PRELIMINARY
GEOTECHNICAL INVESTIGATION

2642, 2646, 2648 NEWTON AVENUE
SAN DIEGO, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

SD LOFTS
SAN DIEGO, CALIFORNIA

AUGUST 13, 2021
PROJECT NO. G2778-52-01

Project No. G2778-52-01
August 13, 2021

SD Lofts
3000 Upas Street, Suite 101
San Diego, California 92104

Attention: Mr. Matthew Segal

Subject: PRELIMINARY GEOTECHNICAL INVESTIGATION
2642, 2646, 2648 NEWTON AVENUE
SAN DIEGO, CALIFORNIA

Dear Mr. Segal:

In accordance with your request and authorization of our Proposal No. LG-21332 dated July 9, 2021, we herein submit the results of our preliminary 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.

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 building(s) and improvements provided the recommendations of this report are incorporated into the design and construction of the planned project.

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 GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our preliminary geotechnical investigation for the proposed development located at 2642, 2646, 2648 Newton Avenue in the City of San Diego, California (see Vicinity Map).

Vicinity Map

The purpose of the preliminary geotechnical investigation is to evaluate the surface and subsurface soil conditions and general site geology, and to identify geotechnical constraints that may affect 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, shallow foundations, concrete slab-on-grade, concrete flatwork, pavement and retaining walls.

We reviewed and the following fault evaluation report in preparation of this report: *Surface Fault Rupture Hazard Evaluation, 2642, 2646 and 2648 Newton Avenue, San Diego, California*, prepared by GDS Incorporated, dated July 28, 2021.

The scope of this investigation included reviewing readily available published and unpublished geologic literature (see *List of References*), performing engineering analyses and preparing this report. We also sampled soil during the excavation of the fault trenches performed for the referenced fault study, and performed laboratory testing. Appendix A presents the fault trench logs prepared by GDS

Incorporated (2021). The details of the laboratory tests and a summary of the test results are shown in Appendix B.

2. SITE AND PROJECT DESCRIPTION

The property is located north of Newton Avenue, about 220 feet east of South 26th Street and south of an existing concrete alleyway in the Barrio Logan area of the City of San Diego, California. Residential properties border the site to the east and west. The property currently consists of a storage area including surface parking, driveways and utilities. Access to the property is from the south on Newton Avenue and on the north from the concrete alleyway. The property appears to have been previously occupied by single-family residences that were removed between 1972 and 1978, and the previous residences were constructed prior to 1953. The site is relatively flat at elevations of about 50 to 55 feet above mean sea level (MSL) at the southwest and northeast, respectively. The Existing Site Map shows the site conditions.

Existing Site Plan

Development plans are not currently available. However, we understand the property may be developed to include an on-grade structure or will be graded to accept historical structures to be moved on-site. We expect the planned buildings would be supported on shallow foundations and a concrete slab-on-grade. We have not been provided with site plans for the project and should update our report with grading plans once they have been prepared.

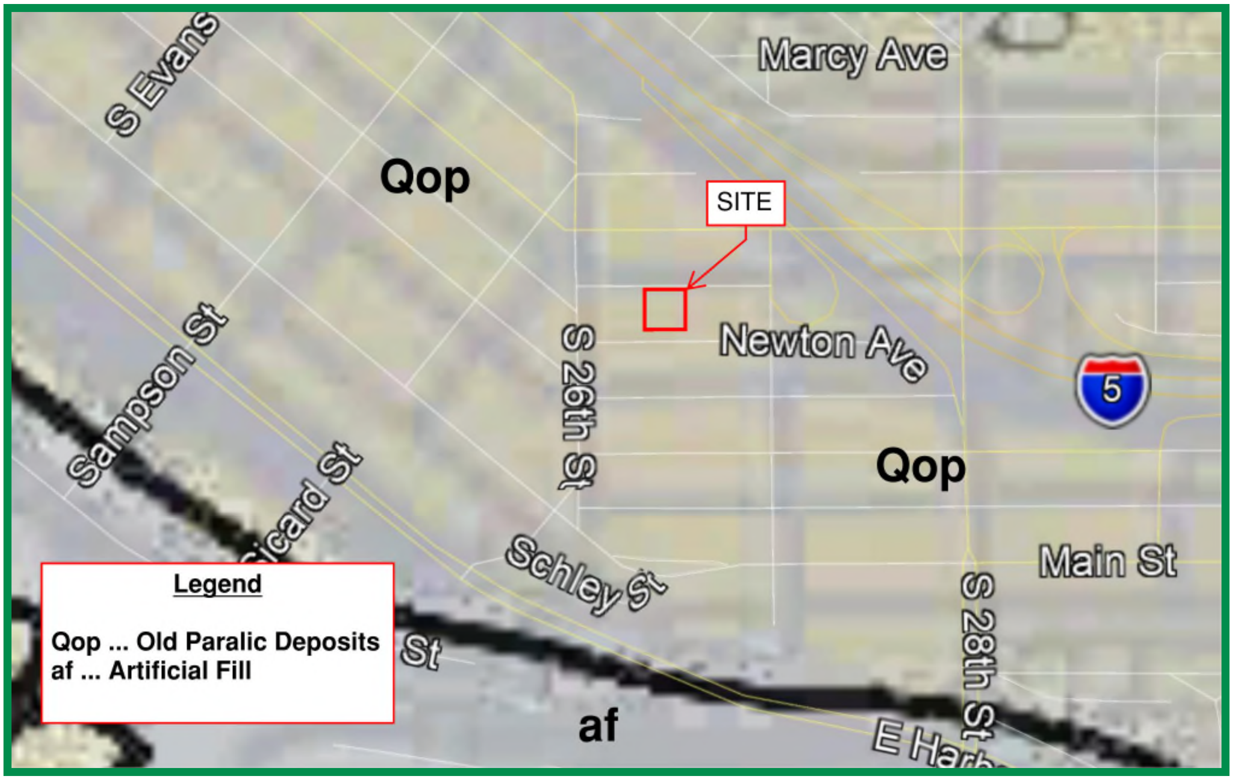
The locations, site descriptions, and proposed development are based on our site reconnaissance, review of published geologic literature, field investigations, and discussions with project personnel. If development plans differ from those described herein, Geocon Incorporated should be contacted for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

3. GEOLOGIC SETTING

The downtown San Diego area is located in the Coastal Plain sub-province 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 late Cretaceous through the Pleistocene with intermittent deposition. Geomorphically, the coastal plain is characterized by a series of twenty-one, stair-stepped marine terraces (younger to the west) that have been dissected by west flowing rivers.

The downtown area is underlain by Pleistocene age Old Paralic Deposits Unit 6 (Qop6, formerly Bay Point Formation) overlying Pliocene age San Diego Formation. Paralic Deposits regionally mapped in the past as the Bay Point Formation include sediments that have estimated ages ranging from about 85,000 to about 600,000 years old (Demere, 1981). This formation is comprised of nearshore marine and non-marine sedimentary units of poorly consolidated, fine- and medium-grained fossiliferous sandstone (Kennedy, 1975). Locally, the Paralic Deposits are believed to have been deposited on the Bird Rock and/or Nestor Terrace abrasion platforms with ages ranging from about 80,000 to 120,000 years (Kern and Rockwell, 1992).

The downtown area is located within a broad structural trough formed by down-warping and normal or oblique-slip along the active Rose Canyon Fault Zone (RCFZ). Pliocene and Pleistocene deposits have accumulated within the basin. During the middle to late Pleistocene, Qop6 was deposited unconformably on the Pliocene age San Diego Formation. The Old Paralic Deposits are generally exposed at sea level, so its total thickness and relationship with the underlying formation is unknown (Demere, 2006). The deposits formed in brackish water estuarine (nearshore marine) and terrestrial environments. Bedding attitudes are generally horizontal or subhorizontal, exceptions being localized undulations and cross-laminations within a horizontally bedded unit. The Regional Geologic Map shows the geologic units in the area of the site.



Regional Geologic Map

Regional geology in the area is predominately controlled by the RCFZ and comprises a broad zone of active and inactive faults (Rockwell, 2010). Faulting along the present trend began during the late Tertiary, approximately 7 million years before present (Lindvall and Rockwell, 1995). The RCFZ is considered a southerly extension of the Newport-Inglewood fault zone. The onshore portion of the fault system extends from La Jolla, where faulting is dominated by strike-slip movement; southward to San Diego Bay, where several faults have oblique movements of both strike-slip and normal faulting to the west and east (Treiman, 2002). The San Diego Bay was created as a down-dropped block within this fault zone.

The major faults comprising the southern end of the RCFZ are the Spanish Bight, Coronado and Silver Strand faults. In addition, there are two active fault zones in the downtown area of San Diego that have been included in the California Geological Survey Earthquake Fault Zone Map: 1) the San Diego Fault zone mapped near First Street, and 2) the Downtown Graben zone mapped roughly between 12th and 16th Streets.

4. SOIL AND GEOLOGIC CONDITIONS

Based on a review of the fault trench logs prepared by GDS Incorporated and our site review of the fault trench, the site is underlain by two surficial soil units (consisting of undocumented fill and topsoil) and one geologic unit (consisting of Old Paralic Deposits). The occurrence, distribution, and description of each unit encountered is shown on the Geologic Map, Figure 1 and on the trench logs in

Appendix A. The Geologic Cross-Section, Figure 2, shows the approximate subsurface relationship between the geologic units. We prepared the geologic cross-section using interpolation between exploratory excavations and observations; therefore, actual geotechnical conditions may vary from those illustrated and should be considered approximate. The surficial soils and geologic unit are described herein in order of increasing age.

4.1 Undocumented Fill (Qudf)

Undocumented fill was encountered during the investigation performed by GDS Incorporated to depths generally ranging from about 1 to 1½ feet, and up to approximately 5 feet in localized areas. The undocumented fill is designated as “fill” on the trench logs by GDS Incorporated. In general, the fill encountered consists of light brown, moist, silty sand with trace gravel and trash/debris consisting of bits of glass and bricks. The fill materials possess a “very low” expansion index (expansion index of 20 or less). The undocumented fill is not considered suitable in its current condition for the support of foundations or structural fill and remedial grading will required. The undocumented fill can be reused for new compacted fill during grading operations provided it is free of roots and debris.

4.2 Topsoil (Qtop)

Holocene-age topsoil is present below the undocumented fill and above the Old Paralic Deposits. The topsoil is designated as “modern soil” on the trench logs by GDS Incorporated. The topsoil was encountered to depths of about 1½ to 4 feet in the fault trenches performed by GDS Incorporated and are characterized as dark reddish brown, moist, clayey, fine sand. The topsoil is likely compressible and possesses a “very low” expansion potential (expansion index of 20 or less). Remedial grading of the topsoil will be necessary in areas to support proposed fill or structures.

