PRELIMINARY UPDATE GEOTECHNICAL INVESTIGATION

MERGE 56 – UNIT 1
SAN DIEGO, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

ALEXANDRIA®

SEPTEMBER 10, 2021
PROJECT NO. 06021-52-14

Project No. 06021-52-14
September 10, 2021

Alexandria Real Estate Equities, Inc.
10996 Torreyana Road, Suite 250
San Diego, California 92121

Attention: Mr. Jason Moorhead

Subject: PRELIMINARY UPDATE GEOTECHNICAL INVESTIGATION
MERGE 56 – UNIT 1
SAN DIEGO, CALIFORNIA

Dear Mr. Moorhead:

In accordance with your request and authorization of our Proposal No. LG-21421 dated September 1, 2021, we herein submit the results of our preliminary update geotechnical investigation for the subject project. We performed our investigation to evaluate the underlying soil and geologic conditions and potential geologic hazards, and to assist in the design of the proposed development and associated improvements.

The accompanying report presents the results of our study and conclusions and recommendations pertaining to geotechnical aspects of the proposed project. The site is suitable for the proposed buildings and improvements provided the recommendations of this report are incorporated into the design and construction of the planned project. We should be contacted to update this report once grading plans for the planned development have been prepared.

Should you have questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

GEOCON INCORPORATED

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PRELIMINARY UPDATE GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our preliminary update geotechnical investigation for the proposed Merge 56 – Unit 1 development located in the Rancho Penasquitos area of the City of San Diego, California (see Vicinity Map).

Vicinity Map

The purpose of this preliminary geotechnical investigation is to review the applicable geotechnical documents and existing geologic information (see List of References), and evaluate the existing geologic conditions and the geologic/geotechnical hazards that may affect the development of the property including faulting, liquefaction and seismic shaking based on the 2019 CBC seismic design criteria. In addition, we provided preliminary recommendations for remedial grading, excavations, foundations, concrete slabs-on-grade and retaining walls. We also reviewed the following geotechnical documents in preparation of this report:

1. *Geotechnical Investigation, Merge 56, San Diego, California*, prepared by Geocon Incorporated, dated December 13, 2018 (Project No. 06021-32-04B).

2. *Final Report of Testing and Observation Services During Verdura MSE Retaining Wall Construction, Merge 56, San Diego, California*, prepared by Geocon Incorporated, dated January 18, 2021 (Project No. 06021-32-09).

3. *As-Graded Geotechnical Conditions, Merge 56 – Unit 1, San Diego, California,* prepared by Geocon Incorporated, dated June 23, 2021 (Project No. 06021-32-09).

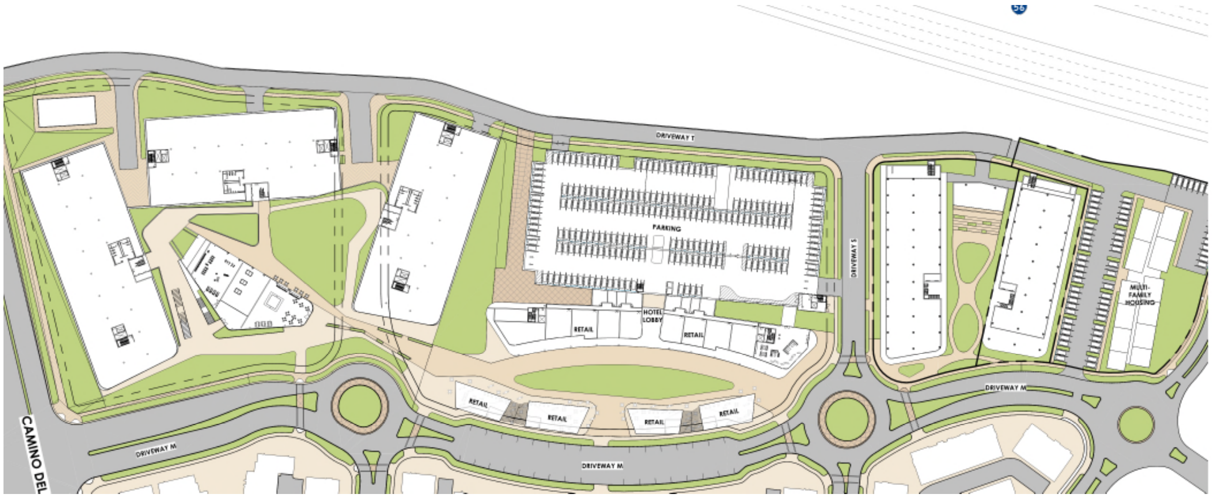
Our scope of services included a review of the referenced geotechnical reports, laboratory tests results, and the preliminary plans. The details of the laboratory tests and a summary of the test results are shown in Appendix A.

2. SITE AND PROJECT DESCRIPTION

The subject property is located south of Highway 56 and east of the Camino Del Sur roadway extension in San Diego, California. The property has recently been sheet-graded and is currently vacant and being used as construction staging. The project site is relatively flat with pad site elevations of about 374 to 386 feet above mean sea level (MSL). A descending slope and retaining wall system up to 50 feet high exists to the north. A subterranean storm water management system is located on the western portion of the site. The Existing Site Map shows the current configuration of the property.

Existing Site Map

Based on the preliminary site plan, we understand the planned development will consist of the construction of 8 office/retail structures, a parking structure, a multi-story housing building and an amenity building with accommodating roadways, utilities and landscaping. The office/retail buildings will be 4 to 6 stories, the parking structure will be 5 levels and the amenity building will be 1 to 2 stories. In addition, four, 1- to 2-story retail buildings will be constructed on the southern portion of the site adjacent to the proposed driveway. We expect the grading will consist of cuts and fills of about 5 feet. The proposed structures will likely be supported on a combination of shallow foundations and cast-in-drilled-hole (CIDH) piles embedded into the underlying formational materials. The Proposed Site Plan shows the current plan development.



Proposed Site Plan

The locations, site descriptions and proposed development are based on our site reconnaissance, review of published geologic literature, previous field investigations and grading, and discussions with project personnel. If development plans differ from those described herein, Geocon Incorporated should be contacted for review of the plans and possible revisions to this report.

3. PREVIOUS GEOTECHNICAL STUDIES AND SITE GRADING

We previously prepared the referenced geotechnical investigation report for the Merge 56 development in 2018, and we recently observed the mass grading for the Merge 56 site including the construction of the mechanically stabilized earth (MSE) retaining walls located to the north. Mass grading of the site development created sheet-graded pads with maximum cuts from natural grade of approximately 20 feet and fill of up to approximately 65 feet deep. The development originally consisted of hillside topography with a generally west flowing drainage course traversing the north end of the site. Elevations ranged from a high of approximately 405 feet above Mean Sea Level (MSL) within the south-central portion of the site to a low of approximately 310 feet MSL at the northwest end of the site within the previously existing drainage course running along the northern end of the site. The general geologic conditions prior to mass grading consisted of surficial soil composed of topsoil, alluvium, colluvium, and shallow landslide deposits overlying formational materials of the Mission Valley Formation/Stadium Conglomerate (undifferentiated).

The recent mass grading operations consisted of canyon clean-outs, subdrain placement, and the removal of unsuitable materials (i.e. loose surficial soil and vegetation) prior to the placement of fill. The formational portion of the site that possessed a cut/fill transition, or where bedrock was exposed within 3 feet of grade, was excavated a minimum of 3 feet below proposed grades and replaced with compacted fill. Subdrains were installed in the canyon areas that consist of 6-inch diameter perforated PVC pipe encapsulated in gravel and filter fabric. The drains were placed at the base of the remedial excavations where a depression occurred in the bedrock topography as shown on the Geologic Map, Figure 1. In addition, the current grades were achieved with the construction of two MSE walls located along the north perimeter of the site, and a subterranean storm water management device (HMP-A) was constructed within the northwest end of the site (see Geologic Map, Figure 1). We understand the grading for Merge 56 – Unit 1 is not complete, and additional grading will be required to achieve proposed grades. The final grading report has not been prepared for the Merge 56 – Unit 1 site.

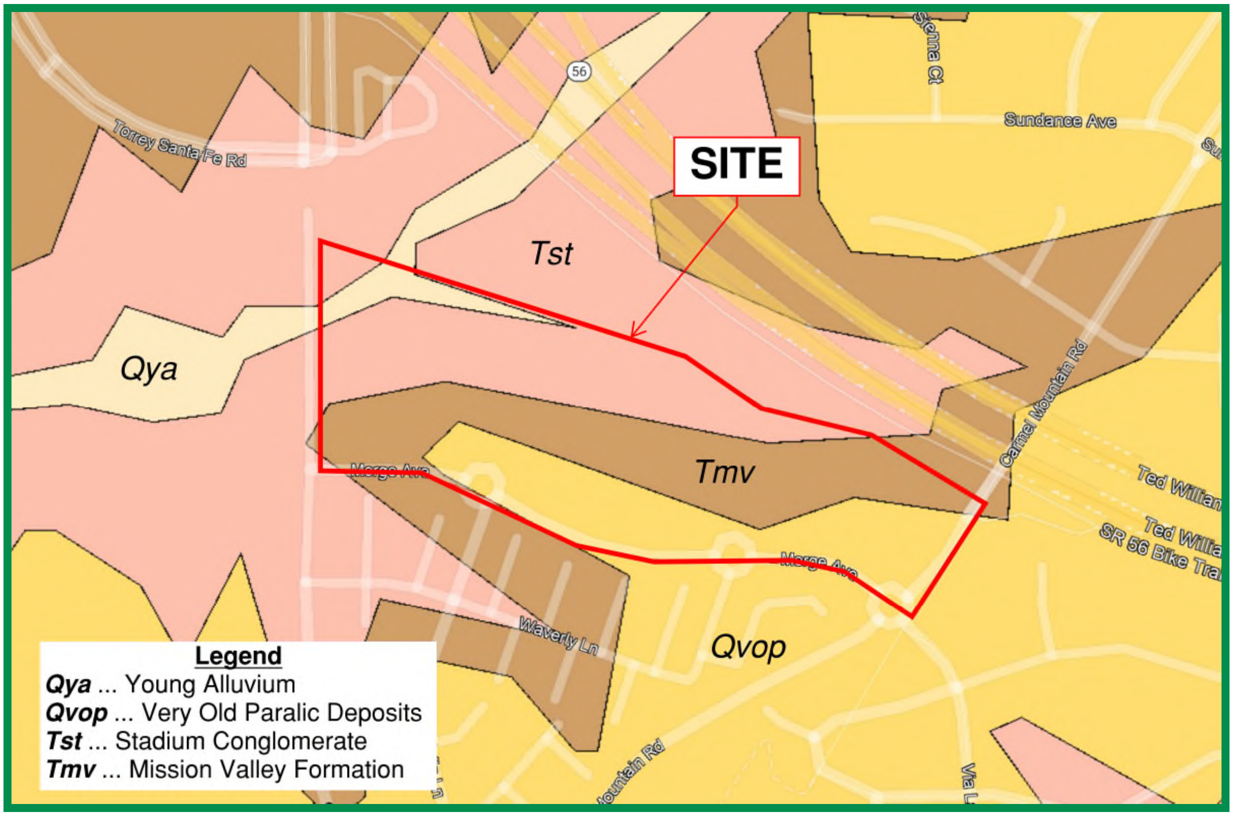
Some rocks were incorporated into deeper portions of the fill on the site. In general, rock fragments greater than 6 inches in maximum dimension were not placed within 3 feet of the proposed finish grade and the zone from 3 to 10 feet below finish grade was limited to material smaller than 12 inches.

Geocon Incorporated provided the testing and observation services during grading operations and construction of the northern MSE walls that consisted of performing laboratory and compaction testing. The field density test results indicate that the fill soil observed was placed at a dry density of at least 90 percent of the laboratory maximum dry density. The Geologic Map, Figure 1 shows the limits of the recent mass grading on-site and the conditions of the site subsequent to mass grading operations.

4. GEOLOGIC SETTING

The site is located in the eastern portion of the coastal plain within the southern portion of the Peninsular Ranges Geomorphic Province of southern California. The Peninsular Ranges is a geologic and geomorphic province that extends from the Imperial Valley to the Pacific Ocean and from the Transverse Ranges to the north and into Baja California to the south. The coastal plain of San Diego County is underlain by a thick sequence of relatively undisturbed and non-conformable sedimentary rocks that thicken to the west and range in age from Upper Cretaceous through the Pleistocene with intermittent deposition. The sedimentary units are deposited on bedrock Cretaceous to Jurassic age igneous and metavolcanic rocks. Geomorphically, the coastal plain is characterized by a series of 21, stair-stepped marine terraces (younger to the west) that have been dissected by west flowing rivers. The coastal plain is a relatively stable block that is dissected by relatively few faults consisting of the potentially active La Nacion Fault Zone and the active Rose Canyon Fault Zone. The Peninsular Ranges Province is also dissected by the Elsinore Fault Zone that is associated with and sub-parallel to the San Andreas Fault Zone, which is the plate boundary between the Pacific and North American Plates.

Sedimentary units make up the geologic sequence encountered on the site and consist of Eocene-age undifferentiated Eocene-age Mission Valley Formation/Stadium Conglomerate (Tst/Tmv, undifferentiated). Quaternary-age Very Old Paralic Deposits were previously mapped (Kennedy and Tan, 2008) to be likely exposed at the higher elevation on portions of the property and were likely excavated out during the site grading operations. The mapped regional geology at the site is shown on the Regional Geologic Map.



