GEOTECHNICAL INVESTIGATION

WAREHOUSE DEVELOPMENT
NORTHWEST CORNER OF
HARVILL AND PLACENTIA AVENUES
MEAD VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

ORBIS REAL ESTATE PARTNERS, LLC
NEWPORT BEACH, CALIFORNIA

NOVEMBER 27, 2019
PROJECT NO. T2891-22-01

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. T2891-22-01
November 27, 2019

Orbis Real Estate Partners, LLC
280 Newport Center Drive, Suite 240
Newport Beach, California 92660

Attention: Mr. Raymond Polverini

Subject: GEOTECHNICAL INVESTIGATION
WAREHOUSE DEVELOPMENT
NORTHWEST CORNER OF HARVILL AND PLACENTIA AVENUES
MEAD VALLEY AREA, RIVERSIDE COUNTY CALIFORNIA

Dear Mr. Polverini:

In accordance with your authorization of Proposal No. IE-2408, Geocon West Inc. (Geocon) herein submits the results of our geotechnical investigation for the subject site. The accompanying report presents the results of our study and conclusions and recommendations pertaining to the geotechnical aspects of the proposed warehouse project. The site is considered suitable for development provided the recommendations of this report are followed.

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

Very truly yours,

GEOCON WEST, INC.

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for the planned warehouse development located northwest of the intersection of Harvill and Placentia Avenues in the Mead Valley area of Riverside County, California (see *Vicinity Map*, Figure 1). The purpose of the geotechnical investigation is to evaluate the surface and subsurface soil conditions and general site geology, and to identify geotechnical constraints that may affect development of the property including faulting, liquefaction and seismic shaking based on the 2016 CBC seismic design criteria. In addition, we provided recommendations for remedial grading, shallow foundations, concrete slab-on-grade, concrete flatwork, preliminary pavement, lateral loading and retaining walls. This investigation also included a review of readily available published and unpublished geologic literature (see *List of References*).

The scope of this investigation included performing a site reconnaissance, field exploration, engineering analyses, and preparing this report. We performed our field investigation on October 10, 2019 by drilling seven small-diameter borings to a maximum depth of approximately 50 feet below the existing ground surface. The *Geologic Map*, Figure 2, presents the approximate locations of the borings. *Appendix A* provides a detailed discussion of the field investigation including logs of the borings. Details of the laboratory tests and a summary of the test results are presented in *Appendix B* and on the boring logs in *Appendix A*.

Recommendations presented herein are based on analyses of data obtained from our site investigation and our understanding of proposed site development. If project details vary significantly from those described herein, Geocon should be contacted to evaluate the necessity for review and possible revision of this report.

2. SITE AND PROJECT DESCRIPTION

The subject site is located at the northwest corner of Harvill and Placentia Avenues, in the Mead Valley area of Riverside County, California. The property is bounded on the east and south by Harvill and Placentia Avenues, respectively, on the north by an existing warehouse, and on the west by vacant land. The majority of the site is currently vacant and appears to be periodically disked for weed abatement. Four residences are present along the western third of the site, west of Sharon Ann Lane, a dirt road that traverses the property in a north-south direction. Based on a review of historical aerial photographs, the houses appear to have begun construction in 1978. Overhead powerlines run along Placentia Avenue. An abandoned power pole was observed in the south-central portion of the property. Access to the property is from Sharon Ann Lane, Placentia and Harvill Avenues. The existing grades range from approximate elevation 1,509 feet above Mean Sea Level (MSL) in the southeast corner to 1,536 feet above MSL in the southwest corner. The property is at latitude 33.824116 and longitude -117.247276.

Based on the *Conceptual Site Plan* prepared by HPA Architecture, we understand that the proposed construction consists of a 274,190-square-foot warehouse, with associated parking and infrastructure. A grading plan was not available at the time of our report, but maximum cuts and fills on the order of 10 feet are anticipated to make finish grade, not including remedial grading.

We expect that the construction will include a concrete cast-in-place or tilt-up building supported on spread footing foundations and with a concrete slab-on-grade floor. We expect column loads will be up to 100 kips and wall loads will be up to 10 kips per linear foot. Preliminary geotechnical recommendations for design of the structure are based on these assumptions and provided herein.

The site descriptions and proposed development are based on a site reconnaissance, review of published geologic literature, our field investigation, a review of the conceptual site plan, and discussions with you. If development plans differ from those described herein, Geocon should be contacted for review of the plans and possible revisions to this report.

3. GEOLOGIC SETTING

The site is located within in the Perris block of the northern Peninsular Ranges Geomorphic Province (Province), defined as a relatively stable area between the Elsinore and San Jacinto fault zones. In the vicinity of the site, the geology consists of massive granitic bedrock and older alluvial fan deposits. The Peninsular Ranges are bounded by the Transverse Ranges (San Gabriel and San Bernardino Mountains) to the north and the Colorado Desert Geomorphic Province to the east. The Province extends westward into the Pacific Ocean and southward to the tip of Baja California. Overall, the Province is characterized by Cretaceous-age granitic rock and a lesser amount of Mesozoic-age metamorphic rock overlain by terrestrial and marine sediments. Faulting within the Province is typically northwest trending and includes the San Andreas, San Jacinto, Elsinore, and Newport-Inglewood faults.

The San Jacinto and Elsinore fault zones are located approximately 10 and 12 miles to the northeast and southwest, respectively. Geologic units within the site consist of very old alluvial fan deposits overlying granitic bedrock of the Val Verde Tonalite.

4. SOIL AND GEOLOGIC CONDITIONS

We observed very old alluvial fan deposits overlying the Val Verde Tonalite at depth during our field investigation. The occurrence, distribution and description of the geologic units encountered are shown on the *Geologic Map*, Figure 2 and the boring logs in *Appendix A*. The surficial soil and geologic units are described herein in order of increasing age.

4.1 Very Old Alluvial Fan Deposits (Qvof)

Very old alluvial fan deposits were observed at the surface throughout the site and were encountered to depths of 41 feet. As observed, the older alluvium consists of dry to moist, medium dense to very dense silty fine to medium sand. Varying amounts of coarse sand and trace amounts of gravel were also observed, along with trace amounts of carbonates.

4.2 Val Verde Tonalite (Kvt)

Val Verde Tonalite was encountered at 41 feet beneath the older alluvium in boring B-1 and likely underlies the site at depth. This granitic bedrock is weathered, strong, coarse-grained, black and white, and micaceous. The tonalite excavated as gravelly sand. Drilling within the bedrock was difficult.

5. GROUNDWATER

We did not encounter groundwater or seepage during the site investigation. According to the California Department of Water Resources, several wells in the area had a groundwater depth measured between 75 and 80 feet below the existing ground surface. It is not uncommon for seepage conditions to develop where none previously existed. Groundwater and seepage are dependent on seasonal precipitation, irrigation, land use, among other factors, and varies as a result. Proper surface drainage will be important to future performance of the project.

6. GEOLOGIC HAZARDS

6.1 Faulting

The numerous faults in southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (Bryant and Hart, 2007). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,700 years). A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years) but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a currently established State of California Alquist-Priolo Earthquake Fault Zone or a Riverside County Fault Hazard Zone for surface fault rupture hazards. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed school is considered low. However, the site is located in the seismically active southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active southern California faults.

According to the *Fault Activity Map of California* (2010), 19 known active faults are located within a search radius of 50 miles from the property. The nearest known active fault is the Casa Loma segment of the San Jacinto fault, located approximately 10 miles northeast of the site, and is the dominant source of potential ground motion. Earthquakes that might occur on these fault zones or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. Table 6.1.1 lists the estimated maximum earthquake magnitude for the most dominant faults in relationship to the site location.

TABLE 6.1.1
KNOWN ACTIVE FAULTS WITHIN 50 MILES OF THE SITE

Fault Name	Maximum Earthquake Magnitude (Mw)	Distance from Site (miles)	Direction from Site
San Jacinto (Casa Loma)	6.9	10	NE
Elsinore Fault (Glen Ivy)	6.8	12	SW
Elsinore (Wildomar)	6.8	13	S
San Jacinto (Claremont)	6.7	13	E
San Andreas (San Bernardino)	7.5	16	N
Chino	6.7	20	NW
San Gorgonio Pass	n/a	23	ENE
San Jacinto (Glen Helen)	6.7	24	N
San Jacinto (Clark)	7.2	24	SE
Whittier	6.8	25	NW
Cucamonga	6.9	28	NNW
Pinto Mountain	7.2	36	NE
San Andreas Fault (North Branch)	7.4	37	ENE
San Andreas Fault (South Branch)	7.5	37	E
Morongo Valley	7.2	41	NE
North Frontal Thrust	7.2	42	NNE
Newport-Inglewood-Rose Canyon	7.1	43	SW
Helendale	7.3	46	NNE
Burnt Mountain	6.5	48	ENE

Historic earthquakes in southern California of magnitude 6.0 and greater, their magnitude, distance, and direction from the site are listed in Table 6.1.2.

TABLE 6.1.2
HISTORIC EARTHQUAKE EVENTS WITH REPECT TO THE SITE

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto	December 25, 1899	6.7	15	ESE
San Jacinto	April 21, 1918	6.8	15	ESE
Loma Linda Area	July 22, 1923	6.3	12	N
Long Beach	March 10, 1933	6.4	44	WSW
Buck Ridge	March 25, 1937	6.0	64	ESE
Imperial Valley	May 18, 1940	6.9	57	ENE
Desert Hot Springs	December 4, 1948	6.0	50	E
Arroyo Salada	March 19, 1954	6.4	77	ESE
Borrego Mountain	April 8, 1968	6.5	83	ESE
San Fernando	February 9, 1971	6.6	83	WNW
Joshua Tree	April 22, 1992	6.1	59	E
Landers	June 28, 1992	7.3	57	ENE
Big Bear	June 28, 1992	6.4	37	NE
Northridge	January 17, 1994	6.7	85	WNW
Hector Mine	October 16, 1999	7.1	82	NE

6.2 Ground Rupture

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

6.3 Liquefaction

Liquefaction typically occurs when a site is located in a zone with seismic activity, onsite soils are cohesionless or silt/clay with low plasticity, groundwater is encountered within 50 feet of the surface, and soil has a relative density less than about 70 percent. If the four previous criteria are met, a seismic event could result in a rapid pore water pressure increase from the earthquake-generated ground accelerations. Due to the lack of a permanent, near-surface groundwater table and the dense to very dense nature of the very old alluvial fan deposits, liquefaction potential for the site is negligible and not a design consideration.