4.3 Old Paralic Deposits (Qop)

The Quaternary-age Old Paralic Deposits exist below the topsoil across the site. The Old Paralic Deposits are designated as “Paleosol (Units A through C)” on the trench logs by GDS Incorporated. These deposits generally consist of medium dense to very dense, dry to moist, yellowish brown, silty to clayey, fine to medium sand with gravel and cobble layers, and cemented zones. The Old Paralic Deposits typically possess a “very low” to “medium” expansion potential (expansion index of 90 or less) and a “S0” sulfate classification. The Old Paralic Deposits are considered acceptable to support the planned fill and foundation loads for the development. However, the upper portions of the Old Paralic Deposits are generally dry to moist and may have a potential for hydroconsolidation settlement and will require remedial grading.

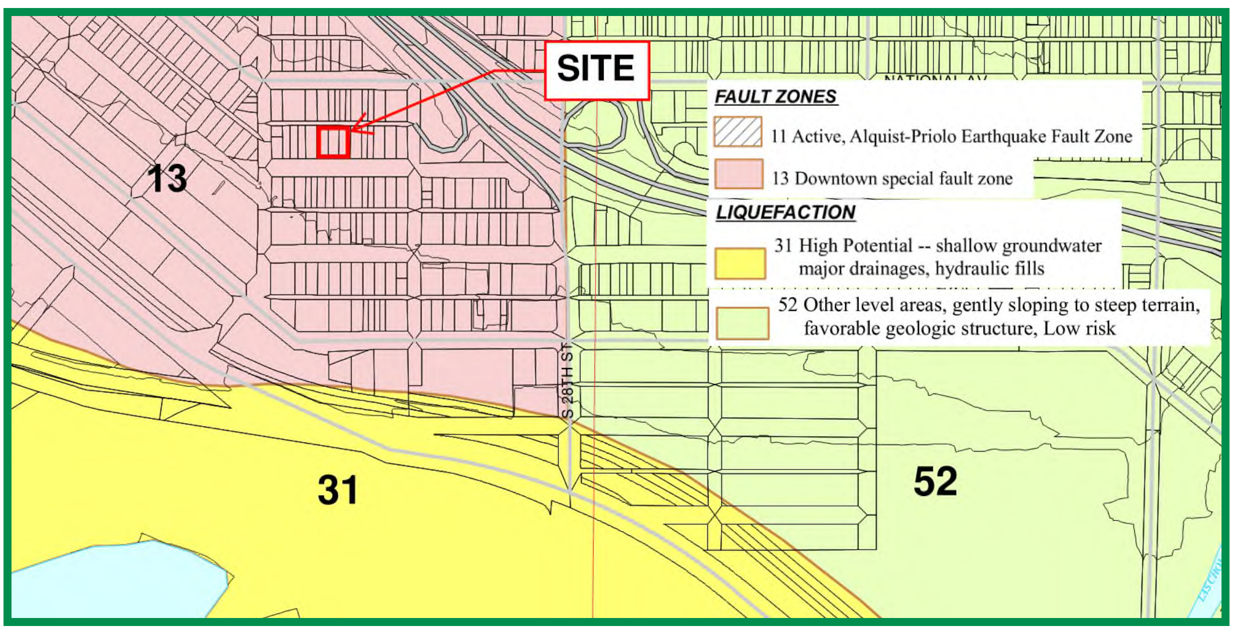
5. GROUNDWATER

Groundwater or seepage was not encountered during the site investigation performed by GDS Incorporated. However, it is not uncommon for shallow seepage conditions to develop where none previously existed when sites are irrigated or infiltration is implemented. Seepage is dependent on seasonal precipitation, irrigation, land use, among other factors, and varies as a result. Proper surface drainage will be important to future performance of the project. We expect groundwater is at an elevation of 0 to 5 feet MSL or at a depth of approximately 45 to 55 feet below existing grade. We do not expect groundwater to be encountered during construction of the proposed development.

6. GEOLOGIC HAZARDS

6.1 Faulting

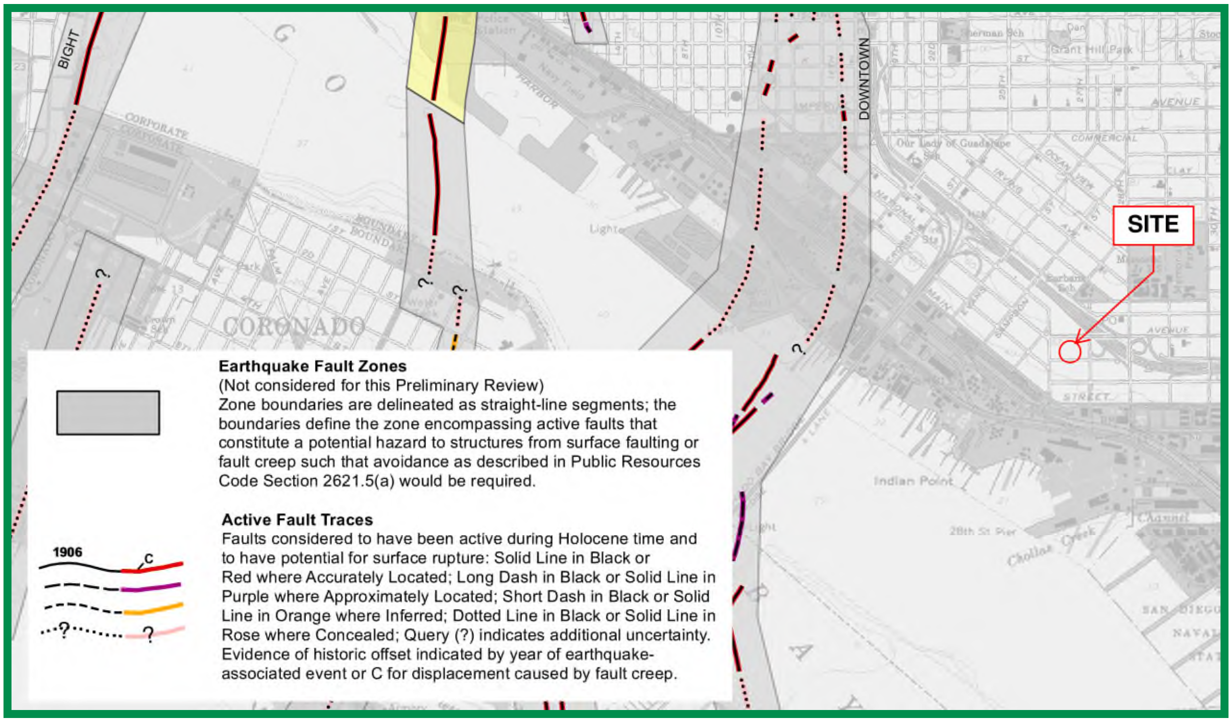
The City of San Diego Seismic Safety Study, Geologic Hazards and Faults, Sheet 13 defines the site with *Hazard Category 13: Downtown special fault zone* (as shown on the Hazard Category Map). A review of geologic literature, the on-site fault evaluation performed by GSD Incorporated, and experience with the soil and geologic conditions in the general area indicate that known active, potentially active, or inactive faults are not located at the site.



Hazard Category Map

In addition, the California Geological Survey (CGS) has issued a revised, draft State of California Earthquake Fault Zone Map for the Point Loma Quadrangle released February 18, 2021 (to be superseded on or about August 17, 2021), which includes portions of the downtown San Diego area.

The subject property is not located within a State of California Earthquake Fault Zone, as shows on the Earthquake Fault Zone Map.



Earthquake Fault Zone Map

We reviewed the fault study report prepared by GDS Incorporated for the site. The fault investigation consisted of excavating two east-west trending fault trenches (Trenches 1 and 2) across the southern end of the site to depths up to approximately 8 feet below existing grade, extending into the underlying Old Paralic Deposits. The locations of the fault trenches are shown on the Geologic Map, Figure 1 and Appendix A presents the fault trench logs. Based on our review of the report and our limited observations of the fault trenches during the field operations, GDS Incorporated did not observe faulting on the property. Therefore, we opine there is no indication of faulting within the immediate properties. Based on our review of the fault logs and our site observations of the trenches, we accept the information presented in the logs.

We opine restrictions on future development at the site are not necessary with respect to the hazard of surface fault rupture. However, a future earthquake originating on a nearby splay of the Rose Canyon Fault could produce very strong near-field ground motions at the site that should be taken into consideration during project design. Also, there is a potential for ground cracking or ground shatter associated with strong ground shaking during an earthquake event on nearby faults to occur beneath the site. The findings of our study are limited to detection of existing seismogenic faults (deep-seated structures) that propagate to the near surface and cannot predict the location of ground cracking associated with strong ground shaking.

6.2 Seismicity

The historic seismicity or instrumental seismic record in the San Diego area indicates that there have been minor earthquakes in the San Diego Bay area, including events in 1964 and 1985 between M3 and 4+ (Treiman, 1993). Surface rupture has not been recorded with any of the seismic activity. Anderson and others (1989) indicate that the greatest peak acceleration recorded in the downtown area (at San Diego Light and Power) was 34 cm/sec^2 (0.03g) produced by an offshore earthquake in 1964 (M 5.6).

Anderson and others (1989) have also estimated recurrence times for major earthquakes that may affect the San Diego Region. By combining geologic data with their model for ground motion attenuation for each earthquake event, they have estimated the recurrence rate of various levels of peak ground acceleration in the San Diego area. The results of their work indicate that peak accelerations of 10 to 20 percent gravity (g) are expected approximately once every 100 years (Anderson and others, 1989). Higher peak accelerations will also occur but with a lower probability of occurrence or higher return period.