Regional Geologic Map

5. SOIL AND GEOLOGIC CONDITIONS

Based on the recent grading performed and the referenced geotechnical documents for the site, the Merge 56 – Unit 1 is underlain by compacted fill overlying Eocene-age Mission Valley Formation/Stadium Conglomerate. The Geologic Map, Figure 1, depicts the approximate lateral limits of the geologic units and location of existing canyon subdrains. We used the grading plans prepared by Latitude 33 Engineering and Planning provided on August 31, 2021 for the base map used in Figure 1. The geologic and soil descriptions are provided herein in order of increasing age.

5.1 Compacted Fill (Qcf and Quc)

The site is underlain by 3 to 65 feet of compacted fill placed during recent mass grading operations in 2020. The fill placed during mass grading is comprised of silty to clayey sands, gravels and sandy clays. We provided testing and observation services during the placement of the fill and we will provide the testing. In general, rock fragments greater than 6 inches in maximum dimension were not placed within 3 feet of the proposed finish grade and the zone from 3 to 10 feet below finish grade was limited to material smaller than 12 inches. Based on previous laboratory testing, we expect the upper 3 feet of fill placed during mass grading on Merge 56 – Unit 1 possesses a "very low" to "low" expansion potential (expansion index of 50 or less). The fill is considered suitable for support of additional fill and/or structural loads in its present condition. However, remedial grading of the upper portion of the fill will be required. The fill materials can be reused for new compacted fill during grading operations provided they are free of roots and debris.

5.2 Mission Valley Formation/Stadium Conglomerate (Tmv/Tst)

The Mission Valley Formation/Stadium Conglomerate (undifferentiated) underlies the compacted fill. The Mission Valley Formation generally consists of dense to very dense, clayey, fine- to medium-grained sandstone. The Stadium Conglomerate typically consists of dense to very dense, light brown to orange brown silty, fine to medium sandstone and gravel/cobble conglomerate. Hard concretionary zones are common in the formational materials. Excavations into formation exposed at grade, or those that extend through the fill and into the formations, may encounter excavation difficulty, potentially cemented and non-rippable areas, and generate oversize material. The oversize material may require special handling techniques, rock-breaking equipment and exportation from the site. The Mission Valley Formation/Stadium Conglomerate is considered suitable to support additional fill or structural loads.

6. GROUNDWATER

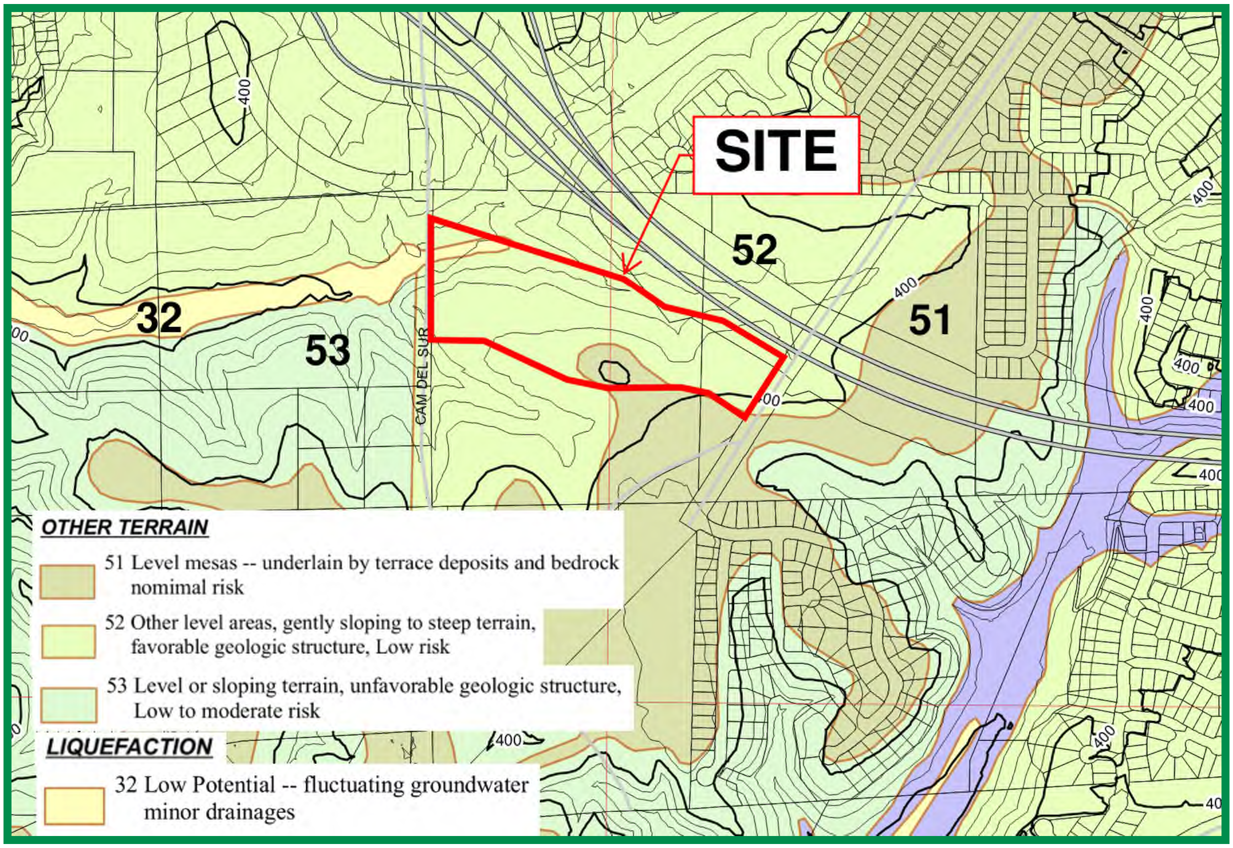
We did not encounter a static, near-surface groundwater table during previous grading operations or during the previous investigation. However, perched groundwater and/or seepage was encountered within alluvial drainage areas during the previous investigation. We do not expect groundwater would significantly affect project development. It is not uncommon for seepage conditions to develop where none previously existed due to the permeability characteristics of the geologic units encountered on site. During the rainy season, seepage conditions may develop that would require special consideration during grading operations. Groundwater elevations are dependent on seasonal precipitation, irrigation and land use, among other factors, and vary as a result. Proper surface drainage will be critical to future performance of the project.

7. GEOLOGIC HAZARDS

7.1 Geologic Hazard Category

The City of San Diego Seismic Safety Study, Geologic Hazards and Faults, Sheet 39 defines the site with *Hazard Category 52: Other level areas, gently sloping to steep terrain, favorable geologic structure, Low risk;* and small portions of the site defined as *Hazard Category 51: Level mesas – underlain by terrace deposits and bedrock, nominal risk,* and *Hazard Category 32: Liquefaction, Low*

Potential – Fluctuating groundwater, minor drainages. (as shown on the Hazard Category Map). Based on a review of the map, a fault does not traverse the planned development area.



Hazard Category Map

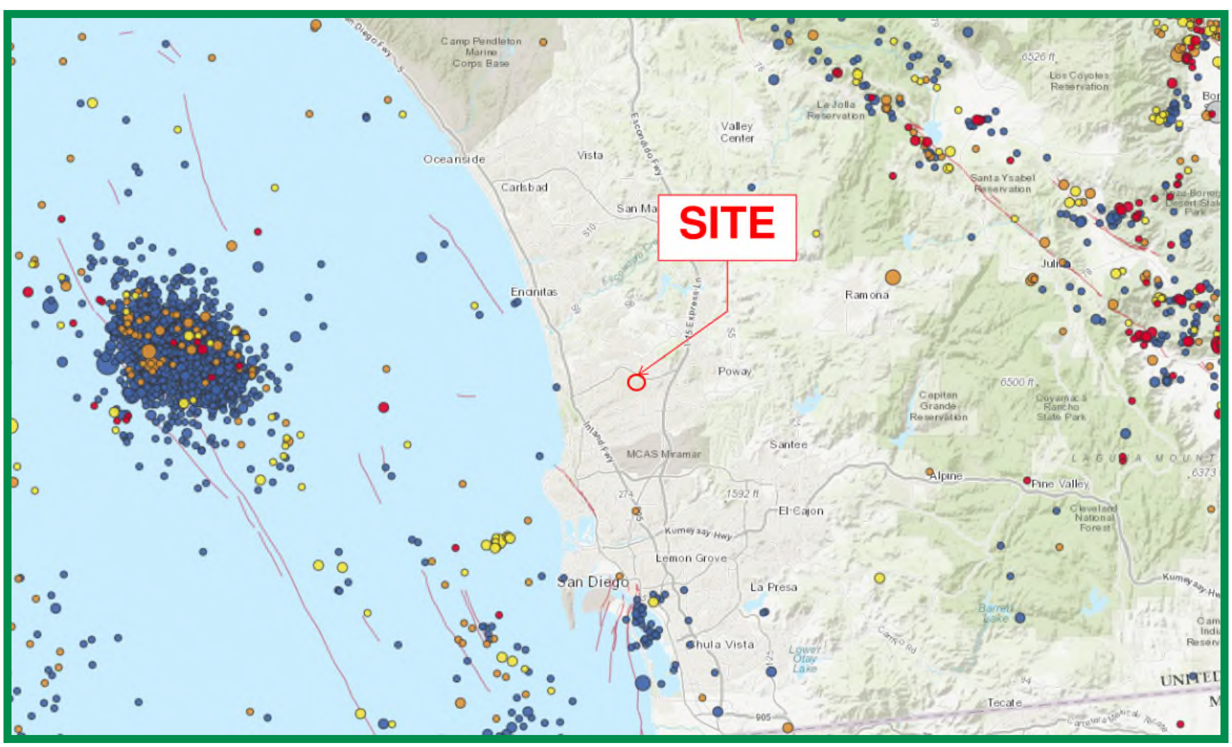
7.2 Faulting and Seismicity

A review of the referenced geologic materials and our knowledge of the general area indicate that the site is not underlain by active, potentially active, or inactive faults. An active fault is defined by the California Geological Survey (CGS) as a fault showing evidence for activity within the last 11,700 years. The site is not located within a State of California Earthquake Fault Zone.

The USGS has developed a program to evaluate the approximate location of faulting in the area of properties. The following figure shows the location of the existing faulting in the San Diego County and Southern California region. The fault traces are shown as solid, dashed and dotted that represent well-constrained, moderately constrained and inferred, respectively. The fault line colors represent fault with ages less than 150 years (red), 15,000 years (orange), 130,000 years (green), 750,000 years (blue) and 1.6 million years (black).

Faults in Southern California

The San Diego County and Southern California region is seismically active. The following figure presents the occurrence of earthquakes with a magnitude greater than 2.5 from the period of 1900 through 2015 according to the Bay Area Earthquake Alliance website.



Earthquakes in Southern California

Considerations important in seismic design include the frequency and duration of motion and the soil conditions underlying the site. Seismic design of structures should be evaluated in accordance with the California Building Code (CBC) guidelines currently adopted by the local agency.

7.3 Liquefaction

Liquefaction typically occurs when a site is located in a zone with seismic activity, onsite soils are cohesionless or silt/clay with low plasticity, groundwater is encountered within 50 feet of the surface and soil densities are less than about 70 percent of the maximum dry densities. If the four previous criteria are met, a seismic event could result in a rapid pore water pressure increase from the earthquake-generated ground accelerations. Due to the lack of a permanent, near-surface groundwater table and the very dense nature of the underlying formational Mission Valley Formation/Stadium Conglomerate, liquefaction potential for the site is considered very low.

7.4 Storm Surge, Tsunamis, and Seiches

Storm surges are large ocean waves that sweep across coastal areas when storms make landfall. Storm surges can cause inundation, severe erosion and backwater flooding along the water front. The site is located approximately 6½ miles from the Pacific Ocean at an elevation of greater than approximately 370 feet MSL; therefore, the potential of storm surges affecting the site is considered low.

A tsunami is a series of long period waves generated in the ocean by a sudden displacement of large volumes of water. Causes of tsunamis include underwater earthquakes, volcanic eruptions, or offshore slope failures. The risk of a tsunami hazard at the site to be very low due to the distance from the Pacific Ocean and the site elevations.

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement. The site is not located near an inland body of water; therefore, we consider the potential for seiches to impact the site low.

7.5 Slope Stability

Existing fill slopes and mechanically stabilized earth (MSE) retaining walls with combined heights up to approximately 25 feet exist on the north of the site. Our referenced reports indicate the slope stability analyses for the existing fill and slopes possesses a calculated factor of safety of at least 1.5 under static conditions for both deep-seated and surficial failure.

Slopes should be landscaped with drought-tolerant vegetation having variable root depths and requiring minimal landscape irrigation. In addition, slopes should be drained and properly maintained to reduce erosion.

7.6 Erosion

The site is relatively flat and is not located adjacent to the Pacific Ocean coast or a free-flowing drainage where active erosion is occurring. Provided the engineering recommendations herein are followed and the project civil engineer prepares the grading plans in accordance with generally-accepted regional standards, we do not expect erosion to be a major impact to site development.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 We did not encounter soil or geologic conditions during our exploration that would preclude the proposed development, provided the recommendations presented herein are followed and implemented during design and construction. We will provide supplemental recommendations if we observe variable or undesirable conditions during construction, or if the proposed construction will differ from that anticipated herein.

8.1.2 With the exception of possible moderate to strong seismic shaking, we did not observe or know of significant geologic hazards to exist on the site that would adversely affect the proposed project.

8.1.3 Based on a review of the referenced documents and our observations during recent grading, the Merge 56 – Unit 1 site is generally underlain by approximately 3 to 65 feet of compacted fill overlying the Mission Valley Formation/Stadium Conglomerate. Remedial grading of the upper portions of the surficial materials should be performed as discussed herein. The Mission Valley Formation/Stadium Conglomerate are considered suitable for the support of proposed fill and structural loads.

8.1.4 We did not encounter groundwater during our subsurface exploration and grading and we do not expect it to be a constraint to project development. However, seepage within surficial soils and formational materials may be encountered during the grading operations, especially during the rainy seasons.