6.4 Expansive Soil

The older alluvium generally consists of silty or clayey sands with lesser amounts of sandy silts and sandy clays. Laboratory testing results indicate samples of the near surface soils exhibits a “very low” expansion potential (expansion index [EI] of 20 or less) with test results showing expansion indices of 0.

6.5 Hydrocompression

Hydrocompression is the tendency of unsaturated soil structure to collapse upon wetting resulting in the overall settlement of the affected soil and overlying foundations or improvements supported thereon. Potentially compressible soils underlying the site are typically removed and recompacted during remedial site grading. However, if compressible soil is left in-place, a potential for settlement due to hydrocompression of the soil exists.

Soils obtained during our investigation were tested for hydrocompression and exhibited a collapse potential of 1.3 to 2.2 percent when loaded to the expected post-grading pressures within the upper 5 feet, resulting to a “slight” (0.1 to 2.0) to “moderate” (2.1 to 6.0) degree of specimen collapse in accordance with ASTM D5333. Soils below 5 feet exhibited a collapse potential of 0.8 to 0.9 percent when loaded to the expected post-grading pressures, resulting in a “slight” degree of specimen collapse in tin accordance with ASTM D5333.

6.6 Seiches and Tsunamis

Seiches are caused by the movement of an inland body of water due to the movement from seismic forces. The site is located approximately 3.8 miles southwest of Lake Perris. In the unlikely event of a seiche, water is anticipated to be confined to the young alluvial valley channel east of Interstate 215.

A tsunami is a series of long-period waves generated in the ocean by a sudden displacement of large volumes of water. Causes of tsunamis include underwater earthquakes, volcanic eruptions, or offshore slope failures. The site is located approximately 36 miles from the Pacific Ocean at an elevation greater than 1,500 feet MSL. Therefore, the risk of tsunamis affecting the site is negligible and not a design consideration.

6.7 Inundation

According to the State of California, Department of Water Resources, *Inundation Map for Perris Dam*, dated April 29, 1975, the site is not within an inundation zone due to dam failure. Therefore, inundation due to dam failure is not a design consideration.

6.8 Landslides

Landslides are not mapped on or near the site. Due to the relatively level topography at the site, we opine that landslides are not present at the property or at a location that could impact the subject site.

6.9 Rock Fall Hazards

Rock falls are not a design consideration due to the lack of natural bedrock slopes above and adjacent to the site.

6.10 Slope Stability

Based on the preliminary site plans, cut and fill slopes will be 30 feet or less at inclinations no steeper than 2:1 (h:v). In general, permanent, cut slopes and graded fill slopes constructed with on-site soils inclined no steeper than 2:1 (h:v) with vertical heights of 30 feet or less will possess Factors of Safety of 1.5 or greater under static loading, 1.1 or greater under pseudo-static loading, and 1.5 or greater for surficial stability (see Figures 3 to 5). Fill keys should be constructed in accordance with the standard grading specifications in *Appendix C*. Grading of fill slopes should be designed in accordance with the requirements of Riverside County and the 2016 California Building Code (CBC).

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 From a geotechnical engineering standpoint, the site is suitable for construction of the proposed warehouse development provided the recommendations presented herein are implemented in design and construction of the project.

7.1.2 Potential geologic hazards at the site include seismic shaking and hydrocompression.

7.1.3 The site is located approximately 10 miles from the nearest active fault. Based on our background research and previous investigation, it is our opinion active or potentially active, do not extend across the site. Risks associated with seismic activity consist of the potential for moderate to strong seismic shaking.

7.1.4 Our field investigation indicates the site is blanketed by very old alluvium underlain by granitic bedrock. The upper portion of the very old alluvium is not considered suitable for the support of compacted fill and settlement-sensitive structures. Remedial grading of the surficial soil will be required as discussed herein. The existing site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed.

7.1.5 Moisture contents in the borings varied with depth. Upper portions of the older alluvium were dry to damp, and moisture increased with depth. Significant moisture conditioning of the soils should be expected during construction. Special handling of the soil should be anticipated, particularly if grading occurs during the rainy season.

7.1.6 Although the majority of on-site soils consist of silty sands, some granular material, having little to no cohesion and subject to caving in unshored excavations, should be expected at the site. It is the responsibility of the contractor to ensure that excavations and trenches are properly shored and maintained in accordance with OSHA rules and regulations to maintain the stability of adjacent existing improvements.

7.1.7 The laboratory tests indicate that the site soils are non-expansive and have a “very low” expansion potential. If medium to highly expansive soils are encountered at the site, they should be exported from the site or selectively graded and placed in the deeper fill areas to allow for the placement of low expansion material at the finish pad grade.

7.1.8 Although a grading plan was not prepared at the time of this report, cuts and fills ranging up to 15 feet are assumed to achieve planned finish grades.

7.1.9 Consolidation testing of samples of the subsurface soils indicates that there is a potential for hydrocompression of the soils beneath the site. Remedial grading will address the collapse potential of the near-surface soils; however, precautionary measures will be needed to mitigate the potential for hydrocollapse of deeper soils. Proper site drainage should be maintained. Landscape planters that saturate the subsurface or stormwater infiltration structures should not be used within 20 feet of the proposed buildings or other on grade improvements. Localized surface settlement should be expected in the vicinity of the stormwater infiltration structures or other areas where water is allowed to infiltrate to the subsurface.

7.1.10 Although not encountered in our exploration, cobbles or corestones within the granitic bedrock may be encountered during site grading and may present difficulty for site excavations. The contractor should be prepared to perform site excavations in these conditions.

7.1.11 We did not encounter groundwater during our investigation and do not expect groundwater would impact site improvements. However, wet conditions and seepage could affect proposed construction if grading and improvement operations occur during or shortly after a rain event.

7.1.12 Proper drainage should be maintained in order to preserve the design properties of the fill in the sheet-graded pad and slope areas.

7.1.13 The planned structures can be supported on a shallow foundation system with a slab-on-grade floor system.

7.1.14 Changes in the design, location or elevation of improvements, as outlined in this report, should be reviewed by this office. Once final grading plans become available, they should be reviewed by this office to evaluate the necessity for review and possible revision of this report.

7.1.15 Recommended grading specifications are provided in *Appendix C*.

7.2 Excavation and Soil Characteristics

7.2.1 Excavation of the very old alluvium should be possible with moderate to heavy effort using conventional heavy-duty equipment. Some difficulty in excavation may be encountered within moderately cemented zones. Although unlikely, excavations extending into the granitic bedrock will require heavy ripping and rock breaking tools.

7.2.2 The soil encountered in the field investigation is considered to be "non-expansive" (expansion index [EI] of less than 20) as defined by 2016 California Building Code (CBC) Section 1803.5.3. Table 7.2.2 presents soil classifications based on the expansion index. Based on the laboratory test results, we expect a majority of the soil encountered will possess a "very low" expansion potential (EI between 0 and 20). Medium to highly expansive soils may be encountered at the site and should not be placed within 4 feet of the proposed foundations, flatwork or paving improvements. Additional testing for expansion potential should be performed during grading and once final grades are achieved.

TABLE 7.2.2
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	ASTM D 4829 Expansion Classification	2016 CBC Expansion Classification
0 – 20	**Very Low**	**Non-Expansive**
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

7.2.3 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. *Appendix B* presents results of the laboratory water-soluble sulfate content tests. The test results indicate the on-site materials at the location tested possess a sulfate content of 0.002 percent (16 parts per million [ppm]) equating to an exposure class of "S0" as defined by 2016 CBC Section 1904.3 and ACI 318. Table 7.2.3 presents a summary of concrete requirements set forth by 2016 CBC Section 1904.3 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

TABLE 7.2.3
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	$0.10 \leq SO_4 < 0.20$	II	0.50	4,000
S2	$0.20 \leq SO_4 \leq 2.00$	V	0.45	4,500
S3	$SO_4 > 2.00$	V+Pozzolan or Slag	0.45	4,500

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

7.2.4 Laboratory testing indicates the site soils have a minimum electrical resistivity of 6,000 ohm-cm, possess 600 ppm chloride, 90 ppm sulfate, and a pH of 8.6. As shown in Table 7.2.4 below, the site would not be classified as "corrosive" to buried improvements, in accordance with the Caltrans Corrosion Guidelines (Caltrans, 2018).

TABLE 7.2.4
CALTRANS CORROSION GUIDELINES

Corrosion Exposure	Resistivity (ohm-cm)	Chloride (ppm)	Sulfate (ppm)	pH
Corrosive	<1,100	**500 or greater**	1,500 or greater	5.5 or less

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

7.3 Seismic Design Criteria

7.3.1 We used the computer program *U.S. Seismic Design Maps*, provided by the California Office of Statewide Health Planning and Development (OSHPD) to evaluate the seismic design criteria. Table 7.3.1 summarizes site-specific design criteria obtained from the 2016 California Building Code (CBC; Based on the 2015 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The short spectral response uses a period of 0.2 second. The building structure and improvements as currently proposed should be designed using a Site Class C in accordance with ASCE 7-10 Section 20.3.1. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2016 CBC and Table 20.3-1 of ASCE 7-10 using blow count data presented on the

boring logs in *Appendix A*. The values presented in Table 7.3.1 are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 7.3.1
2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	C	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.50g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.60g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.00	Table 1613.3.3(1)
Site Coefficient, F_V	1.30	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.50g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.78g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.00g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.52g	Section 1613.3.4 (Eqn 16-40)

7.3.2 Table 7.3.2 presents additional seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_G).

TABLE 7.3.2
2016 CBC SITE ACCELERATION PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Site Class	C	Section 1613.3.2
Mapped MCE_G Peak Ground Acceleration, PGA	0.50g	Figures 2 through 42-7
Site Coefficient, F_{PGA}	1.00	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.50g	Section 11.8.3 (Eqn 11.8-1)

7.3.3 Conformance to the criteria in Tables 7.3.1 and 7.3.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

7.4 Temporary Excavations

7.4.1 The recommendations included herein are provided for temporary excavations. It is the responsibility of the contractor to provide a safe excavation during the construction of the proposed project.

7.4.2 Excavations on the order of 5 to 15 feet in vertical height are expected during grading operations and utility installation. The contractor's competent person should evaluate the necessity for lay back of vertical cut areas. Vertical excavations up to 5 feet may be attempted where loose soils or caving sands are not present, and where not surcharged by existing structures or vehicle/construction equipment loads.