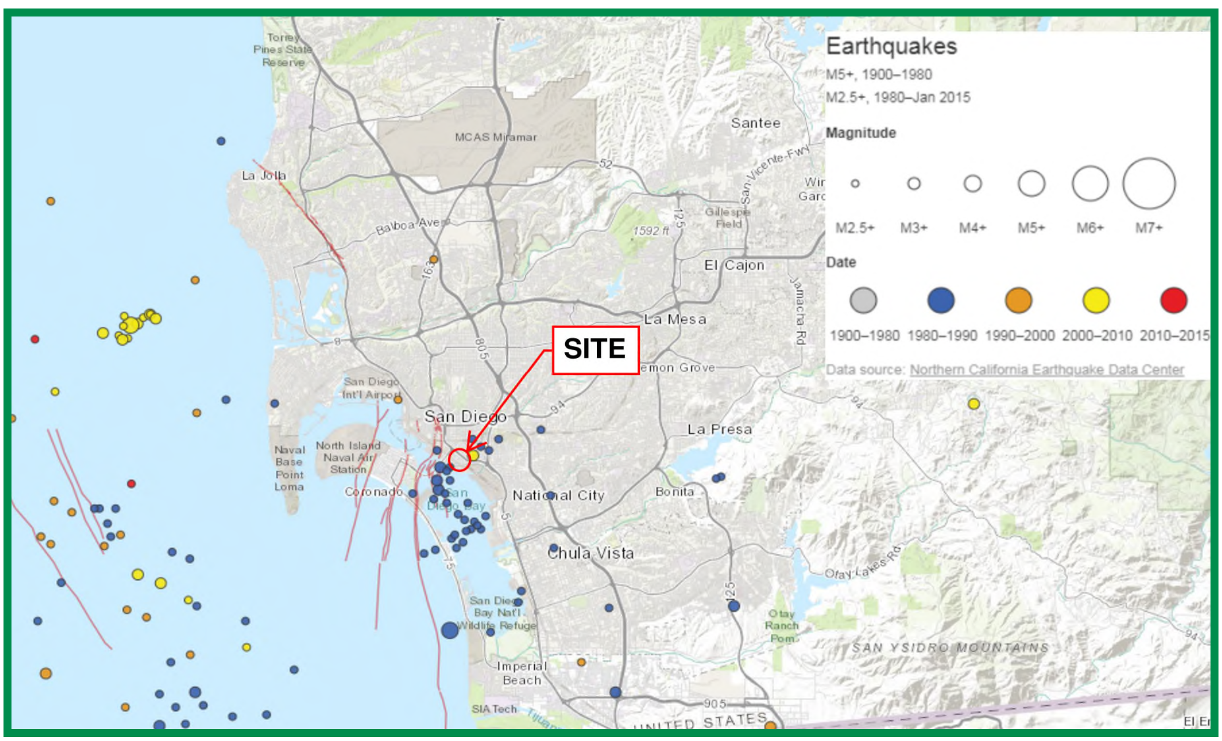
Lindvall and others (1991) have postulated a maximum likely slip rate of about 2 mm per year and a best estimate of about 1.5 mm per year, based on three-dimensional trenching on the Rose Canyon Fault in Rose Canyon several miles north of the site. They found stratigraphic evidence of at least three events during the past 8,100 years. The most recent surface rupture displaces the modern “A” horizon (topsoil), suggesting that this event probably occurred within the past 500 years.

Historically, the Rose Canyon Fault has exhibited low seismicity with respect to earthquakes in excess of magnitude 5.0 or greater. Earthquakes on the Rose Canyon Fault having a maximum magnitude of 7.2 are considered representative of the potential for seismic ground shaking within the property. The “maximum magnitude earthquake” is defined as the maximum earthquake that appears capable of occurring under the presently known tectonic framework.

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.

6.3 Ground Rupture

Ground surface rupture occurs when movement along a fault is sufficient to cause a gap or rupture where the upper edge of the fault zone intersects the ground surface. The potential for ground rupture is considered to be low due to the absence of active faults at the subject site.

6.4 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 dense nature of the underlying Old Paralic Deposits and the depth to the groundwater elevation, liquefaction potential for the site is considered very low.

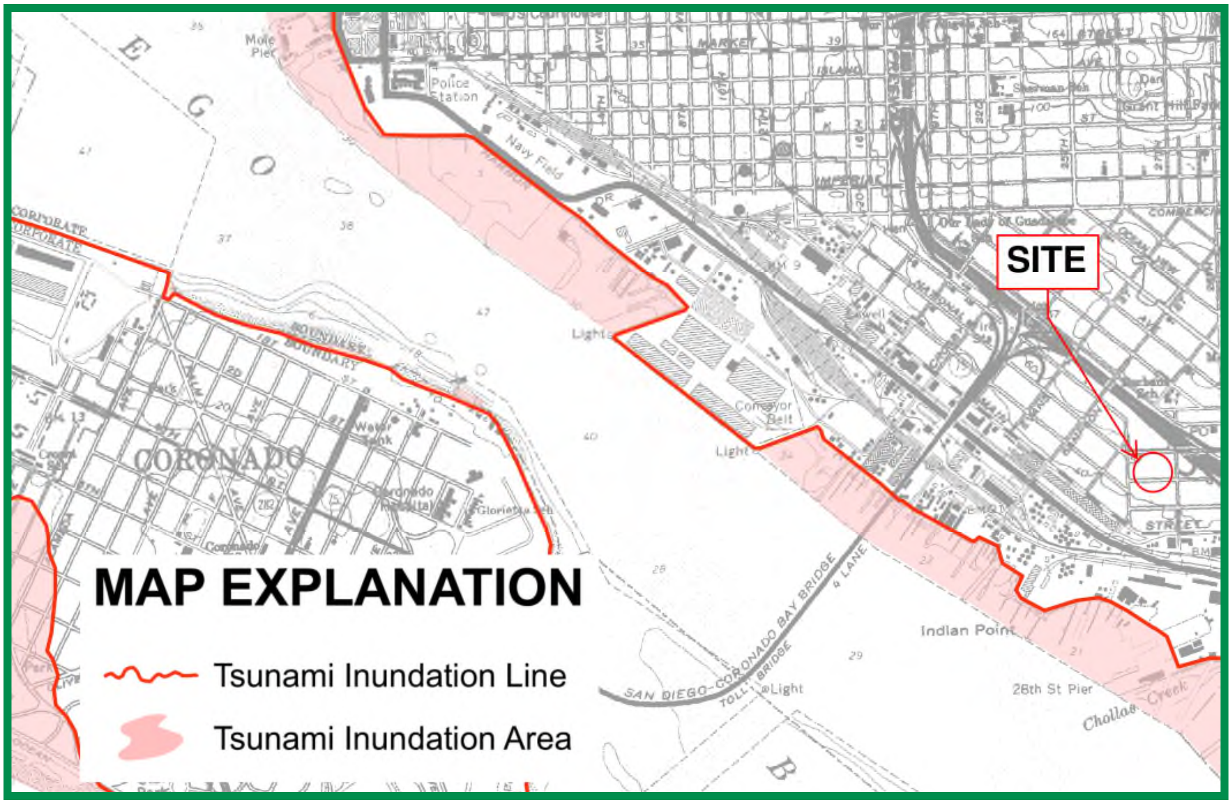
6.5 Storm Surge, Tsunamis, and Seiches

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 0.4 miles from the San Diego Bay, is at an elevation of about 50 feet or greater above Mean Sea Level (MSL) and is protected from ocean waves by the Silver Strand and Coronado to the southwest. Based on historic and predicated wave heights and runout lengths, we opine that the proposed site elevation is sufficient to mitigate the risk; therefore, the potential of storm surges affecting the site is considered low.

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement. The site is located approximately 0.4 miles from the San Diego Bay, is at an elevation of about 50 feet or greater above Mean Sea Level (MSL) and is protected from ocean waves by the Silver Strand and Coronado to the southwest. Based on historic and predicated wave heights and runout lengths, it is our opinion that the proposed site elevation is sufficient to mitigate the risk; therefore, we consider the potential for seiches to impact the site 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 first-order driving force for locally generated tsunamis offshore southern California is expected to be tectonic deformation from large earthquakes (Legg, *et al.*, 2002). The California Geologic Survey (CGS) *Tsunami Inundation Map for Emergency Planning, Point Loma Quadrangle*

(2009), shows the site is not within a tsunami inundation area. Therefore, the potential of tsunamis affecting the site is negligible.



Tsunami Inundation Map – CGS La Jolla Quadrangle

6.6 Landslides

We did not observe evidence of previous or incipient slope instability at the site during our study and the property is relatively flat. Published geologic mapping indicates landslides are not present on or adjacent to the site. Therefore, in our professional opinion, the potential for a landslide is not a significant concern for this project.

6.7 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.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 The soil or geologic conditions encountered during the site exploration performed by GDS Incorporated would not 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.

7.1.2 We expect the planned development will be supported on a shallow foundation system with a concrete slab-on-grade. We should be contacted to provide additional recommendations if subterranean levels are planned.

7.1.3 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.

7.1.4 The site is located within a fault study zone established by the City of San Diego. The site-specific-fault rupture hazard investigation performed by GDS Incorporated indicates that there is no evidence of active, potentially active or inactive faulting observed in the underlying Old Paralic Deposits at the site. We opine active or potentially active faulting does not pass beneath the site and building setbacks will not be required.

7.1.5 The undocumented fill, topsoil and upper dry to damp portions of the Old Paralic Deposits are likely potentially compressible and unsuitable in their present condition for the support of compacted fill or settlement-sensitive improvements. Remedial grading of these materials should be performed as discussed herein. The dense portions of the Old Paralic Deposits are considered suitable for the support of proposed fill and structural loads.

7.1.6 Groundwater was not encountered during the subsurface exploration performed by GDS Incorporated and we do not expect it to be a constraint to project development. We expect groundwater is approximately 45 to 55 feet below existing grades. However, seepage within surficial soils may be encountered during the grading operations, especially during the rainy seasons.

7.1.7 Excavation of the fill, topsoil, and the Old Paralic Deposits should generally be possible with moderate to heavy effort using conventional, heavy-duty equipment during grading and trenching operations. We expect very heavy effort with possible refusal in localized areas for excavations into strongly cemented portions or gravel/cobble layers of the Old Paralic Deposits.

7.1.8 Proper drainage should be maintained in order to preserve the engineering properties of the soil. Recommendations for site drainage are provided herein.

7.1.9 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. We should be contacted to provide an update to this report when development plans have been prepared.

7.1.10 Surface settlement monuments and canyon subdrains will not be required on this project.

7.2 Excavation and Soil Characteristics

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

7.2.2 The soil encountered in the field investigation is considered to be "non-expansive" (expansion index [EI] of 20 or less) as defined by 2019 California Building Code (CBC) Section 1803.5.3. Table 7.2 presents soil classifications based on the expansion index. We expect a majority of the soil encountered during remedial grading will possess a "very low" to "low" expansion potential (EI of 50 or less) in accordance with ASTM D 4829.

TABLE 7.2
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	

7.2.3 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Appendix B presents results of the laboratory water-soluble sulfate content tests. The test result indicates 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.

7.2.4 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.

7.3 Grading

7.3.1 Grading should be performed in accordance with the recommendations provided in this report, the Recommended Grading Specifications contained in Appendix C and the City of San Diego's Grading Ordinance. Geocon Incorporated should observe the grading operations on a full-time basis and provide testing during the fill placement.

7.3.2 Prior to commencing grading, a preconstruction conference should be held at the site with the city 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.

7.3.3 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.

7.3.4 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.