8.1.5 We expect the grading for the planned improvements will consists of additional cuts and fills of about 5 feet. Based on the varying depths of fill below the planned buildings, the proposed structures will likely be supported on a combination of shallow foundations and cast-in-drilled-hole (CIDH) piles embedded into the underlying formational materials, or on shallow foundations embedded into fill.

8.1.6 Excavations of the compacted fill and the Mission Valley Formation/Stadium Conglomerate should generally be possible with moderate to heavy effort using conventional, heavy-duty equipment during grading and trenching operations. In addition, we expect very heavy effort with possible refusal in localized areas for excavations into strongly cemented portions of the Mission Valley/Stadium Conglomerate that may generate oversized material using conventional heavy-duty equipment during the grading operations. Oversized rock (rocks greater than 12-inches in dimension) may be generated within the formational Mission

Valley Formation/Stadium Conglomerate materials that can be incorporated into landscape use or deep compacted fill areas, if available.

8.1.7 Proper drainage should be maintained in order to preserve the engineering properties of the fill in both the building pads and slope areas. Recommendations for site drainage are provided herein.

8.1.8 We should perform a storm water management investigation under a separate report to help evaluate the potential for infiltration on the property. The project civil engineer should use that report to help design the storm water management devices, if planned.

8.1.9 Based on our review of the project plans, we opine the planned development can be constructed in accordance with our recommendations provided herein. We do not expect the planned development will destabilize or result in settlement of adjacent properties if properly constructed.

8.2 Excavation and Soil Characteristics

8.2.1 Excavation of the in-situ soil should be possible with moderate to heavy effort using conventional heavy-duty equipment. Excavation of the formational materials will require very heavy effort and may generate oversized material using conventional heavy-duty equipment during the grading operations. Oversized rock (rocks greater than 12 inches in dimension) may be generated within the formational materials that can be incorporated into landscape use or deep compacted fill areas, if available.

8.2.2 The soil encountered during recent grading is considered to be “non-expansive” and “expansive” (expansion index [EI] of 20 or less and greater than 20, respectively) as defined by 2019 California Building Code (CBC) Section 1803.5.3. We expect a majority of the soil within the upper 5 feet of existing grade within the limits of previous mass grading possesses a “very low” to “low” expansion potential (EI of 50 or less). Table 8.2 presents soil classifications based on the expansion index.

TABLE 8.2
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	ASTM D 4829 Expansion Classification	2019 CBC Expansion Classification
0 – 20	Very Low	Non-Expansive
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

8.2.3 We performed laboratory tests on samples of the site fill materials to evaluate the percentage of water-soluble sulfate content. Appendix A presents results of the laboratory water-soluble sulfate content tests. The test results indicate the on-site materials at the locations tested possess "S0" sulfate exposure to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-14 Chapter 19. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

8.2.4 Geocon Incorporated does not practice in the field of corrosion engineering. Therefore, further evaluation by a corrosion engineer may be performed if improvements susceptible to corrosion are planned.

8.3 Preliminary Grading Recommendations

8.3.1 Grading should be performed in accordance with the recommendations provided in this report, the Recommended Grading Specifications contained in Appendix B and the applicable regulatory agency's grading ordinance. Geocon Incorporated should observe the grading operations on a full-time basis and provide testing during the fill placement.

8.3.2 Prior to commencing grading, a preconstruction conference should be held at the site with the county inspector, developer, grading and underground contractors, civil engineer, and geotechnical engineer in attendance. Special soil handling and/or the grading plans can be discussed at that time.

8.3.3 Site preparation should begin with the removal of deleterious material, debris, and vegetation. The depth of vegetation removal should be such that material exposed in cut areas or soil to be used as fill is relatively free of organic matter. Material generated during stripping and/or site demolition should be exported from the site. Asphalt and concrete should not be mixed with the fill soil unless approved by the Geotechnical Engineer.

8.3.4 Abandoned foundations and buried utilities (if encountered) should be removed and the resultant depressions and/or trenches should be backfilled with properly compacted material as part of the remedial grading.

8.3.5 In general, the upper 1 to 2 feet of compacted fill within areas of planned grading should be removed and properly compacted prior to placing additional fill and/or structural loads. The actual extent and depth of surficial soil requiring removal should be evaluated during the planned geotechnical investigation and during the grading operations. Overly wet soils, as might be encountered in the vicinity of drainages, will require drying and/or mixing with drier soils to facilitate proper compaction.

8.3.6 In addition, office and parking structure building pads should be graded such that 2 feet of compacted fill exists below finish grade. In areas of improvements, the upper 1 to 2 feet of existing material should be processed, moisture conditioned as necessary and recompacted. Undercuts should be sloped a minimum of 1 percent and drained toward the adjacent street or deepest fill.

8.3.7 Ancillary structures and the 1- to 2-story retail buildings can be supported on foundations embedded in compacted fill. The areas should be graded such that there is at least 3 feet of compacted fill below the proposed concrete slabs-on-grade and at least 2 feet of fill below the planned foundations. Table 8.3.1 provides a summary of the grading recommendations.

TABLE 8.3.1
SUMMARY OF GRADING RECOMMENDATIONS

Area	Removal Requirements
Building Pads – Office and Parking Structure	Process Upper 1 to 2 Feet of Existing Materials
	Undercut 2 Feet Below Finish Grade
Ancillary and Retail Buildings	Undercut 2 Feet Below Finish Grade
	Undercut At Least 2 Feet Below Foundations
Areas of Improvements	Process Upper 1 to 2 Feet of Existing Materials
Lateral Grading Limits	10 Feet Outside of Buildings/2 Feet Outside of Improvement Areas, Where Possible
Exposed Bottoms of Remedial Grading	Scarify Upper 12 Inches

8.3.8 We should observe the grading operations and the removal bottoms to check the exposure of the formational materials or competent previously placed fill prior to the placement of compacted fill. Deeper excavations may be required if highly weathered formational materials or saturated or loose fill soil is present at the base of the removals. Fill soil should not be placed until we observe the bottom excavations.

8.3.9 The bottom of the excavations should be sloped at least 1 percent to the adjacent street or deepest fill. Prior to fill soil being placed, the existing ground surface should be scarified, moisture conditioned as necessary, and compacted to a depth of at least 12 inches. Deeper removals may be required if saturated or loose fill soil is encountered. A representative of Geocon should be on-site during removals to evaluate the limits of the remedial grading.

8.3.10 The outer 15 feet (or a distance equal to the height of the slope, whichever is less) of fill slopes should be composed of properly compacted granular "soil" fill to reduce the potential for surficial sloughing. In general, soil with an expansion index of 90 or less and at least 35 percent sand-size particles should be acceptable as "soil" fill. Soil of questionable strength to satisfy surficial stability should be tested in the laboratory for acceptable drained shear strength. The use of cohesionless soil in the outer portion of fill slopes should be avoided. Fill slopes should be overbuilt at least 2 feet and cut back or be compacted by backrolling with a loaded sheepsfoot roller at vertical intervals not to exceed 4 feet to maintain the moisture content of the fill. The slopes should be track-walked at the completion of each slope such that the fill is properly compacted to the face of the finished slope.

8.3.11 The site should then be brought to final subgrade elevations with fill compacted in layers. In general, soil native to the site is suitable for use from a geotechnical engineering standpoint as fill if relatively free from vegetation, debris and other deleterious material. Layers of fill should be about 6 to 8 inches in loose thickness and no thicker than will allow for adequate bonding and compaction. Fill, including backfill and scarified ground surfaces, should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM Test Procedure D 1557. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill. The upper 12 inches of subgrade soil underlying pavement should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content shortly before paving operations.

8.3.12 Import fill (if necessary) should consist of the characteristics presented in Table 8.3.2. Geocon Incorporated should be notified of the import soil source and should perform laboratory testing of import soil prior to its arrival at the site to determine its suitability as fill material.

TABLE 8.3.2
SUMMARY OF IMPORT FILL RECOMMENDATIONS

Soil Characteristic	Values
Expansion Potential	"Very Low" to "Low" (Expansion Index of 50 or less)
Particle Size	Maximum Dimension Less Than 3 Inches
	Generally Free of Debris

8.4 Seismic Design Criteria – 2019 California Building Code

8.4.1 Table 8.4.1 summarizes site-specific design criteria obtained from the 2019 California Building Code (CBC; Based on the 2018 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. We used the computer program *U.S. Seismic Design Maps*, provided by the Structural Engineers Association (SEA) to calculate the seismic design parameters. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2019 CBC and Table 20.3-1 of ASCE 7-16. The values presented herein are for the risk-targeted maximum considered earthquake (MCE_R). Sites designated as Site Class D, E and F may require additional analyses if requested by the project structural engineer and client. Buildings that possess fills less than 20 feet and greater than 20 feet should possess a Site Class C and D, respectively.

TABLE 8.4.1
2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value		2019 CBC Reference
Fill Thickness, T (Feet)	T < 20	T $\geq$ 20	--
Site Class	C	D	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	0.863g	0.863g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.316g	0.316g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.2	1.155	Table 1613.2.3(1)
Site Coefficient, F_V	1.5	1.984*	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.036g	0.997g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.473g	0.626g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.690g	0.664g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.316g	0.417g*	Section 1613.2.4 (Eqn 16-39)

***Note:** Using the code-based values presented in this table, in lieu of a performing a ground motion hazard analysis, requires the exceptions outlined in ASCE 7-16 Section 11.4.8 be followed by the project structural engineer. Per Section 11.4.8 of ASCE/SEI 7-16, a ground motion hazard analysis should be performed for projects for Site Class "E" sites with Ss greater than or equal to 1.0g and for Site Class "D" and "E" sites with S1 greater than 0.2g. Section 11.4.8 also provides exceptions which indicates that the ground motion hazard analysis may be waived provided the exceptions are followed.

8.4.2 Table 8.4.2 presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-16.

TABLE 8.4.2
ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value		ASCE 7-16 Reference
Site Class	C	D	
Mapped MCE_G Peak Ground Acceleration, PGA	0.374g	0.374g	Figure 22-9
Site Coefficient, F_{PGA}	1.2	1.226	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.448	0.458g	Section 11.8.3 (Eqn 11.8-1)

8.4.3 Conformance to the criteria in Tables 8.4.1 and 8.4.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur in the event of a large earthquake. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

8.4.4 The project structural engineer and architect should evaluate the appropriate Risk Category and Seismic Design Category for the planned structures. The values presented herein assume a Risk Category of II and resulting in a Seismic Design Category D. Table 8.4.3 presents a summary of the risk categories in accordance with ASCE 7-16.

TABLE 8.4.3
ASCE 7-16 RISK CATEGORIES

Risk Category	Building Use	Examples
I	Low risk to Human Life at Failure	Barn, Storage Shelter
II	Nominal Risk to Human Life at Failure (Buildings Not Designated as I, III or IV)	Residential, Commercial and Industrial Buildings
III	Substantial Risk to Human Life at Failure	Theaters, Lecture Halls, Dining Halls, Schools, Prisons, Small Healthcare Facilities, Infrastructure Plants, Storage for Explosives/Toxins
IV	Essential Facilities	Hazardous Material Facilities, Hospitals, Fire and Rescue, Emergency Shelters, Police Stations, Power Stations, Aviation Control Facilities, National Defense, Water Storage

8.5 Settlement Due to Fill Loads

8.5.1 Fill soil, even if properly compacted, will experience settlement over the lifetime of the improvements that it supports. The ultimate settlement potential of the fill is a function of the soil classification, placement relative compaction, and subsequent increases in the soil moisture content.

8.5.2 The subject development area is underlain by a maximum fill thickness on the order of 65 fee). The settlement of compacted fill is expected to continue over a relatively extended time period resulting from both gravity loading and hydrocompression upon wetting from rainfall and/or landscape irrigation.

8.5.3 Due to the variable fill thickness, a potential for differential settlement across the proposed buildings exist and special foundation design consideration as discussed herein will be necessary. Based on measured settlement of similar fill depths on other sites and the time period since the fill was placed, we estimate that maximum settlement of the compacted fill will be approximately 0.4 percent for the existing compacted fills resulting in a maximum settlement of about 3 inches. We should estimate the total and differential fill settlement once the grading plans and pad elevations have been prepared. However, based on the Geologic Map, the worst case will be the western building with a total and differential fill thickness of about 50 feet resulting in total and fill settlement of about 2.5 inches.

8.5.4 Deep foundations such as driven piles or drilled piers are the most effective means of reducing the ultimate settlement potential of the proposed structures to a negligible amount. Alternatively, highly reinforced shallow foundation systems and slabs-on-grade may be used for support of the buildings; however, the shallow foundation systems would not eliminate the potential for cosmetic distress related to differential settlement of the underlying fill. Some cosmetic distress should be expected over the life of the structure as a result of long-term differential settlement. The owner, tenants, and future owners should be made aware that cosmetic distress, including separation of caulking at wall joints, small non-structural wall panel cracks, and separation of concrete flatwork is likely to occur.

