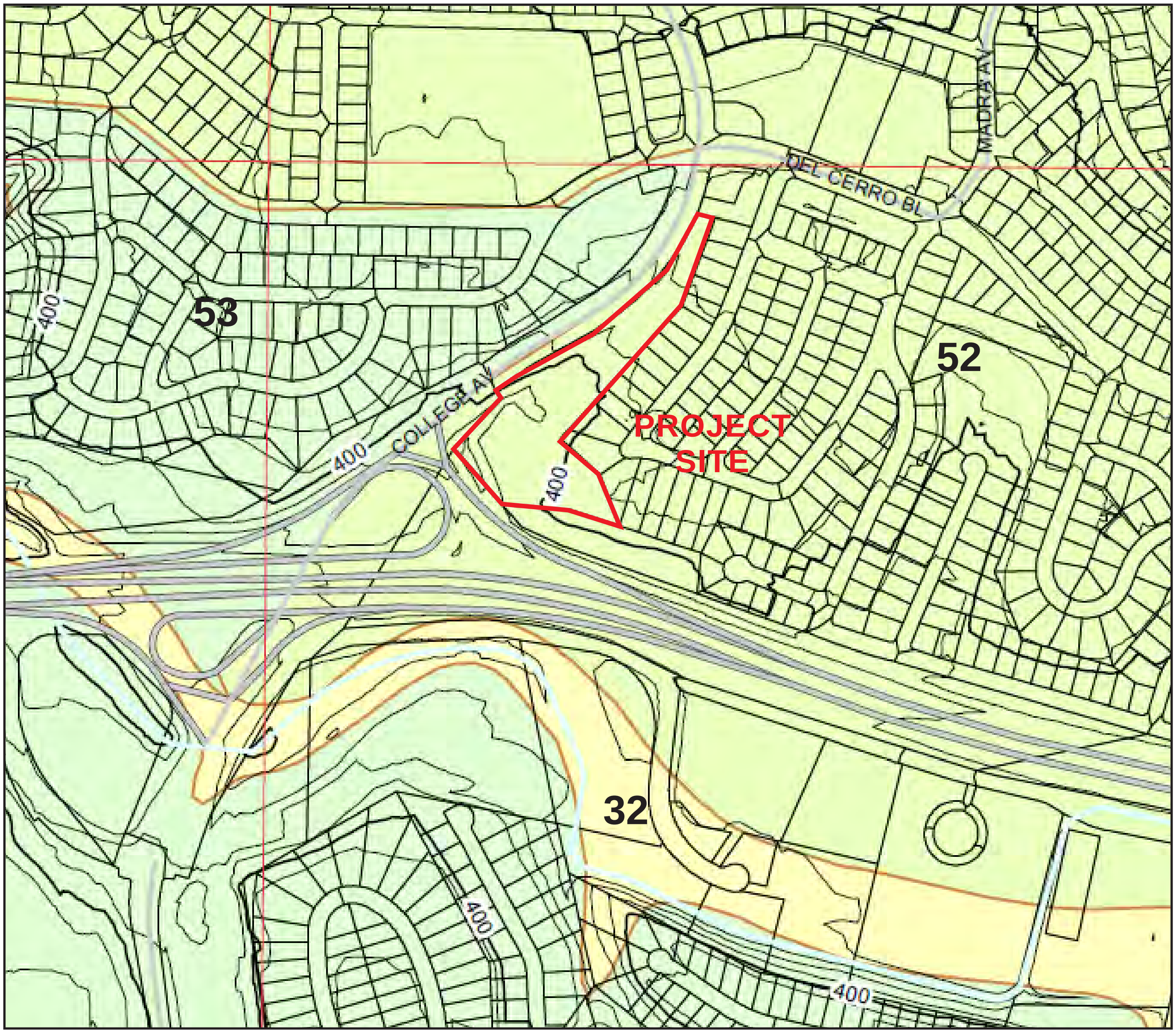
Evidence of landsliding at the site was not observed during our field observations, nor are there any geomorphic features indicative of landsliding noted in our review of published geologic maps. The nearest known landslide is approximately ¾-mile west of the project and developed within exposures of Friars Formation. If the recommendations provided in this report are followed, the likelihood for seismically induced landsliding is considered to be remote.

7.5.6. City of San Diego Seismic Safety Study

As indicated in Figure 3 (excerpted from the San Diego Seismic Safety Study Grid Tile 22), the site is mapped under Geologic Hazard Category 52: Other level areas, gently sloping to steep terrain, favorable geologic structure, Low risk.

7.6. <u>Seismic Design Parameters</u>

Based on our subsurface exploration, the site has been classified as Seismic Site Class D - Default consisting of a stiff soil profile with average SPT N blowcount between 15 and 50 blows per foot and assumed Vs30 of 270 m/s. Table 7.5.7 presents seismic design parameters in accordance with 2019 CBC and mapped spectral acceleration parameters (United States Geological Survey, 2019) utilizing site coordinates of Latitude 32.7805°N and Longitude 117.0640°W. The seismic provisions of the 2019 CBC are significantly different from the previous version and require a site-specific seismic hazard analysis (SHA) for most sites located on Site Class D and E soil conditions which was performed as described in Section 7.7.



CITY OF SAN DIEGO
SEISMIC SAFETY STUDY
PROPOSED ALL PEOPLES CHURCH
COLLEGE AVENUE AND INTERSTATE 8
SAN DIEGO, CALIFORNIA

- 32 Low Potential (liquefaction) – fluctuating groundwater minor drainages.
- 52 Other level areas, gently sloping to steep terrain, favorable geologic structure, Low risk.
- 53 Level or sloping Terrain, unfavorable geologic structure, Low to moderate risk.

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FIGURE 3

SOURCE MAP - CITY OF SAN DIEGO
SEISMIC SAFETY STUDY, GRID TILE 22
DATED APRIL 3, 2008

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TABLE 7.5.7 - 2019 CBC SEISMIC DESIGN PARAMETERS (SITE CLASS D)	
Mapped Spectral Acceleration Parameter at Period of 0.2-Second, S_s	0.889g
Mapped Spectral Acceleration Parameter at Period 1-Second, S_1	0.316g
Site Coefficient, F_a	1.200
Site Coefficient, F_v	N/A[3]
Adjusted MCE_R[1] Spectral Response Acceleration Parameter at Short Period, S_{MS}	1.067g
1-Second Period Adjusted MCE_R[1] Spectral Response Acceleration Parameter, S_{M1}	N/A[3]
Short Period Design Spectral Response Acceleration Parameter, S_{DS}	0.711g
1-Second Period Design Spectral Response Acceleration Parameter, S_{D1}	N/A[3]
Peak Ground Acceleration, PGA_M[2]	0.470g
Seismic Design Category	N/A[3]

Notes: [1] Risk-Targeted Maximum Considered Earthquake
[2] Peak Ground Acceleration adjusted for site effects
[3] Requires Site Specific Ground Motion Hazard Analysis per ASCE 7-16 Section 11.4.8

7.7. Site Specific Ground Motion Hazard Analysis

The site-specific ground motion hazard analysis was performed in accordance with Section 21.1 of ASCE Standard 7-16. Probabilistic and deterministic maximum considered earthquake (MCE) response accelerations were evaluated in order to develop the site-specific design response spectrum. The derivation of the site-specific design response spectra, including the probabilistic and deterministic seismic hazard analyses, are presented in Figure 4, Site-Specific Design Response Spectrum. The detailed analyses and results are described below.

7.7.1. Probabilistic Seismic Hazard Analysis

A site-specific probabilistic seismic hazard analysis was performed to evaluate the spectral response accelerations represented by a 5-percent-damped acceleration response spectrum having a 2 percent probability of exceedance within a 50-year period. The probabilistic seismic hazard analysis was performed using the Java program OpenSHA (http://www.OpenSHA.org), developed jointly by the Southern California Earthquake Center (SCEC) and the United States Geological Survey (USGS). The probabilistic seismic hazard analyses used the next generation attenuation (NGA) relationships by Abrahamson, Silva & Kamai (2014); Boore, Stewart, Seyhan & Atkinson (2014); Campbell and Bozorgnia (2014) and Chiou and Youngs (2014). The resulting median geometric-mean acceleration response spectra were used to create a probabilistic response spectrum based on the average spectral acceleration at each period, and then converted into maximum rotated components of ground motion using applicable scale factors.

7.7.2. Deterministic Seismic Hazard Analysis

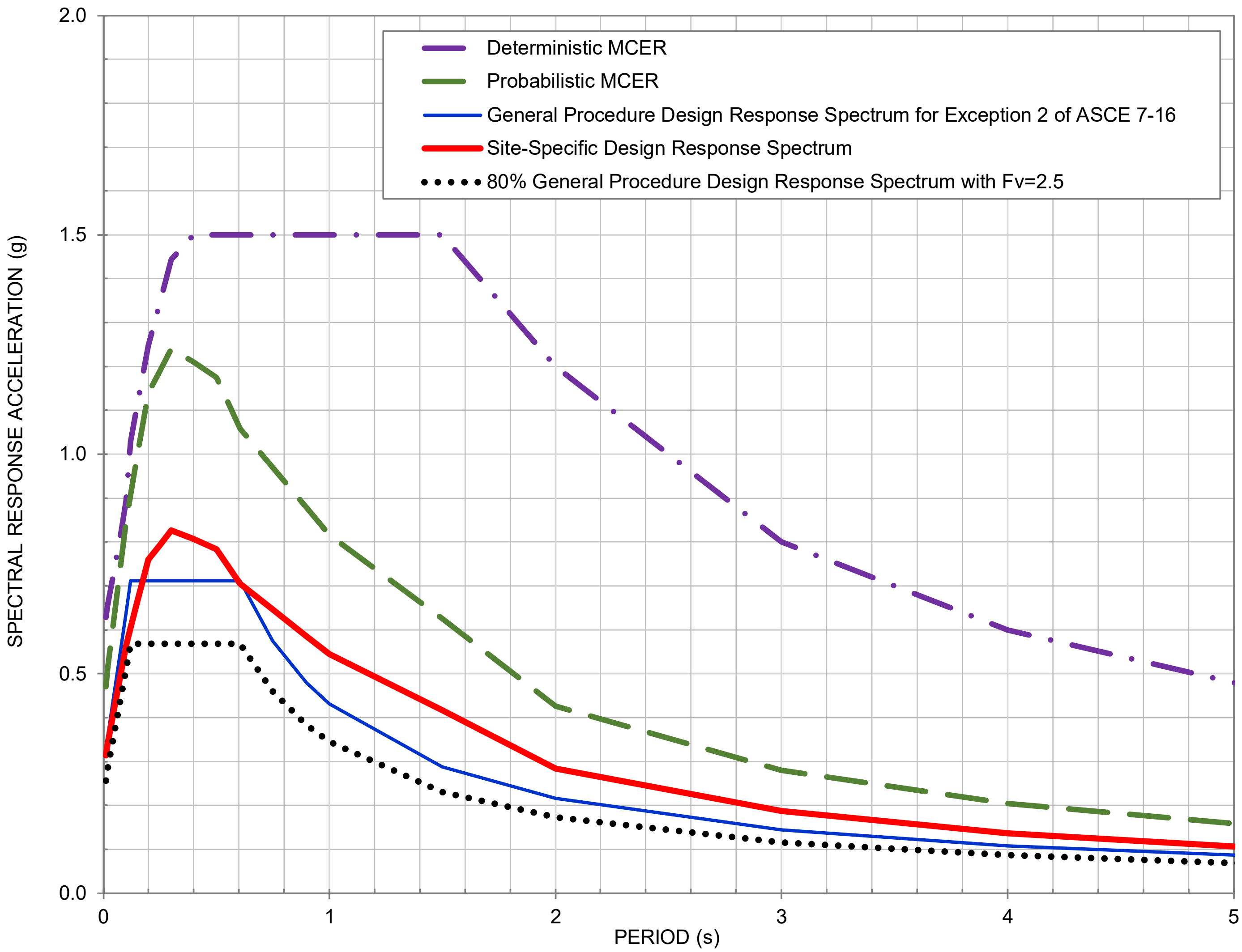
A site-specific deterministic seismic hazard analysis was performed to evaluate the MCE response acceleration. The deterministic MCE response acceleration at specified periods was calculated as the 84th percentile of the maximum rotated component of ground motion computed at each period for characteristic earthquakes on known active faults within the region. Initially we performed an evaluation of potentially damaging earthquake sources by reviewing published geologic maps and sources that contribute to the probabilistic hazard analysis, according to the deaggregation results obtained using the USGS unified hazard tool website (https://earthquake.usgs.gov/hazards/ interactive/). Based on our evaluation, we selected three "controlling" sources and seismic events: the Rose Canyon (Silver Strand section-Downtown Graben fault), Rose Canyon (San Diego section) and Rose Canyon (Silver Strand section) faults. Subsequently we used the NGA Models by Abrahamson, Silva & Kamai (2014); Boore, Stewart, Seyhan & Atkinson (2014); Campbell and Bozorgnia (2014) and Chiou and Youngs (2014) to estimate the ground motion distribution for each earthquake. The 5-percent-damped pseudo-absolute acceleration response spectrum was calculated for each earthquake using an Excel spreadsheet issued by the Pacific Earthquake Engineering Research Center (http://peer.berkeley.edu/ngawest2/ databases/). Earthquake source and site characteristic parameters were evaluated using the California Geological Survey earthquake source database and the CalTrans ARS Online web-based tool (http://dap3.dot.ca.gov/ARS_Online). Distances to faults were evaluated using the USGS unified hazard tool website. The resulting median geometric-mean acceleration response spectra were used to create a deterministic MCE response spectrum based on the greatest spectral acceleration at each period, and then converted into maximum rotated components of ground motion using applicable scale factors. The final deterministic spectral response accelerations were taken to be not lower than the deterministic lower limit as calculated using Figure 21.2-1 of ASCE 7-16, Chapter 21.

7.7.3. Site-Specific Design Response Spectrum

The site-specific MCER spectral response acceleration was calculated at each period to be the lesser of the spectral response accelerations from the probabilistic and deterministic MCE. Finally, the design spectral response acceleration at each period was calculated as two-thirds of the site-specific MCE spectral response acceleration, but not less than 80 percent of the spectral response acceleration evaluated in accordance with Section 11.4.5 of ASCE 7-16. In order to calculate the 80 percent lower limit, mapped values from USGS Seismic Design Maps (http://earthquake.usgs.gov/designmaps/us) were used to calculate SDS, SD1 and the design spectrum in accordance with Section 21.4 of ASCE 7-16. Applicable response spectra data are presented in Table 7.7.3A and on Figure 4, Site-Specific Design Response Spectrum.

PERIOD (s)	SITE-SPECIFIC DESIGN SPECTRAL ACCELERATION Sa, (g)
0.01	0.314
0.02	0.341
0.03	0.369
0.05	0.424
0.075	0.494
0.10	0.563
0.121	0.605
0.15	0.661
0.2	0.759
0.25	0.793
0.3	0.827

PERIOD (s)	SITE-SPECIFIC DESIGN SPECTRAL ACCELERATION Sa, (g)
0.4	0.806
0.5	0.784
0.606	0.705
0.8	0.647
0.9	0.586
1.0	0.544
1.5	0.417
2.0	0.284
3.0	0.187
4.0	0.136
5.0	0.106



Note: See Table 7.7.3A of the report for values of the various curves.

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SITE-SPECIFIC DESIGN RESPONSE SPECTRUM

ALL PEOPLES CHURCH
COLLEGE AVENUE AND INTERSTATE 8
SAN DIEGO, CALIFORNIA

PROJECT NO.	DATE	
1805-05	01/20	**FIGURE 4**

TABLE 7.7.3A
SITE-SPECIFIC DESIGN RESPONSE SPECTRUM DATA

Period (sec)	General Procedure Design Response Spectrum for Exception 2 of ASCE 7-16	Risk Coeff. C_R	Site-Specific Ground Motion Analysis Spectral Accelerations (g)							
			Maximum direction 2%-in-50-yr Probabilistic Spectrum	Probabilistic MCE_R	Maximum direction 84th-percentile Deterministic Spectrum	Deterministic Lower Limit	Deterministic MCE_R	Site Specific MCE_R	80% General Procedure Design Response Spectrum with Fv=2.5	Site-Specific Design Response Spectrum
0.01	0.320	0.896	0.525	0.470	0.544	0.628	0.628	0.470	0.256	0.314
0.02	0.355	0.896	0.571	0.512	0.545	0.656	0.656	0.512	0.284	0.341
0.03	0.390	0.896	0.618	0.553	0.555	0.684	0.684	0.553	0.312	0.369
0.05	0.461	0.896	0.710	0.637	0.622	0.741	0.741	0.637	0.368	0.424
0.075	0.549	0.896	0.826	0.740	0.756	0.811	0.811	0.740	0.439	0.494
0.1	0.637	0.896	0.942	0.844	0.893	0.881	0.893	0.844	0.509	0.563
0.1212	0.711	0.896	1.012	0.907	1.029	0.941	1.029	0.907	0.569	0.605
0.15	0.711	0.896	1.107	0.992	1.109	1.022	1.109	0.992	0.569	0.661
0.2	0.711	0.896	1.271	1.139	1.247	1.163	1.247	1.139	0.569	0.759
0.25	0.711	0.897	1.326	1.189	1.345	1.303	1.345	1.189	0.569	0.793
0.3	0.711	0.898	1.382	1.241	1.407	1.444	1.444	1.241	0.569	0.827
0.4	0.711	0.900	1.344	1.210	1.415	1.500	1.500	1.210	0.569	0.806
0.5	0.711	0.902	1.304	1.175	1.363	1.500	1.500	1.175	0.569	0.784
0.6061	0.711	0.904	1.171	1.058	1.229	1.500	1.500	1.058	0.569	0.705
0.75	0.575	0.906	1.071	0.970	1.135	1.500	1.500	0.970	0.460	0.647
0.9	0.479	0.909	0.966	0.878	1.036	1.500	1.500	0.878	0.383	0.586
1	0.431	0.911	0.896	0.817	0.971	1.500	1.500	0.817	0.345	0.544
1.5	0.287	0.911	0.687	0.625	0.670	1.500	1.500	0.625	0.230	0.417
2	0.216	0.911	0.468	0.426	0.495	1.200	1.200	0.426	0.172	0.284
3	0.144	0.911	0.308	0.280	0.314	0.800	0.800	0.280	0.115	0.187
4	0.108	0.911	0.223	0.204	0.211	0.600	0.600	0.204	0.086	0.136
5	0.086	0.911	0.174	0.159	0.150	0.480	0.480	0.159	0.069	0.106

The site-specific design response parameters are provided in Table 7.7.3B. These parameters were evaluated from Design Response Spectra values presented above in accordance with ASCE 7-16 Section 21.4 guidelines.