7.3.5 We expect the planned building(s) will be supported on a shallow foundation system. The undocumented fill, topsoil and upper portions of the Old Paralic Deposits to at least a depth of 5 feet below existing grade or 2 feet below the proposed foundations (whichever results in a deeper excavation) should be removed and replaced with properly compacted fill. The removals should extend at least 10 feet outside of the proposed foundation system, where possible.

7.3.6 In areas of proposed improvements outside of the building areas, the upper 2 to 3 feet of existing soil should be processed, moisture conditioned as necessary and recompacted.

Deeper removals may be required in areas where loose or saturated materials are encountered. The removals should extend at least 2 feet outside of the improvement area, where possible. Table 7.3.1 provides a summary of the grading recommendations.

TABLE 7.3.1
SUMMARY OF GRADING RECOMMENDATIONS

Area	Removal Requirements
Building Pad	Removal of Undocumented Fill, Topsoil and Upper Portions of the Old Paralic Deposits to at Least a Depth of 5 Feet Below Existing Grade and 2 Feet Below Proposed Foundations
Site Development	Process Upper 2 to 3 Feet of Existing Materials
Grading Limits	10 Feet Outside of Buildings/2 Feet Outside of Improvement Areas, Where Possible
Exposed Bottoms of Remedial Grading	Scarify Upper 12 Inches

7.3.7 The bottom of the excavations should be sloped 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.

7.3.8 The site should then be brought to final subgrade elevations with fill compacted in layers. In general, the existing soil 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 vehicular 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.

7.3.9 Import fill (if necessary) should consist of the characteristics presented in Table 7.3.2. 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 7.3.2
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

7.4 Subdrains

7.4.1 With the exception of retaining wall drains, we do not expect the installation of other subdrains. We should be contacted to provide recommendations for wick drains, if proposed.

7.5 Temporary Excavations

7.5.1 The recommendations included herein are provided for stable excavations. It is the responsibility of the contractor and their competent person to ensure all excavations, temporary slopes and trenches are properly constructed and maintained in accordance with applicable OSHA guidelines in order to maintain safety and the stability of the excavations and adjacent improvements. These excavations should not be allowed to become saturated or to dry out. Surcharge loads should not be permitted to a distance equal to the height of the excavation from the top of the excavation. The top of the excavation should be a minimum of 15 feet from the edge of existing improvements. Excavations steeper than those recommended or closer than 15 feet from an existing surface improvement should be shored in accordance with applicable OSHA codes and regulations.

7.5.2 The stability of the excavations is dependent on the design and construction of the shoring system and site conditions. Therefore, Geocon Incorporated cannot be responsible for site safety and the stability of the proposed excavations.

7.5.3 We should be contacted to provide shoring recommendations if subterranean structures are planned.

7.6 Seismic Design Criteria – 2019 California Building Code

7.6.1 Table 7.6.1 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.

TABLE 7.6.1
2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	C	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.467g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.491g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.200	Table 1613.2.3(1)
Site Coefficient, F_V	1.500*	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.760g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.737g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.173g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.491g*	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.

7.6.2 Table 7.6.2 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 7.6.2
ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.667g	Figure 22-7
Site Coefficient, F_{PGA}	1.200	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.800g	Section 11.8.3 (Eqn 11.8-1)

7.6.3 Conformance to the criteria in Tables 7.6.1 and 7.6.2 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.

7.6.4 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 8.6.3 presents a summary of the risk categories in accordance with ASCE 7-16.

TABLE 7.6.3
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

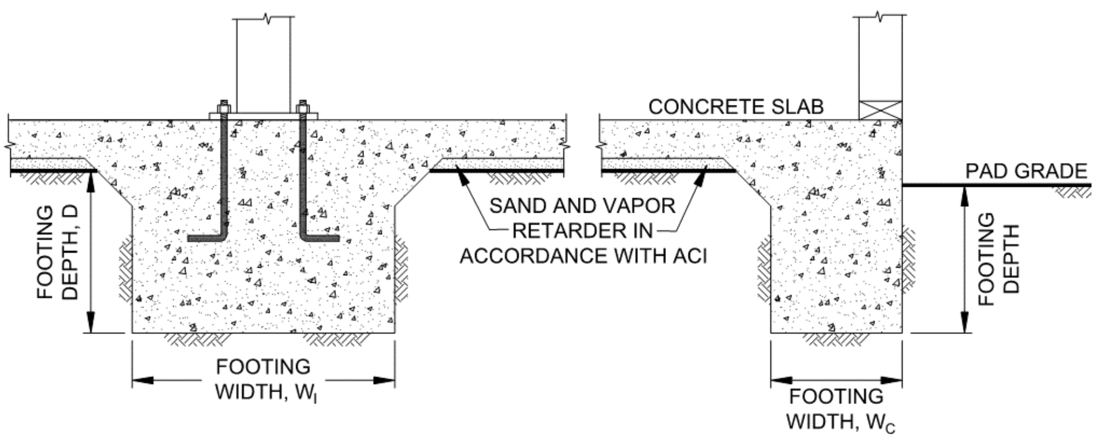
7.7 Shallow Foundations

7.7.1 The proposed structure(s) can be supported on a shallow foundation system founded in properly compacted fill. Foundations for the structure should consist of continuous strip footings and/or isolated spread footings. Table 7.7 provides a summary of the foundation design recommendations.

TABLE 7.7
SUMMARY OF 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 Steel Reinforcement	4 No. 5 Bars, 2 at the Top and 2 at the Bottom
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
Footing Size Used for Settlement	9-Foot Square
Design Expansion Index	50 or less

7.7.2 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.



Wall/Column Footing Dimension Detail

7.7.3 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.

7.7.4 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.

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

7.8 Concrete Slabs-On-Grade

7.8.1 Concrete slabs-on-grade for the structures should be constructed in accordance with Table 7.8.

TABLE 7.8
MINIMUM CONCRETE SLAB-ON-GRADE RECOMMENDATIONS

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

7.8.2 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.

7.8.3 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.

7.8.4 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.

7.8.5 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.

7.8.6 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.

7.8.7 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, stucco 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.

7.9 Exterior Concrete Flatwork

7.9.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations presented in Table 7.9. The recommended steel reinforcement would help reduce the potential for cracking.

TABLE 7.9
MINIMUM CONCRETE FLATWORK RECOMMENDATIONS

Expansion Index, EI	Minimum Steel Reinforcement* Options	Minimum Thickness
EI $\leq$ 50	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.

7.9.2 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.

7.9.3 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.

7.9.4 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.

7.9.5 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.

7.9.6 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.

7.10 Retaining Walls

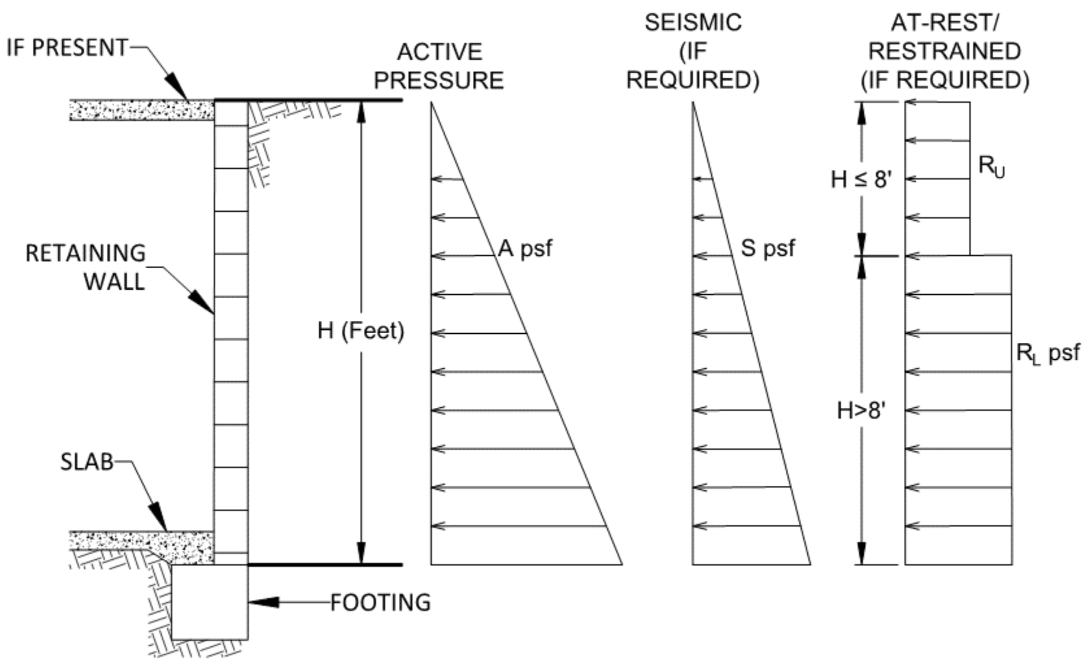
7.10.1 Retaining walls should be designed using the values presented in Table 7.10.1. Soil with an expansion index (EI) of greater than 50 should not be used as backfill material behind retaining walls.

TABLE 7.10.1
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≤50

H equals the height of the retaining portion of the wall

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



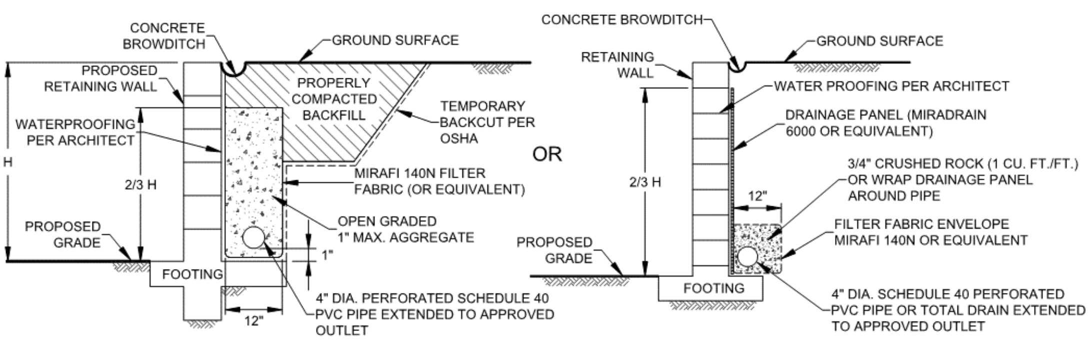
Retaining Wall Loading Diagram

7.10.3 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.

7.10.4 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.

7.10.5 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.

7.10.6 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

7.10.7 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.

7.10.8 In general, wall foundations should be designed in accordance with Table 7.10.2. 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 7.10.2
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

7.10.9 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.

7.10.10 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.

7.10.11 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.

7.11 Lateral Loading

7.11.1 Table 7.11 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 7.11
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.