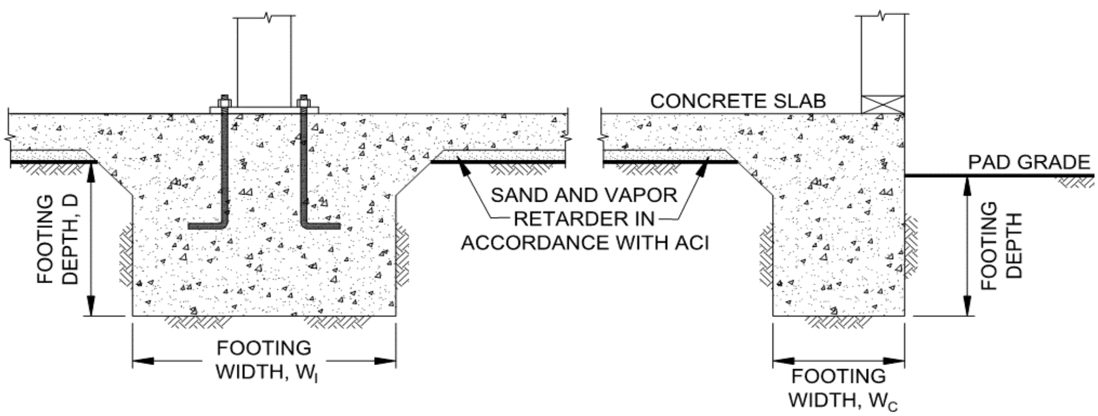
8.6 Shallow Foundations

8.6.1 The proposed structures can be supported on a shallow foundation system founded in the formational materials. The Ancillary and retail structures can be supported in compacted fill. Foundations for the structure should consist of continuous strip footings and/or isolated spread footings. Footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope. Table 8.6 provides a summary of the foundation design recommendations.

TABLE 8.6
SUMMARY OF SHALLOW FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Continuous Foundation Width, W_C	12 inches
Minimum Isolated Foundation Width, W_I	24 inches
Minimum Foundation Depth, D	24 Inches Below Lowest Adjacent Grade
	Minimum 6 Inches Into Formation
Minimum Steel Reinforcement	4 No. 5 Bars, 2 at the Top and 2 at the Bottom
Allowable Bearing Capacity	2,500 psf (Fill) / 6,000 psf (Formation)
Bearing Capacity Increase	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Allowable Bearing Capacity	4,000 psf (Fill) / 9,000 psf (Formation)
Estimated Total Settlement	½ Inch
Estimated Differential Settlement	½ Inch in 40 Feet
Footing Size Used for Settlement	9-Foot Square
Design Expansion Index	50 or less

8.6.2 The foundations should be embedded in accordance with the recommendations herein and the Wall/Column Footing Dimension Detail. The embedment depths should be measured from the lowest adjacent pad grade for both interior and exterior footings. Footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope (unless designed with a post-tensioned foundation system as discussed herein).



Wall/Column Footing Dimension Detail

8.6.3 The bearing capacity values presented herein are for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces.

8.6.4 Overexcavation of the footings and replacement with slurry can be performed in areas where formational materials are not encountered at the bottom of the footing. Minimum two-sack slurry can be placed in the excavations for the conventional foundations to the bottom of proposed footing elevation. The depth of the overexcavation may exceed 10 feet on the southern portion of the site.

8.6.5 Where buildings or other improvements are planned near the top of a slope steeper than 3:1 (horizontal:vertical), special foundations and/or design considerations are recommended due to the tendency for lateral soil movement to occur.

- For fill slopes less than 20 feet high, building footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.
- When located next to a descending 3:1 (horizontal:vertical) fill slope or steeper, the foundations should be extended to a depth where the minimum horizontal distance is equal to H/3 (where H equals the vertical distance from the top of the fill slope to the base of the fill soil) with a minimum of 7 feet but need not exceed 40 feet. The horizontal distance is measured from the outer, deepest edge of the footing to the face of the slope. An acceptable alternative to deepening the footings would be the use of a post-tensioned slab and foundation system or increased footing and slab reinforcement. Specific design parameters or recommendations for either of these alternatives can be provided once the building location and fill slope geometry have been determined.
- Although other improvements, which are relatively rigid or brittle, such as concrete flatwork or masonry walls, may experience some distress if located near the top of a slope, it is generally not economical to mitigate this potential. It may be possible, however, to incorporate design measures that would permit some lateral soil movement without causing extensive distress. Geocon Incorporated should be consulted for specific recommendations.

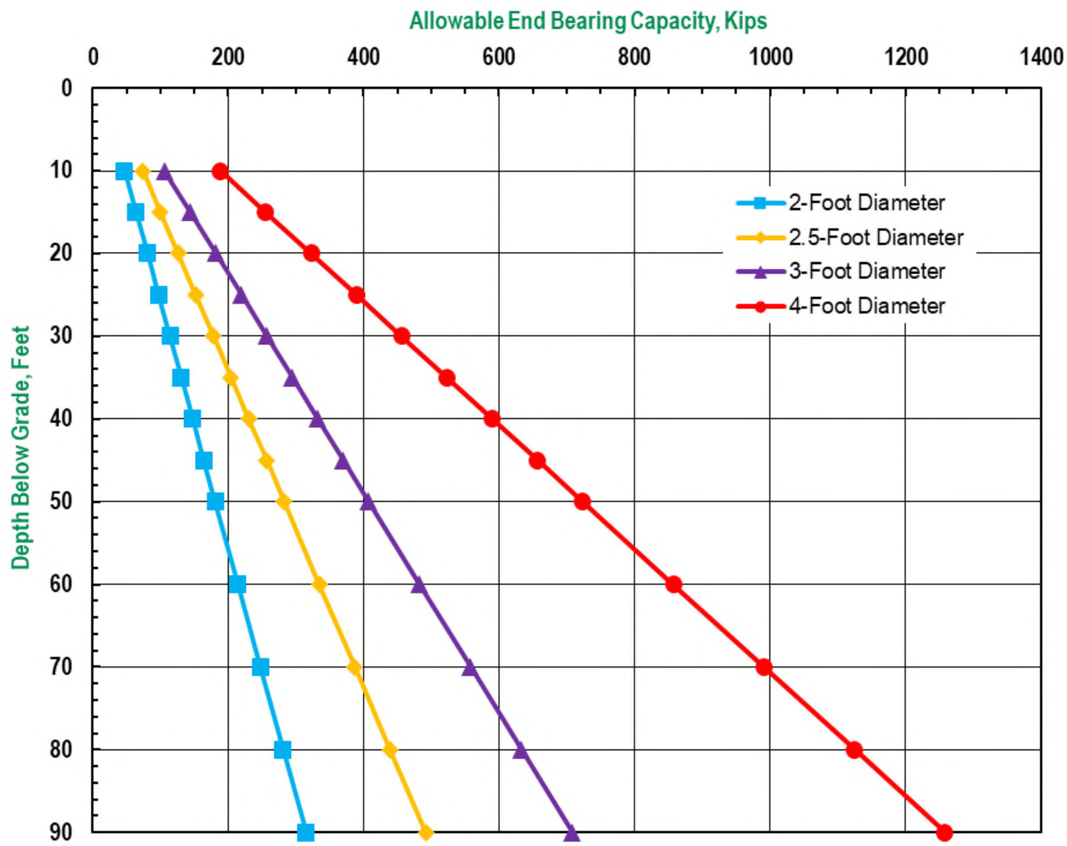
8.6.6 We should observe the foundation excavations prior to the placement of reinforcing steel and concrete to check that the exposed soil conditions are similar to those expected and that they have been extended to the appropriate bearing strata. Foundation modifications may be required if unexpected soil conditions are encountered.

8.6.7 Geocon Incorporated should be consulted to provide additional design parameters as required by the structural engineer.

8.7 Cast-In-Drilled-Hole (CIDH) Piles

8.7.1 We expect cast-in-drilled-hole (CIDH) piles may be used for foundation support due to the differential fill thicknesses below the proposed buildings. The foundation recommendations herein are for CIDH piles and assume that the piles will extend through the fill and into the underlying formational materials.

8.7.2 Piers can be designed to develop support by end bearing within the formational materials and skin friction within the formational materials and the fill soil. The end bearing capacity can be determined by the End Bearing Capacity Chart. These allowable values possess a factor of safety of at least 2 for skin friction and end bearing.



End Bearing Capacity Chart

8.7.3 The CIDH piles can be designed to develop support by end bearing within the formational materials and skin friction within the formational materials and portions of the fill soil using the design parameters presented in Table 8.7.

TABLE 8.7
SUMMARY OF CIDH PILE RECOMMENDATIONS

Parameter	Value
Minimum Pile Diameter	2 Feet
Minimum Pile Spacing	3 Times Pile Diameter
Minimum Foundation Embedment Depth	10 Feet
	3 Feet in Formational Materials
Allowable End Bearing Capacity	Per Chart
Allowable Skin Friction Capacity	200 psf (Fill Materials)
	400 psf (Formational Materials)
Estimated Total Settlement	½ Inch
Estimated Differential Settlement	½ Inch in 40 Feet

8.7.4 The allowable downward capacity may be increased by one-third when considering transient wind or seismic loads. Single pile uplift capacity can be taken as 75 percent of the allowable downward skin friction capacity.

8.7.5 If pile spacing is at least three times the maximum dimension of the pile, no reduction in axial capacity for group effects is considered necessary. If piles are spaced between 2 and 3 pile diameters (center to center), the single pile axial capacity should be reduced by 25 percent. Geocon Incorporated should be contacted to provide single-pile capacity if piles are spaced closer than 2 diameters.

8.7.6 The design tip elevation of the CIDH piles should be determined by the project structural engineer based on the Geologic Map, Figure 1. Some variation should be expected during drilling operations.

8.7.7 The fill materials may encounter large rock and the formational materials may contain gravel, cobble and very dense/cemented zones; therefore, the drilling contractor should expect difficult drilling conditions during excavations for the piles. Because a significant portion of the CIDH piles capacity will be developed by end bearing, the bottom of the borehole should be cleaned of loose cuttings prior to the placement of steel and concrete. Experience indicates that backspinning the auger does not remove loose material and a flat cleanout plate is necessary. Concrete should be placed within the excavation as soon as possible after the auger/cleanout plate is withdrawn to reduce the potential for discontinuities or caving

8.7.8 We can provide a lateral pile capacity analysis using the *LPILE* computer program once the pile type, size, and approximate length has been provided. The total capacity of pile groups should be considered less than the sum of the individual pile capacities for pile spacing of less than 8D (where D is pile diameter) for lateral loads parallel to the pile group and 3D for loads perpendicular to the pile group. The reduction in capacity is based on pile spacing and positioning and can result in group efficiency on the order of 50 percent of the sum of single-pile capacities. We can evaluate the lateral capacity of pile groups using the *GROUP* computer program, if requested.

8.8 Concrete Slabs-On-Grade

8.8.1 Concrete slabs-on-grade for the structures should be constructed in accordance with Table 8.8.

TABLE 8.8
MINIMUM CONCRETE SLAB-ON-GRADE RECOMMENDATIONS

Parameter	Value
Minimum Concrete Slab Thickness	5 inches
Minimum Steel Reinforcement	No. 3 Bars 18 Inches on Center, Both Directions
Typical Slab Underlayment	3 to 4 Inches of Sand/Gravel/Base
Design Expansion Index	50 or less

8.8.2 Slabs that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder. The vapor retarder design should be consistent with the guidelines presented in the American Concrete Institute's (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06). In addition, the membrane should be installed in accordance with manufacturer's recommendations and ASTM requirements and installed in a manner that prevents puncture. The vapor retarder used should be specified by the project architect or developer based on the type of floor covering that will be installed and if the structure will possess a humidity controlled environment.

8.8.3 The bedding sand thickness should be determined by the project foundation engineer, architect, and/or developer. It is common to have 3 to 4 inches of sand in the southern California region. However, we should be contacted to provide recommendations if the bedding sand is thicker than 6 inches. The foundation design engineer should provide appropriate concrete mix design criteria and curing measures to assure proper curing of the slab by reducing the potential for rapid moisture loss and subsequent cracking and/or slab curl. We suggest that the foundation design engineer present the concrete mix design and proper curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plans.

8.8.4 Concrete slabs should be provided with adequate crack-control joints, construction joints and/or expansion joints to reduce unsightly shrinkage cracking. The design of joints should consider criteria of the American Concrete Institute (ACI) when establishing crack-control spacing. Crack-control joints should be spaced at intervals no greater than 12 feet. Additional steel reinforcing, concrete admixtures and/or closer crack control joint spacing should be considered where concrete-exposed finished floors are planned.

8.8.5 Special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be moisturized to maintain a moist condition as would be expected in any such concrete placement.

8.8.6 The concrete slab-on-grade recommendations are based on soil support characteristics only. The project structural engineer should evaluate the structural requirements of the concrete slabs for supporting expected loads.

8.8.7 The recommendations of this report are intended to reduce the potential for cracking of slabs due to expansive soil (if present), differential settlement of existing soil or soil with varying thicknesses. However, even with the incorporation of the recommendations presented herein, foundations, walls and slabs-on-grade placed on such conditions may still exhibit some cracking due to soil movement and/or shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

8.9 Exterior Concrete Flatwork

8.9.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations presented in Table 8.9. The recommended steel reinforcement would help reduce the potential for cracking.

TABLE 8.9
MINIMUM CONCRETE FLATWORK RECOMMENDATIONS

Expansion Index, EI	Minimum Steel Reinforcement* Options	Minimum Thickness
EI $\leq$ 90	6x6-W2.9/W2.9 (6x6-6/6) welded wire mesh	4 Inches
	No. 3 Bars 18 inches on center, Both Directions	

*In excess of 8 feet square.

8.9.2 The subgrade soil should be properly moisturized and compacted prior to the placement of steel and concrete. The subgrade soil should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM D 1557.

8.9.3 Even with the incorporation of the recommendations of this report, the exterior concrete flatwork has a potential to experience some uplift due to expansive soil beneath grade. The steel reinforcement should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork. Additionally, flatwork should be structurally connected to the curbs, where possible, to reduce the potential for offsets between the curbs and the flatwork.

8.9.4 Concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based on the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior to concrete placement. Subgrade soil should be properly compacted and the moisture content of subgrade soil should be verified prior to placing concrete. Base materials will not be required below concrete improvements.