TABLE 7.7.3B
SITE-SPECIFIC SEISMIC DESIGN PARAMETERS

Spectral Response Acceleration 0.2-second period, S_{MS}	1.117g
Spectral Response Acceleration 1-second period, S_{M1}	0.938g
Design Spectral Response Acceleration for short period, S_{DS}	0.745g
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.625g
MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA_M	0.477g

7.8. Non-seismic Geologic Hazards

7.8.1. Mass Wasting

No evidence of mass wasting was observed onsite nor was any noted on the reviewed maps.

7.8.2. Flooding

According to available FEMA maps, the site is not in a FEMA identified flood hazard area.

7.8.3. Subsidence/Ground Fissuring

Due to the presence of the dense underlying materials, the potential for subsidence and ground fissuring due to settlement is unlikely.

8.0 GEOTECHNICAL ENGINEERING

Presented herein is a general discussion of the geotechnical properties of the various soil types and the analytical methods used in this report.

8.1. Soil Characteristics

The materials found in the area of the proposed improvements consist primarily of previously placed undocumented fill soils. Once the planned removals of unsuitable soils (artificial fill, young alluvium, and weathered older alluvium/bedrock) are completed, the proposed structures will be founded upon compacted fill overlying competent Older Alluvium, Stadium Conglomerate, or Santiago Peak Volcanics. In general, these materials exhibit favorable engineering characteristics. Descriptions of the units encountered/anticipated on site can be found in the test pit logs in Appendix B.

8.2. Excavation Characteristics

The onsite soils within the anticipated cut depths are anticipated to be excavatable with conventional grading equipment. Excavations in the cobble rich lenses may necessitate moderate to heavy ripping to efficiently advance. Excavations for deeper utilities and excavations encountering large boulders may require trackhoes. In the southeasterly portion of the site, design cuts are currently proposed to depths on the order of 25 feet below existing grade. Moderately hard to very hard, metavolcanic bedrock will likely be encountered necessitating the use of specialized grading techniques (large excavators with hoe rams, large bulldozers and possibly blasting) to accomplish site grading and overexcavation requirements as outlined in this document.

8.3. Compressibility

Onsite materials that are significantly compressible in their current condition include topsoil, undocumented fill materials, young alluvium, and weathered older alluvium. These materials will require complete removal prior to placement of fill, and where exposed at design grade. Compressibility of unweathered older alluvium, Stadium Conglomerate, and Santiago Peak Volcanics is not a geotechnical design concern for the proposed structures. If the recommended removals are not possible in certain areas due to property line constraints, the improvements in those areas should be designed for increased total and differential settlement potential.

8.4. Collapse Potential/Hydro-Consolidation

Given the removal recommendations presented herein and the age and density of the Older Alluvium, Stadium Conglomerate, and Santiago Peak Volcanics, the potential for hydro-consolidation is considered remote at the subject site.

8.5. Expansion Potential

In general, the onsite soils consist of silty sands with abundant cobbles and some boulders. Minor clayey/silty soils were identified during our subsurface investigation. We anticipate onsite soils will exhibit "Very Low" to "Medium" expansion potential with the majority being in the "Low" range. Final determination of expansion potential for foundation design purposes should be based on testing of the as-graded soil conditions.

8.6. Shear Strength Characteristics

Shear strength testing was not conducted as part of this investigation. Based upon our previous experience with similar soils in the vicinity of the project area, the shear strength parameters presented in Table 8.6 are recommended for compacted fill, Older Alluvium and the bedrock units observed onsite.

TABLE 8.6
SHEAR STRENGTH PARAMETERS

Material	***Cohesion (psf)***	***Friction Angle (degrees)***
Compacted Artificial Fill	150	31
Older Alluvium	100	32
Bedrock (Tst and Jsp)	400	36

8.7. Earthwork Adjustments

The following Table 8.7 presents bulk/shrink values of the various onsite soils for use in estimating earthwork grading quantities.

TABLE 8.7
SHRINK/BULK PARAMETERS

Artificial Fill	Shrink 5-10%
Young Alluvium	Shrink 6-10%
Older Alluvium	Bulk 2-5%
Stadium Conglomerate	Bulk 5-10%
Santiago Peak Volcanics (Rippable)	Bulk 12-18%
Santiago Peak Volcanics (Non-Rippable)	Bulk 18-25%

These values may be used in an effort to balance the earthwork quantities. As is the case with every project, contingencies should be made to adjust the earthwork balance when grading is in progress and actual conditions are better defined.

8.8. Bearing Capacity and Lateral Earth Pressures

Ultimate bearing capacity values were obtained using the graphs and formulas presented in *NAVFAC DM-7.1*. Allowable bearing was determined by applying a factor of safety of at least three (3) to the ultimate bearing capacity.

Static lateral earth pressures were calculated using *Rankine* methods for active and passive cases. If it is desired to use *Coulomb* forces, a separate analysis specific to the application can be conducted.

8.9. Chemical/Resistivity Analyses

Laboratory testing for sulfates, chlorides, and soil resistivity and pH was not conducted. Final design should be based upon representative sampling of the as-graded soils.

8.10. Infiltration Potential

AGS conducted four borehole percolation tests (P-1 through P-4) in accordance with the testing methods described in Appendix D of the BMP Design Manual (2018). Infiltration rates were calculated using the Porchet method. Based on the results of our subsurface investigation and testing, it was determined that the upper portions of the Stadium Conglomerate and Santiago Peak Volcanics onsite possess relatively low infiltration rates. Measured infiltration rates varied between 0.10 in./hr. and 0.39 in./hr. Preliminary design infiltration rates utilizing a factor of safety of 2.0 were determined to be 0.05 in./hr. for Stadium Conglomerate and 0.18 in./hr. for Santiago Peak Volcanics materials which correspond to a “Partial Infiltration” condition. However, it should be noted that the Santiago Peak Volcanics are virtually impermeable and that ‘infiltration’ occurred as water flowing along/through fractures in the bedrock rather than infiltrating vertically through the bedrock.

Current plans do not show proposed infiltration type BMPs for the project site. If future plans include permanent storm water BMPs, additional testing and evaluation may be necessary. It should be noted that a large portion of the site is mantled by deep pre-existing fills in excess of 5 feet in depth. The current City of San Diego Storm Water Standards (2018) considers areas with pre-existing fills greater than 5 feet deep not suitable for infiltration. As such AGS would not recommend storm water infiltration onsite.

9.0 GRADING RECOMMENDATIONS

Development of the subject site as proposed is considered feasible, from a geotechnical standpoint, provided that the conclusions and recommendations presented herein are incorporated into the design and construction of the project. Presented below are specific issues identified by this study as possibly impacting site development. Recommendations to mitigate these issues are presented in the text of this report.

9.1. Site Preparation and Removals

Grading should be accomplished under the observation and testing of the project soils engineer and engineering geologist or their authorized representative in accordance with the recommendations contained herein, the current City of San Diego grading ordinance, and AGS's *Earthwork*

Specifications (Appendix C). All topsoil, undocumented artificial fill, younger alluvium, and weathered older alluvium and bedrock should be removed in structural areas planned to receive fill or where exposed at final grade. Localized areas may require removals up to 25 feet deep. Removals should expose competent Older Alluvium, Stadium Conglomerate or Santiago Peak Volcanics materials. In general, soils removed during remedial grading will be suitable for reuse in compacted fills, provided they do not contain deleterious materials and are properly moisture conditioned.

9.1.1. Stripping and Deleterious Material Removal

Existing vegetation, trash, debris, and other deleterious materials should be removed and wasted from the site prior to removal of unsuitable soils and placement of compacted fill.

9.1.2. Topsoil (No Map Symbol)

Topsoil, if encountered, will require complete removal and recompaction to project specifications if encountered in areas where settlement sensitive structures or improvements are planned. Topsoil onsite is anticipated to be approximately one-half to one foot thick.

9.1.3. Artificial Fill - Undocumented (Map Symbol afu)

In order to mitigate against potential post construction settlement, the undocumented artificial fill at the site will require complete removal and recompaction to project specifications. Estimated removal depths range from 2 to 25 feet. It should be anticipated that specialized grading techniques may be required to efficiently excavate and recompact these unsuitable soils due to existing offsite improvements and presence of oversize rock. Where deep removals are required in proximity to existing offsite improvements, it may be necessary to use large excavators to remove the soils in a trench wise fashion due to the limited access. The soils can then be moisture conditioned to optimum or above, placed and compacted with a sheepsfoot wheel in two (2) foot lifts until design grades are achieved.

9.1.4. Young Alluvium (Map Symbol Qal)

Young alluvium was encountered underlying undocumented fill in the southwest corner of the site extending to an approximate depth of 26 feet. Young alluvium will require complete removal and recompaction to project specifications within a 1:1 downward projection away from site improvements, where possible.

If saturated alluvium is encountered within structural fill areas, additional recommendations for partial removal and surcharge until primary consolidation settlement is completed may be provided based on observed conditions during grading. Settlement monitoring will be required with the use of buried or surface settlement devices. Final determination of alluvium removals and/or monitoring of left-in–place alluvium will be dependent upon exposed field conditions.

9.1.5. Older Alluvium (Map Symbol Qoa)

Older alluvium commonly has a thin highly weathered horizon on the order of 1 to 3 feet thick. The weathered portion of the older alluvium is unsuitable for structural support or placement of fill and should be removed and replaced with compacted fill.

9.1.6. Stadium Conglomerate / Santiago Peak Volcanics (Map Symbols Tst / Jsp)

The weathered portions of Stadium Conglomerate and Santiago Peak Volcanics materials should be removed and compacted within fill areas or where exposed at design grade. Removals are anticipated to be on the order of 1 to 2 feet thick.

9.2. <u>Overexcavation Recommendations</u>

It is recommended that overexcavation of cut/fill transitions located within the structure's footprint should be conducted during grading. The following general overexcavation recommendations are presented below.

9.2.1. Cut/Fill Transitions

Where design grades and/or remedial grading activities create a cut/fill transition, the cut and shallow fill portions of the building pad should be overexcavated to a minimum depth of five (5) feet or 3 feet below the bottom of footing elevation, whichever is deeper, and replaced to design grade with compacted fill. All undercuts should be graded such that a gradient of at least one (1) percent is maintained toward deeper fill areas or the front of the pad. The entire area extending on a 1:1 (H:V) projection away for the building pad should be undercut. Replacement fills should be compacted to project specifications as discussed in Section 9.4.

9.2.2. Steep Cut/Fill Transitions

In order to reduce the differential settlement potential under the proposed structures due to steep cut/fill transitions, we recommend that the cut or shallow fill portion of steep transitions be overexcavated to a depth equal to one-third (H/3) of the deepest fill section (H) within the building pad area. Based on our field observations, the anticipated maximum fill thickness under the parking garage and church building pads will be 20 feet and 32 feet, respectively. Therefore the recommended overexcavation of the cut or shallow fill portion should extend to approximate depths of 7 feet and 11 feet for the parking garage and church building pads, respectively. Additional overexcavation recommendations may be provided during grading based on exposed conditions.

9.2.3. Utility Construction in Hard Rock

In order to facilitate utility construction consideration should be given to undercutting all proposed utility locations in Stadium Conglomerate or Santiago Peak Volcanics a minimum of one (1) foot below the deepest utility. A "Select" fill should be placed within the overexcavation limits consisting of a replacement material with maximum rock size of approximately eight- (8) inch or smaller. This "select" fill should be compacted to project specifications as discussed in Section 9.4.

9.2.4. Removals Along Grading Limits and Property Lines

Removals of unsuitable soils will be required prior to fill placement along the grading limit. Where possible, a 1:1 (H:V) projection, from toe of slope or grading limit, outward to competent materials should be established. Where removals are not possible due to grading limits, property line or easement restrictions, removals should be initiated at the grading boundary (property line, easement, grading limit or outside the improvement) at a 1:1 ratio inward to competent materials. This reduced removal criteria should not be implemented prior to review by the Geotechnical Consultant and approval by the Owner. Where this reduced removal criteria is implemented, special maintenance zones may be necessary. These areas, if present, will need to be identified during grading.

9.3. Construction Staking and Survey

Removal bottoms, keyways, subdrains and backdrains should be surveyed by the civil engineer after approval by the geotechnical engineer/engineering geologist and prior to the placement of fill. Toe stakes should be provided by the civil engineer in order to verify required key dimensions and locations.

9.4. Earthwork Considerations

9.4.1. Compaction Standards

Fill and processed natural ground shall be compacted to a minimum relative compaction of 90 percent of the maximum dry density as determined by ASTM Test Method D1557. Care should be taken that the ultimate grade be considered when determining the compaction requirements for disposal fill areas. Compaction shall be achieved at slightly above the optimum moisture content, and as generally discussed in the attached Earthwork Specifications (Appendix C).

9.4.2. Documentation of Removals and Drains

Removal bottoms, fill keys, backcuts, backdrains and their outlets should be observed and approved by the engineering geologist and/or geotechnical engineer and documented by the civil engineer prior to fill placement.

9.4.3. Treatment of Removal Bottoms

At the completion of removals, the exposed bottom should be scarified to a practical depth, approximately 8-inches, moisture conditioned to above optimum conditions, and compacted in-place to the standards set forth in this report.

9.4.4. Fill Placement

After removals, scarification, and compaction of in-place materials are completed, additional fill may be placed. Fill should be placed in thin lifts [eight- (8) inch bulk], moisture conditioned to above optimum moisture content, mixed, compacted, and tested as grading progresses until final grades are attained.

9.4.5. Benching

Where the natural slope is steeper than 5-horizontal to 1-vertical, and where designated by the project geotechnical engineer or geologist, compacted fill material should be keyed and benched into competent bedrock or firm natural soil.

9.4.6. Mixing

In order to provide thorough moisture conditioning and proper compaction, processing (mixing) of materials is necessary. Mixing should be accomplished prior to, and as part of the compaction of each fill lift. Water trucks or other water delivery means may be necessary for moisture control. Discing may be required when either excessively dry or wet materials are encountered.

9.4.7. Compaction Equipment

Compaction equipment on the project shall include a combination of rubber-tired and sheepsfoot rollers to achieve proper compaction. Adequate water trucks/pulls should be available to provide sufficient moisture and dust control.

9.4.8. Fill Slope Construction

Fill slopes shall be overfilled to an extent determined by the contractor, but not less than two (2) feet measured perpendicular to the slope face, so that when trimmed back to the compacted core, the required compaction is achieved.

Compaction of each fill lift should extend out to the temporary slope face. Backrolling during mass filling at intervals not exceeding four (4) feet in height is recommended unless more extensive overfill is undertaken.

As an alternative to overfilling, fill slopes may be built to the finish slope face in accordance with the following recommendations:

- Compaction of each fill lift shall extend to the face of the slopes.
- Backrolling during mass grading shall be undertaken at intervals not exceeding four (4) feet in height. Backrolling at more frequent intervals may be required.
- Care should be taken to avoid spillage of loose materials down the face of the slopes during grading.
- At completion of mass filling, the slope surface shall be watered, shaped and compacted first with a sheepsfoot roller or track walked with a bulldozer, such that compaction to project standards is achieved to the face slope.

Proper seeding and planting of the slopes should follow as soon as practical, to inhibit erosion and deterioration of the slope surfaces. Proper moisture control will enhance the long-term stability of the finished slope surface.

9.5. Haul Roads

Haul roads, ramp fills, and tailing areas should be removed prior to placement of fill.

