GEOTECHNICAL INVESTIGATION

901 MISSION AVENUE MIXED-USE
OCEANSIDE, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

JPI
SAN DIEGO. CALIFORNIA

MAY 15, 2024
PROJECT NO. G3303-42-01

GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. G3303-42-01
May 15, 2024

JPI
11988 El Camino Real, Suite 200
San Diego, California 92130

Attention: Mr. Conner Kloeppel

Subject: GEOTECHNICAL INVESTIGATION
901 MISSION AVENUE MIXED-USE
OCEANSIDE, CALIFORNIA

Dear Mr. Kloeppel:

In accordance with your request, we herein submit this geotechnical investigation for the subject site. Geocon Incorporated previously performed an investigation on the property (Geocon, 2004) to evaluate geologic conditions and potential geologic hazards at the site. The field data and laboratory test results from that study were used to provide conclusions and recommendations pertinent to geotechnical aspects of currently proposed project. The site is suitable for development provided the recommendations in this report are followed.

If you have any 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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GEOTECHNICAL INVVESTIGATION

1. PURPOSE AND SCOPE

This report provides recommendations for development of a proposed mixed-use project planned for the property located at 901 Mission Avenue between Horne Street and Clementine Street in Oceanside, California (see Site Location Map). Geocon Incorporated previously performed a geotechnical investigation on the property (Geocon, 2004) and the field data and laboratory test results from that study are deemed adequate and applicable to the currently proposed project.

Site Location Map

The scope of this study included reviewing Geocon (2004) and other readily available geologic reports and maps; engineering analysis using data from Geocon (2004); and preparing this report. We herein provide recommendations for remedial grading; foundation design; concrete slab-on-grade; and concrete flatwork.

We conducted our previous field investigation (Geocon, 2004) on December 30, 2003. The approximate locations of the exploratory borings are shown on the Geologic Map, Figure 1.

Appendix A presents the exploratory boring logs and details of the field investigation. Details of the laboratory testing and a summary of test results are provided in Appendix B and on the boring logs in Appendix A.

The recommendations presented herein are based on our analysis of data and our observations obtained during previous investigations and our experience with similar soil and geologic conditions.

2. SITE AND PROJECT DESCRIPTION

The site is bounded by Horne Street to the east, Clementine Street to the west, Mission Avenue to the north, and Seagaze Drive to the south. Site elevation ranges between about 100 feet above Mean Sea Level (MSL) on the western side to about 117 feet MSL to the eastern side.

The property is currently vacant. A paved driveway bisects the north and south halves of the property. Previously, buildings and parking lots occupied the property. In 1982 or 1983, buildings on the northern half of the property were demolished. During our previous investigation (Geocon, 2004), two single-story houses and apartment buildings occupied the southwestern corner of the block. In 2006 or 2007 the remaining structures on the property were demolished.

Based on discussions with you and drawings we have reviewed, we understand that a 6-story apartment building over two podium levels and two levels of subterranean parking are planned.

3. SOIL AND GEOLOGIC CONDITIONS

The site is underlain by undocumented fill overlying Old Paralic Deposits (shown as terrace deposits on the boring logs). Descriptions of the soils encountered during our investigation are presented below and on the boring logs in Appendix A. A Geologic Map and Geologic Cross Sections showing these units are presented as Figures 1 and 2, respectively.

3.1 Undocumented Fill (Qudf)

Undocumented fill soils were encountered in all borings to depths ranging between about 2 to 9 feet below existing grade. The fill generally consisted of silty sand with scattered gravel. The fill is unsuitable to support the planned improvements and will require complete removal. These soils may be reused as fill provided debris is removed.

3.2 Old Paralic Deposits (Qop)

Old Paralic Deposits were encountered beneath the undocumented fill and extended through the full depth explored. The terrace deposit consisted of dense to very dense, silty sand, clayey sand, and sand with silt. Cobbles were encountered at a depth of approximately 20 feet below the ground surface. The Old Paralic Deposits are suitable to support the planned improvements.

4. GROUNDWATER

We did not encounter groundwater during our previous investigation; however, it is not uncommon for groundwater or seepage conditions to develop where none previously existed. Groundwater and seepage conditions are dependent on seasonal precipitation, irrigation, land use, among other factors, and vary as a result. Proper surface drainage will be important to future performance of the project. We do not anticipate ground water will affect the constructability of the project.

5. GEOLOGIC HAZARDS

5.1. Ground Rupture

The USGS (2016) and Kennedy & Tan (2008) show that there are no mapped Quaternary faults crossing or trending toward the site. The site is not located within a currently established Alquist-Priolo Earthquake Fault Zone (CEG, 2021a). The closest active fault is the Newport-Inglewood-Rose Canyon fault zone located approximately 4.5 miles to the west. The risk associated with ground rupture hazard is low.

There are no active faults, potentially active faults, inactive faults, presumed inactive faults, or activity unknown faults at the site or trending toward the site. The risk associated with ground rupture hazard is low

5.2. Seismicity

Considerations important in seismic design include frequency and duration of motion and soil conditions underlying the site. The seismic design of structures should be evaluated in accordance with the 2022 California Building Code currently adopted by the local agency. The risk associated with strong seismic ground motion hazard is high; however, the risk is no greater than that for the site vicinity.

5.3. Liquefaction

Considering the lack of shallow permanent groundwater and the density and age of the underlying geologic units, the risk associated with seismically induced soil liquefaction hazard is low.

5.4. Landslides

No evidence of landsliding was observed during our investigation. Kennedy & Tan (2007) show that there are no landslides mapped at the site or in anarea that could impact the site. The risk associated with ground movement hazard due to landsliding is low.

5.5. Tsunamis and Seiches

The site is not located within a California Tsunami Hazard Area (CGS, 2021b). There are no lakes or reservoirs located near the site. The risk associated with inundation hazard due to tsunami or seiche is low.

5.6. Flooding

The site is designated as Zone X (FEMA, 2019). The risk associated with flooding hazard is low.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 It is our opinion that the site is suitable for the planned improvements, provided the recommendations presented herein are implemented during the design and construction of the project.

6.1.2 The site is underlain by undocumented fill and Old Paralic Deposits. The undocumented fill is not suitable to support settlement-sensitive improvements and will require remedial grading. The Old Paralic Deposits are suitable to support the compacted fill and settlement-sensitive improvements.

6.1.3 We did not encounter groundwater during our investigation. Groundwater will not be a constraint to project development; however, seepage could be encountered during the grading operations, especially during the rainy seasons.

6.1.4 No significant geologic hazards were observed or are known to exist on the site that would adversely impact the site.

6.1.5 The proposed structure can be supported on conventional shallow foundations bearing entirely on undisturbed old paralic deposits (i.e. terrace deposits).

6.2 Excavation and Soil Characteristics

6.2.1 Excavation of the on-site soils should be possible with moderate effort using conventional heavy-duty equipment.

6.2.2 The soil encountered in the field investigation possess a “very low” to “low” expansion potential (expansion index [EI] ≤ 50) as defined by 2022 California Building Code (CBC) Section 1803.5.3. the following table presents soil classifications based on expansion index. We expect a majority of the on-site soil possess a “very low” to “low” expansion potential (EI ≤ 50).

EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	ASTM D 4829 Expansion Classification	2022 CBC Expansion Classification
0 – 20	Very Low	Non-Expansive
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

6.2.3 We performed laboratory tests during our previous investigation on samples of the site soils to evaluate the percentage of water-soluble sulfate content. Appendix B presents results of the laboratory tests. The test results indicate the on-site soils at the locations tested possess a "S0" sulfate exposure to concrete structures as defined by 2022 CBC Section 1904 and ACI 318-08 Sections 4.2 and 4.3. The following table presents a summary of concrete requirements set forth by 2022 CBC Section 1904 and ACI 318. We recommend ACI guidelines be followed when determining the type of concrete used for the project. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) could affect the concentration.

REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Exposure Class		Water-Soluble Sulfate (SO_4) Percent by Weight	Cement Type (ASTM C 150)	Maximum Water to Cement Ratio by Weight[1]	Minimum Compressive Strength (psi)
S0		$SO_4<0.10$	No Type Restriction	n/a	2,500
S1		$0.10 \le SO_4<0.20$	II	0.50	4,000
S2		$0.20 \le SO_4 \le 2.00$	V	0.45	4,500
S3	Option 1	$SO_4>2.00$	V+Pozzolan or Slag	0.45	4,500
	Option 2		V	0.40	5,000

[1] Maximum water to cement ratio limits do not apply to lightweight concrete

6.2.4 Geocon Incorporated does not practice in the field of corrosion engineering; therefore, further evaluation by a corrosion engineer may be needed if improvements susceptible to corrosion are planned.