8.9.5 Where exterior flatwork abuts the structure at entrant or exit points, the exterior slab should be dowelled into the structure's foundation stemwall. This recommendation is intended to reduce the potential for differential elevations that could result from differential settlement or minor heave of the flatwork. Dowelling details should be designed by the project structural engineer.

8.9.6 The recommendations presented herein are intended to reduce the potential for cracking of exterior slabs as a result of differential movement. However, even with the incorporation of the recommendations presented herein, slabs-on-grade will still crack. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, the use of crack control joints and proper concrete placement and curing. Crack control joints should be spaced at intervals no greater than 12 feet. Literature provided by the Portland Concrete Association (PCA) and American Concrete Institute (ACI) present recommendations for proper concrete mix, construction, and curing practices, and should be incorporated into project construction.

8.10 Retaining Walls

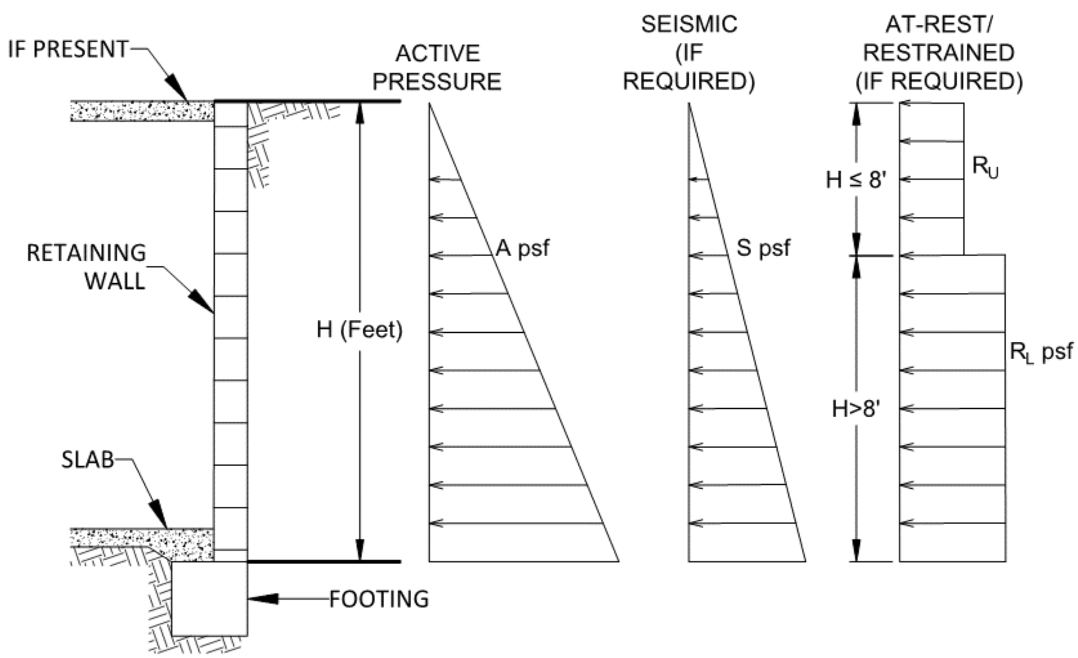
8.10.1 Retaining walls should be designed using the values presented in Table 8.10.1. Soil with an expansion index (EI) of greater than 50 should not be used as backfill material behind retaining walls.

TABLE 8.10.1
RETAINING WALL DESIGN RECOMMENDATIONS

Parameter	Value
Active Soil Pressure, A (Fluid Density, Level Backfill)	35 pcf
Active Soil Pressure, A (Fluid Density, 2:1 Sloping Backfill)	50 pcf
Seismic Pressure, S	15H psf
At-Rest/Restrained Walls Additional Uniform Pressure (0 to 8 Feet High)	7H psf
At-Rest/Restrained Walls Additional Uniform Pressure (8+ Feet High)	13H psf
Expected Expansion Index for the Subject Property	EI$\leq$50

H equals the height of the retaining portion of the wall

8.10.2 The project retaining walls should be designed as shown in the Retaining Wall Loading Diagram.



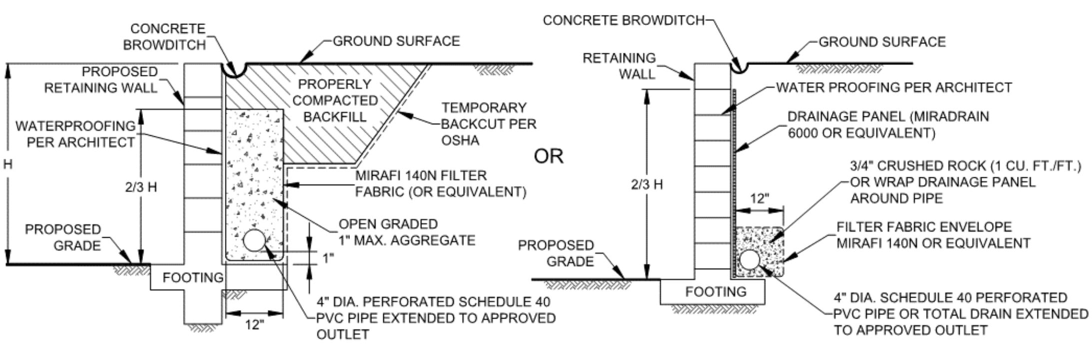
Retaining Wall Loading Diagram

8.10.3 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall) at the top of the wall. Where walls are restrained from movement at the top (at-rest condition), an additional uniform pressure should be applied to the wall. For retaining walls subject to vehicular loads within a horizontal distance equal to two-thirds the wall height, a surcharge equivalent to 2 feet of fill soil should be added.

8.10.4 The structural engineer should determine the Seismic Design Category for the project in accordance with Section 1613.3.5 of the 2019 CBC or Section 11.6 of ASCE 7-10. For structures assigned to Seismic Design Category of D, E, or F, retaining walls that support more than 6 feet of backfill should be designed with seismic lateral pressure in accordance with Section 1803.5.12 of the 2019 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall.

8.10.5 Retaining walls should be designed to ensure stability against overturning sliding, and excessive foundation pressure. Where a keyway is extended below the wall base with the intent to engage passive pressure and enhance sliding stability, it is not necessary to consider active pressure on the keyway.

8.10.6 Drainage openings through the base of the wall (weep holes) should not be used where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted granular (EI of 50 or less) free-draining backfill material with no hydrostatic forces or imposed surcharge load. The retaining wall should be properly drained as shown in the Typical Retaining Wall Drainage Detail. If conditions different than those described are expected, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.



Typical Retaining Wall Drainage Detail

8.10.7 The retaining walls may be designed using either the active and restrained (at-rest) loading condition or the active and seismic loading condition as suggested by the structural engineer. Typically, it appears the design of the restrained condition for retaining wall loading may be adequate for the seismic design of the retaining walls. However, the active earth pressure combined with the seismic design load should be reviewed and also considered in the design of the retaining walls.

8.10.8 In general, wall foundations should be designed in accordance with Table 8.10.2. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure. Therefore, retaining wall foundations should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

TABLE 8.10.2
SUMMARY OF RETAINING WALL FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Retaining Wall Foundation Width	12 inches
Minimum Retaining Wall Foundation Depth	12 Inches
Minimum Steel Reinforcement	Per Structural Engineer
Allowable Bearing Capacity	2,000 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Allowable Bearing Capacity	3,500 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 40 Feet

8.10.9 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls. In the event that other types of walls (such as mechanically stabilized earth [MSE] walls, soil nail walls, or soldier pile walls) are planned, Geocon Incorporated should be consulted for additional recommendations.

8.10.10 It is common to see retaining walls constructed in the areas of the elevator pits. The retaining walls should be properly drained and designed in accordance with the recommendations presented herein. If the elevator pit walls are not drained, the walls should be designed with an increased active pressure with an equivalent fluid density of 90 pcf. It is also common to see seepage and water collection within the elevator pit. The pit should be designed and properly waterproofed to prevent seepage and water migration into the elevator pit.

8.10.11 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependent on the wall height, the type of soil used for backfill, and loads acting on the wall. The retaining walls and improvements above the retaining walls should be designed to incorporate an appropriate amount of lateral deflection as determined by the structural engineer.

8.10.12 Soil contemplated for use as retaining wall backfill, including import materials, should be identified in the field prior to backfill. At that time, Geocon Incorporated should obtain samples for laboratory testing to evaluate its suitability. Modified lateral earth pressures may be necessary if the backfill soil does not meet the required expansion index or shear strength. City or regional standard wall designs, if used, are based on a specific active lateral earth pressure and/or soil friction angle. In this regard, on-site soil to be used as backfill may or may not meet the values for standard wall designs. Geocon Incorporated should be consulted to assess the suitability of the on-site soil for use as wall backfill if standard wall designs will be used.

8.11 Lateral Loading

8.11.1 Table 8.11 should be used to help design the proposed structures and improvements to resist lateral loads for the design of footings or shear keys. The allowable passive pressure assumes a horizontal surface extending at least 5 feet, or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

TABLE 8.11
SUMMARY OF LATERAL LOAD DESIGN RECOMMENDATIONS

Parameter	Value
Passive Pressure Fluid Density	350 pcf
Coefficient of Friction (Concrete and Soil)	0.35
Coefficient of Friction (Along Vapor Barrier)	0.2 to 0.25*

*Per manufacturer's recommendations.

8.11.2 The passive and frictional resistant loads can be combined for design purposes. The lateral passive pressures may be increased by one-third when considering transient loads due to wind or seismic forces.

8.12 Preliminary Pavement Recommendations

8.12.1 We calculated the flexible pavement sections in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4) using an estimated Traffic Index (TI) of 5.0, 5.5, 6.0 and 7.0 for parking stalls, driveways, medium truck traffic areas, and heavy truck traffic areas, respectively. The project civil engineer and owner should review the pavement designations to determine appropriate locations for pavement thickness. The final pavement sections for the parking lot should be based on the R-Value of the subgrade soil encountered at final subgrade elevation. We assumed an

R-Value of 20 (based on previous testing) and 78 for the subgrade soil and base materials, respectively, for the purposes of this preliminary analysis. Table 8.12.1 presents the preliminary flexible pavement sections.

TABLE 8.12.1
PRELIMINARY FLEXIBLE PAVEMENT SECTION

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Parking stalls for automobiles and light-duty vehicles	5.0	20	3	7
Driveways for automobiles and light-duty vehicles	5.5	20	3	9
Medium truck traffic areas	6.0	20	3.5	10
Driveways for heavy truck traffic	7.0	20	4	12

8.12.2 Prior to placing base materials, the upper 12 inches of the subgrade soil should be scarified, moisture conditioned as necessary, and recompacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM D 1557. Similarly, the base material should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Asphalt concrete should be compacted to a density of at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

8.12.3 Base materials should conform to Section 26-1.028 of the *Standard Specifications for The State of California Department of Transportation (Caltrans)* with a ¾-inch maximum size aggregate. The asphalt concrete should conform to Section 203-6 of the *Standard Specifications for Public Works Construction (Greenbook)*.

8.12.4 The base thickness can be reduced if a reinforcement geogrid is used during the installation of the pavement. Geocon should be contact for additional recommendations, if required.

8.12.5 A rigid Portland cement concrete (PCC) pavement section should be placed in roadway aprons and cross gutters. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R-08 Guide for Design and Construction of Concrete Parking Lots using the parameters presented in Table 8.12.2.

TABLE 8.12.2
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	100 pci
Modulus of rupture for concrete, M_R	500 psi
Concrete Compressive Strength	3,000 psi
Traffic Category, TC	A and C
Average daily truck traffic, ADTT	10 and 100

8.12.6 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 8.12.3.

TABLE 8.12.3
RIGID VEHICULAR PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Automobile Parking Stalls (TC=A, ADTT=10)	5.5
Driveways (TC=C, ADTT=100)	7.0

8.12.7 The PCC vehicular pavement should be placed over subgrade soil that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content.

8.12.8 The rigid pavement should also be designed and constructed incorporating the parameters presented in Table 8.12.4.

TABLE 8.12.4
ADDITIONAL RIGID PAVEMENT RECOMMENDATIONS

Subject	Value
Thickened Edge	1.2 Times Slab Thickness
	Minimum Increase of 2 Inches
	4 Feet Wide
Crack Control Joint Spacing	30 Times Slab Thickness
	Max. Spacing of 12 feet for 5.5-Inch-Thick
	Max. Spacing of 15 Feet for Slabs 6 Inches and Thicker
Crack Control Joint Depth	Per ACI 330R-08
	1 Inch Using Early-Entry Saws on Slabs Less Than 9 Inches Thick
Crack Control Joint Width	¼-Inch for Sealed Joints
	⅜-Inch is Common for Sealed Joints
	$^{1}/_{10}$- to $^{1}/_{8}$-Inch is Common for Unsealed Joints

8.12.9 Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed herein.

8.12.10 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. The depth of the crack-control joints should be determined by the referenced ACI report.

8.12.11 To provide load transfer between adjacent pavement slab sections, a butt-type construction joint should be constructed. The butt-type joint should be thickened by at least 20 percent at the edge and taper back at least 4 feet from the face of the slab. As an alternative to the butt-type construction joint, dowelling can be used between construction joints for pavements of 7 inches or thicker. As discussed in the referenced ACI guide, dowels should consist of smooth, 1-inch-diameter reinforcing steel 14 inches long embedded a minimum of 6 inches into the slab on either side of the construction joint. Dowels should be located at the midpoint of the slab, spaced at 12 inches on center and lubricated to allow joint movement while still transferring loads. In addition, tie bars should be installed as recommended in Section 3.8.3 of the referenced ACI guide. The structural engineer should provide other alternative recommendations for load transfer.