9.6. Import Materials

Import soils are anticipated to achieve design site grades and/or as select material for backfill of site retaining walls. Import materials should have similar engineering characteristics as the onsite soils and should be approved by the soil engineer at the source prior to importation to the site.

10.0 CONCLUSIONS AND RECOMMENDATIONS

Construction of the proposed structures and associated improvements is considered feasible, from a geotechnical standpoint, provided that the conclusions and recommendations presented herein are incorporated into the design and construction of the project. Presented below are specific issues identified by this study as possibly affecting site development. Recommendations to mitigate these issues are presented in the text of this report.

10.1. Design Recommendations

Detailed foundation plans are not currently available; however, it is our understanding that the proposed church and parking structures will be supported by a conventional shallow foundation system or a mat foundation placed on compacted fill materials. In addition to the structures, associated parking lots and landscape areas are proposed.

10.1.1. Foundation Design Criteria

The expansion potential of the underlying soils is anticipated to range from "very low" to "medium". For design of shallow foundations supported on compacted fill, the values presented in Table 10.1.1 should be used.

TABLE 10.1.1
SHALLOW FOUNDATION DESIGN PARAMETERS

Minimum Footing Dimensions[1]	• 24 inches in width and 24 inches in depth.
Allowable Bearing Capacity	• 2,500 pounds per square foot (psf). May be increased by 200 psf and 300 psf for each additional foot of foundation width and depth, respectively, up to a maximum of 3,500 psf. • Allowable bearing values may be increased by one-third for transient live loads from wind or seismic forces.
Estimated Static Settlement	• Total settlement: 1.5 inch • Differential settlement: 0.5 inch over 30 feet. • Static settlement of the foundation system is expected to occur on initial application of loading.
Allowable Coefficient of Friction Below Footings	0.35
Lateral Bearing[2] **(Level Condition)**	300 psf/foot of depth to a maximum of 3,000 psf

Notes: 1. Depth of footing embedment should be measured below lowest adjacent finish grade.
2. For resisting lateral forces on footings, lateral bearing and sliding coefficient may be combined with a maximum sliding resistance limited to ½ of dead load.

10.1.2. Mat Foundation

Mat foundations should be designed by the structural engineer and should conform to the 2019 California Building Code. The allowable bearing pressure is an average value applied to the total area of the mat foundation and was used to evaluate the overall static settlement of the foundation. In our model, the mat foundation was assumed to be rigid with respect to the soil. The recommended geotechnical design parameters are presented in Table 10.1.2.

TABLE 10.1.2 RIGID MAT FOUNDATION DESIGN PARAMETERS	
Average Allowable Bearing Capacity	• 2,000 pounds per square foot (psf). 3,000 psf maximum • Allowable bearing values may be increased by one-third for transient live loads from wind or seismic forces.
Estimated Total Static Settlement and Tilting	• Total settlement: 1.0 inch • Differential settlement (tilt): 0.5 inch over 40 feet. • Static settlement of the foundation system is expected to occur on initial application of loading.

Mat foundations typically experience some deflection due to loads placed on the mat and the reaction of the soils underlying the mat. For the approximate flexible design of slab-on-grade mat foundation systems a modulus of subgrade reaction (K_{v1}) of 150 pci is recommended. The modulus of subgrade reaction is based on a unit square foot area and should be adjusted for the planned mat size. The coefficient of subgrade reaction Kb for a mat of a specific width, may be evaluated using the following equation:

$$Kb = Kv_1[(b+1)/2b]^2$$

where b is the width of the foundation.

10.1.3. Foundation Excavations

Foundation excavations should be observed by the geotechnical consultant. Footings should be excavated into compacted fill materials. The excavations should be free of all loose and sloughed materials, be neatly trimmed, and moisture conditioned at the time of concrete placement. Footing excavations should not be allowed to dry back and should be kept moist until concrete is poured.

10.1.4. Isolated Footings

Isolated footings outside the structure footprint should be tied with grade beams to the structure in two orthogonal directions.

10.1.5. Moisture and Vapor Barrier

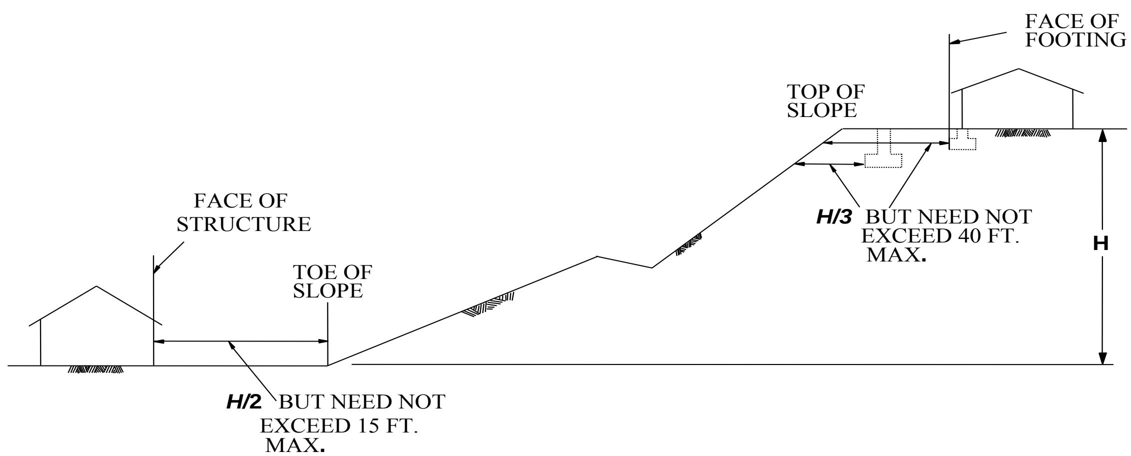
A moisture and vapor retarding system should be placed below the slabs-on-grade in portions of the structure considered to be moisture sensitive. The retarder should be of suitable composition, thickness, strength and low permeance to effectively prevent the migration of water and reduce the transmission of water vapor to acceptable levels. Historically, a 10-mil plastic membrane, such as *Visqueen*, placed between one to four inches of clean sand, has been used for this purpose. More recently, 15-mil polyolefin membrane underlayments (Stego® Wrap or similar material) have been used to lower permeance to effectively prevent the migration of water and reduce the transmission of water vapor to acceptable levels. The use of this system or other systems, materials or techniques can be considered, at the discretion of the designer.

10.1.6. Deepened Footings and Structural Setbacks

It is generally recognized that improvements constructed in proximity to natural slopes or properly constructed, manufactured slopes can, over a period of time, be affected by natural processes including gravity forces, weathering of surficial soils and long-term (secondary) settlement. Most building codes, including the California Building Code (CBC), require that structures be set back or footings deepened, where subject to the influence of these natural processes.

For the subject site, where foundations for structures are to exist in proximity to slopes, the footings should be embedded to satisfy the requirements presented in Figure 5.

FIGURE 5



10.1.7. Concrete Design

Laboratory testing to determine the sulfate concentration of soils at the subject site was not conducted. Final determination should be based on testing of the as-graded soils. It should be noted that some fertilizers have been known to leach sulfates into soils otherwise containing "negligible" sulfate concentrations and increase the sulfate concentrations to potentially detrimental levels. It is incumbent upon the owner to determine whether additional protective measures are warranted to mitigate the potential for increased sulfate concentrations to onsite soils as a result of the future homeowner's actions.

10.1.8. Corrosion

Corrosivity testing was not conducted under the scope of this investigation. Final determination of the corrosivity of onsite soils should be based on testing of the as-graded soils.

10.2. Retaining Walls

It is our understanding that conventional, mechanically stabilized earth (MSE), and/or tieback walls may be part of the proposed development. For preliminary wall design purposes, the following soil parameters can be used for compacted fill materials:

- Unit Weight: 125 pcf, Cohesion: 150 psf, Friction Angle: 31 degrees.

The following earth pressures are recommended for the design of conventional retaining walls onsite utilizing select backfill material having expansion index (EI) of less than 50 and minimum internal friction angle of 31 degrees.

Static Case

TABLE 10.2
LATERAL EARTH PRESSURES

		Rankine Coefficients	Equivalent Fluid Pressure (psf/lin.ft.)
Level Backfill	Active	$K_a = 0.32$	40
	Passive	$K_p = 3.12$	391
	At Rest	$K_o = 0.48$	61
2:1 Backfill	Active	$K_a = 0.50$	63
	At Rest	$K_o = 0.88$	110

Seismic Case

In addition to the above static pressures, unrestrained retaining walls supporting more than 6 feet of backfill height should be designed to resist seismic loading as required by the 2019 CBC. The seismic load can be modeled as a thrust load applied at a point 0.6H above the base of the wall, where H is equal to the height of the wall. This seismic load (in pounds per lineal foot of wall) is represented by the following equation:

$$Pe = \tfrac{3}{8} * \gamma * H^2 * k_h$$

Where: Pe = Seismic thrust load

H = Height of the wall (feet)

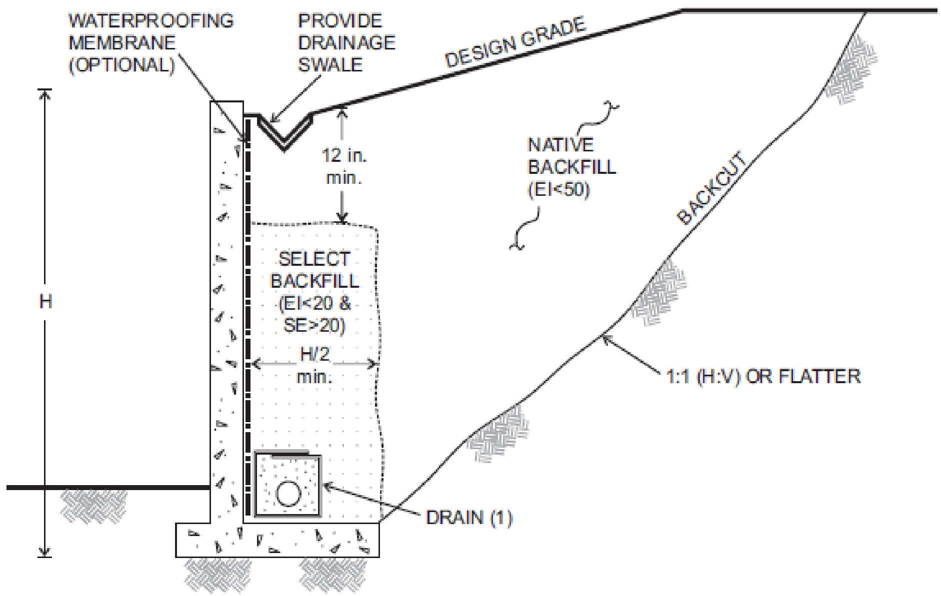
γ = soil density = 125 pcf for compacted fill

k_h = seismic pseudostatic coefficient = 0.5 * peak horizontal ground acceleration / g

The peak horizontal ground acceleration is anticipated to be on the order of 0.477g as discussed in Section 7.7.3. Walls should be designed to resist the combined effects of static pressures and the above seismic thrust load.

The foundations for retaining walls of appurtenant structures structurally separated from the building structures, may bear on properly compacted fill. A bearing value of 2,000 lbs./sq.ft. may be used for design of retaining walls. Retaining wall footings should be designed to resist the lateral forces by passive soil resistance and/or base friction as recommended for foundation lateral resistance. To relieve the potential for hydrostatic pressure wall backfill should consist of a free draining backfill (sand equivalent “SE” >20) and a heel drain should be constructed (see Figure 6). The drain should be placed at the heel of the wall and should consist of a 4-inch diameter perforated pipe (SDR35 or SCHD 40) surrounded by 4 cubic feet of crushed rock (3/4-inch) per lineal foot, wrapped in filter fabric (Mirafi® 140N or equivalent).

FIGURE 6



NOTES: (1) DRAIN: 4-INCH PERFORATED ABS OR PVC PIPE OR APPROVED EQUIVALENT SUBSTITUTE PLACED PERFORATIONS DOWN AND SURROUNDED BY A MINIMUM OF 1 CUBIC FEET OF 3/4 INCH ROCK OR APPROVED EQUIVALENT SUBSTITUTE AND WRAPPED IN MIRAFI 140 FILTER FABRIC OR APPROVED EQUIVALENT SUBSTITUTE

Drainage devices should be installed along the top of the wall backfill and should be sloped to prevent surface water ponding adjacent to the wall. In addition to the wall drainage system, for building perimeter walls extending below the finished grade, the wall should be waterproofed and/or damp-proofed to effectively seal the wall from moisture infiltration through the wall section to the interior wall face.

The wall should be backfilled with granular soils placed in loose lifts no greater than 8-inches thick, at or near optimum moisture content, and mechanically compacted to a minimum 90 percent relative compaction as determined by ASTM Test Method D1557. Flooding or jetting of backfill materials generally do not result in the required degree and uniformity of compaction and, therefore, is not recommended. The soils engineer or his representative should observe the retaining wall footings, backdrain installation and be present during placement of the wall backfill to confirm that the walls are properly backfilled and compacted.

10.3. Utility Trench Excavation

All utility trenches should be shored or laid back in accordance with applicable OSHA standards. Excavations in bedrock areas should be made in consideration of underlying geologic structure. AGS should be consulted on these issues during construction.

10.4. Utility Trench Backfill

Mainline and lateral utility trench backfill should be compacted to at least 90 percent relative compaction as determined by ASTM D1557. Onsite soils will not be suitable for use as bedding material but will be suitable for use in backfill, provided oversized materials are removed. No surcharge loads should be imposed above excavations. This includes spoil piles, lumber, concrete trucks or other construction materials and equipment. Drainage above excavations should be directed away from the banks. Care should be taken to avoid saturation of the soils. Compaction should be accomplished by mechanical means. Jetting of native soils will not be acceptable.

10.5. Exterior Slabs and Walkways

10.5.1. Subgrade Compaction

The subgrade below exterior slabs, sidewalks, driveways, patios, etc. should be compacted to a minimum of 90 percent relative compaction as determined by ASTM D1557.

10.5.2. Subgrade Moisture

The subgrade below exterior slabs, sidewalks, driveways, patios, etc. should be moisture conditioned to a minimum of 110 percent of optimum moisture content (low expansive soils) prior to concrete placement, dependent upon the expansion potential of the subgrade soils.

10.5.3. Slab Thickness

Concrete flatwork and driveways should be designed utilizing four-inch minimum thickness.

10.5.4. Control Joints

Weakened plane joints should be installed on walkways at intervals of approximately eight to ten feet. Exterior slabs should be designed to withstand shrinkage of the concrete.

10.5.5. Flatwork Reinforcement

Consideration should be given to reinforcing any exterior flatwork.

10.5.6. Thickened Edge

Consideration should be given to construct a thickened edge (scoop footing) at the perimeter of slabs and walkways adjacent to landscape areas to minimize moisture variation below these improvements. The thickened edge (scoop footing) should extend approximately eight inches below concrete slabs and should be a minimum of six inches wide.

10.6. Preliminary Pavement Design

For preliminary pavement design, we have assumed an “R” Value of 30 for the onsite subgrade soils. Utilizing City of San Diego Pavement Design Standards Schedule “J” and assuming the subject site is classified equivalent to “Local Residential” (max ADT=1200) which equates to a Traffic Index TI=6.0 the following pavement section is presented below. Additional pavement design recommendations will be provided during grading based on as-graded conditions and R-value testing.

Standard Pavement Section

3-inches Asphalt Concrete
over
8.5-inches Aggregate Base

Pavement subgrade soils should be at or near optimum moisture content and should be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D1557. Aggregate base should be compacted to a minimum of 95 percent relative compaction and should conform with the specifications in Section 26 of the Standard Specifications for the State of California Department of Transportation (Caltrans) or Section 200-2 of the Standard Specifications for Public Works Construction (Green Book). The asphalt concrete should conform to Section 26 of the Caltrans Standard Specifications or Section 203-6 of the Green Book.

11.0 CLOSURE

11.1. Geotechnical Review

As is the case in any grading project, multiple working hypotheses are established utilizing the available data, and the most probable model is used for the analysis. Information collected during the grading and construction operations is intended to evaluate the hypotheses, and some of the assumptions summarized herein may need to be changed as more information becomes available. Some modification of the grading and construction recommendations may become necessary, should the conditions encountered in the field differ significantly than those hypothesized to exist.

AGS should review the grading and foundation plans and sections of the project specifications, to evaluate conformance with the intent of the recommendations contained in this report. If the project description or final design varies from that described in this report, AGS must be consulted regarding the applicability of, and the necessity for, any revisions to the recommendations presented herein. AGS accepts no liability for any use of its recommendations if the project description or final design varies and AGS is not consulted regarding the changes.

11.2. Limitations

This report is based on the project as described and the information obtained from referenced reports and the borings and test pits at the locations indicated on the plans. The findings are based on the review of the field data combined with an interpolation and extrapolation of conditions between and beyond the exploratory excavations. The results reflect an interpretation of the direct evidence obtained. Services performed by AGS have been conducted in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality under similar conditions. No other representation, either expressed or implied, and no warranty or guarantee is included or intended.