6.3 Subdrains

6.3.1 Subdrains will not be required for this project.

6.4 Grading Recommendations

6.4.1 Grading should be performed in accordance with the recommendations provided in this report, the *Recommended Grading Specifications* contained in Appendix C and the local Grading Ordinances. Where the recommendations of this report conflict with Appendix C, the recommendations of this section take precedence.

6.4.2 Earthwork should be observed, and compacted fill tested by Geocon Incorporated.

6.4.3 A preconstruction conference should be held at the site with the City representative, owner, grading and underground contractors, civil engineer, and geotechnical engineer in attendance. Special soil handling and the grading plans can be discussed at that time.

6.4.4 Site preparation should begin with the removal of deleterious material, debris, and vegetation. Material generated during stripping and demolition should be exported from the site and should not be mixed with the fill soil. Existing building foundations and underground improvements within the proposed building areas should be removed.

6.4.5 Abandoned foundations and buried utilities should be removed and the resulting depressions or trenches backfilled with properly compacted soil as part of the remedial grading.

6.4.6 Undocumented fill should be completely removed to expose the underlying Old Paralic Deposits. The removals should extend to the property lines, where practicable. We expect excavations to reach basement level will remove most, if not all of the undocumented fill. Where undocumented fill extends below the basement level, the fill should be removed and recompacted. A representative of Geocon should be on-site during removals to observe the terrace deposits.

6.4.7 Prior to placing fill, the ground surface should be scarified to a depth of at least 12 inches, moisture conditioned, and compacted. Fill soils should then be placed and compacted in layers to the design finish-grade elevation. The fill layers should be no thicker than will allow for adequate bonding and compaction. Fill (including scarified ground surfaces) should be compacted to at least 90 percent of maximum dry density (as determined by ASTM D 1557) at or slightly above optimum moisture content.

6.4.8 The following table provides a summary of the grading recommendations.

SUMMARY OF GRADING RECOMMENDATIONS

Area	Removal Requirements
Site	Complete removal of undocumented fill.
Lateral Grading Limits	To property limits, where practicable.
Compaction Requirements	At least 90 percent relative compaction near or slightly above optimum moisture content.
Exposed Removal Bottoms	Scarify Upper 12 Inches
Expansion Potential	“Very Low” to “Low” (Expansion Index of 50 or less)

6.4.9 Imported soil should have the characteristics presented on the following table. We should be notified of the imported soil source and should be provided with samples in order to perform testing of imported soil prior to its arrival at the site to evaluate its suitability as fill material.

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

6.5 Excavation Slopes and Shoring

6.5.1 The recommendations included herein are provided for stable excavations. Geocon Incorporated is not responsible for site safety and the stability of the proposed excavations. It is the contractor’s responsibility to ensure that all excavations, temporary slopes, and trenches are properly constructed and maintained in accordance with applicable OSHA guidelines in order to maintain the safety and stability of the excavations and adjacent improvements. The walls of the 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 unless the shoring system is designed to accommodate the surcharge loading. The top of sloped excavations should be a minimum of 15 feet from the edge of existing improvements. Vertical excavations should be shored in accordance with applicable OSHA codes and regulations.

6.5.2 The design of temporary shoring is governed by soil and groundwater conditions, and by the depth and width of the excavated area. Continuous support of the excavation face can be provided by a system of soldier piles and wood lagging or other applicable techniques. Excavations exceeding 15 feet may require soil nails, tieback anchors or internal bracing to provide additional wall restraint.

6.5.3 The condition of existing buildings, streets, sidewalks, and other structures/improvements around the perimeter of the planned excavation should be documented prior to the start of shoring and excavation work. Special attention should be given to documenting cracks or other indications of differential settlement within the adjacent structures, pavements, and other improvements. Underground utilities that are sensitive to settlement should be videotaped prior to construction to check the integrity of pipes. Monitoring points should be established on the shoring wall and adjacent building. The points should be monitored weekly during the excavation work and on a monthly thereafter until the podium floor is finished. Inclinometers should be installed and monitored behind any shoring sections that will be advanced deeper than 30 feet below the existing ground surface.

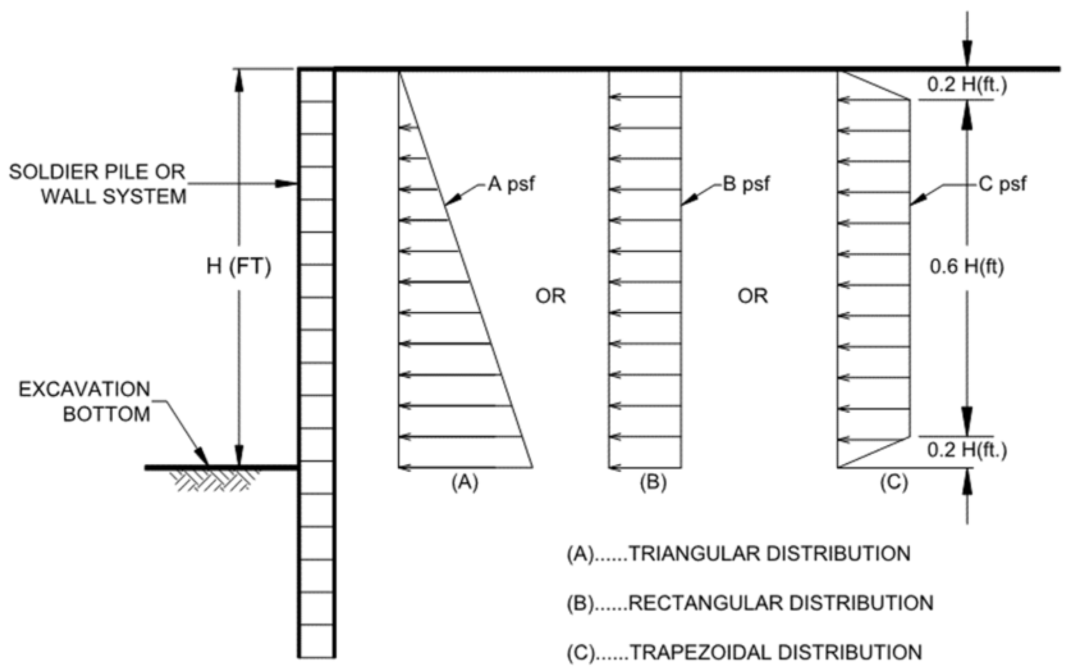
6.5.4 In general, ground conditions are suited for soldier-pile-and-tieback-anchor walls. However, gravel, cobble, and oversized material could be encountered that might be difficult to drill. Additionally, if cohesionless sands are encountered, some raveling could occur along the unsupported portions of excavations.

6.5.5 Temporary shoring should be designed in conformance with FHWA-IF-99-015 using a lateral pressure envelope acting on the back of the shoring as presented in the following table assuming a level backfill. The distributions are shown on the Active Pressures for Temporary Shoring. Cantilevered shoring should use the triangular distribution and multi-braced systems (such as tieback anchors and rakers) should use the trapezoidal or rectangular distributions. The project shoring engineer should determine the applicable soil pressure distribution for the design of the temporary shoring system. Additional lateral earth pressure due to the surcharging effects from construction equipment, sloping backfill, stockpiles, adjacent structures, and traffic loads should be considered during design of the shoring system.

SUMMARY OF TEMPORARY SHORING WALL RECOMMENDATIONS

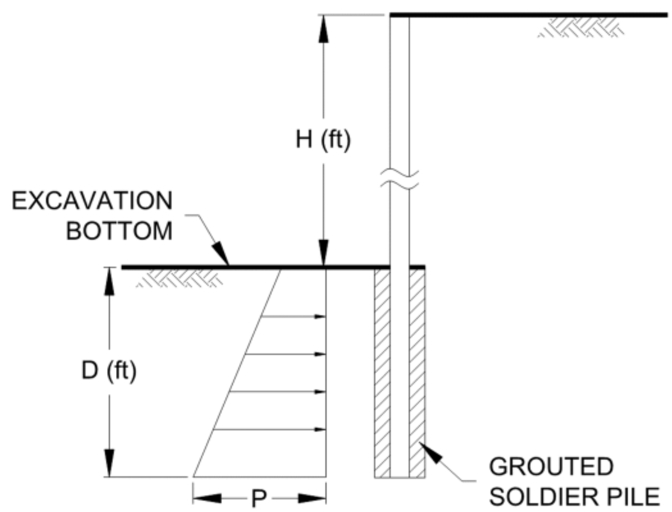
Parameter	Value
Triangular Distribution, A	25H psf
Rectangular Distribution, B	16H psf
Trapezoidal Distribution, C	20H psf
Passive Pressure, P	350D + 500 psf
Effective Zone Angle, E	30 degrees
Maximum Design Lateral Movement	1 Inch
Maximum Design Vertical Movement	½ Inch
Maximum Design Retained Height, H	15 Feet

H equals the height of the retaining portion of the wall in feet
D equals the embedment depth of the retaining wall in feet



Active Pressures on Temporary Shoring

6.5.6 The passive resistance can be assumed to act over a width of three pile diameters. Typically, soldier piles are embedded a minimum of 0.5 times the maximum height of the excavation (this depth is to include footing excavations) if tieback anchors are not employed. The project structural engineer should determine the actual embedment depth.