7.4.3 Vertical excavations greater than 5 feet will require sloping measures in order to provide a stable excavation. We expect that sufficient space is available to complete the majority of the required earthwork for this project using sloping measures. If necessary, compound excavation, slot-cutting, and or shoring recommendations will be provided in an addendum.

7.4.4 Where sloped embankments are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The contractor's personnel should inspect the soil exposed in the cut slopes during excavation so that modifications of the slopes can be made if variations in the soil conditions occur. Excavations should be stabilized within 30 days of initial excavation.

7.5 Grading

7.5.1 Grading should be performed in accordance with the recommendations provided in this report, the *Recommended Grading Specifications* contained in *Appendix C* and Riverside County Standards.

7.5.2 Prior to commencing grading, a pre-construction conference should be held at the site with the owner/developer, city inspector, grading contractor, civil engineer, and geotechnical engineer in attendance. Special soil handling requirements can be discussed at that time.

7.5.3 Site preparation should begin with the removal of previous structures and infrastructure, deleterious material, debris, buried trash, and vegetation. The depth of removal should be such that material exposed in cut areas or soil to be used as fill is relatively free of organic matter. Material generated during stripping and/or site demolition should be exported from the site.

7.5.4 The upper portion of the very old alluvium in the building areas should be removed to expose competent older alluvium. Based on our findings, we expect the existing soils within approximately 5 feet of the existing ground surface will require remedial excavation and proper compaction. Areas of loose, dry, or compressible soils will require additional excavation and processing prior to fill placement. Removals should extend at least 3 feet below the bottom of the planned foundations, and the excavations should be extended laterally a minimum distance of 5 feet beyond the building footprint or for a distance equal to the depth of removal, whichever is greater. Where the lateral over-excavation is not possible, structural setbacks or deepened footings may be required.

7.5.5 Removals in pavement and walkway areas should extend at least 2 feet beneath the pavement or flatwork subgrade elevation.

7.5.6 The actual depth of removal should be evaluated by the engineering geologist during grading operations. Deeper excavations may be required if dry, loose, soft, or porous materials are present at the base of the removals. The bottom of the excavations should be scarified to a depth of at least 1 foot, moisture conditioned as necessary, and properly compacted.

7.5.7 The site should then be brought to final subgrade elevations with fill compacted in layers. In general, soil native to the site is suitable for use as fill if free from vegetation, debris and other deleterious material. Layers of fill should be about 6 to 8 inches in loose thickness and no thicker than will allow for adequate bonding and compaction. Fill, including backfill and scarified ground surfaces, should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density at 0 to 2 percent above optimum moisture content, as determined in accordance with ASTM D 1557. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill. The upper 12 inches of subgrade soil underlying pavement should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density at 0 to 2 percent above optimum moisture content shortly before paving operations.

7.5.8 Import fill soil (if necessary) should consist of granular materials with a “very low” expansion potential (EI of less than 20), free of deleterious material and rock fragments larger than 6 inches and should be compacted as recommended herein. Geocon should be notified of the import soil source and should perform laboratory testing of import soil prior to its arrival at the site to determine its suitability as fill material.

7.5.9 Foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer, prior to placing fill, steel, gravel or concrete.

7.6 Utility Trench Backfill

7.6.1 Utility trenches should be properly backfilled in accordance with the requirements of Riverside County and the latest edition of the *Standard Specifications for Public Works Construction* (Greenbook). The pipes should be bedded with well graded crushed rock or clean sands (Sand Equivalent greater than 30) to a depth of at least one foot over the pipe. The use of open graded rock is only acceptable if used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. Backfill of utility trenches should not contain rocks greater than 3 inches in diameter. The use of 2-sack slurry and controlled low strength material (CLSM) are also acceptable as backfill. However, consideration should be given to the possibility of differential settlement where the slurry ends and earthen backfill begins. These transitions should be minimized and additional stabilization should be considered at these transitions.

7.6.2 Trench excavation bottoms must be observed and approved in writing by the Geotechnical Engineer, prior to placing bedding materials, fill, gravel, or concrete.

7.7 Earthwork Grading Factors

7.7.1 Estimates of shrinkage factors are based on empirical judgments comparing the material in its existing or natural state as encountered in the exploratory excavations to a compacted state. Variations in natural soil density and in compacted fill density render shrinkage value estimates very approximate. As an example, the contractor can compact the fill to a dry density of 90 percent or higher of the laboratory maximum dry density. Thus, the contractor has an approximately 10 percent range of control over the fill volume. Based on our experience and the densities measured during our investigation, the shrinkage of alluvium soil is expected to be up to 10 percent when compacted to at least 90 percent of the laboratory maximum dry density. This estimate is for preliminary quantity estimates only. Due to the variations in the actual shrinkage/bulking factors, a balance area should be provided to accommodate variations

7.8 Foundation and Concrete Slab-On-Grade Recommendations

7.8.1 The foundation recommendations presented herein are for the proposed buildings subsequent to the recommended grading assuming that the buildings are founded in soils with a low expansion potential. If soils with a medium or high expansion potential are placed within 4 feet of finish grade, then Geocon should be contacted for additional recommendations. The proposed structure can be supported on a shallow foundation system bearing in newly placed compacted fill.

7.8.2 Foundations for the structure should consist of either continuous strip footings and/or isolated spread footings. Continuous footings should be at least 18 inches wide and extend at least 18 inches below lowest adjacent pad grade. Isolated spread footings should have a minimum width of 24 inches and should also extend at least 18 inches below lowest adjacent pad grade. A wall/column footing dimension detail depicting footing embedment is provided on Figure 6.

7.8.3 From a geotechnical engineering standpoint, concrete slabs-on-grade for the structure should be at least 4 inches thick and be reinforced with at least No. 3 steel reinforcing bars placed 24 inches on center in both directions. 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 slab for supporting equipment and storage loads. A thicker concrete slab may be required for heavier loading conditions. To reduce the effects of differential settlement on the foundation system, thickened slabs and/or an increase in steel reinforcement can provide a benefit to reduce concrete cracking.

7.8.4 Steel reinforcement for continuous footings should consist of at least two No. 4 steel reinforcing bars placed horizontally in the footings, one near the top and one near the bottom. Steel reinforcement for the spread footings should be designed by the project structural engineer.

7.8.5 The recommendations presented herein are based on soil characteristics only (EI of 50 or less) and are not intended to replace steel reinforcement required for structural considerations.

7.8.6 Foundations may be designed for an allowable soil bearing pressure of 3,500 pounds per square foot (psf) (dead plus live load). The value presented herein is for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces.

7.8.7 The maximum expected static settlement for the planned structures supported on conventional foundation systems with the above allowable bearing pressure and deriving support in engineered fill is estimated to be 1 inch and to occur below the heaviest loaded structural element. Differential settlement is estimated to be on the order of ½ inch over a horizontal distance of 40 feet. Once the design and foundation loading configuration proceeds to a more finalized plan, the estimated settlements within this report should be reviewed and revised, if necessary

7.8.8 Once the design and foundation loading configuration proceeds to a more finalized plan, the estimated settlements within this report should be reviewed and revised, if necessary.

7.8.9 Slabs-on-grade that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) and should be installed in general conformance with ASTM E1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the California Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Green Building Code, the concrete slab-on-grade may be underlain by a vapor retarder over 4 inches of clean sand (sand equivalent greater than 30), since the sand will serve as a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

7.8.10 The bedding sand thickness should be evaluated by the project foundation engineer, architect, and/or developer. However, we should be contacted to provide recommendations if the bedding sand is thicker than 4 inches. Placement of 3 inches and 4 inches of sand is common practice in southern California for 5-inch and 4-inch thick slabs, respectively. The foundation engineer should provide appropriate concrete mix design criteria and curing measures that may be utilized to assure proper curing of the slab to reduce the potential for rapid moisture loss and subsequent cracking and/or slab curl.

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

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

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

7.9 Concrete Flatwork

7.9.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations herein. Slab panels should be a minimum of 4 inches thick and, when in excess of 8 feet square, should be reinforced with No. 3 reinforcing bars spaced 24 inches on center in each direction to reduce the potential for wide cracking. In addition, concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior to concrete placement. Subgrade soil should be properly compacted and the moisture content of subgrade soil should be checked prior to placing concrete.

7.9.2 Even with the incorporation of the recommendations within this report, the exterior concrete flatwork has a likelihood of experiencing some movement due to swelling or settlement; therefore, the steel reinforcement should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork. Additionally, flatwork should be structurally connected to the curbs, where possible, to reduce the potential for offsets between the curbs and the flatwork.

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

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

7.10 Conventional Retaining Walls

7.10.1 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 10 feet. In the event that walls higher than 10 feet or other types of walls are planned, Geocon should be consulted for additional recommendations.

7.10.2 Retaining walls not restrained at the top and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 40 pounds per cubic foot (pcf). Where the backfill will be inclined at no steeper than 2:1 (horizontal to vertical), an active soil pressure of 65 pcf is recommended. These soil pressures assume that the backfill materials within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall possess an EI of 50 or less. For walls where backfill materials do not conform to the criteria herein, Geocon should be consulted for additional recommendations.

7.10.3 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls with a level backfill surface should be designed for a soil pressure equivalent to the pressure exerted by a fluid density of 60 pcf.

7.10.4 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, proposed retaining walls in excess of 6 feet in height should be designed with seismic lateral pressure (Section 1803.5.12 of the 2016 CBC).

7.10.5 A seismic load of 10 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2016 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure. The earth pressure is based on half of two-thirds of PGA_M calculated from ASCE 7-10 Section 11.8.3.

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

7.10.7 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and waterproofed as required by the project architect. The soil immediately adjacent to the backfilled retaining wall should be composed of free draining material completely wrapped in Mirafi 140N (or equivalent) filter fabric for a lateral distance of 1 foot for the bottom two-thirds of the height of the retaining wall. The upper one-third should be backfilled with less permeable compacted fill to reduce water infiltration. Alternatively, a drainage panel, such as a Miradrain 6000 or equivalent, can be placed along the back of the wall. A typical drain detail for each option is shown on Figure 7. The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted backfill (EI of 20 or less) with no hydrostatic forces or imposed surcharge load. If conditions different than those described are expected or if specific drainage details are desired, Geocon should be contacted for additional recommendations.