7.11.2 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.

7.12 Preliminary Pavement Recommendations

7.12.1 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 have assumed an R-Value of 15 and 78 for the subgrade soil and base materials, respectively, for the purposes of this preliminary analysis. Table 7.12.1 presents the preliminary flexible pavement sections.

TABLE 7.12.1
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	15	3	8
Driveways for automobiles and light-duty vehicles	5.5	15	3	10
Medium truck traffic areas	6.0	15	3.5	11
Driveways for heavy truck and fire truck traffic	7.0	15	4	13

7.12.2 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.

7.12.3 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 7.12.2.

TABLE 7.12.2
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	50 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

7.12.4 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 7.12.3.

TABLE 7.12.3
RIGID VEHICULAR PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Automobile Parking Stalls (TC=A)	6.0
Driveways and Fire Truck Traffic (TC=C)	7.5

7.12.5 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.

7.12.6 The rigid pavement should also be designed and constructed incorporating the parameters presented in Table 7.12.4.

TABLE 7.12.4
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

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

7.12.8 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.

7.12.9 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.

7.12.10 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.

7.13 Site Drainage and Moisture Protection

7.13.1 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.

7.13.2 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.

7.13.3 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.

7.13.4 We should prepare a storm water infiltration feasibility report of storm water management devices are planned. We expect a no infiltration condition is likely due to the presence of hydrocollapse potential.

7.14 Grading and Foundation Plan Review

7.14.1 Geocon Incorporated should review the grading and building foundation plans for the project prior to final design submittal to evaluate if additional analyses and/or recommendations are required.

7.15 Testing and Observation Services During Construction

7.15.1 Geocon Incorporated should provide geotechnical testing and observation services during the grading operations, foundation construction, utility installation, retaining wall backfill and pavement installation. Table 7.15 presents the typical geotechnical observations we would expect for the proposed improvements.

TABLE 7.15
EXPECTED GEOTECHNICAL TESTING AND OBSERVATION SERVICES

Construction Phase	Observations	Expected Time Frame
Grading	Base of Removal	Part Time During Removals
	Geologic Logging	Part Time to Full Time
	Fill Placement and Soil Compaction Operations	Full Time
Foundations	Foundation Excavation Observations	Part Time
Utility Backfill	Fill Placement and Soil Compaction Operations	Part Time to Full Time
Retaining Wall Backfill	Fill Placement and Soil Compaction Operations	Part Time to Full Time
Subgrade for Sidewalks, Curb/Gutter and Pavement	Soil Compaction Operations	Part Time
Pavement Construction	Base Placement and Compaction	Part Time
	Asphalt Concrete Placement and Compaction	Full Time

LIMITATIONS AND UNIFORMITY OF CONDITIONS

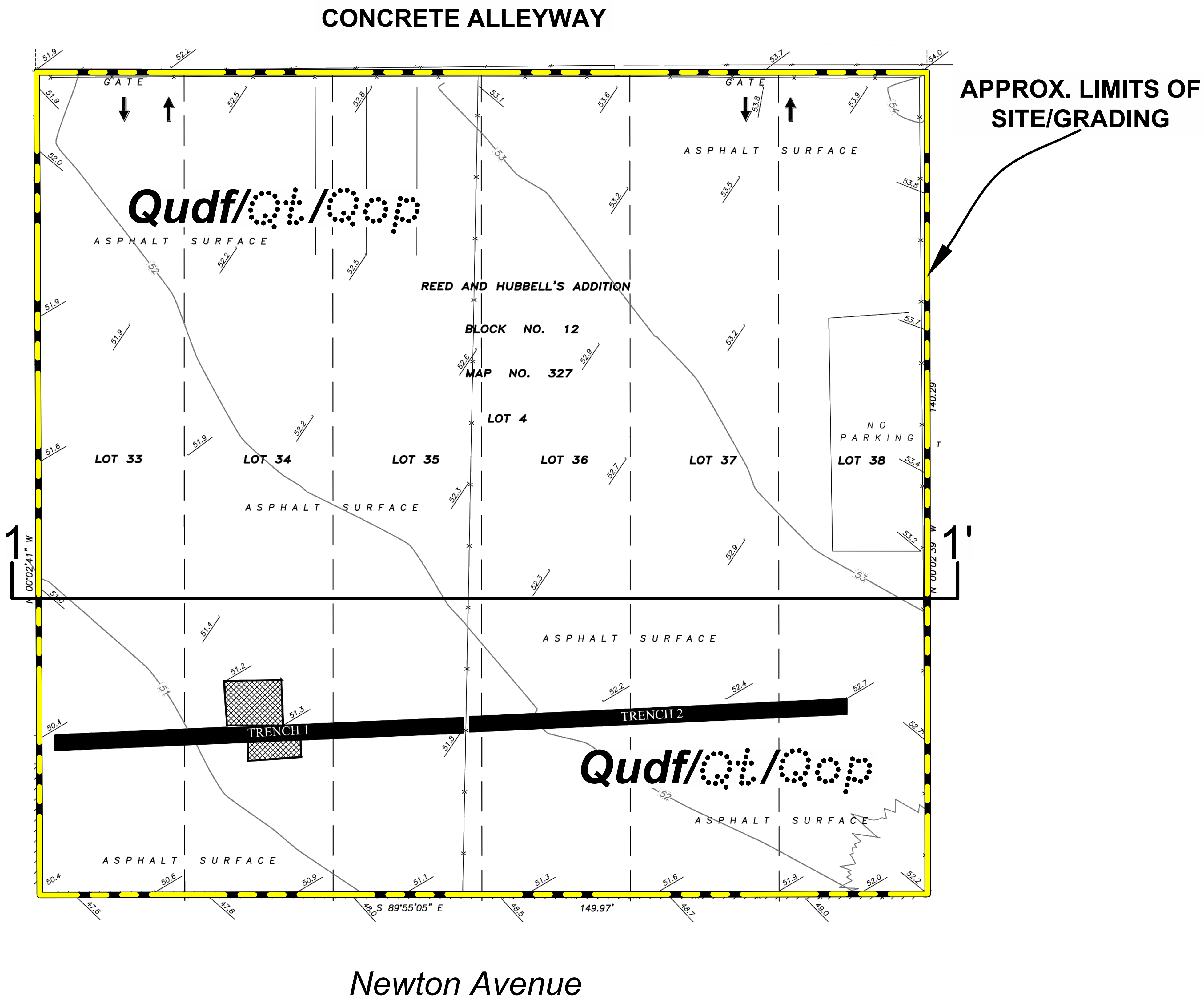
1. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development 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.

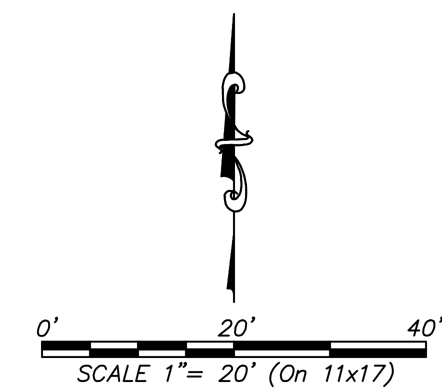
2. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction 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.

2642, 2646, 2648 NEWTON AVENUE
SAN DIEGO, CALIFORNIA





GEOCON LEGEND

Qudf	........UNDOCUMENTED FILL
Qt	........TOPSOIL (Dotted Where Buried)
Qop	........OLD PARALIC DEPOSITS (Dotted Where Buried)
1 — 1'	........APPROX. LOCATION OF GEOLOGIC CROSS-SECTION
	........FAULT STUDY TRENCH (GDS Incorporated, 2021)
	........REMEDIAL EXCAVATIONS (GDS Incorporated, 2021)

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GEOLOGIC MAP

PROJECT NO. G2778 - 52 - 01
FIGURE 1
DATE 08 - 13 - 2021

Plotted:08/13/2021 12:06PM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G2778-52-01 Newton Ave\SHEETS\G2778-52-01 GeoMap.dwg

2642, 2646, 2648 NEWTON AVENUE
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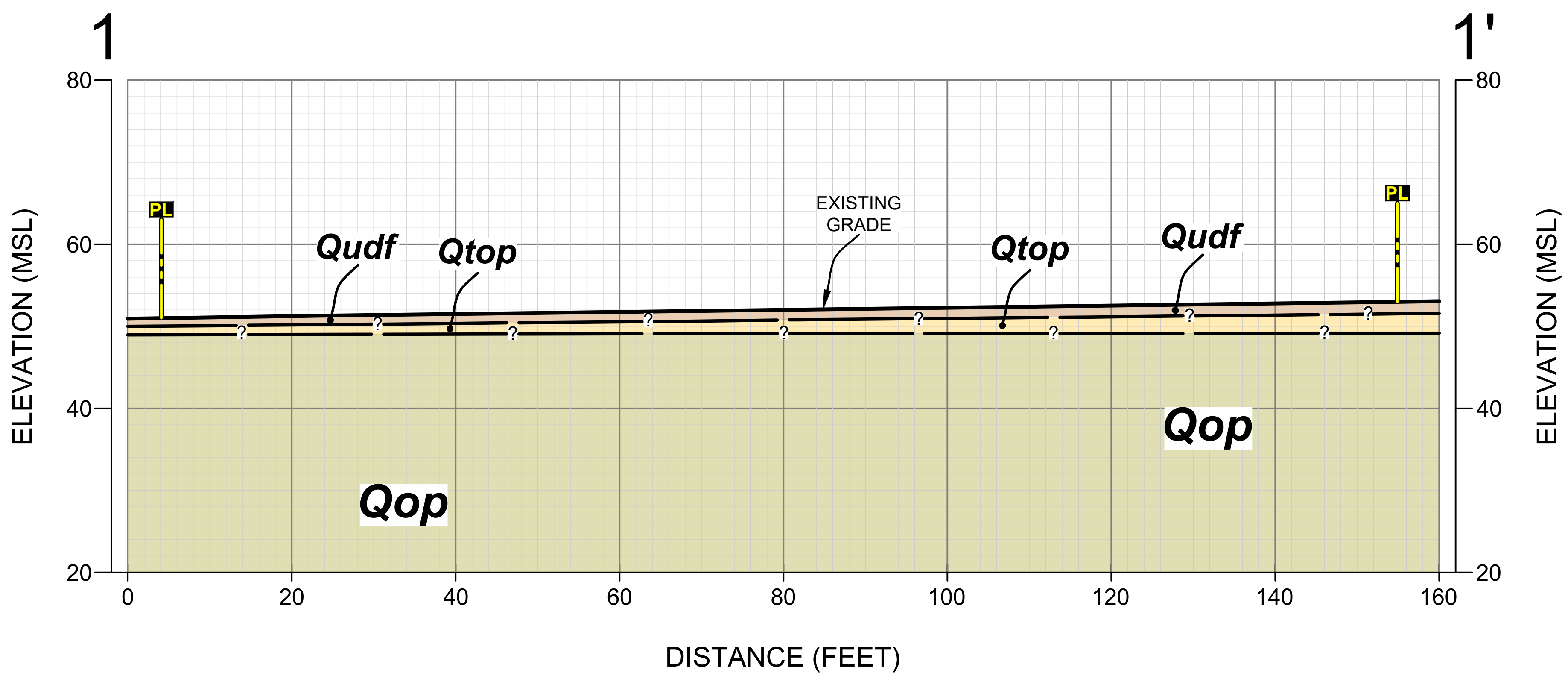

GEOLOGIC CROSS-SECTION 1-1'

SCALE: 1" = 20' (Vert. = Horiz.)

GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL

Qtop ....TOPSOIL

Qop ....OLD PARALIC DEPOSITS (Dotted Where Buried)

?........APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)

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

GEOLOGIC CROSS - SECTION

PROJECT NO. G2778 - 52 - 01
FIGURE 2
DATE 08 - 13 - 2021

Plotted:08/13/2021 12:14PM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G2778-52-01 Newton Ave\SHEETS\G2778-52-01 Cross-Section.dwg

APPENDIX A

APPENDIX A

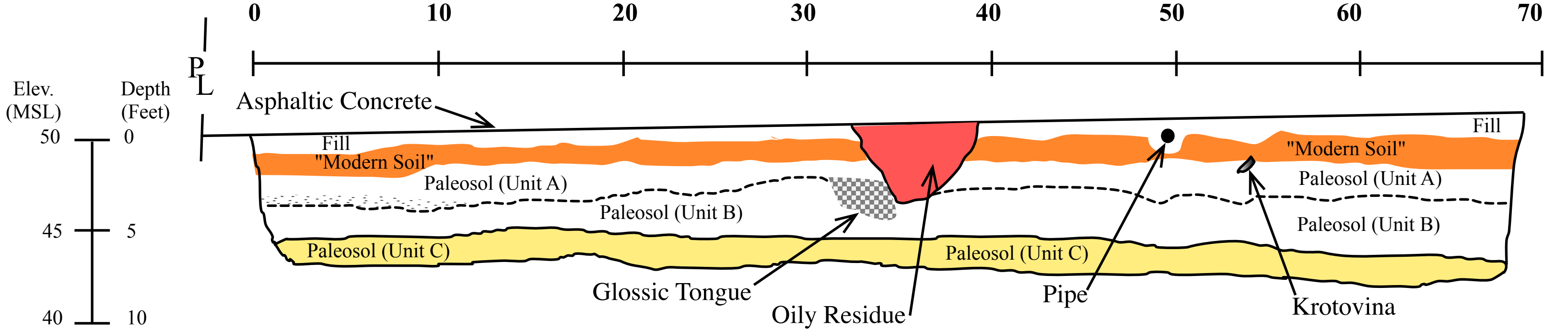
FAULT TRENCH LOGS (GDS INCORPORATED, 2021)

FOR

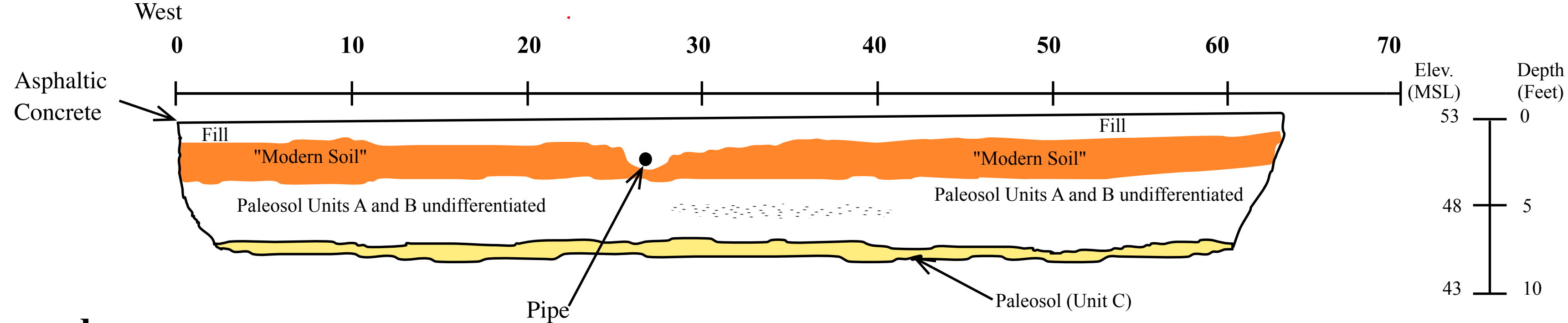
2642, 2646, 2648 NEWTON AVENUE
SAN DIEGO, CALIFORNIA

PROJECT NO. G2778-52-01

Trench 1 - View to North



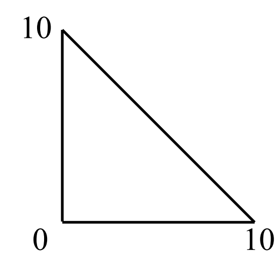
Trench 2 - View to North



Legend:

Fill	Dominantly light brown, moist, fine to medium silty SAND, trace to 5% gravel, debris including bits of glass, bricks, etc.
Modern Soil	Dark reddish brown, moist, clayey fine SAND, trace manganese nodules, some carbonate occurring as blebs or fine stringers. Abrupt contact to underlying unit.
Paleosol (Unit A)	Light brown, dry to moist, clayey fine to medium SAND, locally strong angular blocky structure, clay films on ped surfaces, carbonates occurring as nodules or blebs. Some zones of fine to coarse SAND (as shown). Gradational, wavy contact to underlying unit.
Paleosol (Unit B)	Olive brown, dry to moist, silty fine to medium SAND, manganese staining. Dominantly abrupt contact underlying unit.
Paleosol (Unit C)	Light brown, dominantly poorly graded fine to medium SAND, moist, loose.

Scale



NOTES:
Scale as shown
Elevations, directions, dimensions, and locations are approximate.

GDS INC.
PROFESSIONAL GEOLOGIC CONSULTANCY

Figure 3: Trench Logs
2642, 2646, 2648 Newton Avenue
San Diego, California

APPENDIX A

PHOTOGRAPHIC DOCUMENTATION

Trench 1 - View to North

West

Match Line

Match Line

East

NOTE:
NTS
Directions and locations are approximate

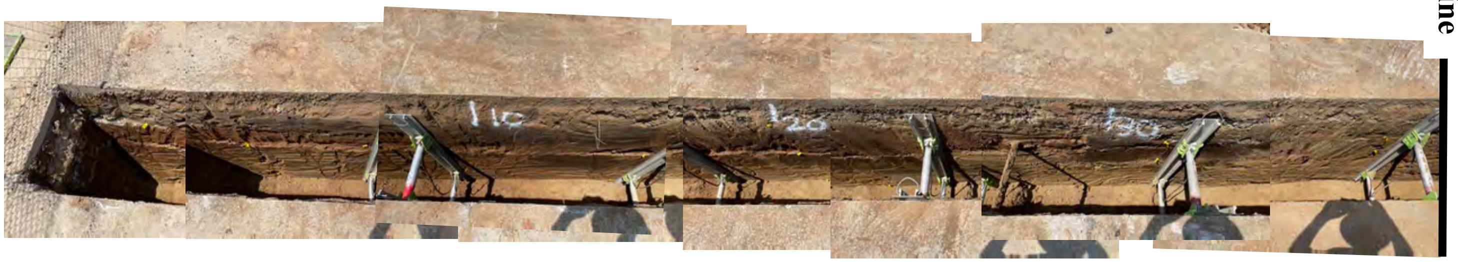
GDS INC.
PROFESSIONAL GEOLOGIC CONSULTANCY

Appendix A: Photomosaic of Trench 1
2642, 2646, 2648 Newton Avenue
San Diego, California

Trench 2 - View to North

West

Match Line



Match Line

East

NOTE:
NTS
Directions and locations are approximate

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Appendix A: Photomosaic of Trench 2
2642, 2646, 2648 Newton Avenue
San Diego, California

APPENDIX B

APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. We tested selected soil samples for in-place moisture content, maximum density/optimum moisture content, direct shear strength, expansion index, water-soluble sulfate and gradation characteristics. The results of our laboratory tests are presented herein.

SUMMARY OF LABORATORY IN-PLACE MOISTURE CONTENT TEST RESULTS ASTM D 2216

Sample No.	Sample Depth (feet)	Geologic Unit	Moisture Content (% dry wt.)
3	1-3	Qudf/Qt	10.2

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

Sample No.	Sample Depth (feet)	Description (Geologic Unit)	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
1	0-3	Brown, Clayey, fine to medium SAND (Qudf/Qt)	128.9	9.6

SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS ASTM D 4829

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	2019 CBC Expansion Classification	ASTM Soil Expansion Classification
	Before Test	After Test				
1	8.4	16.4	116.4	15	Non-Expansive	Very Low
2	8.3	14.0	117.3	0	Non-Expansive	Very Low