Passive Pressures on Temporary Shoring

6.5.7 We should observe the drilled shafts for the soldier piles prior to the placement of the pile to check that the exposed soil conditions are similar to those expected and that excavation extends to an appropriate bearing strata and design depth.

6.5.8 It is essential that the shoring system be designed to of the amount of lateral wall displacement. Earth pressures acting on a lagging wall can cause movement of the shoring toward the excavation and result in ground subsidence outside of the excavation.

6.5.9 Survey points should be established at the top of the pile on at least 20 percent of the soldier piles. An additional point located at an intermediate point between the top of the pile and the base of the excavation should be monitored on at least 20 percent of the piles if tieback anchors will be used. These points should be monitored on a weekly basis during excavation work and on a monthly basis thereafter until the permanent support system is constructed.

6.5.10 The project civil engineer should provide the approximate location, depth, and pipe type of the underground utilities to the shoring engineer to help select the shoring type and shoring design. The shoring system should be designed to limit horizontal soldier pile movement to a maximum of 1 inch. The amount of horizontal deflection can be assumed to be essentially zero along the Active Zone and Effective Zone boundary. The magnitude of movement for intermediate depths and distances from the shoring wall can be linearly interpolated.

6.5.11 Lagging should keep pace with excavation. The excavation should not be advanced deeper than 5 feet below the bottom of lagging at any time or as determined by the shoring design engineer. Unlagged portions should only be allowed to stand for short periods of time in order to decrease the risk of soil instability and should never be left unsupported overnight. Proper backfilling should be conducted when necessary between the back of lagging and excavation sidewalls to reduce sloughing in this zone and all voids should be filled by the end of each day. It may be necessary to backfill with slurry to help prevent future lateral movement behind the supported excavation. Further, the excavation should not be advanced further than four feet below a row of tiebacks prior to those tiebacks being proof tested and locked off unless otherwise specific by the shoring engineer. Surface sloughing can occur during the excavation process.

6.6 Seismic Design Criteria – 2022 California Building Code

6.6.1 The following table summarizes site-specific design criteria obtained from the 2022 California Building Code (CBC; Based on the 2021 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. We used SEAOC (2020) 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 2022 CBC and Table 20.3-1 of ASCE 7-16. We assigned the site class based on exploratory borings performed on the property and the results of a ReMi line performed for the adjacent property to the west (CTE 2023). The values presented herein are for the risk-targeted maximum considered earthquake (MCE_R).

2022 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2022 CBC Reference
Site Class	C	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.052g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.384g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.2	Table 1613.2.3(1)
Site Coefficient, F_V	1.5	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.262g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.576	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.841g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.384g	Section 1613.2.4 (Eqn 16-39)

*See following paragraph.

6.6.2 The following table 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.

ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Site Class	C	Section 1613.2.2 (2022 CBC)
Mapped MCE_G Peak Ground Acceleration, PGA	0.462g	Figure 22-7
Site Coefficient, F_{PGA}	1.2	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.554g	Section 11.8.3 (Eqn 11.8-1)

6.6.3 Conformance to the seismic design criteria presented in the above tables does not constitute a 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.

6.6.4 The project structural engineer or architect should determine 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. The following table presents a summary of the risk categories in accordance with ASCE 7-16.

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 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

6.7 Shallow Foundations

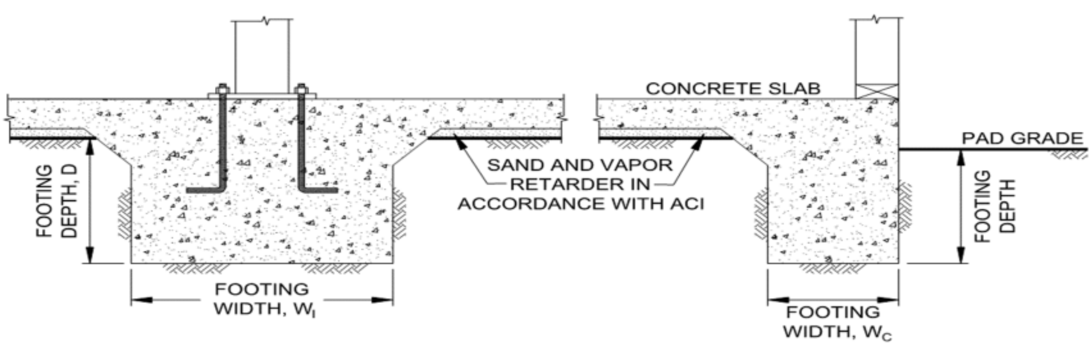
6.7.1 The proposed structure can be supported on shallow foundations bearing on undisturbed Old Paralic Deposits. Where fill is encountered below building footings, the footings should be deepened to extend through the fill to bear on the underlying Old Paralic Deposits. As an alternative to deepening footings, fill beneath the footing can be removed and the resulting excavation filled with a minimum 2-sack cement-slurry back to footing bottom.

6.7.2 Foundations should consist of continuous strip footings and isolated spread footings. The following table provides a summary of the foundation design recommendations. The allowable bearing pressure below for the basement walls assumes footings are at least 10 feet below finish grade.

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	18 Inches Below Lowest Adjacent Grade
Minimum Concrete Reinforcement	(4) No. 5 Bars, 2 at the Top and 2 at the Bottom
Allowable Bearing Capacity for the Building (Footings Bering on Old Paralic Deposits)	6,000 psf
Allowable Bearing Pressure (Ancillary Structures bearing on Compacted Fill)	2,500 psf
Bearing Capacity Increase for Ancillary Structures	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Allowable Bearing Pressure (Ancillary Structures)	4,000 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 40 Feet
Footing Size Used for Settlement	8-Foot Square
Design Expansion Index	50 or less

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



Wall/Column Footing Dimension Detail

6.7.4 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.

6.7.5 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.

6.7.6 We should be consulted to provide additional design parameters as required by the structural engineer.

6.8 Interior Concrete Slabs on Grade

6.8.1 Interior concrete slabs on grade for the structure should be constructed in accordance with the following table.

MINIMUM INTERIOR CONCRETE SLAB-ON-GRADE RECOMMENDATIONS

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

6.8.2 A vapor retarder should underlie slabs that receive moisture-sensitive floor coverings or could be used to store moisture-sensitive materials. 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). The vapor retarder should be installed in a manner that prevents puncture in accordance with manufacturer's requirements and ASTM recommendations. The project architect or developer should specify the vapor retarder used based on the type of floor covering that will be installed and if the structure will possess a humidity-controlled environment.

6.8.3 The project foundation engineer, architect, or developer should determine the bedding sand thickness. It is common to have 3 to 4 inches of sand in the southern California region. We should be contacted to provide recommendations if the bedding sand is thicker than 6 inches.

6.8.4 The foundation-design engineer should provide appropriate concrete-mix-design criteria and curing measures to assure proper curing of the slab including reducing potential rapid moisture loss which could result in shrinkage cracking and slab curl.

6.8.5 The foundation-design engineer should present the concrete-mix design and proper concrete-curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plans.

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

6.8.7 Subgrade presaturation prior to placing concrete is not deemed necessary; however, the subgrade soil should be moisturized to maintain a moist condition as would be expected in any such concrete placement.

6.8.8 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.

6.8.9 The recommendations of this report are intended to reduce potential cracking of slabs due to expansive soil, differential settlement of existing soil or soil with varying thicknesses. Even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade placed on such conditions may still exhibit 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 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.

6.9 Exterior Concrete Flatwork

6.9.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations presented in the following table. The recommended reinforcing steel could help reduce potential cracking.

MINIMUM CONCRETE FLATWORK RECOMMENDATIONS

Expansion Index, EI	Minimum Concrete Reinforcement* Options	Minimum Thickness
EI ≤ 50	6x6-W2.9/W2.9 (6x6-6/6) welded wire mesh	4 Inches
	No. 3 Bars 24 inches on center, Both Directions	

*In excess of 8 feet square.

6.9.2 If subgrade soils have an expansion index greater than 20, the City of Oceanside requires a minimum of 6 inches of Class 2 base beneath curbs, gutters, and sidewalks. Although the expansion index performed during our previous geotechnical was less than 20, additional expansion index tests should be performed on subgrade soils to evaluate if base is required beneath curbs, gutters, and sidewalks.