The recommendations presented in this report are based on the assumption that an appropriate level of field review will be provided by geotechnical engineers and engineering geologists who are familiar with the design and site geologic conditions. That field review shall be sufficient to confirm that geotechnical and geologic conditions exposed during grading are consistent with the geologic representations and corresponding recommendations presented in this report. AGS should be notified of any pertinent changes in the project plans or if subsurface conditions are found to vary from those described herein. Such changes or variations may require a re-evaluation of the recommendations contained in this report.

The data, opinions, and recommendations of this report are applicable to the specific design of this project as discussed in this report. They have no applicability to any other project or to any other location, and any and all subsequent users accept any and all liability resulting from any use or reuse of the data, opinions, and recommendations without the prior written consent of AGS.

AGS has no responsibility for construction means, methods, techniques, sequences, or procedures, or for safety precautions or programs in connection with the construction, for the acts or omissions of the CONTRACTOR, or any other person performing any of the construction, or for the failure of any of them to carry out the construction in accordance with the final design drawings and specifications.

APPENDIX A

REFERENCES

REFERENCES

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APPENDIX B

SUBSURFACE INVESTIGATION

Project College Ave/I8
Date Excavated Dec. 2014
Logged by FE
Equipment Cat 328D

LOG OF TEST PITS

Test Pit No.	Depth (ft.)	USCS	Description
EX-1	0.0 – 2.0	SM	**Artificial Fill – Undocumented (afu):** SILTY SAND with abundant rounded COBBLES to 4-in. diameter, yellowish brown, very moist, loose; some clay.
	2.0 – 9.5		**Stadium Conglomerate (Tst):** COBBLE CONGLOMERATE, rounded volcanic and metamorphic clasts to 6-in. diameter in a SILTY SANDSTONE matrix, light brownish yellow, moderately hard.
			TOTAL DEPTH 9.5 FT. NO GROUNDWATER, NO CAVING.
EX-2	0.0 – 3.5	SM	**Artificial Fill – Undocumented (afu):** SILTY SAND, light reddish brown, very moist, loose; with some rounded cobbles to 8-in. diameter; minor seepage at 3.5 ft.
	3.5 – 7.0		**Santiago Peak Volcanics (Jsp):** METAVOLCANIC BEDROCK, light gray to gray on fresh surfaces, slightly to moderately weathered, moderately hard to hard; jointed, manganese oxide along joint surfaces. @5 ft. N 60° E, Vertical - Joint N 5° W, 75° SW - Joint @6 ft. Hard, slightly weathered
			TOTAL DEPTH 7.0 FT. MINOR SEEPAGE AT 3.5 FT., NO CAVING.

Test Pit No.	Depth (ft.)	USCS	Description
EX-3	0.0 – 22.0	SW	**Artificial Fill – Undocumented (afu):** SAND with COBBLES, fine to coarse grained, brown, moist, loose; with some clay and silt.
		SM	@2 ft. SILTY SAND, pale yellow to light gray, slightly moist, moderately dense; abundant rounded COBBLES to 8-in. diameter.
	22.0 – 26.0	CL/ML	**Alluvium (Qal):** CLAYEY SILT, brown, very moist, stiff; some fine grained sand and angular gravel.
	26.0 – 27.0		**Santiago Peak Volcanics (Jsp):** METAVOLCANIC BEDROCK, reddish brown, moderately weathered, hard.
			TOTAL DEPTH 27.0 FT. NO GROUNDWATER, CAVING AT 3 FT.
EX-4	0.0 – 8.0	SW	**Artificial Fill – Undocumented (afu):** SAND with COBBLES, fine to coarse grained, pale yellow, slightly moist, loose. @4 ft. Moderately dense.
	6.5 – 13.0		**Older Alluvium (Qoa):** SILTY SAND, fine grained, yellow, slightly moist to moist, moderately dense to dense; some clay. @10 ft. Some ¼ to ½-in. thick SILTY CLAY lenses, olive, moist, stiff; slightly plastic.
			TOTAL DEPTH 13.0 FT. NO GROUNDWATER, NO CAVING.

Test Pit No.	Depth (ft.)	USCS	Description
EX-5	0.0 – 21.0	SM	**Artificial Fill – Undocumented (afu):** SILTY SAND, reddish brown, moist, loose.
		SW	@2.5 ft. SAND with COBBLES, light gray, slightly moist, medium dense; with some silt and clay.
	21.0 – 22.0	ML	**Alluvium (Qal):** CLAYEY to SANDY SILT, dark grayish brown, moist to very moist, stiff; abundant subangular gravel.
	22.0 – 23.0		**Older Alluvium (Qoa):** SILTY SAND, fine grained, yellow, slightly moist, moderately dense to dense.
			TOTAL DEPTH 23.0 FT. NO GROUNDWATER, NO CAVING.
EX-6	0.0 – 10.0	SW	**Artificial Fill – Undocumented (afu):** SAND with COBBLES, pale yellow to light gray; with some silt and clay.
	10.0 – 15.0		**Older Alluvium (Qoa):** Interbedded CLAYEY fine grained SAND and SILTY CLAY, yellow and olive, moist, dense/stiff.
	15.0 – 15.5		**Santiago Peak Volcanics (Jsp):** METAVOLCANIC BEDROCK, brownish yellow, highly weathered, abundant clay development, soft to moderately hard. @15.5 ft. Slightly weathered, hard.
			TOTAL DEPTH 15.5 FT. NO GROUNDWATER, NO CAVING.

Test Pit No.	Depth (ft.)	USCS	Description
EX-7	0.0 – 20.0	SM	**Artificial Fill – Undocumented (afu):** Angular, gray metavolcanic clasts from 8-in. to 4-ft. diameter in a SILTY SAND matrix, fine to coarse grained, yellowish brown, moist, loose. @6 ft. Some rounded cobbles to 5-in. diameter. @8 ft. Some rounded cobbles to 7-in. diameter. @19 ft. Some rounded cobbles to 10-in. diameter.
	20.0 – 24.5		**Older Alluvium (Qoa):** Fine SANDY SILT, red, slightly moist, stiff; some 1/16-in. paleo root holes. @22 ft. Some clay; no visible porosity.
			TOTAL DEPTH 24.5 FT. NO GROUNDWATER, CAVING AT 5 FT.
EX-8	0.0 – 4.5	SM	**Artificial Fill – Undocumented (afu):** SILTY SAND, fine to coarse grained, reddish brown, moist, loose; abundant rounded cobbles to 3-in. diameter.
	4.5 – 12.5		**Stadium Conglomerate (Tst):** COBBLE CONGLOMERATE, rounded cobbles to 3-in. diameter in a SILTY SANDSTONE matrix, yellow, slightly moist, hard.
			TOTAL DEPTH 12.5 FT. NO GROUNDWATER, NO CAVING.

Test Pit No.	Depth (ft.)	USCS	Description
EX-9	0.0 – 10.0	SM	**Artificial Fill – Undocumented (afu):** SILTY SAND with COBBLES, dark brown and yellowish brown, moist, loose; some 4-in. thick asphalt slabs. @4 ft. Some angular metavolcanic clasts to 2-ft. diameter. @8 ft. Trash debris.
	10.0 – 12.5		**Stadium Conglomerate (Tst):** COBBLE CONGLOMERATE, rounded volcanic and metamorphic clasts to 3-in. diameter, in a SILTY SANDSTONE matrix, light yellow, slightly moist, moderately hard. @11 ft. Hard.
			TOTAL DEPTH 12.5 FT. NO GROUNDWATER, CAVING AT 4 FT.
EX-10	0.0 – 22.0	SW	**Artificial Fill – Undocumented (afu):** GRAVELY SAND, reddish brown, moist, loose; with some rounded cobbles to 3-in. diameter; few metavolcanic clasts to 18-in. diameter.425
		SM-ML	@11 ft. SILTY SAND and CLAYEY SILT, dark gray, moist to very moist, firm to medium dense; some organics.
	22.0 – 22.5		**Santiago Peak Volcanics (Jsp):** METAVOLCANIC BEDROCK, moderately weathered, hard.
			TOTAL DEPTH 15.5 FT. NO GROUNDWATER, CAVING AT 6 FT.

APPENDIX C

GENERAL EARTHWORK SPECIFICATIONS

AND GRADING GUIDELINES

GENERAL EARTHWORK SPECIFICATIONS

I. General

A. General procedures and requirements for earthwork and grading are presented herein. The earthwork and grading recommendations provided in the geotechnical report are considered part of these specifications, and where the general specifications provided herein conflict with those provided in the geotechnical report, the recommendations in the geotechnical report shall govern. Recommendations provided herein and in the geotechnical report may need to be modified depending on the conditions encountered during grading.

B. The contractor is responsible for the satisfactory completion of all earthwork in accordance with the project plans, specifications, applicable building codes, and local governing agency requirements. Where these requirements conflict, the stricter requirements shall govern.

C. It is the contractor's responsibility to read and understand the guidelines presented herein and in the geotechnical report as well as the project plans and specifications. Information presented in the geotechnical report is subject to verification during grading. The information presented on the exploration logs depict conditions at the particular time of excavation and at the location of the excavation. Subsurface conditions present at other locations may differ, and the passage of time may result in different subsurface conditions being encountered at the locations of the exploratory excavations. The contractor shall perform an independent investigation and evaluate the nature of the surface and subsurface conditions to be encountered and the procedures and equipment to be used in performing his work.

D. The contractor shall have the responsibility to provide adequate equipment and procedures to accomplish the earthwork in accordance with applicable requirements. When the quality of work is less than that required, the Geotechnical Consultant may reject the work and may recommend that the operations be suspended until the conditions are corrected.

E. Prior to the start of grading, a qualified Geotechnical Consultant should be retained to observe grading procedures and provide testing of the fills for conformance with the project specifications, approved grading plan, and guidelines presented herein. All remedial removals, clean-outs, removal bottoms, keyways, and subdrain installations should be observed and documented by the Geotechnical Consultant prior to placing fill. It is the contractor's responsibility to appraise the Geotechnical Consultant of their schedules and notify the Geotechnical Consultant when those areas are ready for observation.

F. The contractor is responsible for providing a safe environment for the Geotechnical Consultant to observe grading and conduct tests.

II. Site Preparation

A. Clearing and Grubbing: Excessive vegetation and other deleterious material shall be sufficiently removed as required by the Geotechnical Consultant, and such materials shall be properly disposed of offsite in a method acceptable to the owner and governing agencies. Where applicable, the contractor may obtain permission from the Geotechnical Consultant, owner, and governing agencies to dispose of vegetation and other deleterious materials in designated areas onsite.

B. Unsuitable Soils Removals: Earth materials that are deemed unsuitable for the support of fill shall be removed as necessary to the satisfaction of the Geotechnical Consultant.

C. Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipelines, other utilities, or other structures located within the limits of grading shall be removed and/or abandoned in accordance with the requirements of the governing agency and to the satisfaction of the Geotechnical Consultant.

D. Preparation of Areas to Receive Fill: After removals are completed, the exposed surfaces shall be scarified to a depth of approximately 8 inches, watered or dried, as needed, to achieve a generally uniform moisture content that is at or near optimum moisture content. The scarified materials shall then be compacted to the project requirements and tested as specified.

E. All areas receiving fill shall be observed and approved by the Geotechnical Consultant prior to the placement of fill. A licensed surveyor shall provide survey control for determining elevations of processed areas and keyways.

III. Placement of Fill

A. Suitability of fill materials: Any materials, derived onsite or imported, may be utilized as fill provided that the materials have been determined to be suitable by the Geotechnical Consultant. Such materials shall be essentially free of organic matter and other deleterious materials, and be of a gradation, expansion potential, and/or strength that is acceptable to the Geotechnical Consultant. Fill materials shall be tested in a laboratory approved by the Geotechnical Consultant, and import materials shall be tested and approved prior to being imported.

B. Generally, different fill materials shall be thoroughly mixed to provide a relatively uniform blend of materials and prevent abrupt changes in material type. Fill materials derived from benching should be dispersed throughout the fill area instead of placing the materials within only an equipment-width from the cut/fill contact.

C. Oversize Materials: Rocks greater than 8 inches in largest dimension shall be disposed of offsite or be placed in accordance with the recommendations by the Geotechnical Consultant in the areas that are designated as suitable for oversize rock placement. Rocks that are smaller than 8 inches in largest dimension may be utilized in the fill provided that they are not nested and are their quantity and distribution are acceptable to the Geotechnical Consultant.

D. The fill materials shall be placed in thin, horizontal layers such that, when compacted, shall not exceed 6 inches. Each layer shall be spread evenly and shall be thoroughly mixed to obtain a near uniform moisture content and uniform blend of materials.

E. Moisture Content: Fill materials shall be placed at or above the optimum moisture content or as recommended by the geotechnical report. Where the moisture content of the engineered fill is less than recommended, water shall be added, and the fill materials shall be blended so that a near uniform moisture content is achieved. If the moisture content is above the limits specified by the Geotechnical Consultant, the fill materials shall be aerated by discing, blading, or other methods until the moisture content is acceptable.

F. Each layer of fill shall be compacted to the project standards in accordance to the project specifications and recommendations of the Geotechnical Consultant. Unless otherwise specified by the Geotechnical Consultant, the fill shall be compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM Test Method: D1557.

G. Benching: Where placing fill on a slope exceeding a ratio of 5 to 1 (horizontal to vertical), the ground should be keyed or benched. The keyways and benches shall extend through all unsuitable materials into suitable materials such as firm materials or sound bedrock or as recommended by the Geotechnical Consultant. The minimum keyway width shall be 15 feet and extend into suitable materials, or as recommended by the geotechnical report and approved by the Geotechnical Consultant. The minimum keyway width for fill over cut slopes is also 15 feet, or as recommended by the geotechnical report and approved by the Geotechnical Consultant. As a general rule, unless otherwise recommended by the Geotechnical Consultant, the minimum width of the keyway shall be equal to 1/2 the height of the fill slope.

H. Slope Face: The specified minimum relative compaction shall be maintained out to the finish face of fill and stabilization fill slopes. Generally, this may be achieved by overbuilding the slope and cutting back to the compacted core. The actual amount of overbuilding may vary as field conditions dictate. Alternately, this may be achieved by backrolling the slope face with suitable equipment or other methods that produce the designated result. Loose soil should not be allowed to build up on the slope face. If present, loose soils shall be trimmed to expose the compacted slope face.

I. Slope Ratio: Unless otherwise approved by the Geotechnical Consultant and governing agencies, permanent fill slopes shall be designed and constructed no steeper than 2 to 1 (horizontal to vertical).

J. Natural Ground and Cut Areas: Design grades that are in natural ground or in cuts should be evaluated by the Geotechnical Consultant to determine whether scarification and processing of the ground and/or overexcavation is needed.

K. Fill materials shall not be placed, spread, or compacted during unfavorable weather conditions. When grading is interrupted by rain, filing operations shall not resume until the Geotechnical Consultant approves the moisture and density of the previously placed compacted fill.

IV. Cut Slopes

A. The Geotechnical Consultant shall inspect all cut slopes, including fill over cut slopes, and shall be notified by the contractor when cut slopes are started.

B. If adverse or potentially adverse conditions are encountered during grading, the Geotechnical Consultant shall investigate, evaluate, and make recommendations to mitigate the adverse conditions.

C. Unless otherwise stated in the geotechnical report, cut slopes shall not be excavated higher or steeper than the requirements of the local governing agencies. Short-term stability of the cut slopes and other excavations is the contractor's responsibility.

V. Drainage

A. Backdrains and Subdrains: Backdrains and subdrains shall be provided in fill as recommended by the Geotechnical Consultant and shall be constructed in accordance with the governing agency and/or recommendations of the Geotechnical Consultant. The location of subdrains, especially outlets, shall be surveyed and recorded by the Civil Engineer.

B. Top-of-slope Drainage: Positive drainage shall be established away from the top of slope. Site drainage shall not be permitted to flow over the tops of slopes.

C. Drainage terraces shall be constructed in compliance with the governing agency requirements and/or in accordance with the recommendations of the Geotechnical Consultant.

D. Non-erodible interceptor swales shall be placed at the top of cut slopes that face the same direction as the prevailing drainage.

VI. Erosion Control

A. All finish cut and fill slopes shall be protected from erosion and/or planted in accordance with the project specifications and/or landscape architect's recommendations. Such measures to protect the slope face shall be undertaken as soon as practical after completion of grading.

B. During construction, the contractor shall maintain proper drainage and prevent the ponding of water. The contractor shall take remedial measures to prevent the erosion of graded areas until permanent drainage and erosion control measures have been installed.

VII. Trench Excavation and Backfill

A. Safety: The contractor shall follow all OSHA requirements for safety of trench excavations. Knowing and following these requirements is the contractor's responsibility. All trench excavations or open cuts in excess of 5 feet in depth shall be shored or laid back. Trench excavations and open cuts exposing adverse geologic conditions may require further evaluation by the Geotechnical Consultant. If a contractor fails to provide safe access for compaction testing, backfill not tested due to safety concerns may be subject to removal.