8.12.12 Concrete curb/gutter should be placed on soil subgrade compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Cross-gutters that receives vehicular should be placed on subgrade soil compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Base materials should not be placed below the curb/gutter, or cross-gutters so water is not able to migrate from the adjacent parkways to the pavement sections. Where flatwork is located directly adjacent to the curb/gutter, the concrete flatwork should be structurally connected to the curbs to help reduce the potential for offsets between the curbs and the flatwork.

8.13 Site Drainage and Moisture Protection

8.13.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2019 CBC 1804.4 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

8.13.2 In the case of basement walls or building walls retaining landscaping areas, a water-proofing system should be used on the wall and joints, and a Miradrain drainage panel (or similar) should be placed over the waterproofing. The project architect or civil engineer should provide detailed specifications on the plans for all waterproofing and drainage.

8.13.3 Underground utilities should be leak free. Utility and irrigation lines should be checked periodically for leaks, and detected leaks should be repaired promptly. Detrimental soil movement could occur if water is allowed to infiltrate the soil for prolonged periods of time.

8.13.4 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes can be used. In addition, where landscaping is planned adjacent to the pavement, construction of a cutoff wall along the edge of the pavement that extends at least 6 inches below the bottom of the base material should be considered.

8.13.5 We should prepare a storm water infiltration feasibility report of storm water management devices are planned. We expect the site possesses a “no infiltration” condition due to the presence of vernal pools in the formational materials and the presence of compacted fill.

8.14 Update Geotechnical Report

8.14.1 We should prepare an updated geotechnical report once grading plans have been prepared for the planned improvements.

8.14.2 The update geotechnical report will present our findings, conclusions, and updated geotechnical recommendations for the proposed structures and improvements.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

2. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Incorporated should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon Incorporated.

3. This report is issued with the understanding that it is the responsibility of the owner or his representative to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

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

GEOCON LEGEND

Quc ....COMPACTED FILL IN UNDERCUT AREA

Qcf ....COMPACTED FILL

Tmv/Tst ....MISSION VALLEY FORMATION/STADIUM CONGLOMERATE (Undifferentiated; Dotted Where Buried)

379 ....APPROX. ELEVATION AT BASE OF FILL (In Feet)

339 ....APPROX. SUBDRAIN AND SUBDRAIN ELEVATION (In Feet)

........APPROX. LOCATION OF GEOLOGIC CONTACT (Dotted Where Buried)

GEOLOGIC MAP

MERGE 56 - UNIT 1
SAN DIEGO, CALIFORNIA

GEOCON INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

SCALE 1" = 50'	DATE 09 - 10 - 2021
PROJECT NO. 06021 - 52 - 14	FIGURE 1
SHEET 1 OF 1	

Plotted:09/10/2021 9:52AM | By:RUBEN AGUILAR | File Location:Y:\PROJECTS\06021-52-14 Merge 56 Unit 1 (ARE)\SHEETS\06021-52-14 GeologicMap-Unit-01.dwg

APPENDIX A

APPENDIX A

PREVIOUS LABORATORY TESTING

We performed laboratory tests during previous mass grading for Merge 56 – Unit 1 in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected soil samples were tested for in-place dry density and moisture content, maximum density and optimum moisture content, direct shear strength, expansion index, and water soluble sulfate. The results of our previous laboratory tests are presented herein.

SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS ASTM D 1557

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
1	Brown, Silty fine to coarse SAND with some gravel	130.9	7.8
2	Brown, Silty fine SAND with trace gravel	120.5	11.9
3	Yellowish-brown Sandy GRAVEL	125.1	11.0
4	Dark brown, Clayey fine to medium SAND with trace gravel	119.8	12.3
5	Brown, fine to coarse Sandy CLAY with some gravel	118.5	13.0
6	Brown, fine to coarse Sandy CLAY with gravel	115.2	14.7
7	Yellowish-brown, Clayey, fine to medium SAND with trace gravel	124.9	11.5
8	Yellowish-brown, Clayey, fine to medium SAND with trace gravel	123.5	11.6
9	Yellowish-brown, Silty, fine to medium SAND with trace gravel	118.5	13.1
10	Dark brown, Clayey fine to coarse SAND with trace gravel	123.2	11.6
11	Dark brown, Clayey fine to coarse SAND with trace gravel	125.6	10.7
12	Yellowish-brown, fine to coarse Sandy GRAVEL with little silt	123.1	11.4
13	Brown, Silty Fine SAND	117.0	13.7
14	Reddish-brown, Silty fine to coarse SAND with gravel	131.6	8.3
15	Light grayish brown, Clayey fine to medium SAND with little gravel	116.7	13.6
16	Orange-brown, Clayey, fine to medium SAND with trace gravel	124.3	11.0

SUMMARY OF LABORATORY DIRECT SHEAR TEST RESULTS
ASTM D 3080

Sample No.*	Dry Density (pcf)	Moisture Content (%)		Unit Cohesion (psf) Peak (Ultimate)	Angle of Shear Resistance (degrees) Peak (Ultimate)
		Initial	Final		
9	106.4	13.6	20.0	600 [500]	28 [30]
12	112.2	11.4	19.1	700 [600]	27 [28]
13	105.3	13.7	20.1	630 [470]	26 [28]
14	118.1	8.1	13.7	600 [500]	32 [32]
16	112.1	11.2	17.2	600 [580]	30 [30]

*Samples were remolded to approximately 90 percent of maximum dry density at near optimum moisture content.

SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829

Sample No.	Lot No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index
		Before Test	After Test		
EI-1	Lot 6	11.0	21.5	106.3	36
EI-2	Lot 4	11.0	18.8	107.4	25
EI-3	Lot 4	10.1	18.4	110.1	24
EI-4	Lot 4	10.3	23.1	107.5	41
EI-5	Lot 4	10.3	19.1	107.4	39
EI-6	Lot 4	10.4	19.4	107.3	31
EI-7	Lot 3	10.8	19.4	108.2	36
EI-8	Lot 3	10.1	18.3	109.7	21
EI-9	Lot 2	10.7	19.1	107.4	26
EI-10	Lot 2	10.7	18.1	106.8	11

SUMMARY OF LABORATORY WATER-SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST NO. 417

Sample No.	Lot No.	Water-Soluble Sulfate (%)	Sulfate Severity
EI-1	Lot 6	0.025	S0
EI-2	Lot 4	0.027	S0
EI-3	Lot 4	0.036	S0
EI-4	Lot 4	0.003	S0
EI-5	Lot 4	0.038	S0
EI-6	Lot 4	0.044	S0
EI-7	Lot 3	0.044	S0
EI-8	Lot 3	0.037	S0
EI-9	Lot 2	0.036	S0
EI-10	Lot 2	0.014	S0

APPENDIX B

APPENDIX B

RECOMMENDED GRADING SPECIFICATIONS

FOR

MERGE 56 – UNIT 1
SAN DIEGO, CALIFORNIA

PROJECT NO. 06021-52-14

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

1.1 These Recommended Grading Specifications shall be used in conjunction with the Geotechnical Report for the project prepared by Geocon. The recommendations contained in the text of the Geotechnical Report are a part of the earthwork and grading specifications and shall supersede the provisions contained hereinafter in the case of conflict.

1.2 Prior to the commencement of grading, a geotechnical consultant (Consultant) shall be employed for the purpose of observing earthwork procedures and testing the fills for substantial conformance with the recommendations of the Geotechnical Report and these specifications. The Consultant should provide adequate testing and observation services so that they may assess whether, in their opinion, the work was performed in substantial conformance with these specifications. It shall be the responsibility of the Contractor to assist the Consultant and keep them apprised of work schedules and changes so that personnel may be scheduled accordingly.

1.3 It shall be the sole responsibility of the Contractor to provide adequate equipment and methods to accomplish the work in accordance with applicable grading codes or agency ordinances, these specifications and the approved grading plans. If, in the opinion of the Consultant, unsatisfactory conditions such as questionable soil materials, poor moisture condition, inadequate compaction, and/or adverse weather result in a quality of work not in conformance with these specifications, the Consultant will be empowered to reject the work and recommend to the Owner that grading be stopped until the unacceptable conditions are corrected.

2. DEFINITIONS

2.1 **Owner** shall refer to the owner of the property or the entity on whose behalf the grading work is being performed and who has contracted with the Contractor to have grading performed.

2.2 **Contractor** shall refer to the Contractor performing the site grading work.

2.3 **Civil Engineer** or **Engineer of Work** shall refer to the California licensed Civil Engineer or consulting firm responsible for preparation of the grading plans, surveying and verifying as-graded topography.

2.4 **Consultant** shall refer to the soil engineering and engineering geology consulting firm retained to provide geotechnical services for the project.

2.5 **Soil Engineer** shall refer to a California licensed Civil Engineer retained by the Owner, who is experienced in the practice of geotechnical engineering. The Soil Engineer shall be responsible for having qualified representatives on-site to observe and test the Contractor's work for conformance with these specifications.

2.6 **Engineering Geologist** shall refer to a California licensed Engineering Geologist retained by the Owner to provide geologic observations and recommendations during the site grading.

2.7 **Geotechnical Report** shall refer to a soil report (including all addenda) which may include a geologic reconnaissance or geologic investigation that was prepared specifically for the development of the project for which these Recommended Grading Specifications are intended to apply.

3. MATERIALS

3.1 Materials for compacted fill shall consist of any soil excavated from the cut areas or imported to the site that, in the opinion of the Consultant, is suitable for use in construction of fills. In general, fill materials can be classified as *soil* fills, *soil-rock* fills or *rock* fills, as defined below.

3.1.1 **Soil fills** are defined as fills containing no rocks or hard lumps greater than 12 inches in maximum dimension and containing at least 40 percent by weight of material smaller than ¾ inch in size.

3.1.2 **Soil-rock fills** are defined as fills containing no rocks or hard lumps larger than 4 feet in maximum dimension and containing a sufficient matrix of soil fill to allow for proper compaction of soil fill around the rock fragments or hard lumps as specified in Paragraph 6.2. **Oversize rock** is defined as material greater than 12 inches.

3.1.3 **Rock fills** are defined as fills containing no rocks or hard lumps larger than 3 feet in maximum dimension and containing little or no fines. Fines are defined as material smaller than ¾ inch in maximum dimension. The quantity of fines shall be less than approximately 20 percent of the rock fill quantity.

3.2 Material of a perishable, spongy, or otherwise unsuitable nature as determined by the Consultant shall not be used in fills.

3.3 Materials used for fill, either imported or on-site, shall not contain hazardous materials as defined by the California Code of Regulations, Title 22, Division 4, Chapter 30, Articles 9 and 10; 40CFR; and any other applicable local, state or federal laws. The Consultant shall not be responsible for the identification or analysis of the potential presence of hazardous materials. However, if observations, odors or soil discoloration cause Consultant to suspect the presence of hazardous materials, the Consultant may request from the Owner the termination of grading operations within the affected area. Prior to resuming grading operations, the Owner shall provide a written report to the Consultant indicating that the suspected materials are not hazardous as defined by applicable laws and regulations.

3.4 The outer 15 feet of *soil-rock* fill slopes, measured horizontally, should be composed of properly compacted *soil* fill materials approved by the Consultant. *Rock* fill may extend to the slope face, provided that the slope is not steeper than 2:1 (horizontal:vertical) and a soil layer no thicker than 12 inches is track-walked onto the face for landscaping purposes. This procedure may be utilized provided it is acceptable to the governing agency, Owner and Consultant.

3.5 Samples of soil materials to be used for fill should be tested in the laboratory by the Consultant to determine the maximum density, optimum moisture content, and, where appropriate, shear strength, expansion, and gradation characteristics of the soil.

3.6 During grading, soil or groundwater conditions other than those identified in the Geotechnical Report may be encountered by the Contractor. The Consultant shall be notified immediately to evaluate the significance of the unanticipated condition.

4. CLEARING AND PREPARING AREAS TO BE FILLED

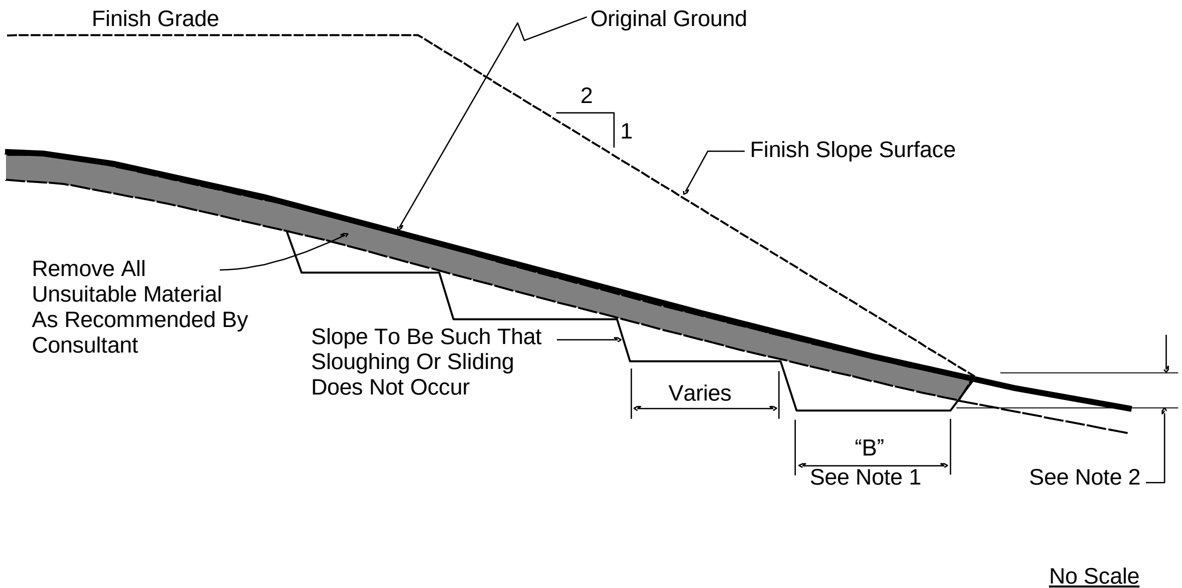
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material and shall be performed in areas to be graded. Roots and other projections exceeding 1½ inches in diameter shall be removed to a depth of 3 feet below the surface of the ground. Borrow areas shall be grubbed to the extent necessary to provide suitable fill materials.