6.9.3 The subgrade soil should be properly moisturized and compacted prior to the placement of reinforcing 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.

6.9.4 Even with the incorporation of the recommendations presented in this report, the exterior concrete flatwork can experience differential movement. Flatwork should be structurally connected to the curbs, where practical, to reduce potential offset between the curb and the flatwork.

6.9.5 Concrete flatwork should be provided with crack-control joints. The project structural engineer should determine the spacing of the crack-control joints based on the slab thickness and the intended use. American Concrete Institute (ACI) criteria should be used when establishing crack control spacing.

6.9.6 Subgrade soil for exterior slabs that are not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior section of this report.

Subgrade soil should be properly compacted, and the moisture content of subgrade soil should be verified prior to placing concrete.

6.9.7 Where exterior flatwork abuts the structure at entrance or exit points, the exterior slab should be dowelled into the foundation stem wall. This recommendation is intended to reduce potential differential elevations that could result from differential settlement or minor heave of the flatwork. The project structural engineer should provide dowelling details.

6.9.8 The recommendations presented herein are intended to reduce potential cracking of exterior slabs resulting from differential movement; however, even with the incorporation of the recommendations presented herein, slabs-on-grade could still crack. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics. Their occurrence may be 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. The Portland Concrete Association (PCA) and American Concrete Institute (ACI) provide recommendations for proper concrete mix, construction, and curing practices, and should be incorporated into project construction.

6.10 Retaining Walls

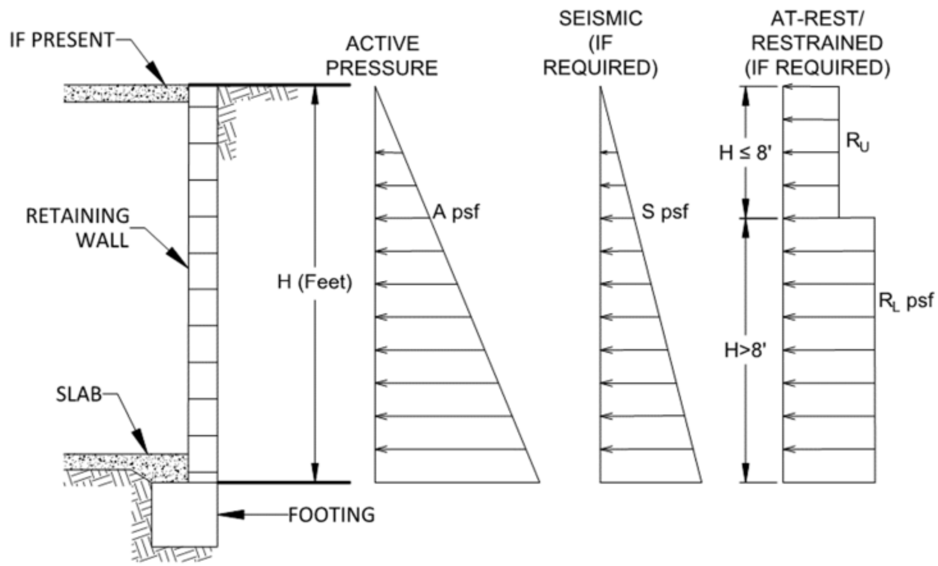
6.10.1 Walls that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall) at the top of the wall should be designed using the values presented in the following table. Soil with an expansion index (EI) greater than 50 should not be used as backfill material behind retaining walls.

RETAINING WALL DESIGN RECOMMENDATIONS

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

H equals the height of the retaining portion of the wall in feet.

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



Retaining Wall Loading Diagram

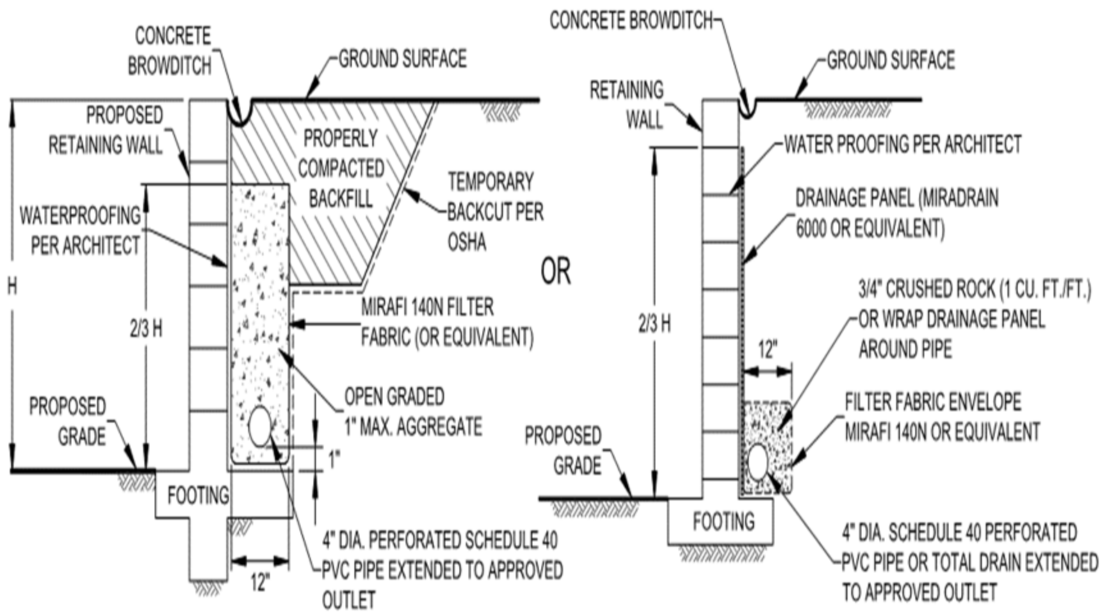
6.10.3 Where walls are restrained from movement at the top (at-rest condition), an additional uniform pressure should be applied to the wall.

6.10.4 The structural engineer should determine the Seismic Design Category for the project in accordance with Section 1613.3.5 of the 2022 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 2022 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.

6.10.5 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.

6.10.6 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.

6.10.7 The recommendations herein assume a properly compacted, granular (EI of 50 or less), free-draining backfill soil and that 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 should be contacted for additional recommendations.



Typical Retaining Wall Drainage Detail

6.10.8 In general, wall foundations should be designed in accordance with the following table. 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.

SUMMARY OF RETAINING WALL FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Retaining Wall Foundation Width	12 inches
Minimum Retaining Wall Foundation Depth	12 Inches
Minimum Concrete Reinforcement	Per Structural Engineer
Allowable Bearing Capacity	2,500 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Allowable Bearing Capacity	4,000 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 40 Feet

6.10.9 The recommendations presented herein are applicable for the design of rigid concrete or masonry retaining walls. Should other types of walls (such as mechanically stabilized earth [MSE] walls) be planned, Geocon should be consulted for additional recommendations.

6.10.10 Soil contemplated for use as retaining wall backfill, including imported soils, should be identified in the field prior to backfill. At that time, Geocon should be provided 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.

6.10.11 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 should be consulted to assess the suitability of the on-site soil for use as wall backfill if standard wall designs are used.

6.11 Lateral Loading

6.11.1 The following table provides design parameters to resist lateral loads and 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 soil in areas not covered by floor slabs or pavement should not be included in the determination of passive resistance.

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.

6.11.2 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.

6.12 Preliminary Pavement Recommendations

6.12.1 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 30 and 78 for the subgrade soil and base materials, respectively, for the purposes of this preliminary analysis. The following table presents the preliminary flexible pavement sections.

PRELIMINARY FLEXIBLE PAVEMENT SECTION

Location	Assumed Traffic Index	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Parking Stalls for Automobiles and Light-Duty Vehicles	5.0	3	5.5
Driveways for Automobiles and Light-Duty Vehicles	5.5	3	7
Medium Truck Traffic Areas	6.0	3.5	7.5
Driveways for Heavy Truck Traffic	7.0	4	9.5

6.12.2 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.

6.12.3 Base materials should conform to Section 26-1.02B of the *Standard Specifications for The State of California Department of Transportation (Caltrans)* with a ¾-inch maximum size aggregate. Asphalt concrete should conform to Section 203-6 of the *Standard Specifications for Public Works Construction (Greenbook)*.

6.12.4 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 330-21 *Commercial Concrete Parking Lots and Site Paving Design and Construction – Guide.* The following table provides the traffic categories and design parameters used for the calculations for 20-year design life.

TRAFFIC CATEGORIES

Traffic Category	Description	Reliability (%)	Slabs Cracked at End of Design Life (%)
A	Car Parking Areas and Access Lanes	60	15
E	Heavy Truck Traffic	75	15

6.12.5 We used the parameters presented in the following table to calculate the pavement design sections. We should be contacted to provide updated design sections, if necessary.

RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of Subgrade Reaction, k	150 pci
Modulus of Rupture for Concrete, M_R	500 psi
Concrete Compressive Strength	3,000 psi
Concrete Modulus of Elasticity, E	3,150,000 psi

6.12.6 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented on the following table.

RIGID PAVEMENT DESIGN THICKNESS

Traffic Category	Trucks Per Day	Portland Cement Concrete, T (Inches)
A = Car Parking Areas and Access Lanes	1	5
Heavy Truck Traffic	<10	6.5

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

6.12.8 The recommendations of the referenced ACI guidelines should be followed for joint and joint spacings.

6.12.9 Reinforcing steel will not be necessary within the concrete for geotechnical purposes.

6.12.10 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement. The location, spacing and depth of the crack-control joints should be in accordance with ACI guidelines.

6.12.11 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. Subgrade for cross-gutters 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. 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.

6.12.12 Subgrade for cross-gutters 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.

6.12.13 Base materials should not be placed below the curb/gutter, or cross-gutters to mitigate water infiltration from the adjacent parkways into the pavement sections. However, the City of Oceanside may require it where subgrade soils have an expansion index greater than 20.

6.12.14 Where flatwork is located adjacent to the curb/gutter, the concrete flatwork should be structurally connected to the curbs to reduce potential offsets between the curbs and the flatwork.

6.12.15 The performance of pavement is highly dependent on providing positive surface drainage away from the edge of the pavement. Ponding of water on or adjacent to the pavement will likely result in pavement distress and subgrade failure. Drainage from landscaped areas should be directed to controlled drainage structures. Landscape areas adjacent to the edge of asphalt pavements are not recommended due to the potential for surface or irrigation water to infiltrate the underlying permeable aggregate base and cause distress. Where such a condition cannot be avoided, consideration should be given to incorporating measures that will significantly reduce the potential for subsurface water migration into the aggregate base. If planter islands are planned, the perimeter curb should extend at least 6 inches below the level of the base materials.

6.13 Site Drainage and Moisture Protection

6.13.1 Adequate site drainage is critical to reduce the risk 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 2022 CBC 1804.4. 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.

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

6.13.3 Underground utilities should be leak free. Utility and irrigation lines should be checked at appropriate intervals for leaks. Detected leaks should be promptly repaired. Detrimental soil movement could occur if water is allowed to infiltrate the soil for prolonged periods of time.

6.13.4 Landscaping planters adjacent to paved areas are not recommended due to the risk for surface or irrigation water to infiltrate the pavement subgrade and base course. Area drains to collect excess irrigation water and transmit it to drainage structures, or impervious above-grade planter boxes should be used. 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 where landscaping is planned next to pavement.

6.14 Grading and Foundation Plan Review

6.14.1 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.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

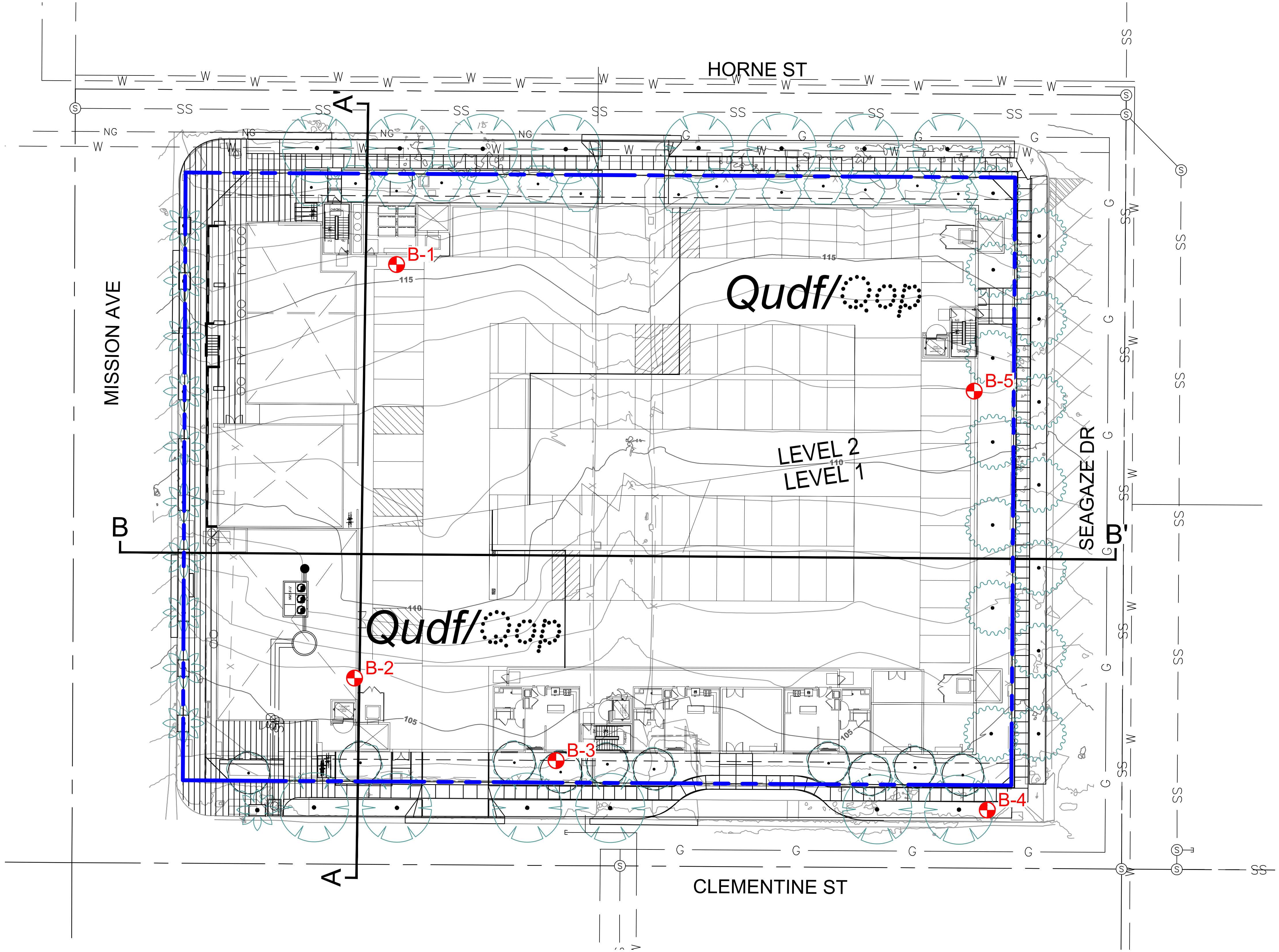
1. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development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

901 MISSION AVENUE
SAN DIEGO, CALIFORNIA



0' 20' 40' 60' 80'
SCALE 1"=40'(on 11x17)

GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL

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

B-5 ....APPROX. LOCATION OF BORING

A — A' ....APPROX. LOCATION OF GEOLOGIC CROSS SECTION

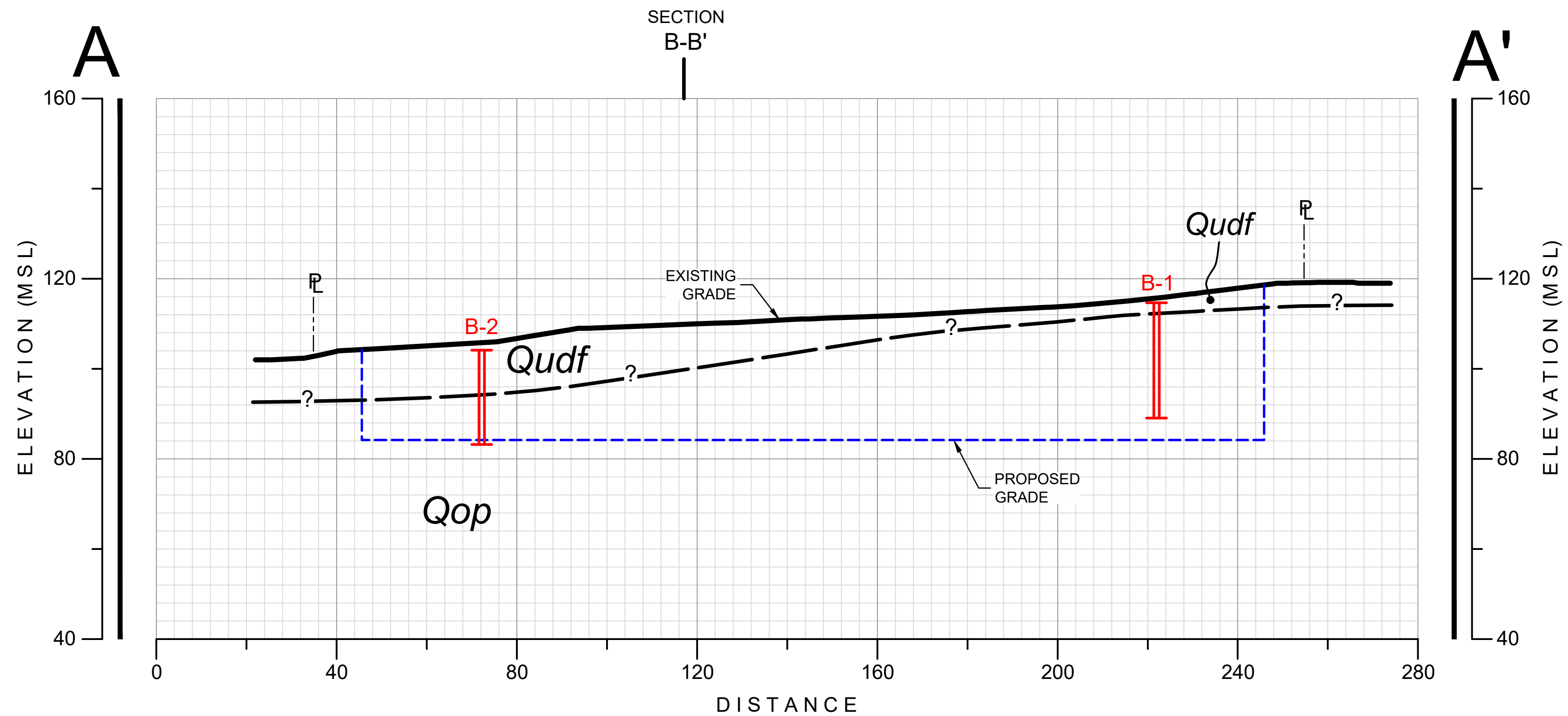
GEOCON INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

GEOLOGIC MAP

PROJECT NO. G3303 - 42 - 01
FIGURE 1
DATE 05 - 15 - 2024

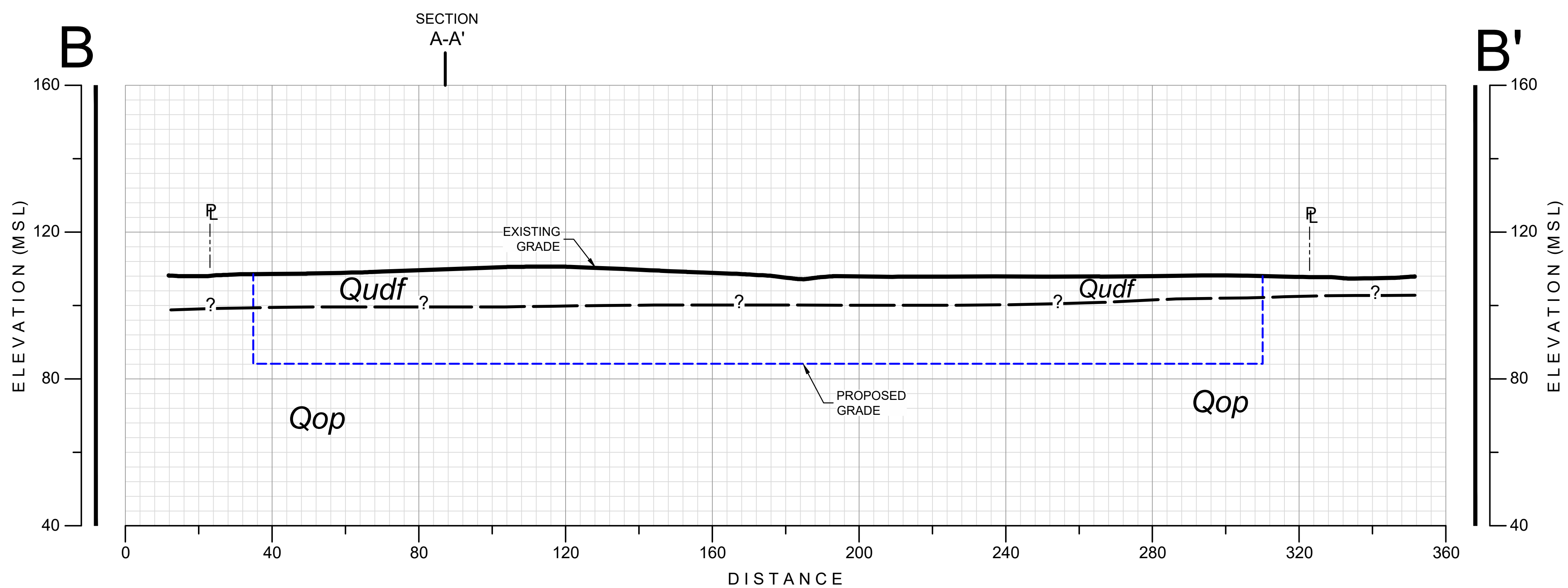
Plotted:05/15/2024 7:52AM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G3303-42-01 (901 Mission Avenue)\SHEETS\G3303-42-01 Geo Map.dwg

901 MISSION AVENUE
SAN DIEGO, CALIFORNIA



GEOLOGIC CROSS-SECTION A-A'

SCALE: 1" = 40' (Vert. = Horiz.)



GEOLOGIC CROSS-SECTION B-B'

SCALE: 1" = 40' (Vert. = Horiz.)

GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL

Qop ....OLD PARALIC DEPOSITS

B-2 ....APPROX. LOCATION OF BORING

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

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

GEOLOGIC CROSS SECTION

PROJECT NO. G3303 - 42 - 01
FIGURE 2
DATE 05 - 15 - 2024

Plotted:05/15/2024 7:44AM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G3303-42-01 (901 Mission Avenue)\SHEETS\G3303-42-01 XSections.dwg

APPENDIX A

APPENDIX A
FIELD INVESTIGATION

Our previous field investigation was performed on December 30, 2003, and consisted of a site reconnaissance and drilling five exploratory small-diameter borings at the approximate locations shown on Figure 1. The small-diameter borings were drilled to depths varying from 10½ to 25 feet using a CME drill rig equipped with hollow-stem auger. Relatively undisturbed samples were obtained by driving a California Sampler 12 inches with blows from a 140-pound hammer falling 30 inches. This split-tube sampler was equipped with 1-inch-high by $2^{3}/_{8}$-inch-diameter brass sampler rings to facilitate sample removal and testing. Disturbed bulk samples were obtained from the boring's cuttings.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the American Society for Testing and Materials (ASTM) Practice for Description and Identification of Soils (Visual-Manual Procedure D 2488). The logs of the exploratory borings are presented on Figures A-1 through A-5. The logs depict the various soil types encountered and indicate the depths at which samples were obtained.

PROJECT NO. 07213-22-01

BORING B 1

ELEV. (MSL.) **114'** DATE COMPLETED **12-30-2003**

EQUIPMENT **CME 75**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B1-1			SM	**UNDOCUMENTED FILL** Loose, damp, moderate yellowish brown to moderate brown Silty SAND. Scattered gravel			
2	B1-2					88		
4					**TERRACE DEPOSIT** Dense, damp to moist, moderate brown Silty SAND, Slightly porous. Very dense below 5 feet, non-porous			
6	B1-3					63		2.0
8								
10	B1-4			SM		68/6	96.5	5.9
12								
14								
16	B1-5				-Mottled moderate brown and light brown	50/4		
18					-Becomes pale yellow brown below 15 feet			
20					-Abundant cobbles below 20 feet -2 hours to advance 5 feet			
22								
24								

BORING TERMINATED AT 25 FEET (REFUSAL)
Backfilled with approximately 7 cubic feet of bentonite chips

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

07213-22-01.GPJ

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

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

PROJECT NO. 07213-22-01

BORING B 2

ELEV. (MSL.) **103'** DATE COMPLETED **12-30-2003**

EQUIPMENT **CME 75**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B2-1				**UNDOCUMENTED FILL** Loose to medium dense, moist, moderate brown Silty SAND; scattered gravel			
2	B2-2					92	3.9	112.1
4								
6	B2-3			SM		52		
8								
10	B2-4				**TERRACE DEPOSITS** Very dense, moist, moderate brown, Silty SAND, scattered cobbles	90	126.7	10.4
12								
14				SM				
16	B2-5					88	109.1	7.3
					-Very dense, moist, moderate brown, poorly graded SAND with silt			
18								
	B2-6				-With silt and gravel; (top of cobble layer?)	50/4		