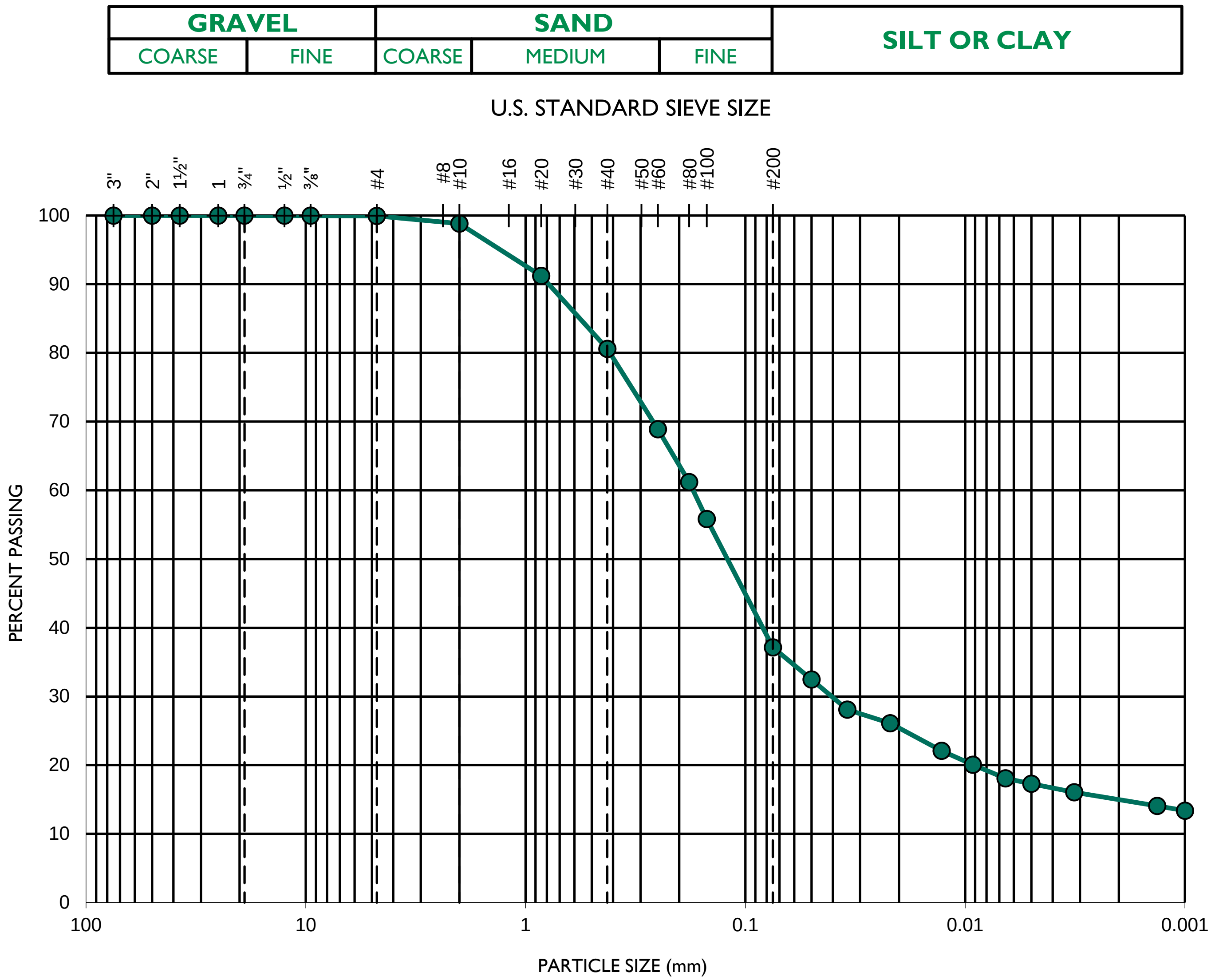
SUMMARY OF LABORATORY WATER-SOLUBLE SULFATE TEST RESULTS CALIFORNIA TEST NO. 417

Sample No.	Depth (feet)	Geologic Unit	Water-Soluble Sulfate (%)	ACI 318 Sulfate Exposure
1	0-3	Qudf/Qt	0.004	S0
2	5-7	Qop	0.016	S0

SAMPLE NO.: I

SAMPLE DEPTH (FT.): 0-3'

GEOLOGIC UNIT: Qudf/Qt



TEST DATA					
D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	C_c	C_u	SOIL DESCRIPTION
--	0.04119	0.17327	--	--	Silty Clayey SAND

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SIEVE ANALYSES - ASTM D 135 & D 422

2642, 2646, 2648 NEWTON AVENUE

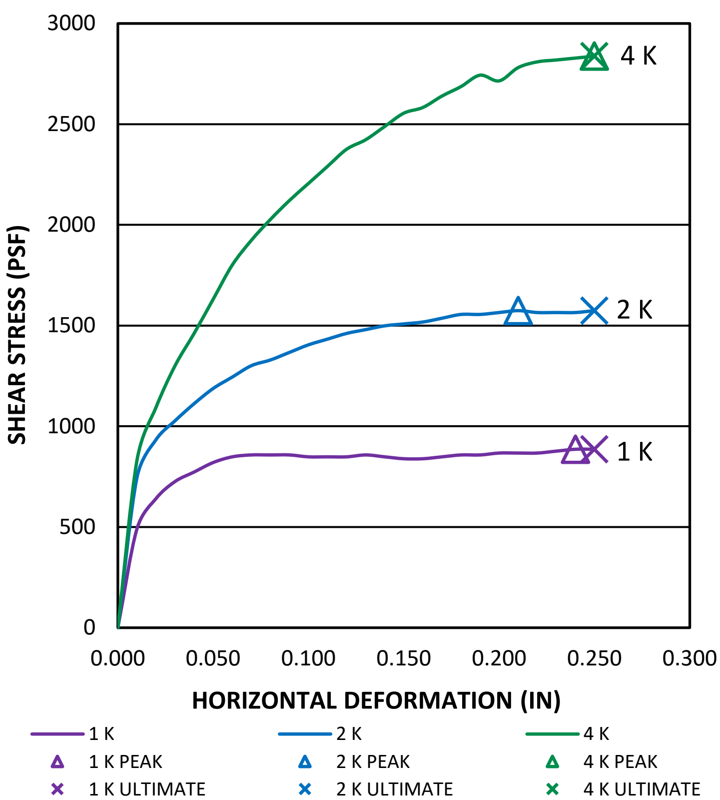
PROJECT NO.: G2778-52-01

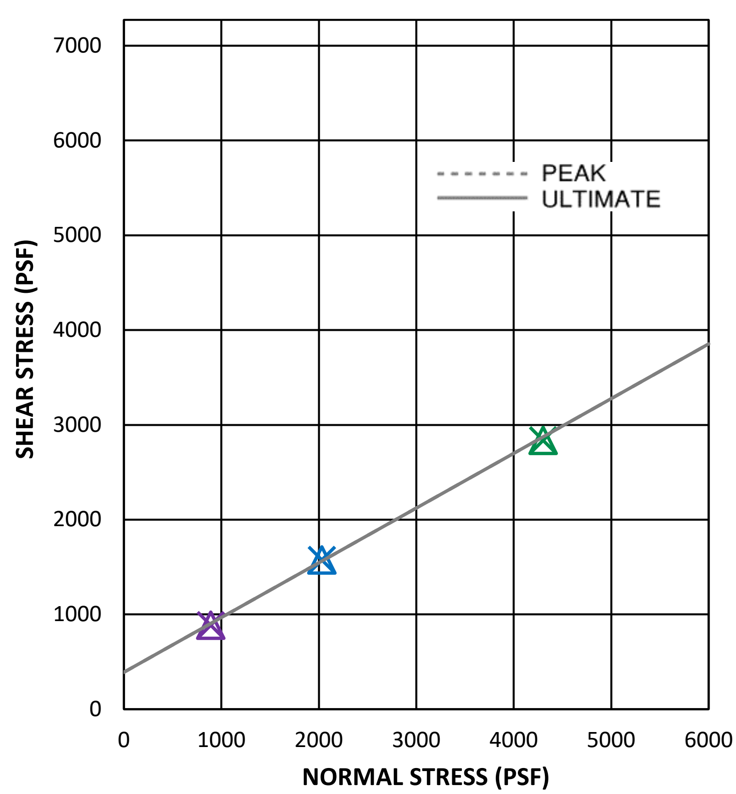
SAMPLE NO.:	I	GEOLOGIC UNIT:	Qudf/Qt
SAMPLE DEPTH (FT):	0-3'	NATURAL/REMOLDED:	R

INITIAL CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
ACTUAL NORMAL STRESS (PSF):	890	2030	4300	--
WATER CONTENT (%):	10.3	9.5	9.1	9.6
DRY DENSITY (PCF):	115.0	116.4	116.6	116.0

AFTER TEST CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
WATER CONTENT (%):	15.6	15.4	15.6	15.5
PEAK SHEAR STRESS (PSF):	886	1574	2838	--
ULT.-E.O.T. SHEAR STRESS (PSF):	886	1574	2838	--

RESULTS		
PEAK	COHESION, C (PSF)	390
	FRICTION ANGLE (DEGREES)	30
ULTIMATE	COHESION, C (PSF)	390
	FRICTION ANGLE (DEGREES)	30





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DIRECT SHEAR - ASTM D 3080

2642, 2646, 2648 NEWTON AVE

PROJECT NO.: G2778-52-01

APPENDIX C

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

2642, 2646, 2648 NEWTON AVENUE
SAN DIEGO, CALIFORNIA

PROJECT NO. G2778-52-01

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

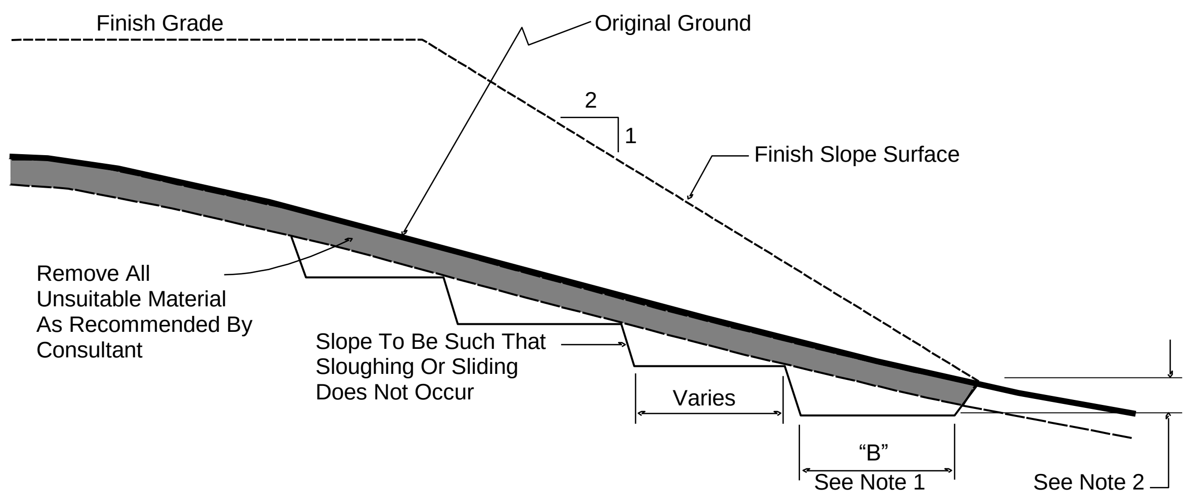
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material 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



No Scale

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.