4.2 Asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility or in an acceptable area of the project evaluated by Geocon and the property owner. Concrete fragments that are free of reinforcing steel may be placed in fills, provided they are placed in accordance with Section 6.2 or 6.3 of this document.

4.3 After clearing and grubbing of organic matter and other unsuitable material, loose or porous soils shall be removed to the depth recommended in the Geotechnical Report. The depth of removal and compaction should be observed and approved by a representative of the Consultant. The exposed surface shall then be plowed or scarified to a minimum depth of 6 inches and until the surface is free from uneven features that would tend to prevent uniform compaction by the equipment to be used.

4.4 Where the slope ratio of the original ground is steeper than 5:1 (horizontal:vertical), or where recommended by the Consultant, the original ground should be benched in accordance with the following illustration.

TYPICAL BENCHING DETAIL



DETAIL NOTES: (1) Key width "B" should be a minimum of 10 feet, or sufficiently wide to permit complete coverage with the compaction equipment used. The base of the key should be graded horizontal, or inclined slightly into the natural slope.

(2) The outside of the key should be below the topsoil or unsuitable surficial material and at least 2 feet into dense formational material. Where hard rock is exposed in the bottom of the key, the depth and configuration of the key may be modified as approved by the Consultant.

4.5 After areas to receive fill have been cleared and scarified, the surface should be moisture conditioned to achieve the proper moisture content, and compacted as recommended in Section 6 of these specifications.

5. COMPACTION EQUIPMENT

5.1 Compaction of *soil* or *soil-rock* fill shall be accomplished by sheepsfoot or segmented-steel wheeled rollers, vibratory rollers, multiple-wheel pneumatic-tired rollers, or other types of acceptable compaction equipment. Equipment shall be of such a design that it will be capable of compacting the *soil* or *soil-rock* fill to the specified relative compaction at the specified moisture content.

5.2 Compaction of *rock* fills shall be performed in accordance with Section 6.3.

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

6.1 *Soil* fill, as defined in Paragraph 3.1.1, shall be placed by the Contractor in accordance with the following recommendations:

6.1.1 *Soil* fill shall be placed by the Contractor in layers that, when compacted, should generally not exceed 8 inches. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to obtain uniformity of material and moisture in each layer. The entire fill shall be constructed as a unit in nearly level lifts. Rock materials greater than 12 inches in maximum dimension shall be placed in accordance with Section 6.2 or 6.3 of these specifications.

6.1.2 In general, the *soil* fill shall be compacted at a moisture content at or above the optimum moisture content as determined by ASTM D 1557.

6.1.3 When the moisture content of *soil* fill is below that specified by the Consultant, water shall be added by the Contractor until the moisture content is in the range specified.

6.1.4 When the moisture content of the *soil* fill is above the range specified by the Consultant or too wet to achieve proper compaction, the *soil* fill shall be aerated by the Contractor by blading/mixing, or other satisfactory methods until the moisture content is within the range specified.

6.1.5 After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted by the Contractor to a relative compaction of at least 90 percent. Relative compaction is defined as the ratio (expressed in percent) of the in-place dry density of the compacted fill to the maximum laboratory dry density as determined in accordance with ASTM D 1557. Compaction shall be continuous over the entire area, and compaction equipment shall make sufficient passes so that the specified minimum relative compaction has been achieved throughout the entire fill.

6.1.6 Where practical, soils having an Expansion Index greater than 50 should be placed at least 3 feet below finish pad grade and should be compacted at a moisture content generally 2 to 4 percent greater than the optimum moisture content for the material.

6.1.7 Properly compacted *soil* fill shall extend to the design surface of fill slopes. To achieve proper compaction, it is recommended that fill slopes be over-built by at least 3 feet and then cut to the design grade. This procedure is considered preferable to track-walking of slopes, as described in the following paragraph.

6.1.8 As an alternative to over-building of slopes, slope faces may be back-rolled with a heavy-duty loaded sheepsfoot or vibratory roller at maximum 4-foot fill height intervals. Upon completion, slopes should then be track-walked with a D-8 dozer or similar equipment, such that a dozer track covers all slope surfaces at least twice.

6.2 *Soil-rock* fill, as defined in Paragraph 3.1.2, shall be placed by the Contractor in accordance with the following recommendations:

6.2.1 Rocks larger than 12 inches but less than 4 feet in maximum dimension may be incorporated into the compacted *soil* fill, but shall be limited to the area measured 15 feet minimum horizontally from the slope face and 5 feet below finish grade or 3 feet below the deepest utility, whichever is deeper.

6.2.2 Rocks or rock fragments up to 4 feet in maximum dimension may either be individually placed or placed in windrows. Under certain conditions, rocks or rock fragments up to 10 feet in maximum dimension may be placed using similar methods. The acceptability of placing rock materials greater than 4 feet in maximum dimension shall be evaluated during grading as specific cases arise and shall be approved by the Consultant prior to placement.

6.2.3 For individual placement, sufficient space shall be provided between rocks to allow for passage of compaction equipment.

6.2.4 For windrow placement, the rocks should be placed in trenches excavated in properly compacted *soil* fill. Trenches should be approximately 5 feet wide and 4 feet deep in maximum dimension. The voids around and beneath rocks should be filled with approved granular soil having a Sand Equivalent of 30 or greater and should be compacted by flooding. Windrows may also be placed utilizing an "open-face" method in lieu of the trench procedure, however, this method should first be approved by the Consultant.

6.2.5 Windrows should generally be parallel to each other and may be placed either parallel to or perpendicular to the face of the slope depending on the site geometry. The minimum horizontal spacing for windrows shall be 12 feet center-to-center with a 5-foot stagger or offset from lower courses to next overlying course. The minimum vertical spacing between windrow courses shall be 2 feet from the top of a lower windrow to the bottom of the next higher windrow.

6.2.6 Rock placement, fill placement and flooding of approved granular soil in the windrows should be continuously observed by the Consultant.

6.3 *Rock* fills, as defined in Section 3.1.3, shall be placed by the Contractor in accordance with the following recommendations:

6.3.1 The base of the *rock* fill shall be placed on a sloping surface (minimum slope of 2 percent). The surface shall slope toward suitable subdrainage outlet facilities. The *rock* fills shall be provided with subdrains during construction so that a hydrostatic pressure buildup does not develop. The subdrains shall be permanently connected to controlled drainage facilities to control post-construction infiltration of water.

6.3.2 *Rock* fills shall be placed in lifts not exceeding 3 feet. Placement shall be by rock trucks traversing previously placed lifts and dumping at the edge of the currently placed lift. Spreading of the *rock* fill shall be by dozer to facilitate *seating* of the rock. The *rock* fill shall be watered heavily during placement. Watering shall consist of water trucks traversing in front of the current rock lift face and spraying water continuously during rock placement. Compaction equipment with compactive energy comparable to or greater than that of a 20-ton steel vibratory roller or other compaction equipment providing suitable energy to achieve the required compaction or deflection as recommended in Paragraph 6.3.3 shall be utilized. The number of passes to be made should be determined as described in Paragraph 6.3.3. Once a *rock* fill lift has been covered with *soil* fill, no additional *rock* fill lifts will be permitted over the *soil* fill.

6.3.3 Plate bearing tests, in accordance with ASTM D 1196, may be performed in both the compacted *soil* fill and in the *rock* fill to aid in determining the required minimum number of passes of the compaction equipment. If performed, a minimum of three plate bearing tests should be performed in the properly compacted *soil* fill (minimum relative compaction of 90 percent). Plate bearing tests shall then be performed on areas of *rock* fill having two passes, four passes and six passes of the compaction equipment, respectively. The number of passes required for the *rock* fill shall be determined by comparing the results of the plate bearing tests for the *soil* fill and the *rock* fill and by evaluating the deflection variation with number of passes. The required number of passes of the compaction equipment will be performed as necessary until the plate bearing deflections are equal to or less than that determined for the properly compacted *soil* fill. In no case will the required number of passes be less than two.

6.3.4 A representative of the Consultant should be present during *rock* fill operations to observe that the minimum number of "passes" have been obtained, that water is being properly applied and that specified procedures are being followed. The actual number of plate bearing tests will be determined by the Consultant during grading.

6.3.5 Test pits shall be excavated by the Contractor so that the Consultant can state that, in their opinion, sufficient water is present and that voids between large rocks are properly filled with smaller rock material. In-place density testing will not be required in the *rock* fills.

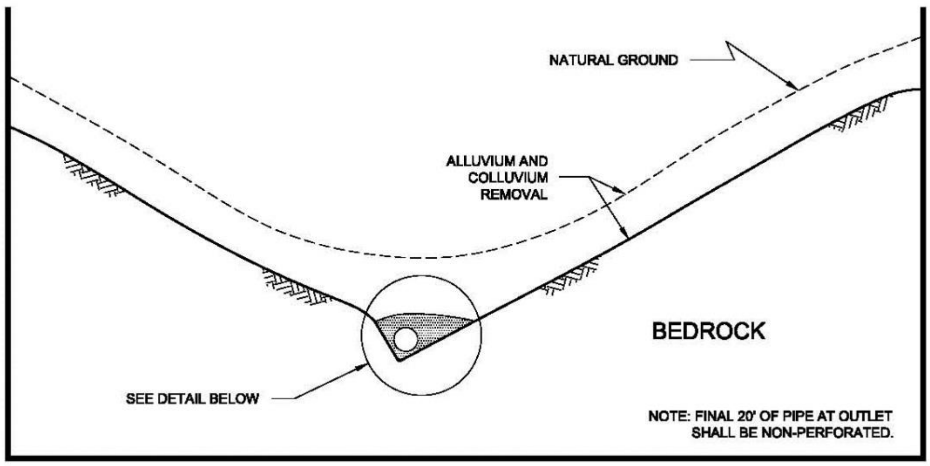
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant prior to commencing grading. The gradation of the graded filter material will be determined at the time the *rock* fill is being excavated. Materials typical of the *rock* fill should be submitted to the Consultant in a timely manner, to allow design of the graded filter prior to the commencement of *rock* fill placement.

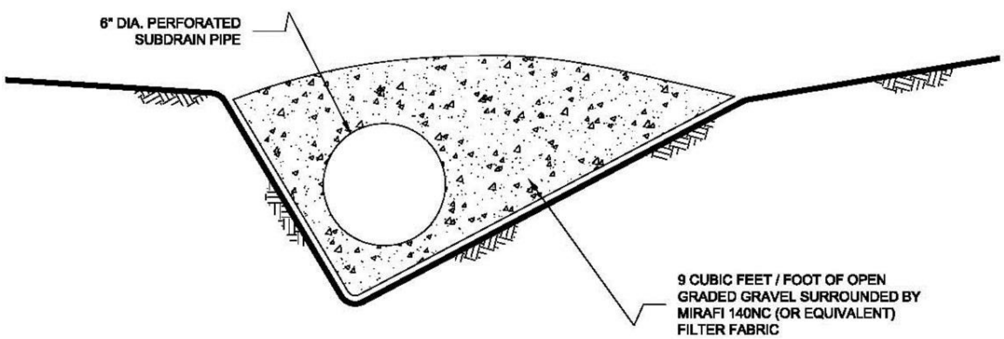
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. SUBDRAINS

7.1 The geologic units on the site may have permeability characteristics and/or fracture systems that could be susceptible under certain conditions to seepage. The use of canyon subdrains may be necessary to mitigate the potential for adverse impacts associated with seepage conditions. Canyon subdrains with lengths in excess of 500 feet or extensions of existing offsite subdrains should use 8-inch-diameter pipes. Canyon subdrains less than 500 feet in length should use 6-inch-diameter pipes.

TYPICAL CANYON DRAIN DETAIL





NOTES:

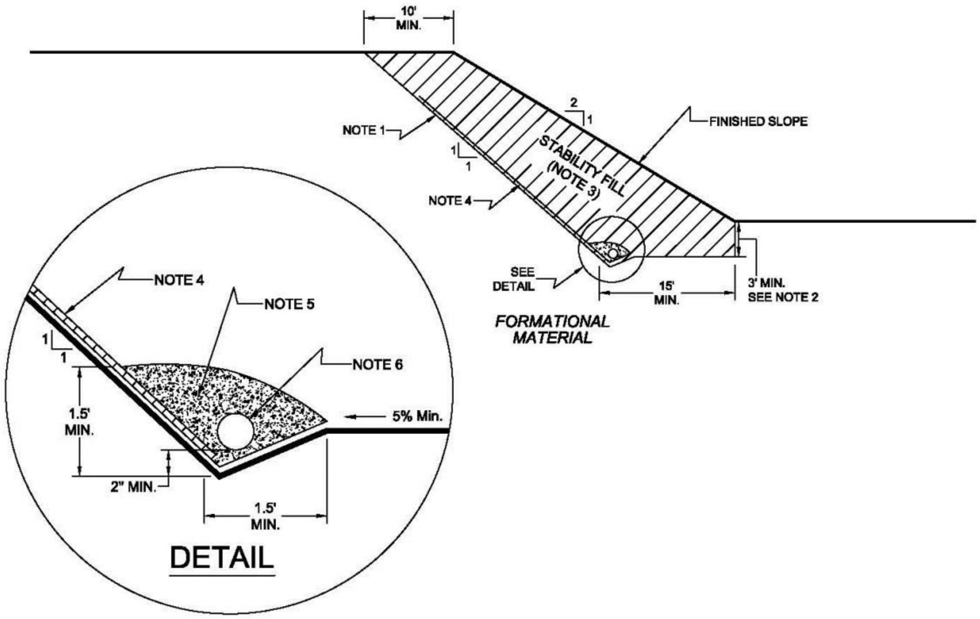
1......6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

2......6-INCH DIAMETER, SCHEDULE 40 PVC PERFORATED PIPE FOR FILLS LESS THAN 100-FEET IN DEPTH OR A PIPE LENGTH SHORTER THAN 500 FEET.