B. Bedding: Bedding materials shall be non-expansive and have a Sand Equivalent greater than 30. Where permitted by the Geotechnical Consultant, the bedding materials can be densified by jetting.

C. Backfill: Jetting of backfill materials is generally not acceptable. Where permitted by the Geotechnical Consultant, the bedding materials can be densified by jetting provided the backfill materials are granular, free-draining and have a Sand Equivalent greater than 30.

VIII. Geotechnical Observation and Testing During Grading

A. Compaction Testing: Fill shall be tested by the Geotechnical Consultant for evaluation of general compliance with the recommended compaction and moisture conditions. The tests shall be taken in the compacted soils beneath the surface if the surficial materials are disturbed. The contractor shall assist the Geotechnical Consultant by excavating suitable test pits for testing of compacted fill.

B. Where tests indicate that the density of a layer of fill is less than required, or the moisture content not within specifications, the Geotechnical Consultant shall notify the contractor of the unsatisfactory conditions of the fill. The portions of the fill that are not within specifications shall be reworked until the required density and/or moisture content has been attained. No additional fill shall be placed until the last lift of fill is tested and found to meet the project specifications and approved by the Geotechnical Consultant.

C. If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as adverse weather, excessive rock or deleterious materials being placed in the fill, insufficient equipment, excessive rate of fill placement, results in a quality of work that is unacceptable, the consultant shall notify the contractor, and the contractor shall rectify the conditions, and if necessary, stop work until conditions are satisfactory.

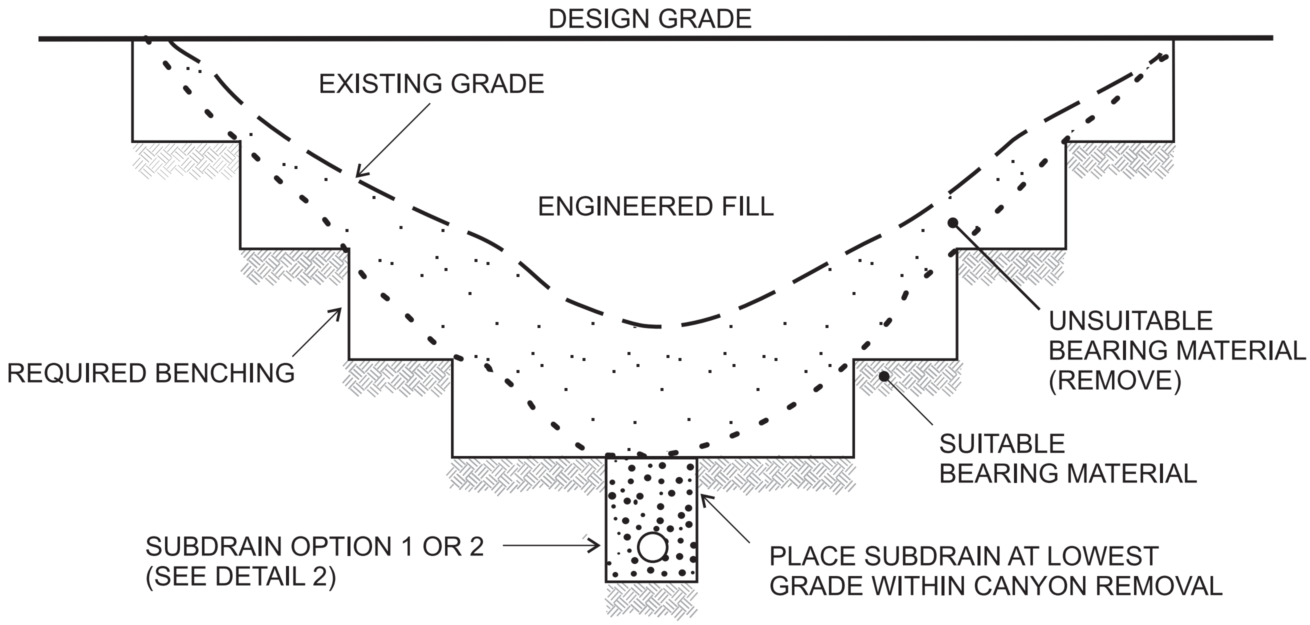
D. Frequency of Compaction Testing: The location and frequency of tests shall be at the Geotechnical Consultant's discretion. Generally, compaction tests shall be taken at intervals not exceeding two feet in fill height and 1,000 cubic yards of fill materials placed.

E. Compaction Test Locations: The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of the compaction test locations. The contractor shall coordinate with the surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations. Alternately, the test locations can be surveyed and the results provided to the Geotechnical Consultant.

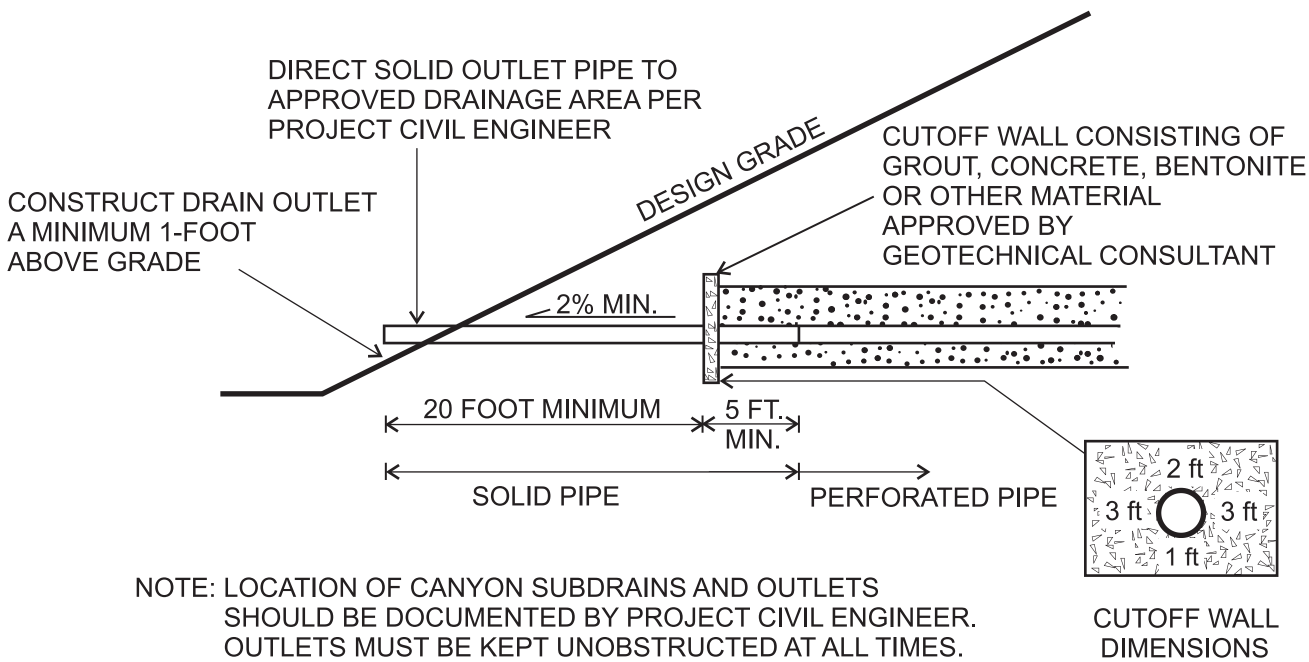
F. Areas of fill that have not been observed or tested by the Geotechnical Consultant may have to be removed and recompacted at the contractor's expense. The depth and extent of removals will be determined by the Geotechnical Consultant.

G. Observation and testing by the Geotechnical Consultant shall be conducted during grading in order for the Geotechnical Consultant to state that, in his opinion, grading has been completed in accordance with the approved geotechnical report and project specifications.

H. Reporting of Test Results: After completion of grading operations, the Geotechnical Consultant shall submit reports documenting their observations during construction and test results. These reports may be subject to review by the local governing agencies.



CANYON SUBDRAIN PROFILE



NOTE: LOCATION OF CANYON SUBDRAINS AND OUTLETS SHOULD BE DOCUMENTED BY PROJECT CIVIL ENGINEER. OUTLETS MUST BE KEPT UNOBSTRUCTED AT ALL TIMES.

CANYON SUBDRAIN TERMINUS

VER 1.0

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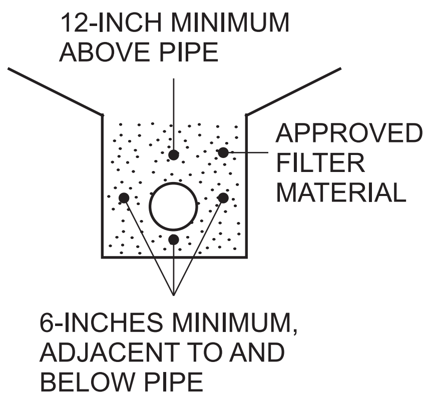
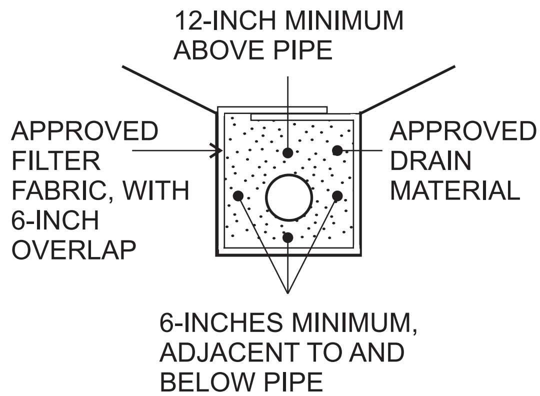
AGS ADVANCED GEOTECHNICAL SOLUTIONS	CANYON SUBDRAIN	DETAIL 1



OPTION 1

FILTER MATERIAL: MINIMUM VOLUME OF 9 CUBIC FEET PER LINEAL FOOT OF CALTRANS CLASS 2 PERMEABLE MATERIAL



OPTION 2

DRAIN MATERIAL: MINIMUM VOLUME OF 9 CUBIC FEET PER LINEAL FOOT OF 3/4-INCH MAX ROCK OR APPROVED EQUIVALENT SUBSTITUTE

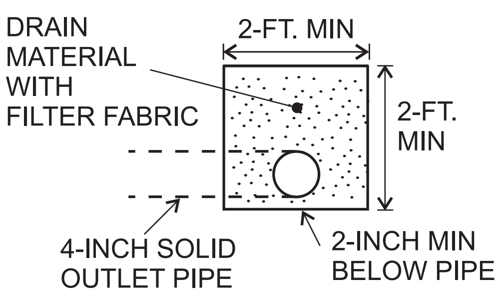
FILTER FABRIC: MIRAFI 140 FILTER FABRIC OR APPROVED EQUIVALENT SUBSTITUTE

PIPE: 6 OR 8-INCH ABS OR PVC PIPE OR APPROVED SUBSTITUTE WITH A MINIMUM OF 8 PERFORATIONS (1/4-INCH DIAMETER) PER LINEAL FOOT IN BOTTOM HALF OF PIPE

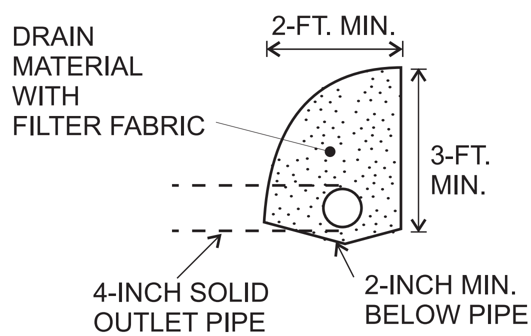
(ASTM D2751, SDR-35 OR ASTM D3034, SDR-35
ASTM D1527, SCHD. 40 OR ASTM D1785, SCHD. 40)

NOTE: CONTINUOUS RUN IN EXCESS OF 5OO FEET REQUIRES 8-INCH DIAMETER PIPE (ASTM D3034, SDR-35, OR ASTM D1785, SCHD. 40)

CANYON SUBDRAIN



OPTION 1



OPTION 2

DRAIN MATERIAL: GRAVEL TRENCH TO BE FILLED WITH 3/4-INCH MAX ROCK OR APPROVED EQUIVALENT SUBSTITUTE

FILTER FABRIC: MIRAFI 140 FILTER FABRIC OR EQUIVALENT SUBSTITUTE WITH A MINIMUM 6-INCH OVERLAP

PIPE: 4-INCH ABS OR PVC PIPE OR APPROVED EQUIVALENT SUBSTITUTE WITH A MINIMUM OF 8 PERFORATIONS (1/4-INCH DIAMETER) PER LINEAL FOOT IN BOTTOM HALF OF PIPE

(ASTM D2751, SDR-35 OR ASTM D3034, SDR-35
ASTM D1527, SCHD. 40 OR ASTM D1785, SCHD. 40)

BUTTRESS/STABILIZATION DRAIN

VER 1.0

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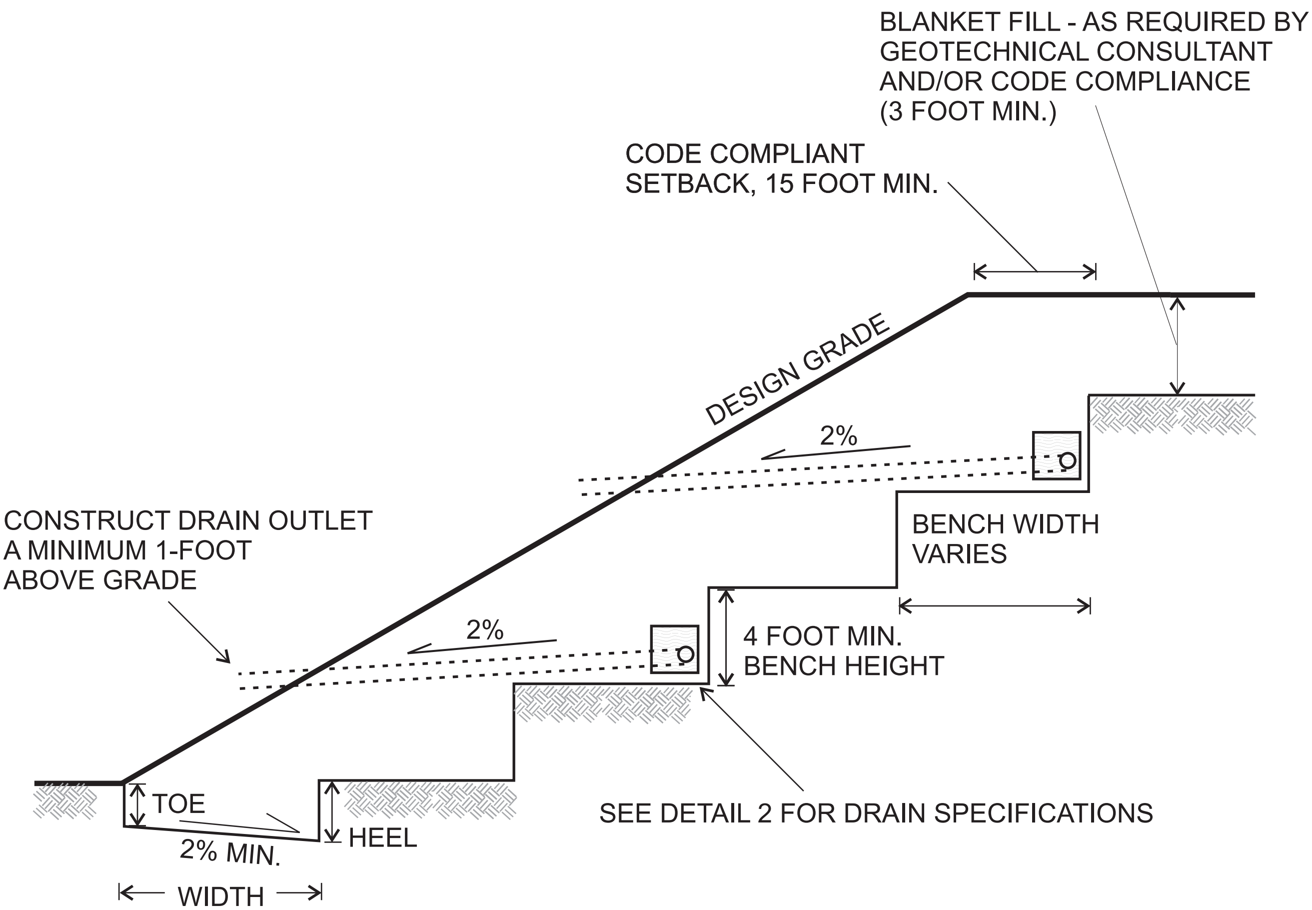
AGS ADVANCED GEOTECHNICAL SOLUTIONS	DRAIN SPECIFICATIONS	DETAIL 2



CODE COMPLIANT KEYWAY
WITH MINIMUM DIMENSIONS:

TOE 2 FOOT MIN.
HEEL 3 FOOT MIN.
WIDTH 15 FOOT MIN.