BORING TERMINATED AT 19½ FEET

Backfilled with soil cuttings mixed with 1x94 lb sack of cement

Figure A-2,
Log of Boring B 2, Page 1 of 1

07213-22-01.GPJ

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

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

PROJECT NO. 07213-22-01

BORING B 3

ELEV. (MSL.) 101' DATE COMPLETED 12-30-2003

EQUIPMENT CME 75

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B3-1			SM	**UNDOCUMENTED FILL** Loose, moist, moderate brown, Silty SAND			
2	B3-2			SM	**TERRACE DEPOSITS** Dense, moist, moderate brown Silty SAND	58	116.2	6.7
4								
6	B3-3					55	120.9	9.5
8								
10	B3-4				-Very dense, moist, moderate brown, poorly graded SAND with Silt	50/6		

BORING TERMINATED AT 10½ FEET
Backfilled with soil cuttings mixed with 1x94 lb sack of cement

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

07213-22-01.GPJ

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

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

BORING B 4

ELEV. (MSL.) **100'** DATE COMPLETED **12-30-2003**

EQUIPMENT **CME 75**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B4-1				**UNDOCUMENTED FILL** Medium dense to loose, moist to wet, moderate brown Silty SAND			
2	B4-2			SM		28	120.4	10.7
4								
6	B4-3			SM	**TERRACE DEPOSITS** -Medium dense, moist, moderate brown, Silty SAND	24		
8								
10	B4-4			SP-SM	Dense, moist, moderate brown poorly graded SAND with Silt	38	107.8	7.4

BORING TERMINATED AT 11 FEET
Backfilled with soil cuttings mixed with ½x94 lb sack of cement

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

07213-22-01.GPJ

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

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

PROJECT NO. 07213-22-01

BORING B 5

ELEV. (MSL.) **109'** DATE COMPLETED **12-30-2003**

EQUIPMENT **CME 75**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B5-1			SM	**UNDOCUMENTED FILL** Loose, moist, dark yellowish brown and light brown Silty SAND; scattered gravel			
2	B5-2					50/6		
				SM	**TERRACE DEPOSITS** Very dense, moist, moderate brown Silty SAND			
4								
	B5-3					50/4	109.6	6.8
6								
8								
				SP-SM	Very dense, moist, grayish orange and dark yellowish orange SAND with Silt			
10	B5-4					98	106.5	3.6
12								
				SC	Very dense, moist, moderate to light brown Clayey SAND			
14								
	B5-5					50/6	109.8	7.4
16								
18								
	B5-6					78		
20					BORING TERMINATED AT 20 FEET Backfilled with soil cuttings mixed with 1x94 lb sack of cement			

Figure A-5,
Log of Boring B 5, Page 1 of 1

07213-22-01.GPJ

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

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

APPENDIX B

APPENDIX B
LABORATORY TESTING

Presented below are the results of laboratory testing performed as part of Geocon's 2004 geotechnical investigation. Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Samples were subjected to drained direct shear testing, expansion index testing, R-value testing, and laboratory maximum dry density and optimum moisture content tests. One sample was tested for its corrosivity characteristics. Results of all laboratory tests are presented on the following tables. In situ moisture and dry density tests are presented on the boring logs (Appendix A).

SUMMARY OF LABORATORY DIRECT SHEAR TEST RESULTS
ASTM D 3080

Sample No.	Dry Density (pcf)	Moisture Content (%)	Unit Cohesion (psf)	Angle of Shear Resistance (degrees)
B1-1 and B5-1 combined*	119.0	9.4	344	38
B1-5	113.3	6.6	197	40

* Sample remolded to 90 percent relative compaction at approximately optimum moisture content.

SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829

Sample No.	Moisture Content		Dry Density (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
B1-1 and B5-1 combined	7.8	12.9	118.7	0

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

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
B1-1 and B5-1 combined	Brown Silty SAND	134.2	7.8

SUMMARY OF LABORATORY RESISTANCE VALUE TEST RESULTS
ASTM D 2844

Sample No.	Description	R-Value
B1-1 and B5-1 combined	Brown Silty SAND	64

SUMMARY OF LABORATORY PH AND RESISTIVITY TEST RESULTS
CALIFORNIA TEST METHOD NO. 643

Sample No.	pH	MINIMUM RESISTIVITY (OHM-CENTIMETERS)
B1-1 and B5-1 combined	6.8	175.8

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

Sample No.	WATER-SOLUBLE SULFATE (%)
B1-1 and B5-1 combined	0.015

APPENDIX C

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

901 MISSION AVENUE MIXED-USE
OCEANSIDE, CALIFORNIA

PROJECT NO. G3303-42-01

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

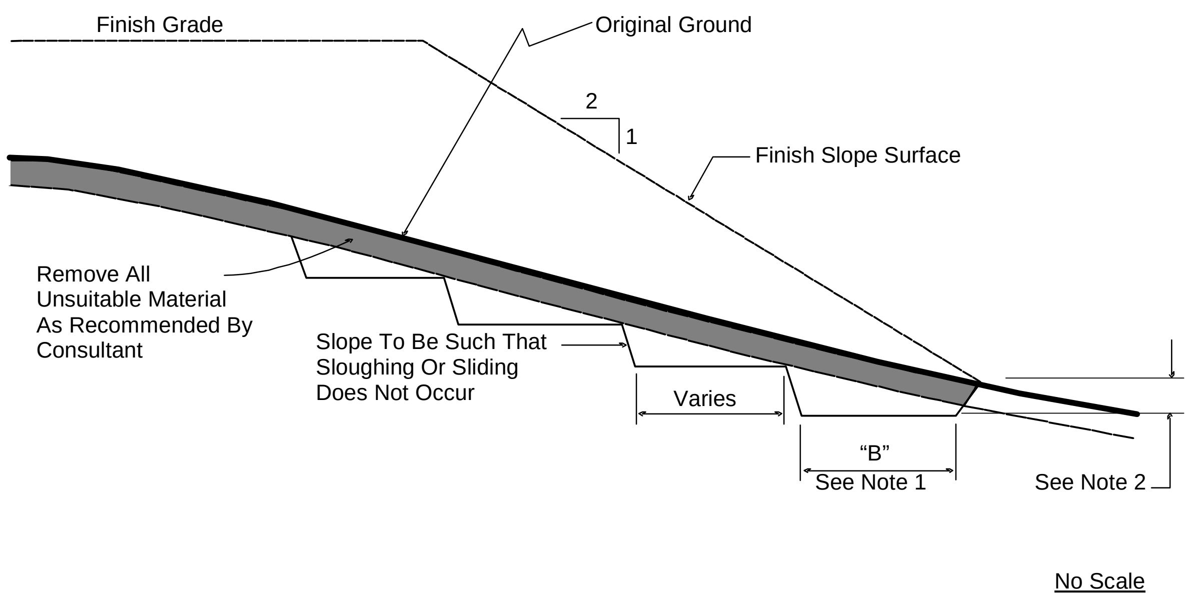
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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



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.

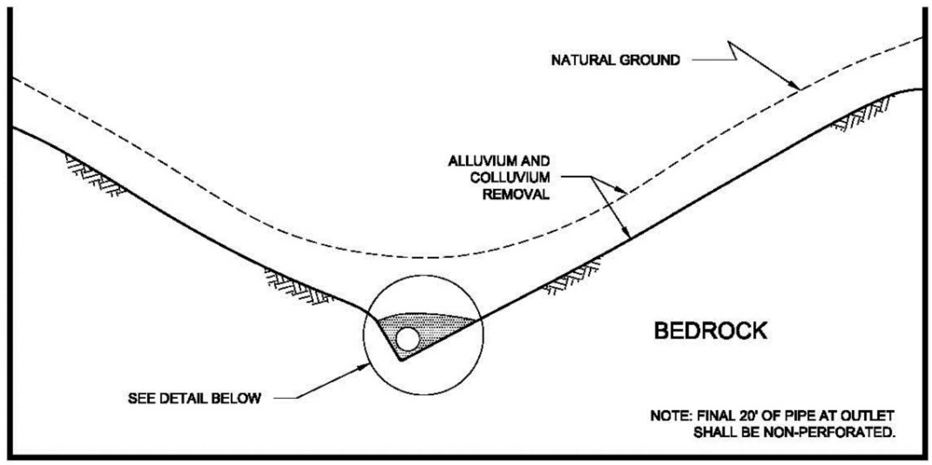
6.3.6 To reduce the potential for “piping” of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