NO SCALE

7.2 Slope drains within stability fill keyways should use 4-inch-diameter (or lager) pipes.

TYPICAL STABILITY FILL DETAIL



NOTES:

1.....EXCAVATE BACKCUT AT 1:1 INCLINATION (UNLESS OTHERWISE NOTED).

2.....BASE OF STABILITY FILL TO BE 3 FEET INTO FORMATIONAL MATERIAL, SLOPING A MINIMUM 5% INTO SLOPE.

3.....STABILITY FILL TO BE COMPOSED OF PROPERLY COMPACTED GRANULAR SOIL.

4.....CHIMNEY DRAINS TO BE APPROVED PREFABRICATED CHIMNEY DRAIN PANELS (MIRADRAIN G200N OR EQUIVALENT) SPACED APPROXIMATELY 20 FEET CENTER TO CENTER AND 4 FEET WIDE. CLOSER SPACING MAY BE REQUIRED IF SEEPAGE IS ENCOUNTERED.

5.....FILTER MATERIAL TO BE 3/4-INCH, OPEN-GRADED CRUSHED ROCK ENCLOSED IN APPROVED FILTER FABRIC (MIRAFI 140NC).

6.....COLLECTOR PIPE TO BE 4-INCH MINIMUM DIAMETER, PERFORATED, THICK-WALLED PVC SCHEDULE 40 OR EQUIVALENT, AND SLOPED TO DRAIN AT 1 PERCENT MINIMUM TO APPROVED OUTLET.

NO SCALE

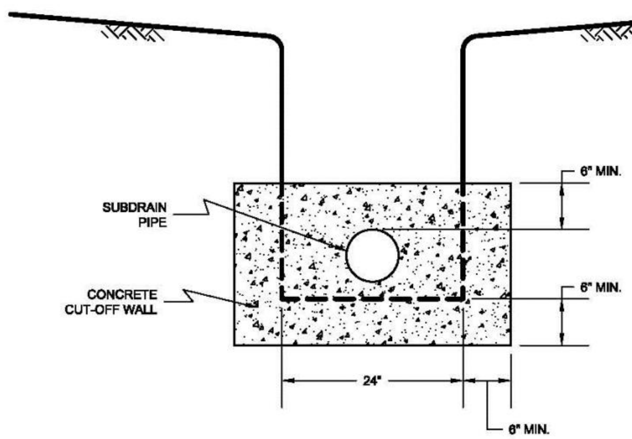
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing 40-scale grading plans.

7.4 *Rock* fill or *soil-rock* fill areas may require subdrains along their down-slope perimeters to mitigate the potential for buildup of water from construction or landscape irrigation. The subdrains should be at least 6-inch-diameter pipes encapsulated in gravel and filter fabric. *Rock* fill drains should be constructed using the same requirements as canyon subdrains.

7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

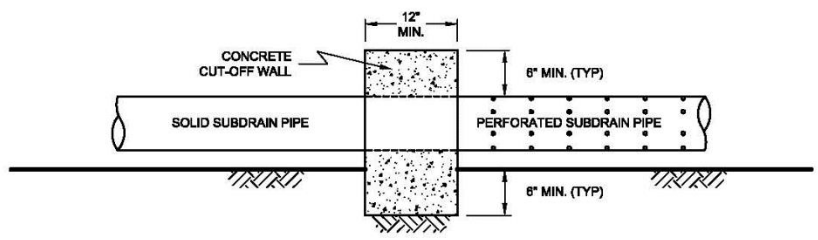
TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



NO SCALE

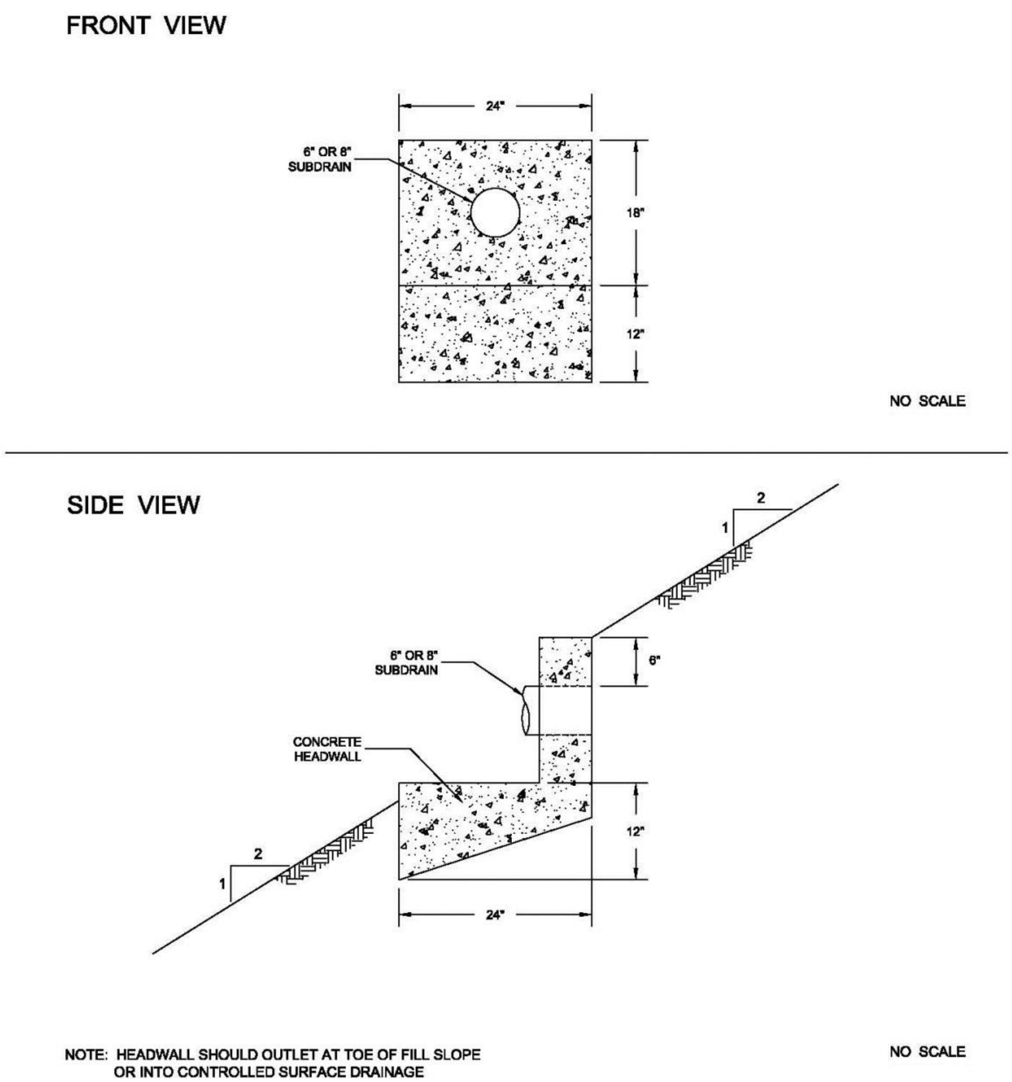
SIDE VIEW



NO SCALE

7.6 Subdrains that discharge into a natural drainage course or open space area should be provided with a permanent headwall structure.

TYPICAL HEADWALL DETAIL



7.7 The final grading plans should show the location of the proposed subdrains. After completion of remedial excavations and subdrain installation, the project civil engineer should survey the drain locations and prepare an "as-built" map showing the drain locations. The final outlet and connection locations should be determined during grading operations. Subdrains that will be extended on adjacent projects after grading can be placed on formational material and a vertical riser should be placed at the end of the subdrain. The grading contractor should consider videoing the subdrains shortly after burial to check proper installation and functionality. The contractor is responsible for the performance of the drains.

8. OBSERVATION AND TESTING

8.1 The Consultant shall be the Owner's representative to observe and perform tests during clearing, grubbing, filling, and compaction operations. In general, no more than 2 feet in vertical elevation of *soil* or *soil-rock* fill should be placed without at least one field density test being performed within that interval. In addition, a minimum of one field density test should be performed for every 2,000 cubic yards of *soil* or *soil-rock* fill placed and compacted.

8.2 The Consultant should perform a sufficient distribution of field density tests of the compacted *soil* or *soil-rock* fill to provide a basis for expressing an opinion whether the fill material is compacted as specified. Density tests shall be performed in the compacted materials below any disturbed surface. When these tests indicate that the density of any layer of fill or portion thereof is below that specified, the particular layer or areas represented by the test shall be reworked until the specified density has been achieved.

8.3 During placement of *rock* fill, the Consultant should observe that the minimum number of passes have been obtained per the criteria discussed in Section 6.3.3. The Consultant should request the excavation of observation pits and may perform plate bearing tests on the placed *rock* fills. The observation pits will be excavated to provide a basis for expressing an opinion as to whether the *rock* fill is properly seated and sufficient moisture has been applied to the material. When observations indicate that a layer of *rock* fill or any portion thereof is below that specified, the affected layer or area shall be reworked until the *rock* fill has been adequately seated and sufficient moisture applied.

8.4 A settlement monitoring program designed by the Consultant may be conducted in areas of *rock* fill placement. The specific design of the monitoring program shall be as recommended in the Conclusions and Recommendations section of the project Geotechnical Report or in the final report of testing and observation services performed during grading.

8.5 We should observe the placement of subdrains, to check that the drainage devices have been placed and constructed in substantial conformance with project specifications.

8.6 Testing procedures shall conform to the following Standards as appropriate:

8.6.1 Soil and Soil-Rock Fills:

8.6.1.1 Field Density Test, ASTM D 1556, *Density of Soil In-Place By the Sand-Cone Method.*

8.6.1.2 Field Density Test, Nuclear Method, ASTM D 6938, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth).*

8.6.1.3 Laboratory Compaction Test, ASTM D 1557, *Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-Pound Hammer and 18-Inch Drop.*

8.6.1.4. Expansion Index Test, ASTM D 4829, *Expansion Index Test*.

9. PROTECTION OF WORK

9.1 During construction, the Contractor shall properly grade all excavated surfaces to provide positive drainage and prevent ponding of water. Drainage of surface water shall be controlled to avoid damage to adjoining properties or to finished work on the site. The Contractor shall take remedial measures to prevent erosion of freshly graded areas until such time as permanent drainage and erosion control features have been installed. Areas subjected to erosion or sedimentation shall be properly prepared in accordance with the Specifications prior to placing additional fill or structures.

9.2 After completion of grading as observed and tested by the Consultant, no further excavation or filling shall be conducted except in conjunction with the services of the Consultant.

10. CERTIFICATIONS AND FINAL REPORTS

10.1 Upon completion of the work, Contractor shall furnish Owner a certification by the Civil Engineer stating that the lots and/or building pads are graded to within 0.1 foot vertically of elevations shown on the grading plan and that all tops and toes of slopes are within 0.5 foot horizontally of the positions shown on the grading plans. After installation of a section of subdrain, the project Civil Engineer should survey its location and prepare an *as-built* plan of the subdrain location. The project Civil Engineer should verify the proper outlet for the subdrains and the Contractor should ensure that the drain system is free of obstructions.

10.2 The Owner is responsible for furnishing a final as-graded soil and geologic report satisfactory to the appropriate governing or accepting agencies. The as-graded report should be prepared and signed by a California licensed Civil Engineer experienced in geotechnical engineering and by a California Certified Engineering Geologist, indicating that the geotechnical aspects of the grading were performed in substantial conformance with the Specifications or approved changes to the Specifications.

LIST OF REFERENCES

1. *2019 California Building Code, California Code of Regulations, Title 24, Part 2, based on the 2018 International Building Code*, prepared by California Building Standards Commission, dated July 2019.

2. American Concrete Institute, *ACI 318-11, Building Code Requirements for Structural Concrete and Commentary*, dated August, 2011.

3. American Concrete Institute, *ACI 330-08, Guide for the Design and Construction of Concrete Parking Lots,* dated June, 2008.

4. American Society of Civil Engineers (ASCE), *ASCE 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, 2017.

5. California Department of Conservation, Division of Mines and Geology, *Probabilistic Seismic Hazard Assessment for the State of California,* Open File Report 96-08, 1996.

6. *City of San Diego Seismic Safety Study, Geologic Hazards and Faults,* 2008 edition, Map Sheet 39.

7. Historical Aerial Photos. *http://www.historicaerials.com*

8. Jennings, C. W., 1994, California Division of Mines and Geology, *Fault Activity Map of California and Adjacent Areas*, California Geologic Data Map Series Map No. 6.

9. Kennedy, M. P., and S. S. Tan, 2008, *Geologic Map of the San Diego 30'x60' Quadrangle, California*, USGS Regional Map Series Map No. 3, Scale 1:100,000.

10. SEAOC web application, *OSHPD Seismic Design Maps*, https://seismicmaps.org/.

11. Special Publication 117A, *Guidelines For Evaluating and Mitigating Seismic Hazards in California 2008*, California Geological Survey, Revised and Re-adopted September 11, 2008.

12. Unpublished reports, aerial photographs, and maps on file with Geocon Incorporated.