NOTES:

1. DRAIN OUTLETS TO BE PROVIDED EVERY 100 FEET CONNECT TO PERFORATED DRAIN PIPE BY “L” OR “T” AT A MINIMUM 2% GRADIENT.

2. THE NECESSITY AND LOCATION OF ADDITIONAL DRAINS SHALL BE DETERMINED IN THE FIELD BY THE GEOTECHNICAL CONSULTANT. UPPER STAGE OUTLETS SHOULD BE EMPTIED ONTO CONCRETE TERRACE DRAINS.

3. DRAIN PIPE TO EXTEND FULL LENGTH OF STABILIZATION/BUTTRESS WITH A MINIMUM GRADIENT OF 2% TO SOLID OUTLET PIPES.

4. LOCATION OF DRAINS AND OUTLETS SHOULD BE DOCUMENTED BY PROJECT CIVIL ENGINEER. OUTLETS MUST BE KEPT UNOBSTRUCTED AT ALL TIMES.

VER 1.0

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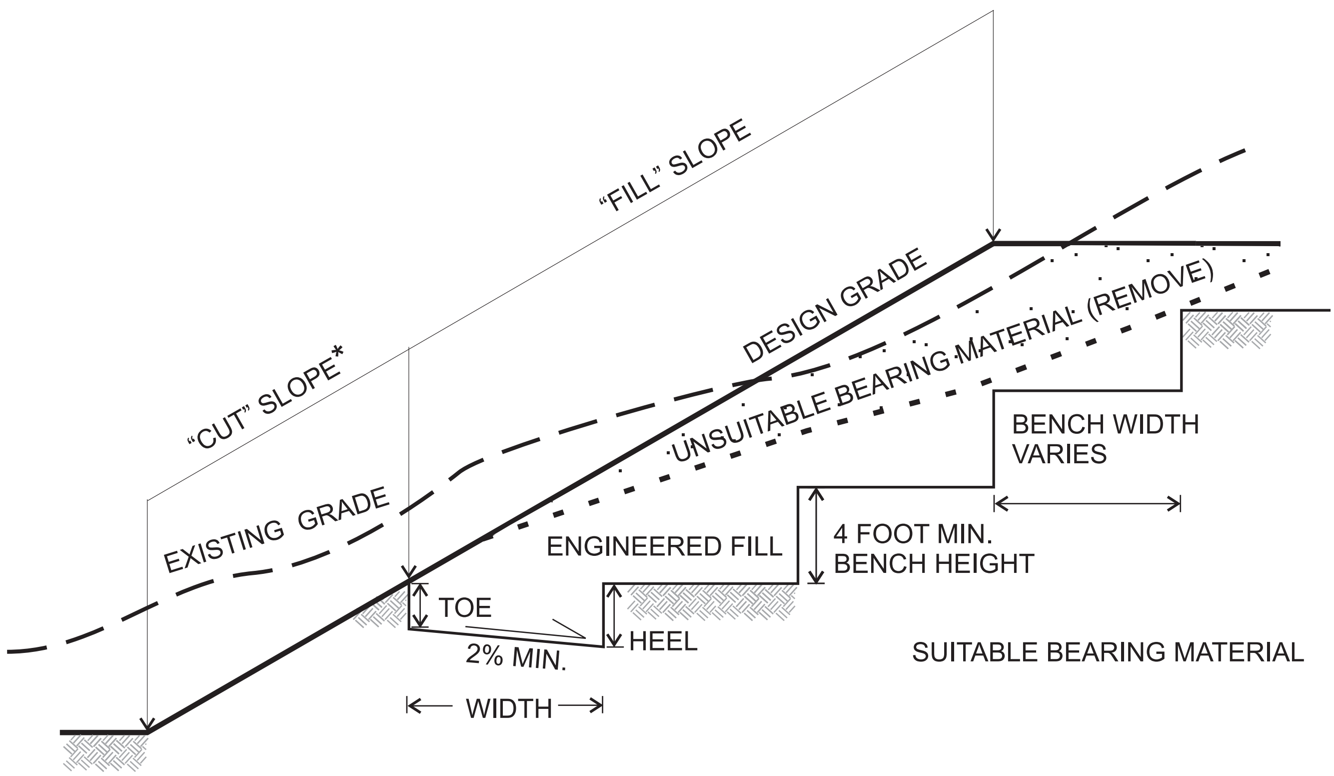
AGS ADVANCED GEOTECHNICAL SOLUTIONS	STABILIZATION/BUTTRESS FILL	DETAIL 3

* THE “CUT” PORTION OF THE SLOPE SHALL BE EXCAVATED AND EVALUATED BY THE GEOTECHNICAL CONSULTANT PRIOR TO CONSTRUCTING THE “FILL” PORTION



SUITABLE
BEARING MATERIAL

CODE COMPLIANT KEYWAY
WITH MINIMUM DIMENSIONS:

TOE: 2 FOOT MIN.
HEEL: 3 FOOT MIN.
WIDTH: 15 FOOT MIN.

NOTES:

1. THE NECESSITY AND LOCATION OF DRAINS SHALL BE DETERMINED IN THE FIELD BY THE GEOTECHNICAL CONSULTANT

2. SEE DETAIL 2 FOR DRAIN SPECIFICATIONS

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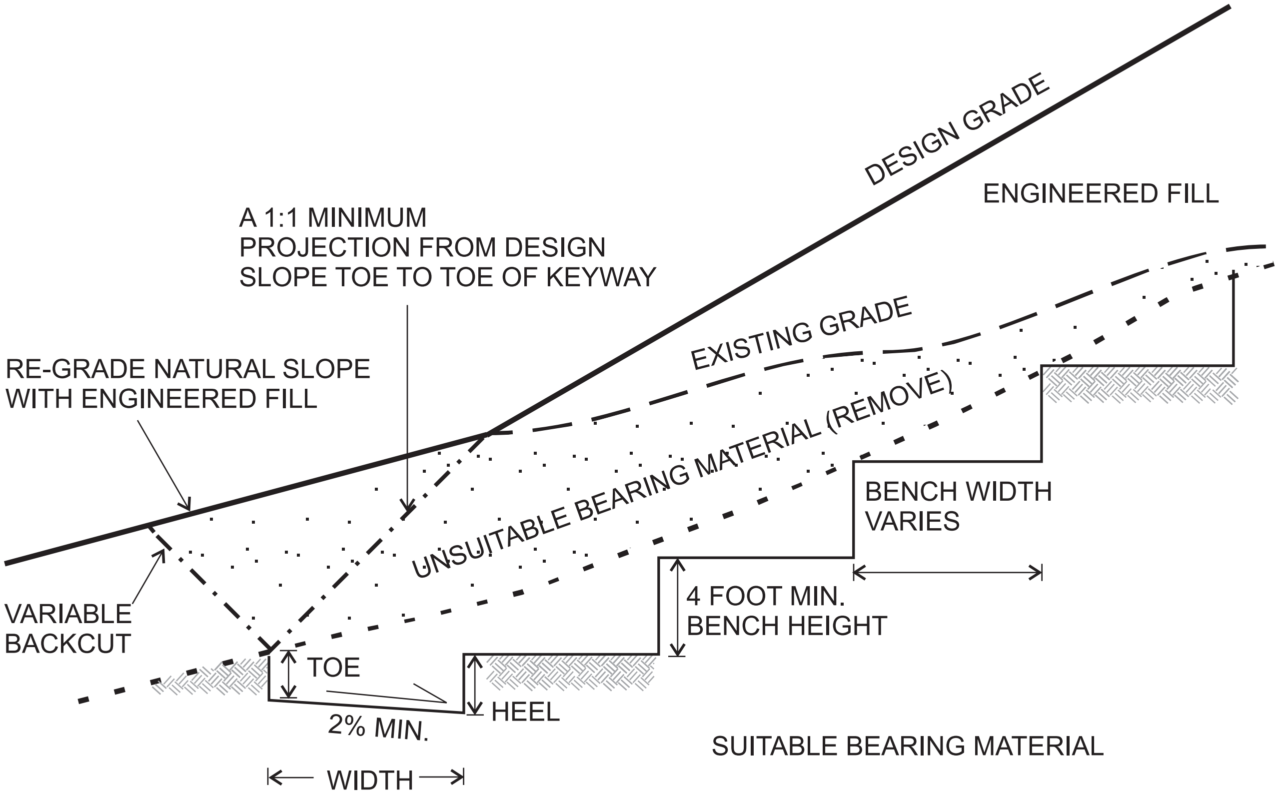
ADVANCED GEOTECHNICAL SOLUTIONS

FILL OVER CUT SLOPE

DETAIL 4



CODE COMPLIANT KEYWAY
WITH MINIMUM DIMENSIONS:

TOE: 2 FOOT MIN.
HEEL: 3 FOOT MIN.
WIDTH: 15 FOOT MIN.

NOTES:

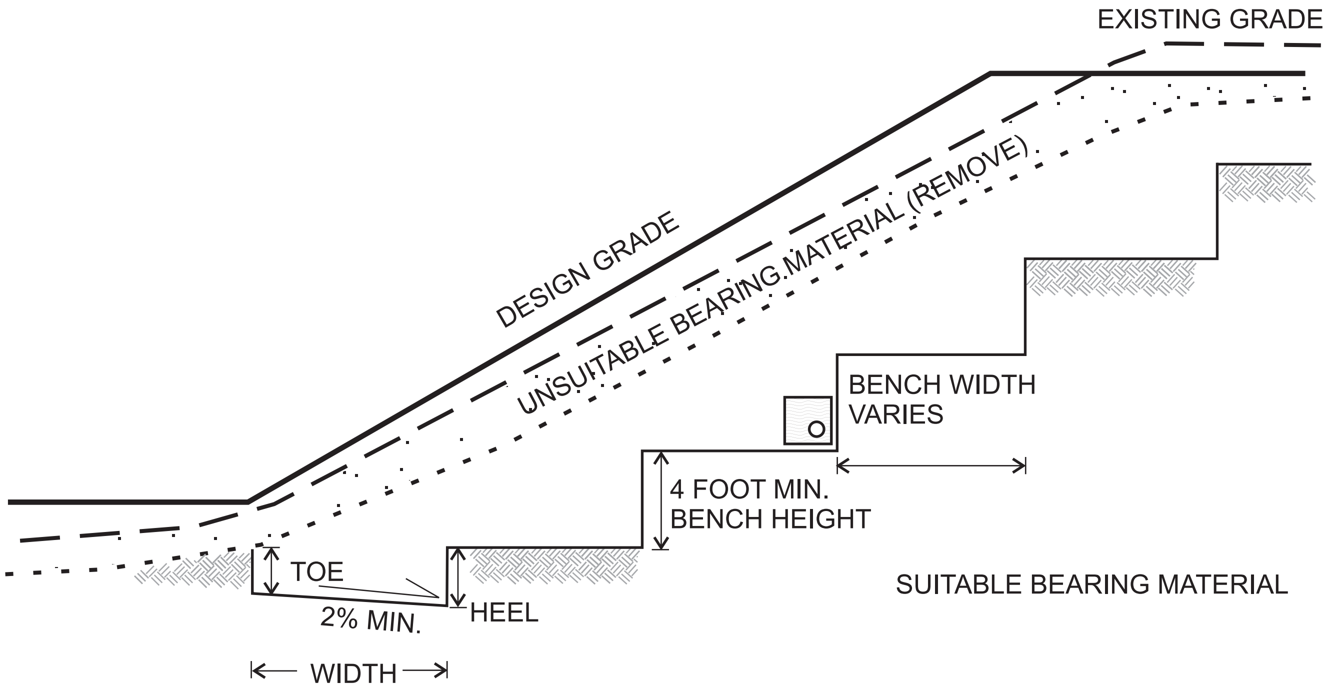
1. WHEN THE NATURAL SLOPE APPROACHES OR EXCEEDS THE DESIGN GRADE SLOPE RATIO, SPECIAL RECOMMENDATIONS ARE NECESSARY BY THE GEOTECHNICAL CONSULTANT

2. THE GEOTECHNICAL CONSULTANT WILL DETERMINE THE REQUIREMENT FOR AND LOCATION OF SUBSURFACE DRAINAGE SYSTEMS.

3. MAINTAIN MINIMUM 15 FOOT HORIZONTAL WIDTH FROM FACE OF SLOPE TO BENCH/BACKCUT

VER 1.0 NTS

AGS ADVANCED GEOTECHNICAL SOLUTIONS	FILL OVER NATURAL SLOPE	DETAIL 5



CODE COMPLIANT KEYWAY
WITH MINIMUM DIMENSIONS:

TOE: 2 FOOT MIN.
HEEL: 3 FOOT MIN.
WIDTH: 15 FOOT MIN.

NOTES:

1. MAINTAIN MINIMUM 15 FOOT HORIZONTAL WIDTH FROM FACE OF SLOPE TO BENCH/BACKCUT
2. SEE DETAIL 2 FOR DRAIN SPECIFICATIONS

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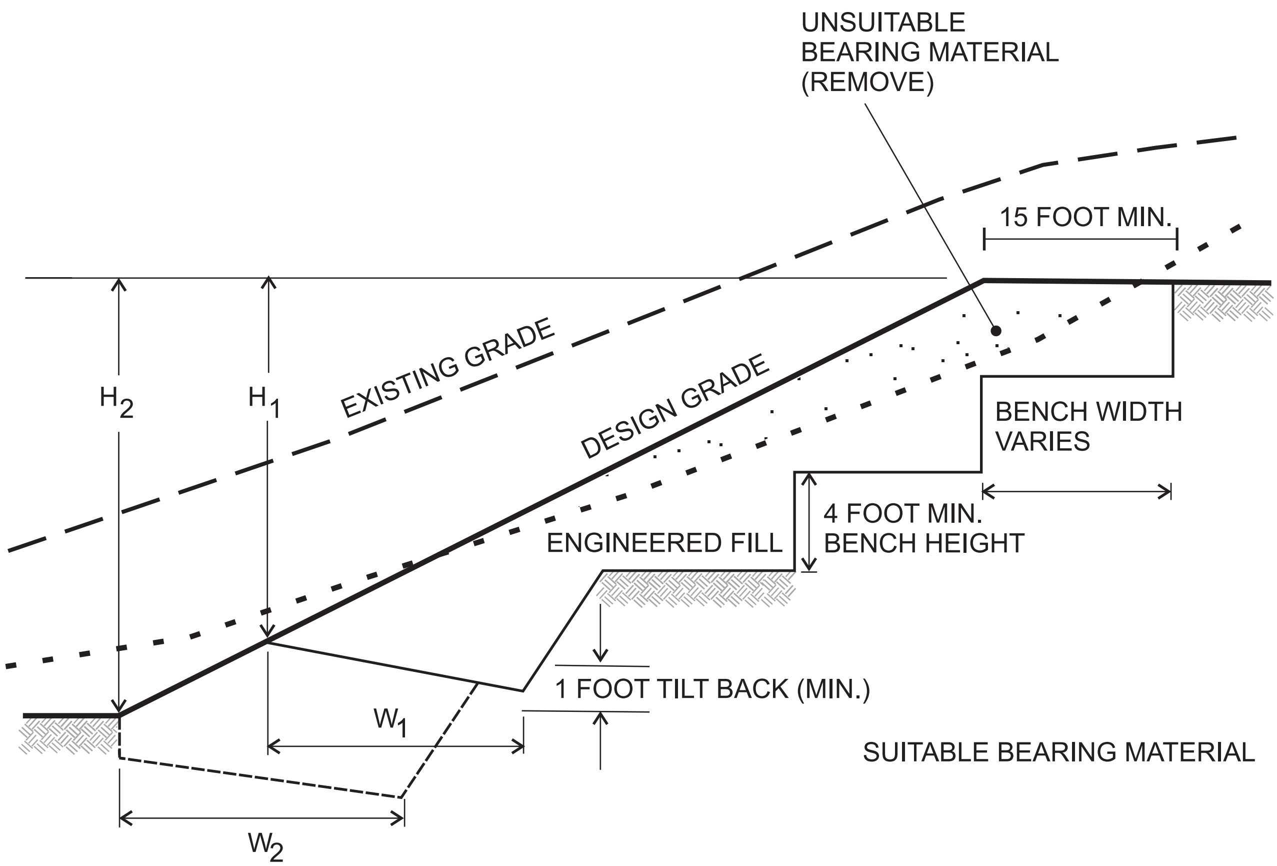
ADVANCED GEOTECHNICAL SOLUTIONS

SKIN FILL CONDITION

DETAIL 6



NOTES:

1. IF RECOMMENDED BY THE GEOTECHNICAL CONSULTANT, THE REMAINING CUT PORTION OF THE SLOPE MAY REQUIRE REMOVAL AND REPLACEMENT WITH AN ENGINEERED FILL

2. “W” SHALL BE EQUIPMENT WIDTH (15 FEET) FOR SLOPE HEIGHT LESS THAN 25 FEET. FOR SLOPES GREATER THAN 25 FEET, “W” SHALL BE DETERMINED BY THE GEOTECHNICAL CONSULTANT. AT NO TIME SHALL “W” BE LESS THAN H/2