7.10.8 Wall foundations should be designed in accordance with the above foundation recommendations.

7.11 Lateral Loading

7.11.1 To resist lateral loads, a passive pressure exerted by an equivalent fluid density of 300 pounds per cubic foot (pcf) should be used for the design of footings or shear keys. The allowable passive pressure assumes a horizontal surface extending at least 5 feet, or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

7.11.2 If friction is to be used to resist lateral loads, an allowable coefficient of friction between soil and concrete of 0.40 should be used for design. The friction coefficient may be reduced depending on the vapor barrier or waterproofing material used for construction in accordance with the manufacturer's recommendations.

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

7.12 Preliminary Pavement Recommendations

7.12.1 We calculated the flexible pavement sections in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4) and County of Riverside specifications using a range of Traffic Indices. The project civil engineer and owner should evaluate the final Traffic Index for the pavements and review the pavement designations to determine appropriate locations for pavement thickness. Based on the laboratory testing of the onsite soils, we have used a preliminary R-value of 21 for the subgrade soils for the purposes of this analysis. The final pavement sections should be based on the R-value of the subgrade soil encountered at final subgrade elevation. Table 7.12.1 presents the preliminary flexible pavement sections.

TABLE 7.12.1
PRELIMINARY FLEXIBLE PAVEMENT SECTION

Location	Assumed Traffic Index	Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Driveways for automobiles and light-duty vehicles	5.5	21	3.0	9.0
Medium truck traffic areas	6.0		3.5	9.5
Driveways for heavy truck and fire truck traffic	7.0		4.0	12.0
Collector Roadways	8.0		5.0	13.0

7.12.2 Prior to placing base materials, the upper 12 inches of the subgrade soil should be scarified, moisture conditioned as necessary, and recompacted to a dry density of at least 95 percent of the laboratory maximum dry density at 0 to 2 percent 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 at 0 to 2 percent above optimum moisture content. Asphalt concrete should be compacted to a density of at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

7.12.3 Base materials should conform to Section 26-1.028 of the *Standard Specifications for The State of California Department of Transportation (Caltrans)*. The asphalt concrete should conform to Section 203-6 of the *Standard Specifications for Public Works Construction (Greenbook)*.

7.12.4 A rigid Portland cement concrete (PCC) pavement section should be placed in heavy truck areas, driveway aprons, and cross gutters. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R *Guide for Design and Construction of Concrete Parking Lots* using the parameters presented in Table 7.12.4.

TABLE 7.12.4
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	200 pci
Modulus of rupture for concrete, M_R	500 psi
Traffic Category, TC	C and D
Average daily truck traffic, ADTT	100 and 700

7.12.5 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 7.12.5.

TABLE 7.12.5
RIGID PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Automobile Parking Stalls (TC=C)	6.5
Heavy Truck and Fire Lane Areas (TC=D)	7.5

7.12.6 The PCC 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 at 0 to 2 percent above optimum moisture content. This pavement section is based on a minimum concrete compressive strength of approximately 3,000 psi (pounds per square inch).

7.12.7 A thickened edge or integral curb should be constructed on the outside of concrete slabs subjected to wheel loads. The thickened edge should be 1.2 times the slab thickness or a minimum thickness of 2 inches, whichever results in a thicker edge, and taper back to the recommended slab thickness 4 feet behind the face of the slab (e.g., 6-inch and 7.5-inch-thick slabs would have an 8- and 9.5-inch-thick edge, respectively). Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed herein.

7.12.8 In order to control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab in accordance with the referenced ACI report.

7.12.9 The performance of pavements is highly dependent on providing positive surface drainage away from the edge of the pavement. Ponding of water on or adjacent to the pavement surfaces 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.

7.13 Temporary Excavations

7.13.1 Excavations on the order of 5 to 15 feet below the existing ground surface are expected for construction of the proposed utility improvements; and we expect that the proposed utilities will be installed with conventional cut-and-cover methods.

7.13.2 The excavations will expose previously placed fill and older alluvial soils which are suitable for vertical excavations up to 5 feet where loose soils or caving sands are not present and where not surcharged by adjacent traffic or structures.

7.13.3 Vertical excavations greater than 5 feet will require sloping measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments should be designed by the contractor's competent person in accordance with OSHA regulations.

7.13.4 Where there is insufficient space for sloped excavations, shoring or trench shields should be used to support excavations. Shoring may also be necessary where sloped excavation could remove vertical or lateral support of existing improvements, including existing utilities and adjacent structures. Recommendations for temporary shoring can be provided in an addendum if needed.

7.13.5 Where sloped embankments are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The contractor's competent person should inspect the soils exposed in the cut slopes during excavation in accordance with OSHA regulations so that modifications of the slopes can be made if variations in the soil conditions occur.

7.14 Site Drainage and Moisture Protection

7.14.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2016 CBC 1804.4 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

7.14.2 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.

7.14.3 Landscape planters that saturate the subsurface should not be used within 20 feet of the proposed structure or other settlement sensitive on grade improvements. Localized surface settlement should be anticipated in areas where water is allowed to infiltrate into the subsurface.

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

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

7.14.6 If not properly constructed, there is a potential for distress to improvements and properties located hydrologically down gradient or adjacent to infiltration areas. Factors such as the amount of water to be detained, its residence time, and soil permeability have an important effect on seepage transmission and the potential adverse impacts that may occur if the storm water management features are not properly designed and constructed. We have not performed a hydrogeology study at the site. Down-gradient and adjacent structures may be subjected to seeps, movement of foundations and slabs, or other impacts as a result of water infiltration.

7.15 Grading and Foundation Plan Review

7.15.1 Geocon should review the project grading and foundation plans prior to final design submittal to verify that the plans have been prepared in substantial conformance with the recommendations of this report and to provide additional analyses or recommendations, if necessary.

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 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.

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

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

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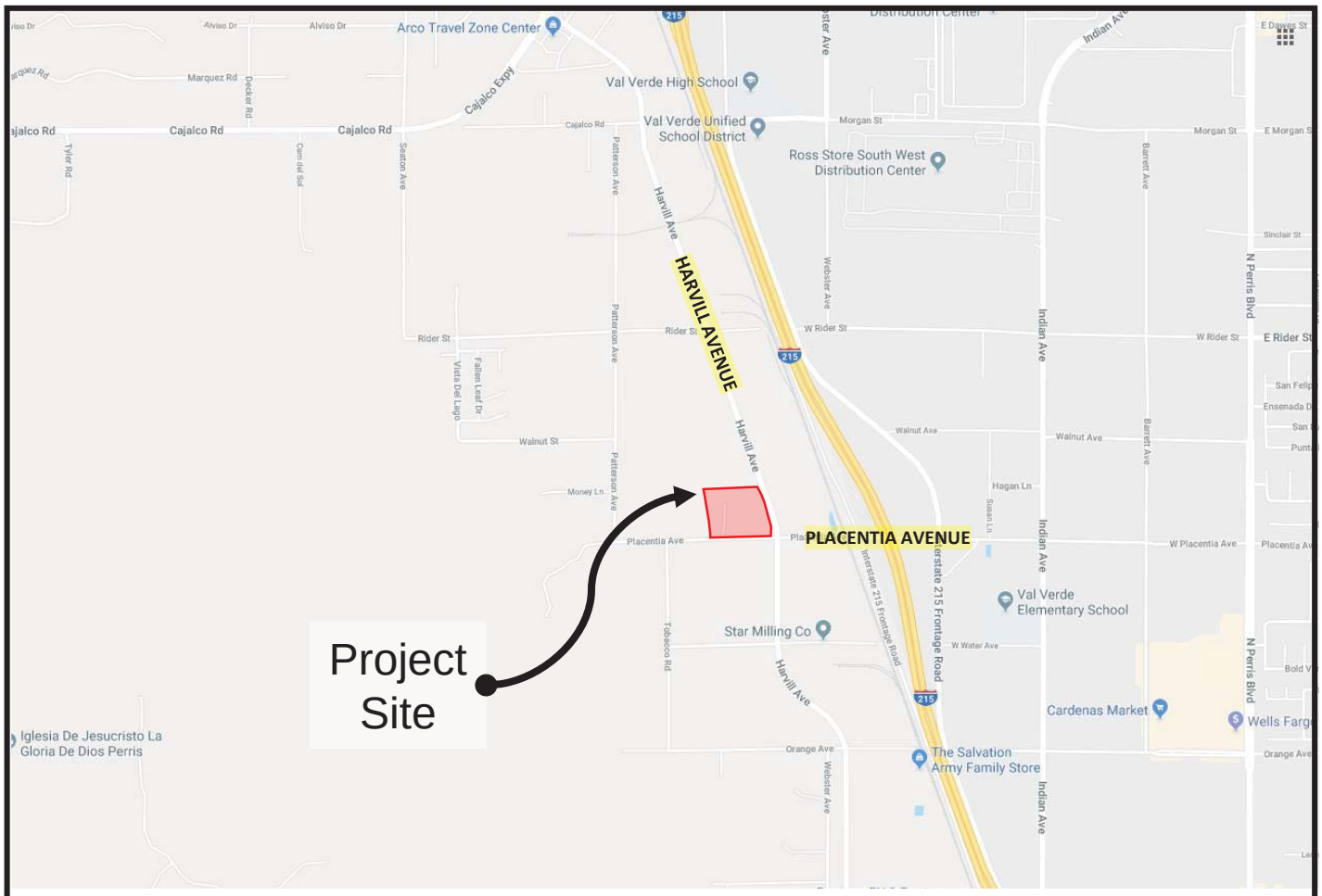

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1000 ft

SOURCE: Google Maps, 2019

VICINITY MAP

GEOCON WEST, INC.