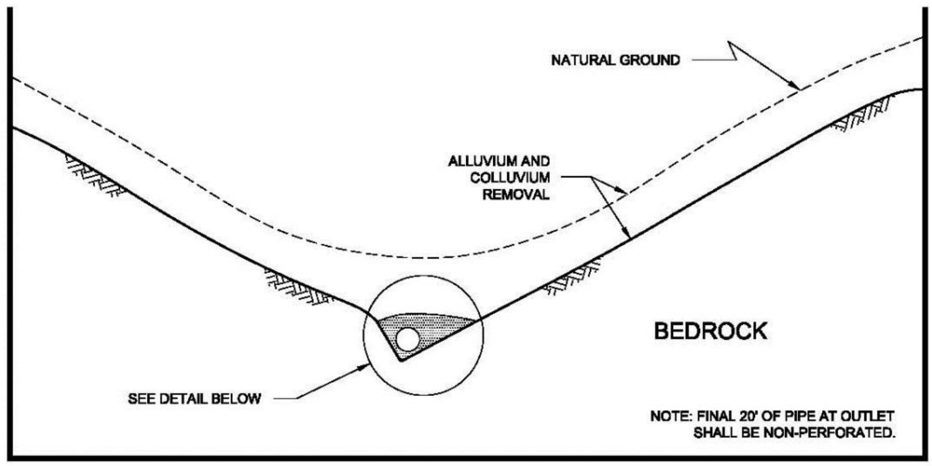
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant 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.

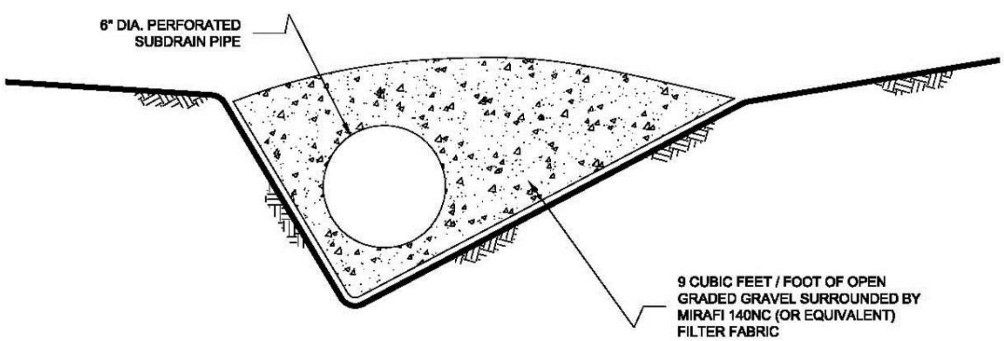
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

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:

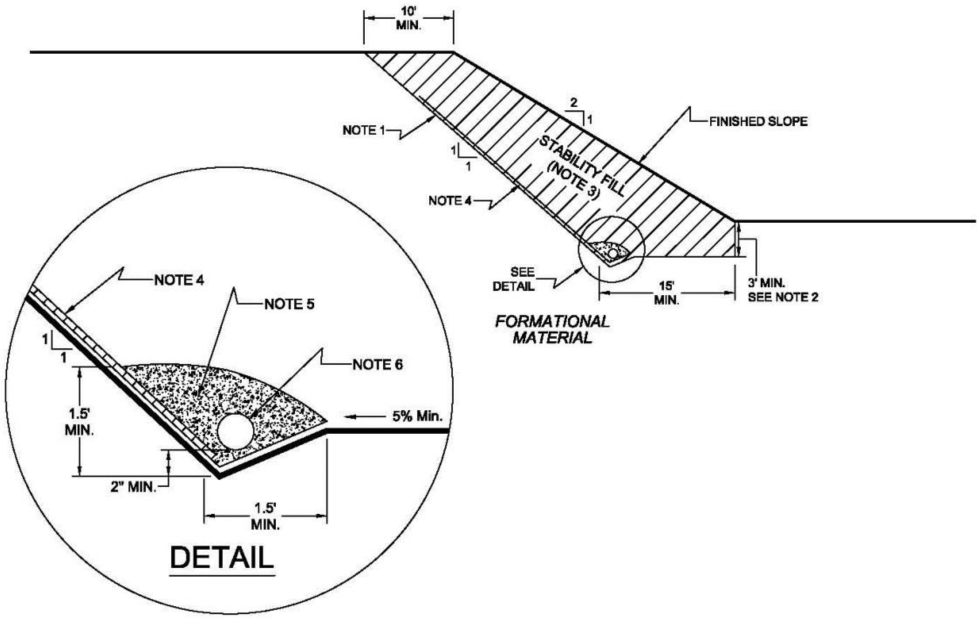
1......6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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.

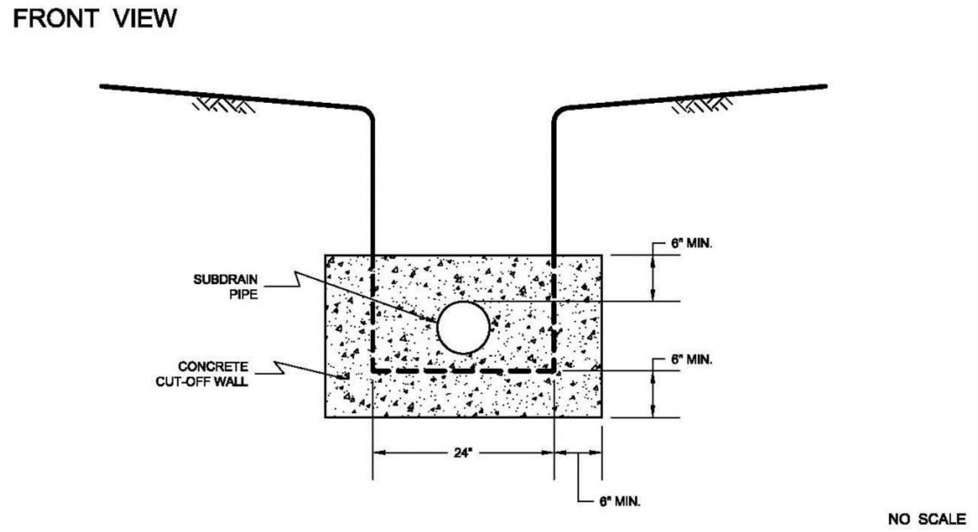
NO SCALE

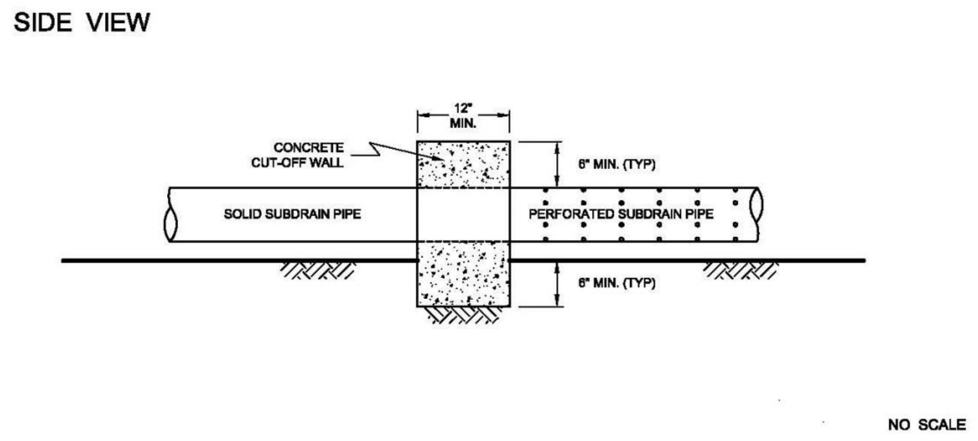
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

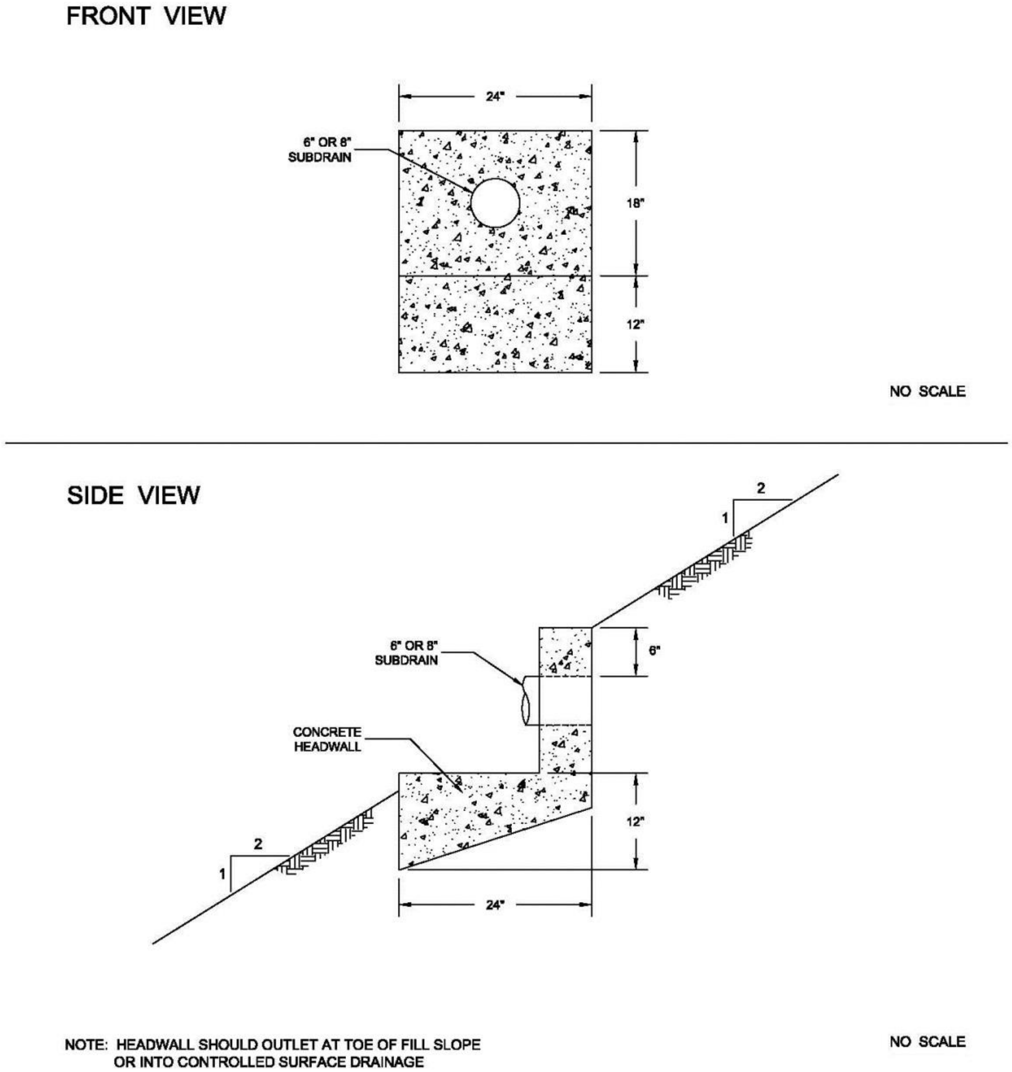
TYPICAL CUT OFF WALL DETAIL





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.

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