3. DRAINS WILL BE REQUIRED (SEE DETAIL 2)

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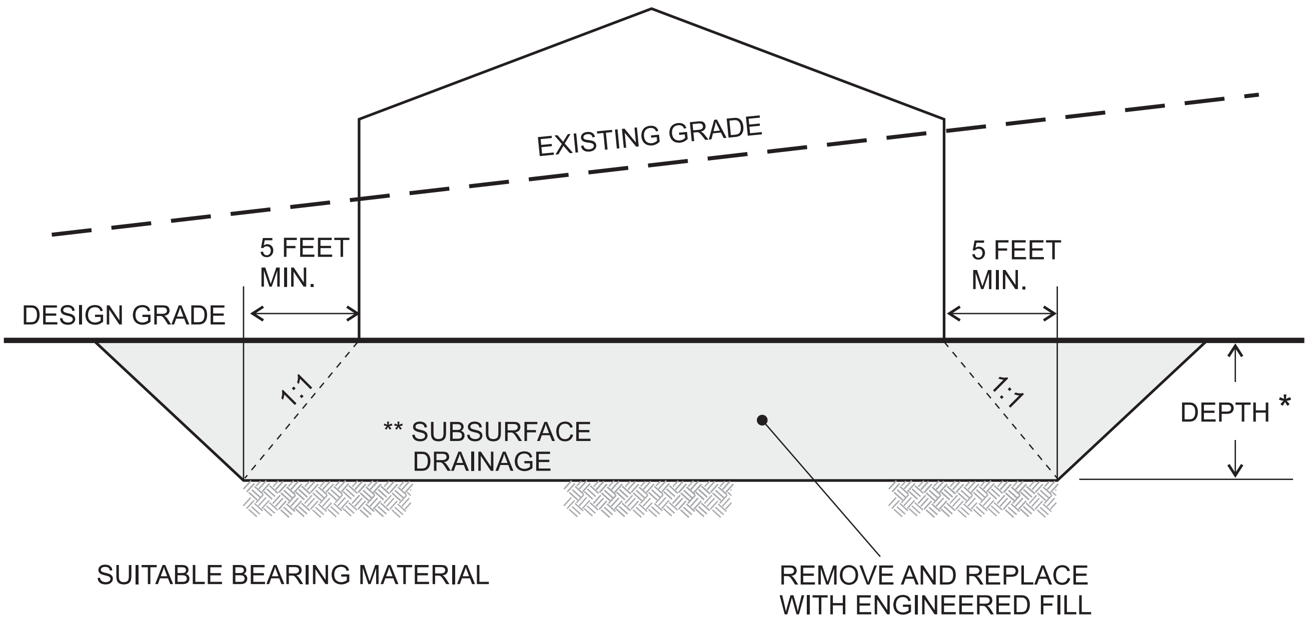
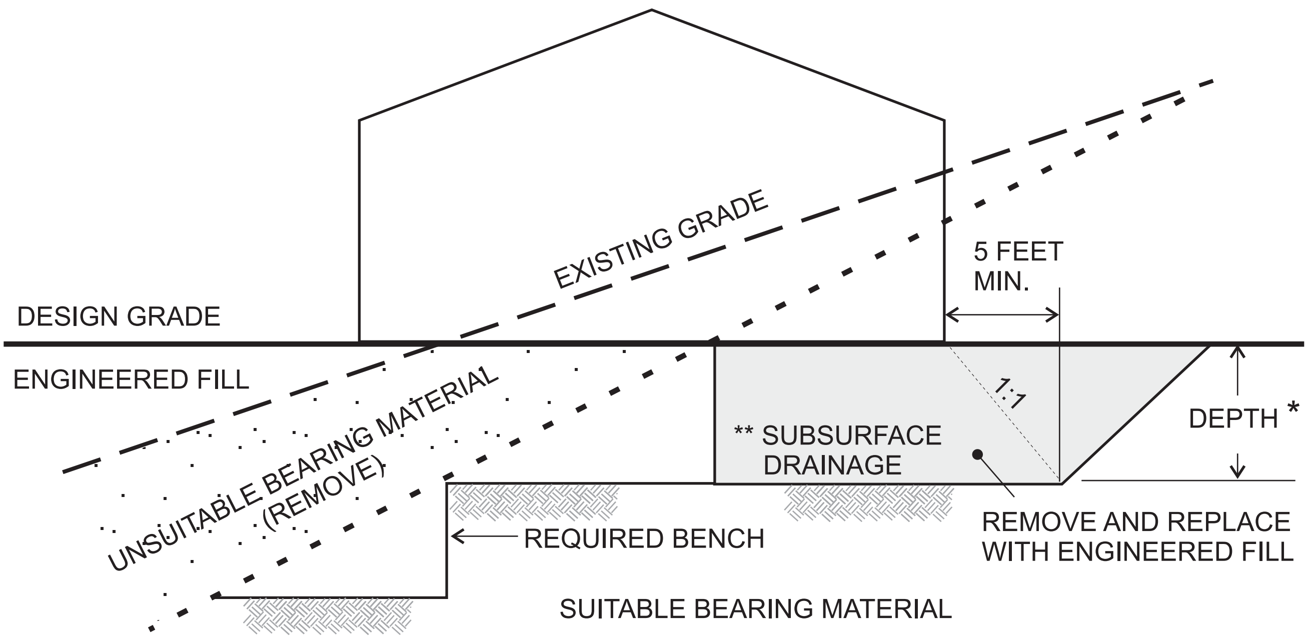
AGS ADVANCED GEOTECHNICAL SOLUTIONS	PARTIAL CUT SLOPE STABILIZATION	DETAIL 7



CUT LOT OVEREXCAVATION



CUT-FILL LOT OVEREXCAVATION

NOTES:

* SEE REPORT FOR RECOMMENDED DEPTHS, DEEPER OVEREXCAVATION MAY BE REQUIRED BY THE GEOTECHNICAL CONSULTANT BASED ON EXPOSED FIELD CONDITIONS

** CONSTRUCT EXCAVATION TO PROVIDE FOR POSITIVE DRAINAGE TOWARDS STREETS, DEEPER FILL AREAS OR APPROVED DRAINAGE DEVICES BASED ON FIELD CONDITIONS

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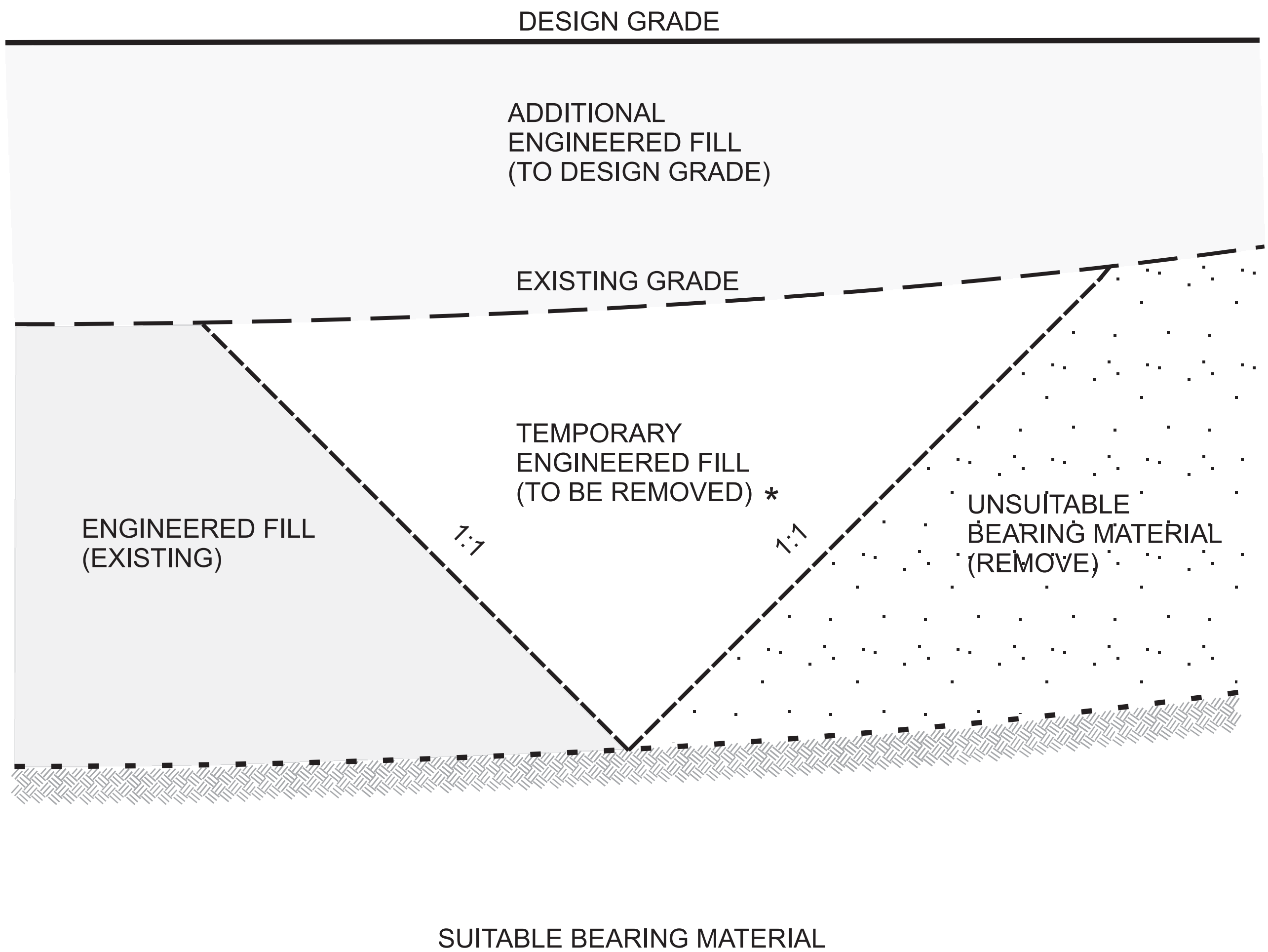
AGS ADVANCED GEOTECHNICAL SOLUTIONS	CUT & CUT-FILL LOT OVEREXCAVATION	DETAIL 8



* REMOVE BEFORE PLACING ADDITIONAL ENGINEERED FILL

TYPICAL UP-CANYON PROFILE

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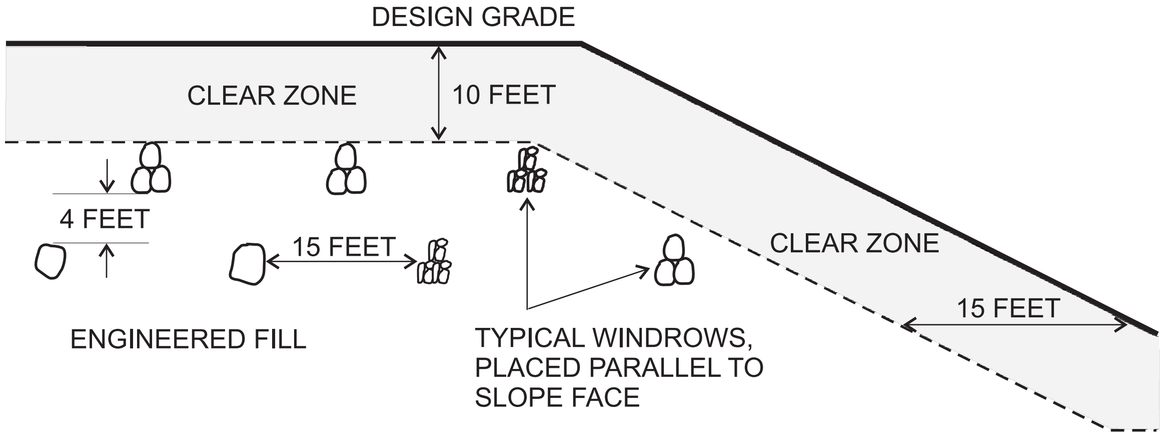
ADVANCED GEOTECHNICAL SOLUTIONS

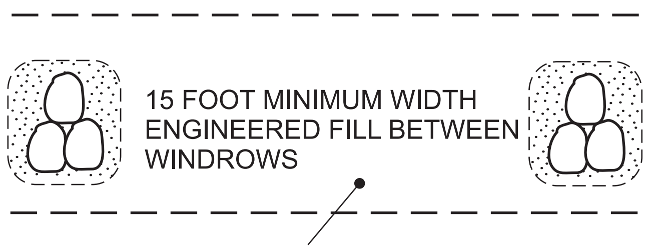
REMOVAL ADJACENT TO EXISTING FILL

DETAIL 9



CLEAR ZONE DIMENSIONS FOR REFERENCE ONLY, ACTUAL DEPTH, WIDTH, WINDROW LENGTH, ETC. TO BE BASED ON ELEVATIONS OF FOUNDATIONS, UTILITIES OR OTHER STRUCTURES PER THE GEOTECHNICAL CONSULTANT OR GOVERNING AGENCY APPROVAL

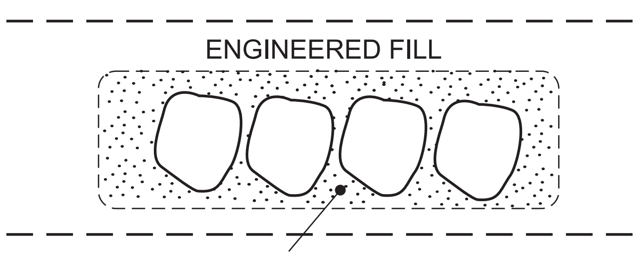
OVERSIZED MATERIAL DISPOSAL PROFILE



HORIZONTALLY PLACED ENGINEERED FILL, FREE OF OVERSIZED MATERIALS AND COMPACTED TO MINIMUM PROJECT STANDARDS

COMPACT ENGINEERED FILL ABOVE OVERSIZED MATERIALS TO FACILITATE "TRENCH" CONDITION PRIOR TO FLOODING GRANULAR MATERIALS

WINDROW CROSS-SECTION



GRANULAR MATERIAL APPROVED BY THE GEOTECHNICAL CONSULTANT AND CONSOLIDATED IN-PLACE BY FLOODING

WINDROW PROFILE

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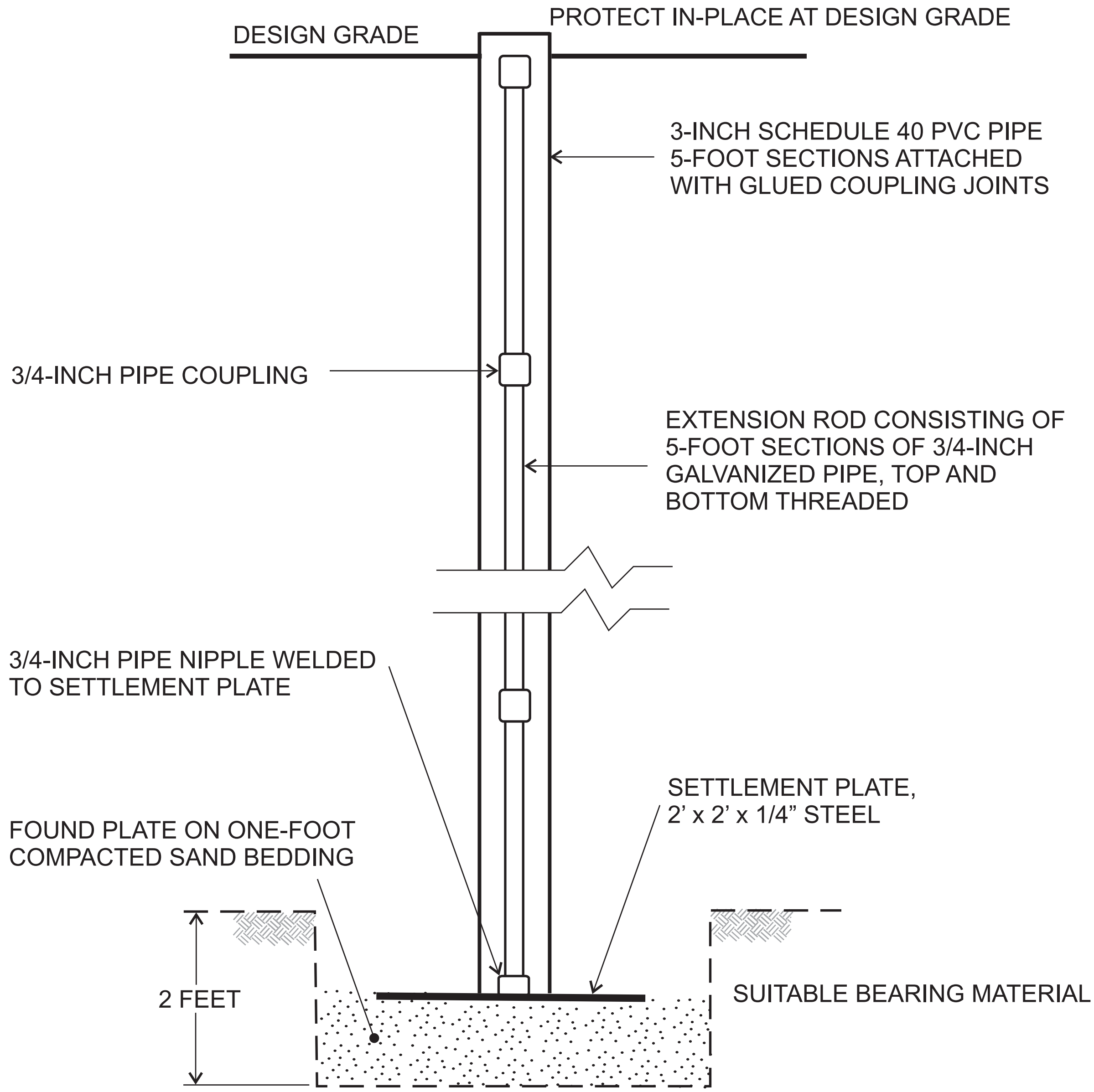
ADVANCED GEOTECHNICAL SOLUTIONS

OVERSIZED MATERIAL DISPOSAL CRITERIA

DETAIL 10



NOTES:

1. SETTLEMENT PLATE LOCATIONS SHALL BE SUFFICIENTLY IDENTIFIED BY THE CONTRACTOR AND BE READILY VISIBLE TO EQUIPMENT OPERATORS.