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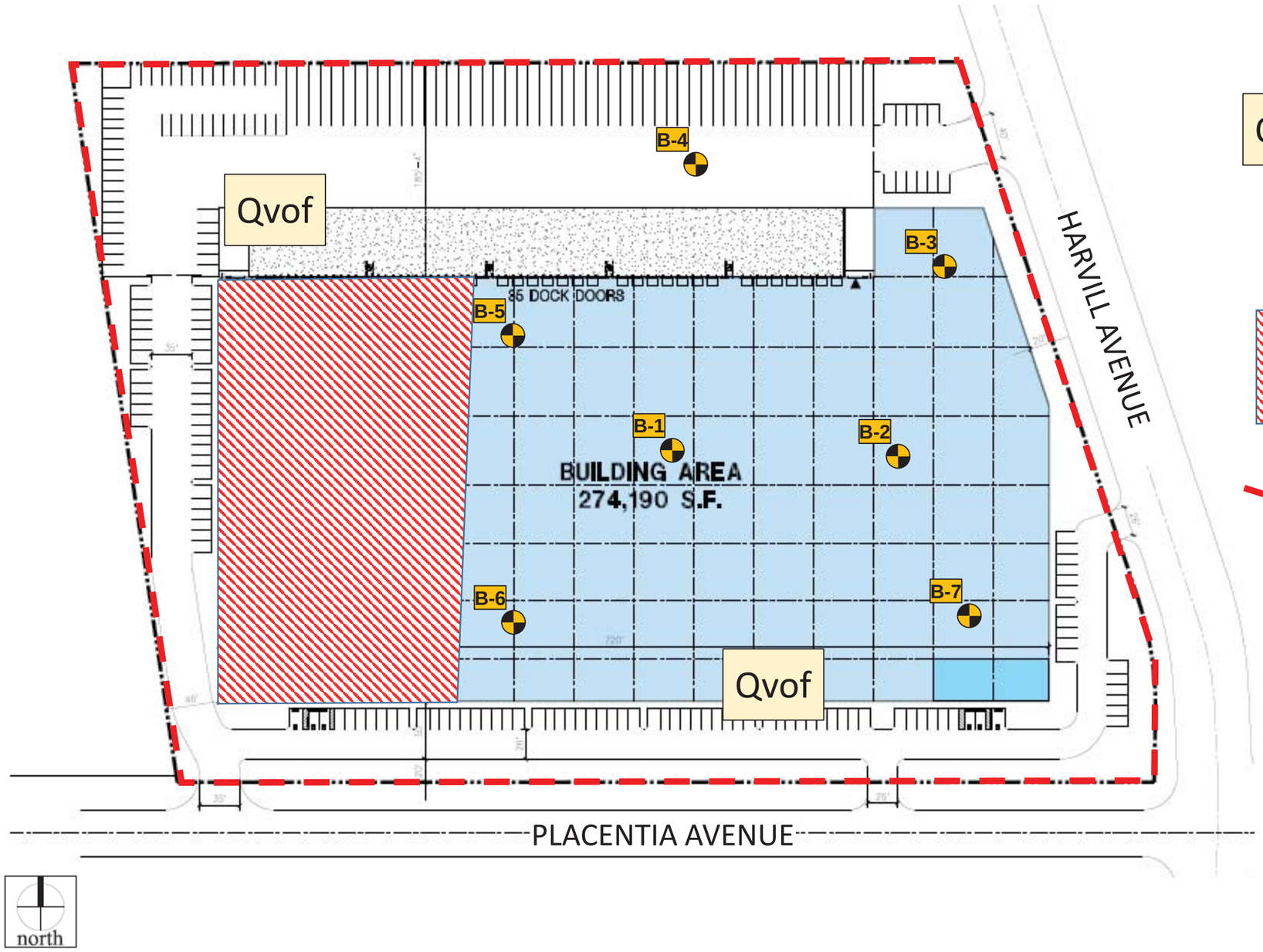
WAREHOUSE DEVELOPMENT
NORTHWEST CORNER OF
HARVILL AND PLACENTIA AVENUES
MEAD VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

LCW		

NOVEMBER 2019	PROJECT NO. T2891-22-01	FIG. 1



GEOCON LEGEND

Locations are approximate

Qvof ..... VERY OLD ALLUVIAL FAN DEPOSITS

B-7 ..... BORING LOCATION

..... EXISTING RESIDENCES

..... LIMITS OF THIS REPORT

Source: HPA Architecture. *Conceptual Site Plan Placentia Ave. & Patterson Ave riverside, CA*, dated June 7th, 2018

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GEOLOGIC MAP

WAREHOUSE DEVELOPMENT
NORTHWEST CORNER OF
HARVILL AND PLACENTIA AVENUES
MEAD VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

LCW			NOVEMBER 2019	PROJECT NO. T2891-22-01	FIG. 2

ASSUMED CONDITIONS:

SLOPE HEIGHT	H	=	30 feet
SLOPE INCLINATION	2.0 : 1.0 (Horizontal : Vertical)		
TOTAL UNIT WEIGHT OF SOIL	γ_t	=	130 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	=	34 degrees
APPARENT COHESION	C	=	108 pounds per square foot
NO SEEPAGE FORCES			

ANALYSIS:

$\lambda_{cf} = \dfrac{\gamma H \tan\phi}{C}$ EQUATION (3-3), REFERENCE 1

$FS = \dfrac{NcfC}{\gamma H}$ EQUATION (3-2), REFERENCE 1

λ_{cf} = 24.4 CALCULATED USING EQ. (3-3)

Ncf = 66 DETERMINED USING FIGURE 10, REFERENCE 2

FS = 1.8 FACTOR OF SAFETY CALCULATED USING EQ. (3-2)

REFERENCES:

1……Janbu, N., Stability Analysis of Slopes with Dimensionless Parameters, Harvard Soil Mechanics Series No. 46,1954

2……Janbu, N., Discussion of J.M. Bell Dimensionless Parameters for Homogeneous Earth Slpes, Journal of Soil Mechanicx and Foundation Design, No. SM6, November 1967

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SLOPE STABILITY ANALYSIS

WAREHOUSE DEVELOPMENT
NORTHWEST CORNER OF
HARVILL AND PLACENTIA AVENUES
MEAD VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

NOVEMBER 2019 | PROJECT NO. T2891-22-01 | FIG. 3

ASSUMED CONDITIONS:

SLOPE HEIGHT	H	=	30 feet
SLOPE INCLINATION	2.0 : 1.0 (Horizontal : Vertical)		
TOTAL UNIT WEIGHT OF SOIL	γ_t	=	130 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	=	34 degrees
APPARENT COHESION	C	=	108 pounds per square foot
PSEUDOSTATIC COEFFICIENT	k_h	=	0.15
PSEUDOSTATIC INCLINATION	1.4 : 1.0 (Horizontal : Vertical)		
PSEUDOSTATIC UNIT WEIGHT	γ_{ps}	=	131 pounds per cubic foot

NO SEEPAGE FORCES

ANALYSIS:

$\lambda_{cf} = \dfrac{\gamma H \tan\phi}{C}$ EQUATION (3-3), REFERENCE 1

$FS = \dfrac{NcfC}{\gamma H}$ EQUATION (3-2), REFERENCE 1

λ_{cf} = 24.6 CALCULATED USING EQ. (3-3)

Ncf = 51 DETERMINED USING FIGURE 10, REFERENCE 2

FS = 1.4 FACTOR OF SAFETY CALCULATED USING EQ. (3-2)

REFERENCES:

1……Janbu, N., Stability Analysis of Slopes with Dimensionless Parameters, Harvard Soil Mechanics Series No. 46,1954

2……Janbu, N., Discussion of J.M. Bell Dimensionless Parameters for Homogeneous Earth Slpes, Journal of Soil Mechanicx and Foundation Design, No. SM6, November 1967

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SLOPE STABILITY ANALYSIS - WITH SEISMIC

WAREHOUSE DEVELOPMENT
NORTHWEST CORNER OF
HARVILL AND PLACENTIA AVENUES
MEAD VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

NOVEMBER 2019 | PROJECT NO. T2891-22-01 | FIG. 4

ASSUMED CONDITIONS:

SLOPE HEIGHT	H	=	Infinte
SLOPE INCLINATION	2.0 : 1.0 (Horizontal : Vertical)		
SLOPE ANGLE	i	=	26.6 °
DEPTH OF SATURATION	Z	=	2.5 feet
UNIT WEIGHT OF WATER	γ_w		62.4 pounds per cubic foot
TOTAL UNIT WEIGHT OF SOIL	γ_t	=	130 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	=	34 degrees
APPARENT COHESION	C	=	108 pounds per square foot

SLOPE SATURATED TO VERTICAL DEPTH Z BELOW SLOPE FACE.
SEEPAGE FORCES PARALLEL TO SLOPE FACE.

ANALYSIS:

$$FS = \frac{C + (\gamma_t - \gamma_w) Z \cdot \cos^2 i \cdot \tan \phi}{\gamma_t \cdot Z \cdot \sin i \cdot \cos i} = 1.5$$

REFERENCES:

1……Haefeli, R. *The Stability of Slopes Acted Upon by Parallel Seepage,* Proc. Second International Conference, SMFE, Rotterdam, 1948, 1, 57-62.

2……Skempton, A. W., and F. A. Delory, *Stability of Natural Slopes in London Clay,* Proc. Fourth International Conference, SMFE, London, 1957, 2, 378-81.

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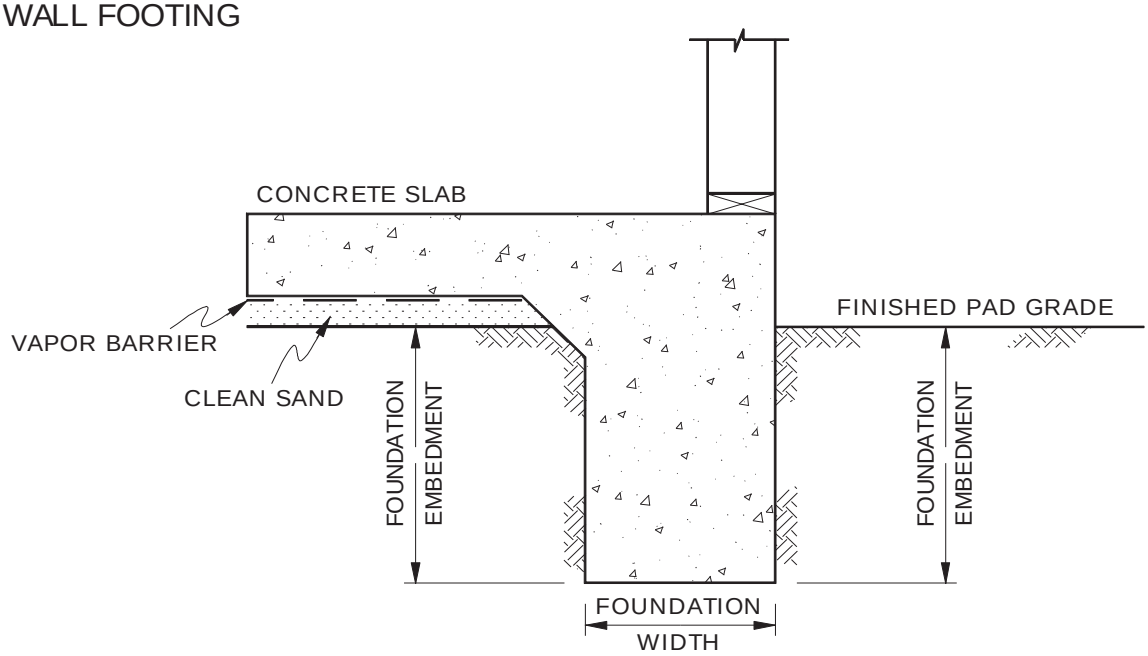
SURFICIAL SLOPE STABILITY