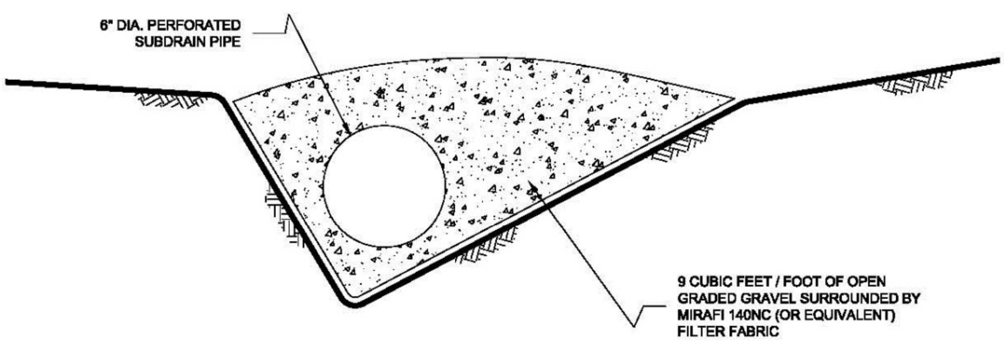
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

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:

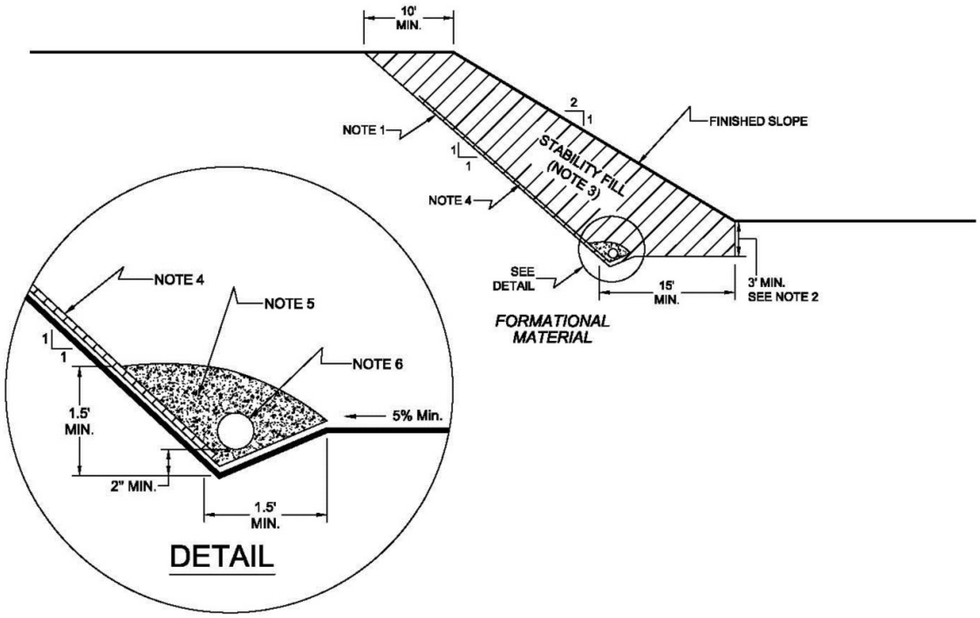
1......8-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

NO SCALE

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

TYPICAL STABILITY FILL DETAIL



NOTES:

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

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

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

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

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

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

NO SCALE

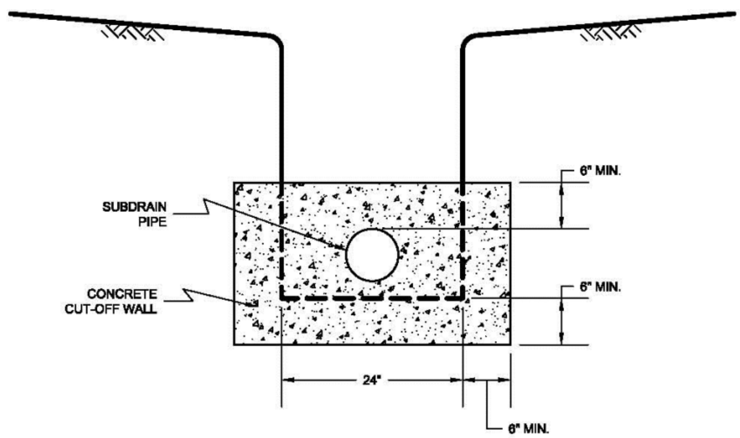
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

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

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

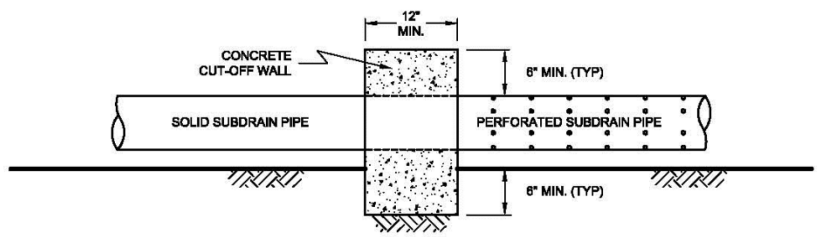
TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



NO SCALE

SIDE VIEW

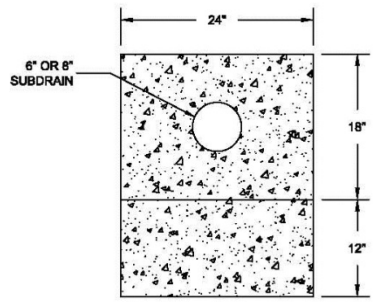


NO SCALE

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

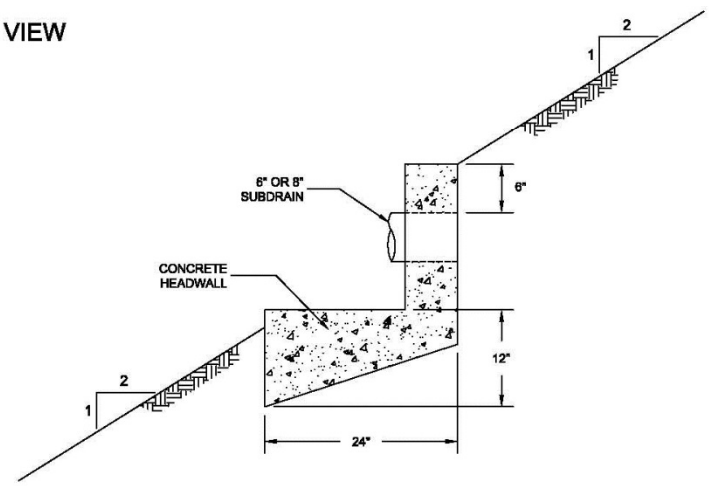
TYPICAL HEADWALL DETAIL

FRONT VIEW



NO SCALE

SIDE VIEW



NOTE: HEADWALL SHOULD OUTLET AT TOE OF FILL SLOPE
OR INTO CONTROLLED SURFACE DRAINAGE

NO SCALE

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

8. OBSERVATION AND TESTING

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

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

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

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

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

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

8.6.1 Soil and Soil-Rock Fills:

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

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

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

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

9. PROTECTION OF WORK

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

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

10. CERTIFICATIONS AND FINAL REPORTS

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

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

LIST OF REFERENCES

CGS (2021a), *EQ Zapp: California Earthquake Hazards Zone Application*, online map that queries California Geological Survey mapped earthquake hazard zones, https://www.conservation.ca.gov/cgs/geohazards/eq-zapp, accessed April 2, 2024;

CGS (2021b), *Tsunami Inundation Maps and Data,* web application for accessing tsunami inundation hazard, https://www.conservation.ca.gov/cgs/tsunami/maps, accessed April 2, 2024;

CTE (2023), *Geotechnical Investigation, Proposed 801 – 815 Mission Avenue Mult-Family Residence, 801 – 815 Mission Avenue, Oceanside, California,* CTE Job No. 4830.2200116, dated August 15, 2023;

FEMA (2019), *Flood Map Service Center*, FEMA website, https://msc.fema.gov/portal/home, flood map number 06073C1613H, effective December 20, 2019, accessed May 4, 2024;

Geocon Incorporated (2004), *Geotechnical Investigation, Mission Avenue Mixed-use Project, Mission Avenue at Horne Street, Oceanside, California*, Project No. 07213-22-01, dated January 9, 2004;

Kennedy, M. P., and Tan, S. S., (2007), *Geologic Map of the Oceanside 30' x 60' Quadrangle, California*, USGS Regional Geologic Map Series, 1:100,000 Scale, Map No. 3;

SEAOCC (2020), *Seismic Design Maps*, website interface that queries the U.S. Geological Survey (USGS) web servers and retrieves the seismic design variables using ASCE 7-16, ASCE 7-10, ASCE 41-13, ASCE 41-17, IBC 2015, IBC 2012, NEHRP-2015, and NEHRP 2009 seismic design map data, http://seismicmaps.org, accessed April 2, 2024;

USGS (2019), *Quaternary Fault and Fold Database of the United States*: U.S. Geological Survey website, https://www.usgs.gov/natural-hazards/earthquake-hazards/faults, accessed April 2, 2024;