2. CONTRACTOR SHALL MAINTAIN ADEQUATE HORIZONTAL CLEARANCE FOR EQUIPMENT OPERATION AND SHALL BE RESPONSIBLE FOR REPAIRING ANY DAMAGE TO SETTLEMENT PLATE DURING SITE CONSTRUCTION.

3. A MINIMUM 5-FOOT ZONE ADJACENT TO SETTLEMENT PLATE/EXTENSION RODS SHALL BE ESTABLISHED FOR HAND-HELD MECHANICAL COMPACTION OF ENGINEERED FILL. ENGINEERED FILL SHALL BE COMPACTED TO MINIMUM PROJECT STANDARD.

4. ELEVATIONS OF SETTLEMENT PLATE AND ALL EXTENSION ROD PLACEMENT SHALL BE DOCUMENTED BY PROJECT CIVIL ENGINEER OR SURVEYOR.

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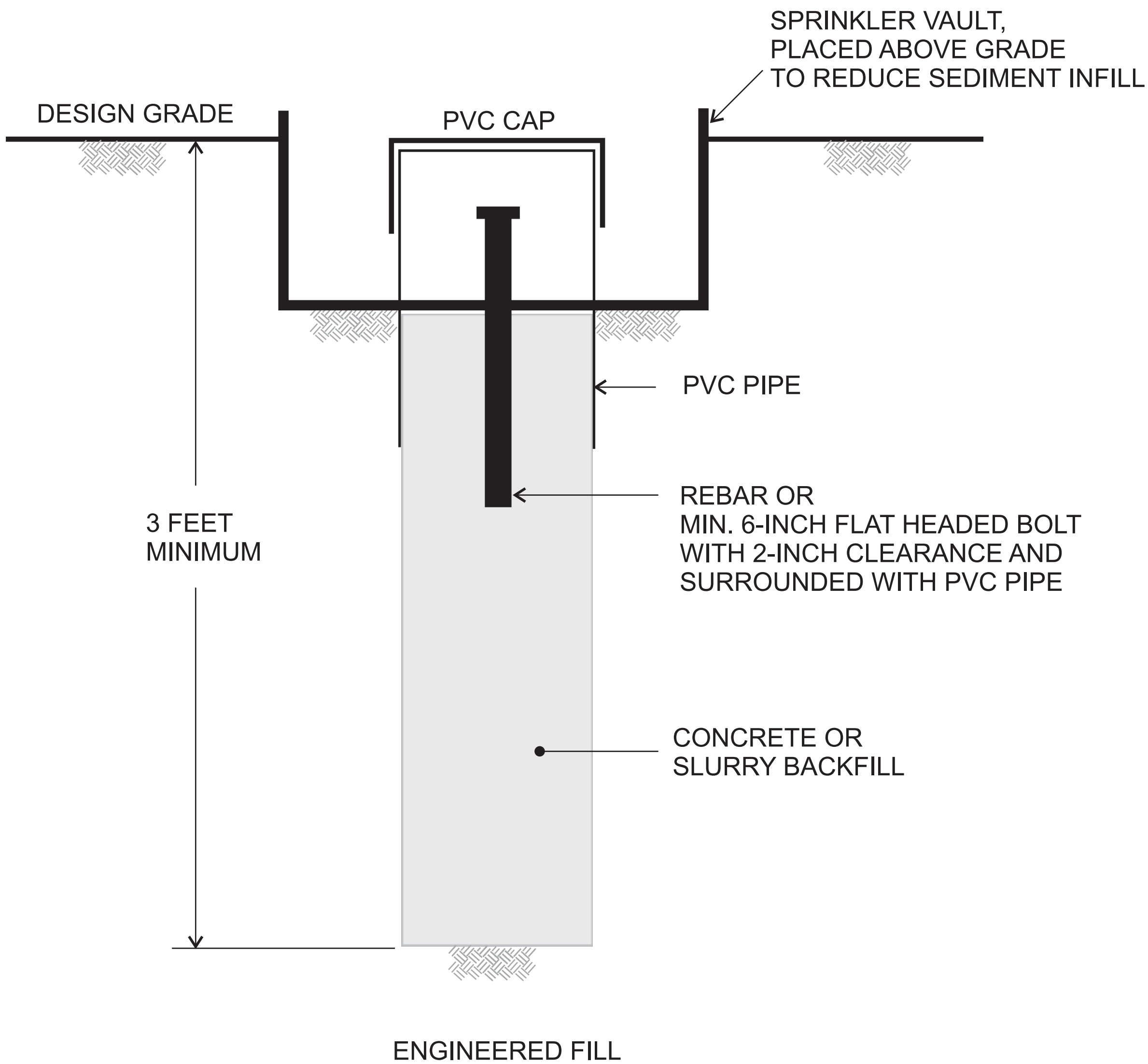
AGS ADVANCED GEOTECHNICAL SOLUTIONS	SETTLEMENT PLATE	DETAIL 11



NOTES:

1. SETTLEMENT MONUMENT LOCATIONS SHALL BE SUFFICIENTLY IDENTIFIED AND BE READILY VISIBLE TO EQUIPMENT OPERATORS.

2. ELEVATIONS OF SURFACE MONUMENTS SHALL BE DOCUMENTED BY PROJECT CIVIL ENGINEER OR SURVEYOR.

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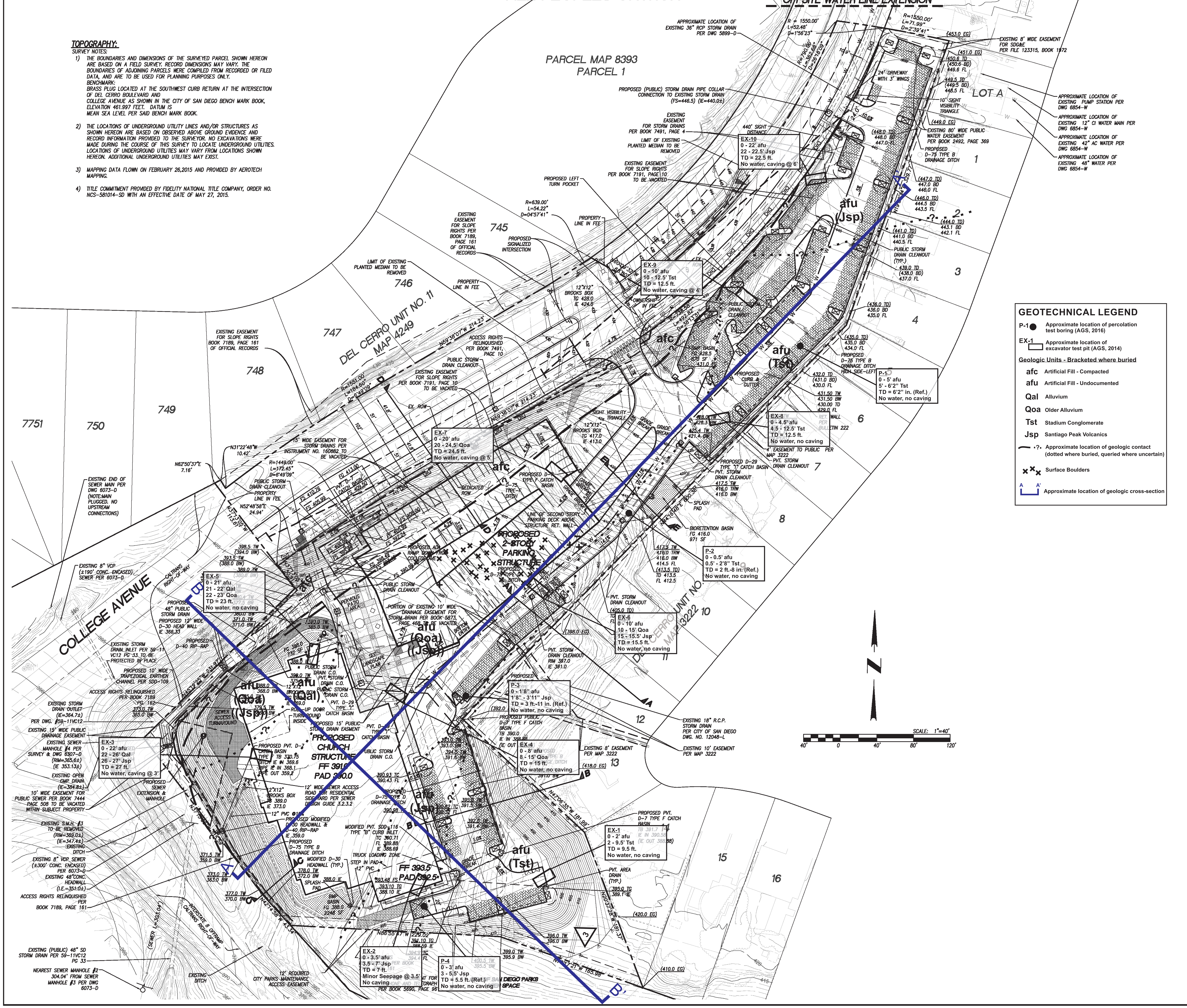
AGS ADVANCED GEOTECHNICAL SOLUTIONS	SETTLEMENT MONUMENT	DETAIL 12

PRELIMINARY GRADING PLAN
SITE DEVELOPMENT PERMIT NO. 92338, PLANNED DEVELOPMENT PERMIT NO. 92339, EASEMENT VACATION NO. 92340, AND TENTATIVE MAP NO. ______
ALL PEOPLES CHURCH

SEE SHEET 6 FOR CONTINUATION OF OFFSITE WATER LINE EXTENSION



LEGEND

- BOUNDARY LINE
- ADJACENT PROPERTY LINE
- EASEMENT LINE
- RIGHT OF WAY
- EXISTING STREET CENTERLINE
- EXISTING IMPROVEMENTS
- EXISTING CONTOUR
- EXISTING CURB & GUTTER
- EXISTING SEWER MAIN
- EXISTING STORM DRAIN MAIN
- EXISTING CURB INLET
- EXISTING WATER MAIN
- EXISTING WATER VALVE
- EXISTING FIRE HYDRANT
- EXISTING STREET LIGHT
- EXISTING STRUCTURE
- EXISTING VEGETATION
- PROPOSED IMPROVEMENTS
- PROPOSED ROAD CENTERLINE
- LIMIT OF WORK LINE
- CURB AND GUTTER
- 24' RESIDENTIAL DRIVEWAY
- PROPOSED CONTOURS
- PROPOSED SPOT ELEVATION
- RETAINING WALL
- SDRSD D-75 BROW DITCH
- BIORETENTION BASIN
- STORM DRAIN HEADWALL
- STORM DRAIN CLEANOUT
- AREA DRAIN
- STORM DRAIN INLET
- TYPE F CATCH BASIN
- RIP RAP ENERGY DISSIPATOR PER SDD-104
- PVC STORM DRAIN (SIZE VARIES)
- WATER SERVICE
- TRAFFIC SIGNAL
- PARKING GARAGE WALL
- CUT/FILL SLOPE
- PERMEABLE PAVERS
- PROPOSED RIP-RAP
- TREE PER LANDSCAPE PLANS

PREPARED BY:

PASCO LARET SUITER & ASSOCIATES
CIVIL ENGINEERING + LAND PLANNING + LAND SURVEYING
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REGISTERED PROFESSIONAL ENGINEER WILLIAM GREGG MACK No. 73620 Exp. 12/31/20 CIVIL STATE OF CALIFORNIA

WILLIAM GREGG MACK R.C.E. 73620
EXPIRATION: 12/31/2020 DATE

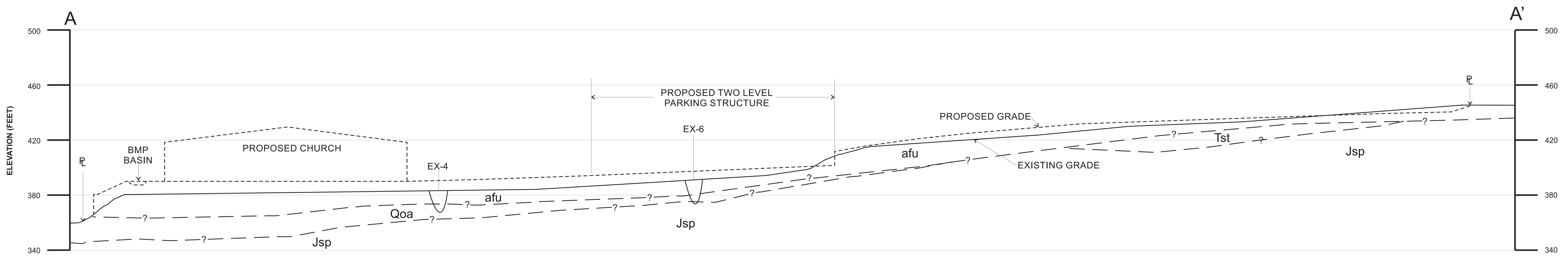
PROJECT NAME: ALL PEOPLES CHURCH
PROJECT ADDRESS: NORTHEAST CORNER OF COLLEGE AVENUE & INTERSTATE 8, SAN DIEGO, CALIFORNIA 92120
PROJECT TRACKING SYSTEM NUMBER: 636444
INTERNAL ORDER NUMBER: PENDING
SHEET TITLE: PRELIMINARY GRADING PLAN
SHEET NUMBER: C 2.0 OF 7

DRAWN BY: GJK
CHECKED BY: W. MACK
ORIGINAL DATE: 04-22-2019
REVISIONS:
1. 03-17-2020 11. ____
2. ____ 12. ____

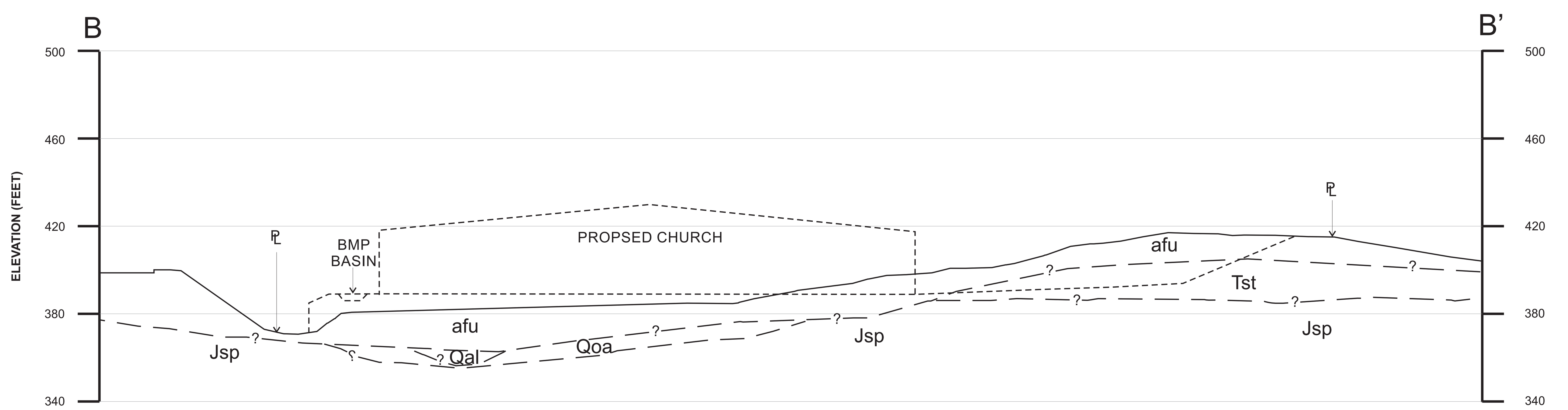
PLATE 1
Geologic Map and Exploration Location Plan

AGS ADVANCED GEOTECHNICAL SOLUTIONS, INC.

Project:	Report:	Date:
P/W 1805-05	1805-05-B-3	Oct. 2020



CROSS-SECTION A-A'
SCALE 1"=40.0' H&V



CROSS-SECTION B-B'
SCALE 1"=40.0' H&V

PLATE 2
Geologic Cross-Sections A-A' & B-B'

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ADVANCED GEOTECHNICAL SOLUTIONS, INC.

Project:	Report:	Date:
P/W 1805-05	1805-05-B-3	Oct. 2020