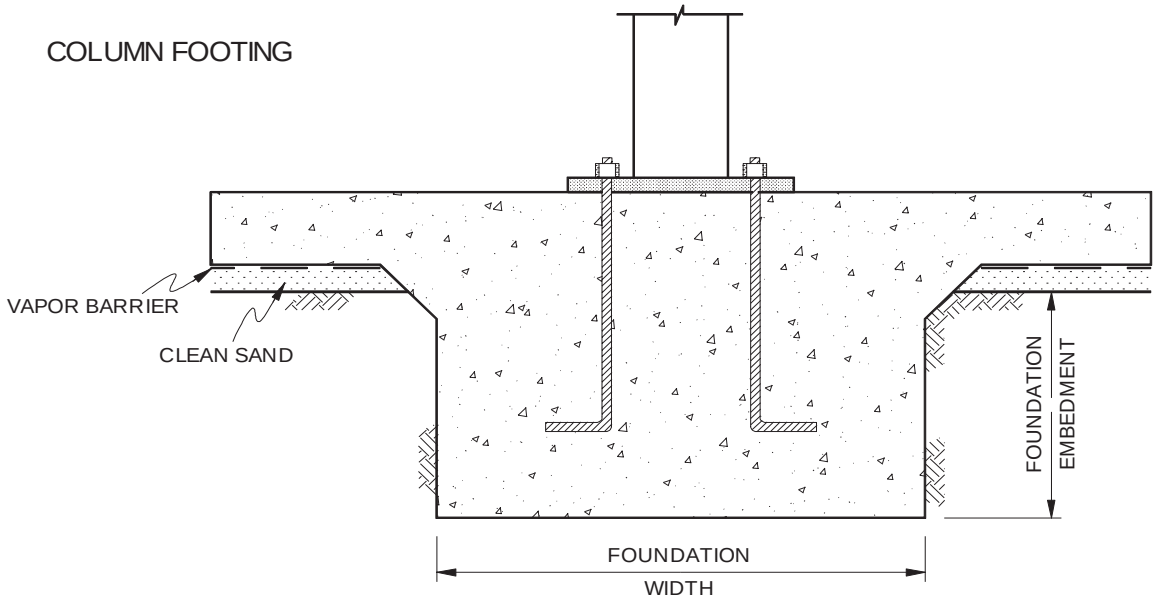
WAREHOUSE DEVELOPMENT
NORTHWEST CORNER OF
HARVILL AND PLACENTIA AVENUES
MEAD VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

NOVEMBER 2019 | PROJECT NO. T2891-22-01 | FIG. 5

WALL FOOTING



COLUMN FOOTING



NOTE: SEE REPORT FOR FOUNDATION WIDTH AND DEPTH RECOMMENDATION

NO SCALE

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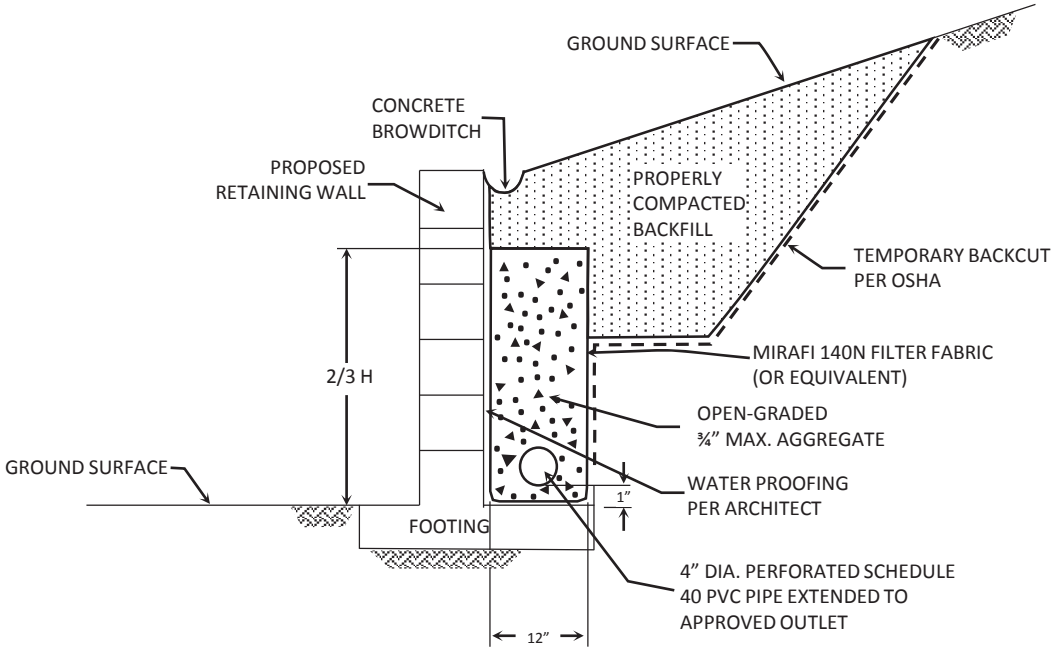
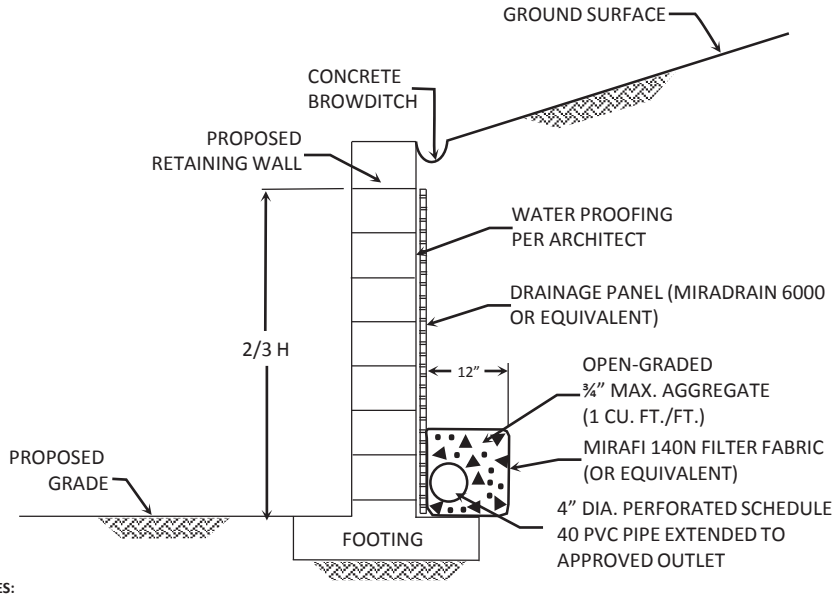
WALL / COLUMN FOOTING DETAIL

WAREHOUSE DEVELOPMENT
NORTHWEST CORNER OF
HARVILL AND PLACENTIA AVENUES
MEAD VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

NOVEMBER 2019	PROJECT NO. T2891-22-01	FIG. 6





NOTES:

DRAIN SHOULD BE UNFORMLY SLOPED TO GRAVITY OUTLET OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPIMG

CONCRETE BROW DITCH RECOMMENDED FOR SLOPE HEIGHTS GREATER THAN 6 FEET

NO SCALE

TYPICAL RETAINING WALL DRAIN DETAIL

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WAREHOUSE DEVELOPMENT
NORTHWEST CORNER OF
HARVILL AND PLACENTIA AVENUES
MEAD VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

LCW		

NOVEMBER 2019	PROJECT NO. T2891-22-01	FIG. 7

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

Field work for our investigation included a site reconnaissance, subsurface exploration, and soil sampling. The *Geologic Map*, Figure 2 presents the locations of the exploratory borings. Boring logs and an explanation of the geologic units encountered are presented in figures following the text in this appendix. We located the borings in the field using existing reference points. Therefore, actual boring locations may deviate slightly. We performed a field investigation on October 10, 2019 which consisted of drilling 7 exploratory borings to a maximum depth of approximately 50.3 feet below existing grade with a CME 75 drill rig equipped with 8-inch-diameter hollow-stem auger.

We collected bulk and relatively undisturbed samples from the borings by driving a 3-inch O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound hammer falling 30 inches on an auto hammer. The California Modified Sampler was equipped with 1-inch high by $2^{3}/_{8}$-inch inside diameter brass sampler rings to facilitate removal and testing. Relatively undisturbed samples and bulk samples of disturbed soils were transported to our laboratory for testing. The type of sample is noted on the exploratory boring logs.

The samplers were driven 18 inches into the bottom of the excavations. Blow counts are recorded for every 6 inches the sampler is driven. The penetration resistances shown on the boring logs are shown in terms of blows per foot. The values indicated on the boring logs are the sum of the last 12 inches of the sampler if driven 18 inches. If the sampler was not driven for 18 inches, an approximate value is calculated in term of blows per foot or the final 6-inch interval is reported. These values are not to be taken as N-values, adjustments have not been applied. We estimated elevations shown on the boring logs from a topographic map.

We visually examined the soil conditions encountered within the borings, classified, and logged in general accordance with the Unified Soil Classification System (USCS). Logs of the borings are presented on Figures A-1 through A-7. The logs depict the general soil and geologic conditions encountered and the depth at which we obtained the samples.

PROJECT NO. T2891-22-01

BORING B-1

ELEV. (MSL.) **1521 ft** DATE COMPLETED **10/10/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B-1@0-5'			SM	**VERY OLD ALLUVIAL FAN DEPOSITS (Qvof)** Silty SAND, medium dense, dry, dark reddish brown; fine to medium sand; trace coarse sand; some mica; trace pinhole porosity; weeds at surface			
2	B-1@2.5'					31	117.8	4.2
4					-Becomes damp; light reddish brown			
6	B-1@5'				-Becomes dense	70	115.7	5.0
8	B-1@7.5'				-Becomes very dense; moist; trace clay	90	128.5	9.1
10	B-1@10'					78		
12								
14								
16	B-1@15'				-Becomes dense; some clay	71		
18								
20	B-1@20'				-Becomes very dense	63		
22								
24								
26	B-1@25'				-No clay	94-11"		
28								

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

T2843-22-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. T2891-22-01

BORING B-1

ELEV. (MSL.) **1521 ft** DATE COMPLETED **10/10/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B-1@30'				-Becomes medium dense	21		
32								
34								
36	B-1@355'				-Becomes very dense; increase in coarse sand and mica; slow advance	50-4"		
38								
40	B-1@40'					50-5"		
42					**VAL VERDE TONALITE (Kvt)** Weathered, strong, black and white, GRANITIC BEDROCK; coarse-grained; micaceous; excavates as a gravlley sand			
44								
46	B-1@455'					50-2"		
48								
50	B-1@50'					50-4"		
					Total Depth 51.3' Groundwater not encountered Penetration resistance for 130-lb hammer falling 30" by auto-hammer Backfilled with cuttings 10/10/2019			

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

T2843-22-01 BORING LOGS.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.

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PROJECT NO. T2891-22-01

BORING B-2

ELEV. (MSL.) **1516 ft** DATE COMPLETED **10/10/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**VERY OLD ALLUVIAL FAN DEPOSITS (Qvof)** Silty SAND, medium dense, dry, reddish brown; fine to medium sand; some coarse sand; some mica; trace pinhole porosity; weeds at surface			
2	B-2@2.5'				-Becomes damp	23	116.5	4.8
4								
6	B-2@5' B-2@5-10'				-Becomes dense, moist, dark reddish brown; some carbonate stringers	68	117.9	7.9
8	B-2@7.5'				-Becomes very dense, moist, dark brown; few coarse sand	75	132.4	8.9
10	B-2@10'				-Becomes dense	57		
12								
14								
16	B-2@15'				-Becomes very dense	84		
18								
20	B-2@20'					50-6"		

Total Depth 20.5'
Groundwater not encountered
Penetration resistance for 130-lb hammer falling 30" by auto-hammer
Backfilled with cuttings 10/10/2019

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

T2843-22-01 BORING LOGS.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.

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PROJECT NO. T2891-22-01

BORING B-3

ELEV. (MSL.) **1514 ft** DATE COMPLETED **10/10/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**VERY OLD ALLUVIAL FAN DEPOSITS (Qvof)** Silty SAND, medium dense, dry, reddish brown; fine to medium sand; some coarse sand; some mica; weeds at surface			
2	B-3@2.5'				-Becomes damp	37	119.9	4.6
4								
6	B-3@5' B-3@5-10'				-Becomes fine to coarse sand; trace gravel	22	115.3	4.4
8	B-3@7.5'				-Beocomes moist; dark reddish brown; some carbonate stringers	50		
10	B-3@10'				-Becomes very dense; fine to medium sand	50-6"		
12								
14								
16	B-3@15'				-Becomes dense	56		
18								
20	B-3@20'				-Becomes very dense	50-6"		

Total Depth 21.5'
Groundwater not encountered
Penetration resistance for 130-lb hammer falling 30" by auto-hammer
Backfilled with cuttings 10/10/2019

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

T2843-22-01 BORING LOGS.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.

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PROJECT NO. T2891-22-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING B-4** ELEV. (MSL.) **1517 ft** DATE COMPLETED **10/10/2019** EQUIPMENT **HOLLOW STEM AUGER** BY: **PDT**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	B-4@0-5'			SM	**VERY OLD ALLUVIAL FAN DEPOSITS (Qvof)** Silty SAND, medium dense, dry, dark reddish brown; fine to medium sand; trace coarse sand; some mica; weeds at surface			
2								
4								
					Total Depth 5' Groundwater not encountered Backfilled with cuttings 10/10/2019			

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

T2843-22-01 BORING LOGS.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.

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PROJECT NO. T2891-22-01

BORING B-5

ELEV. (MSL.) **1522 ft** DATE COMPLETED **10/10/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B-5@0-5'			SM	**VERY OLD ALLUVIAL FAN DEPOSITS (Qvof)** Silty SAND, medium dense, dry, yellowish brown; fine to medium sand; few coarse sand; some mice; weeds at surface			
2	B-5@2.5'					49	111.2	4.6
4								
6	B-5@5'				-Becomes damp, brown	19	111.2	4.0
8	B-5@7.5'				-Becomes very dense; moist; dark reddish brown	86		
10	B-5@10'				-Becomes dense; trace carbonate stringers	67		
12								
14								
16	B-5@15'				-Becomes yellowish brown	69		
18								
20	B-5@20'				-Trace carbonate stringers	58		
					Total Depth 21.5' Groundwater not encountered Penetration resistance for 130-lb hammer falling 30" by auto-hammer Backfilled with cuttings 10/10/2019			

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

T2843-22-01 BORING LOGS.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.

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PROJECT NO. T2891-22-01

BORING B-6

ELEV. (MSL.) **1526 ft** DATE COMPLETED **10/10/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**VERY OLD ALLUVIAL FAN DEPOSITS (Qvof)** Silty SAND, dense, dry, brown; fine to medium sand; few coarse sand; some mica; weeds at surface			
2	B-6@2.5'				-Becomes damp	58	119.5	4.0
4								
6	B-6@5' B-6@5-10'				-Becomes medium dense, reddish brown; trace carbonate stringers	27	117.4	4.6
8	B-6@7.5'				-Becomes moist, grayish brown	74	136.2	5.9
10	B-6@10'					63	121.2	11.5
12								
14								
16	B-6@155'				-Becomes medium dense; some clay	44		
18								
20	B-6@20'				-Becomes yellowish brown	50-4"		

Total Depth 20.8'
Groundwater not encountered
Penetration resistance for 130-lb hammer falling 30" by auto-hammer
Backfilled with cuttings 10/10/2019

Figure A-6,
Log of Boring B-6, Page 1 of 1

T2843-22-01 BORING LOGS.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.

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BORING B-7

ELEV. (MSL.) **1512 ft** DATE COMPLETED **10/10/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**VERY OLD ALLUVIAL FAN DEPOSITS (Qvof)** Silty SAND, medium dense, dry, brown; fine to medium sand; few coarse sand; some mica; weeds at surface			
2	B-7@2.5'					55	139.0	7.1
4								
6	B-7@5'				-Becomes very dense, moist, dark reddish brown	87	122.3	5.3
8	B-7@7.5'				-Trace carbonate stringers	50-6"	123.2	5.9
10	B-7@10'				-Becomes dense	70		
12								
14								
16	B-7@15'				-Becomes very dense; reddish brown	50-6"		
18								
20	B-7@20'				-Becomes dense; trace clay; trace carbonate stringers	59		

Total Depth 21.5'
Groundwater not encountered
Penetration resistance for 130-lb hammer falling 30" by auto-hammer
Backfilled with cuttings 10/10/2019

Figure A-7,
Log of Boring B-7, Page 1 of 1

T2843-22-01 BORING LOGS.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.

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APPENDIX B

APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with current generally accepted test methods of ASTM International (ASTM) or other suggested procedures. We analyzed selected soil samples for in-situ density and moisture content, maximum dry density and optimum moisture content, expansion index, corrosivity, grain size distribution, consolidation characteristics, R-values, and direct shear strength. The results of the laboratory tests are presented on Figures B-1 through B-8. The in-place dry density and moisture content of the samples tested are presented on the boring logs in *Appendix A*.

SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS ASTM D1557

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% of dry wt.)
B-1 @ 0-5’	Silty SAND (SM) dark reddish brown	134.5	7.5
B-5 @ 0-5’	Silty SAND (SM), yellowish brown	134.5	7.0

SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS ASTM D4829

Sample No.	Moisture Content		After Test Dry Density (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
B-6 @ 5-10’	7.5	14.6	118.5	0

SUMMARY OF CORROSIVITY TEST RESULTS

Sample No.	Chloride Content (ppm)	Sulfate Content (%)	pH	Resistivity (ohm-centimeter)
B-5 @ 0-5’	600	0.002	8.6	6,000

Chloride content determined by California Test 422.
Water-soluble sulfate determined by California Test 417.
Resistivity and pH determined by California Test 643.

SUMMARY OF LABORATORY R-VALUE TEST RESULTS ASTM D2844

Sample No.	R-Value
B-4 @ 0-5’	21

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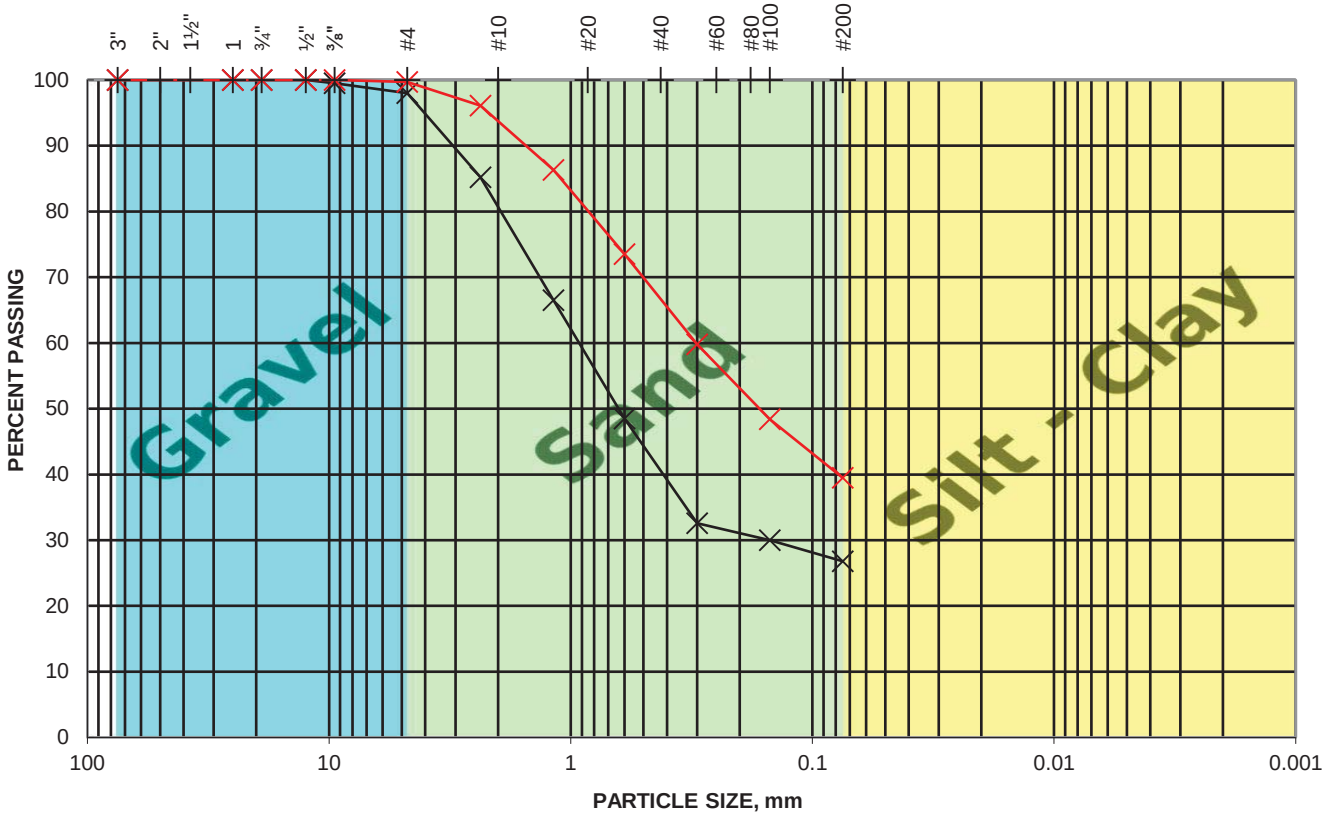
PDT

LABORATORY TEST RESULTS

WAREHOUSE DEVELOPMENT
NORTHWEST CORNER OF
HARVILL AND PLACENIA AVENUES
MEAD VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

NOVEMBER 2019 | PROJECT NO. T2891-22-01 | FIG B-1



SAMPLE ID	SAMPLE DESCRIPTION
B-3 @ 5-10'	SM - Silty Sand with trace gravel
B-4 @ 0-5'	SM - Silty Sand

GEOCON
WEST, INC.
GEOTECHNICAL ENVIRONMENTAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562-7065
PHONE 951-304-2300 FAX 951-304-2392

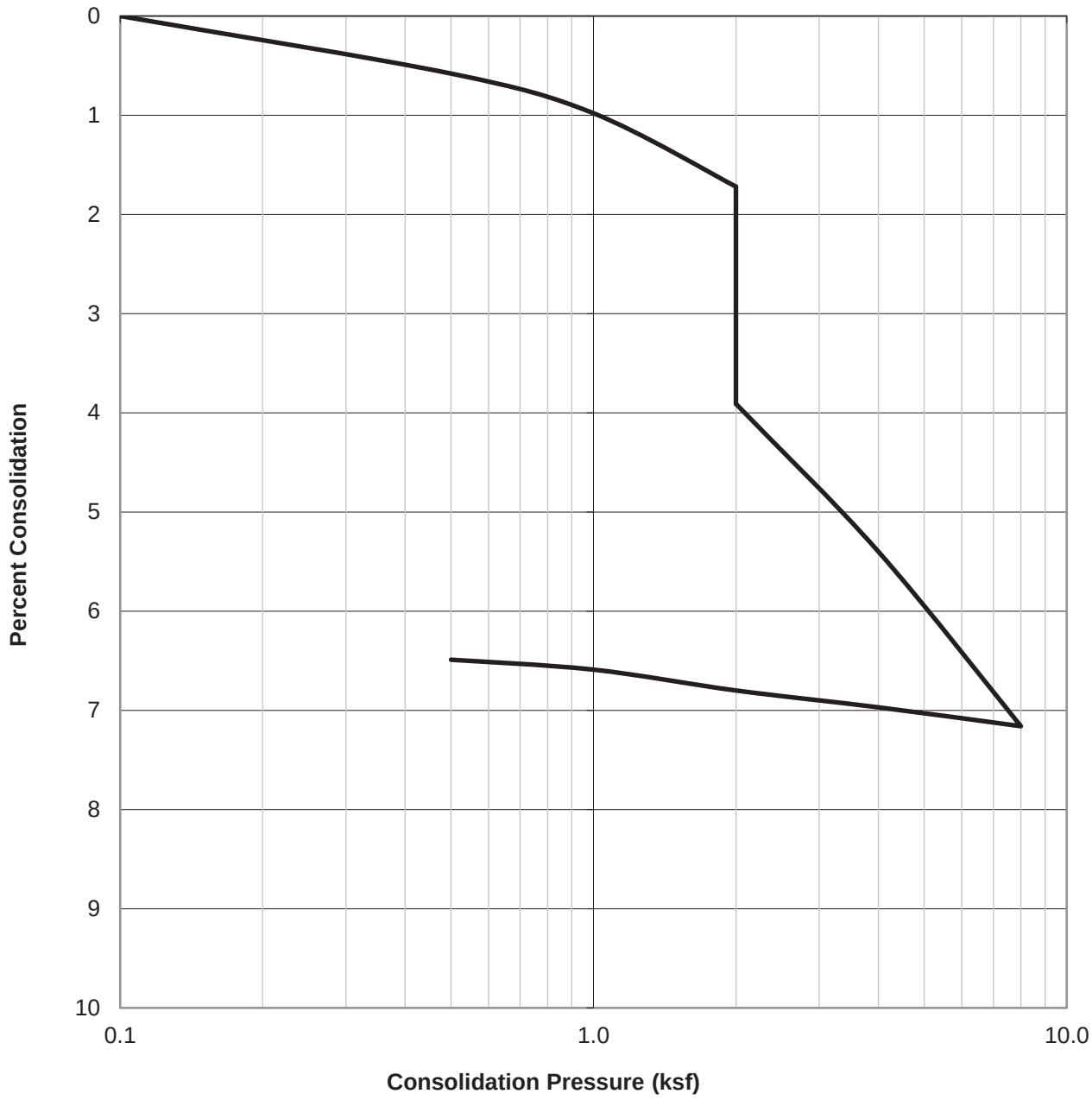
PDT

GRAIN SIZE DISTRIBUTION

WAREHOUSE DEVELOPMENT
NORTHWEST CORNER OF
HARVILL AND PLACENIA AVENUES
MEAD VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

NOVEMBER, 2019 | PROJECT NO. T2891-22-01 | FIG B-2

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@5'	Silty SAND (SM), dark reddish brown	115.7	5.0	13.7

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by:

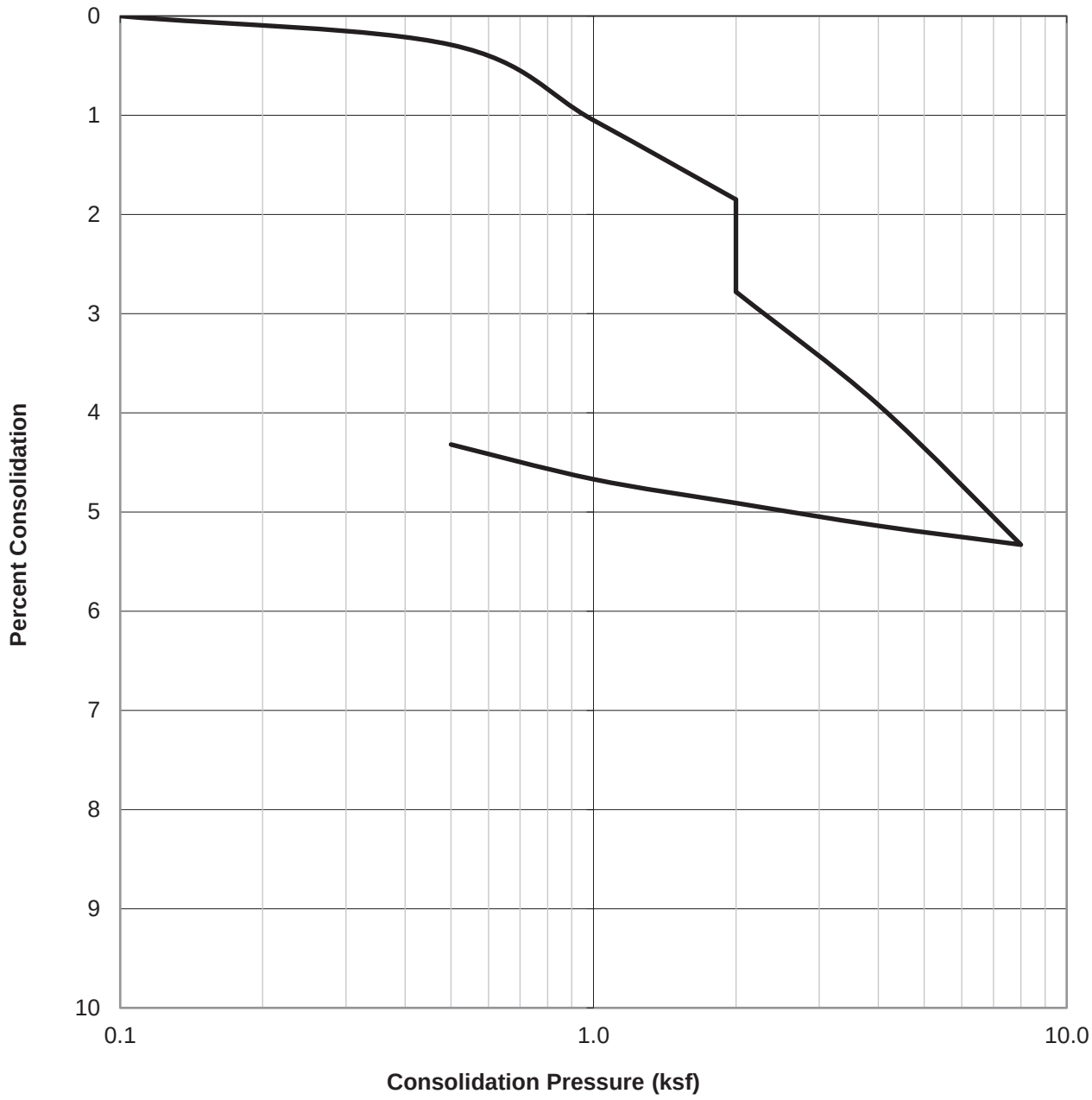
Project No.: T2891-22-01

WAREHOUSE DEVELOPMENT
MEAD VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

Nov 19

Figure B3

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@7.5'	Silty SAND (SM), light reddish brown	128.5	9.1	12.1

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by:

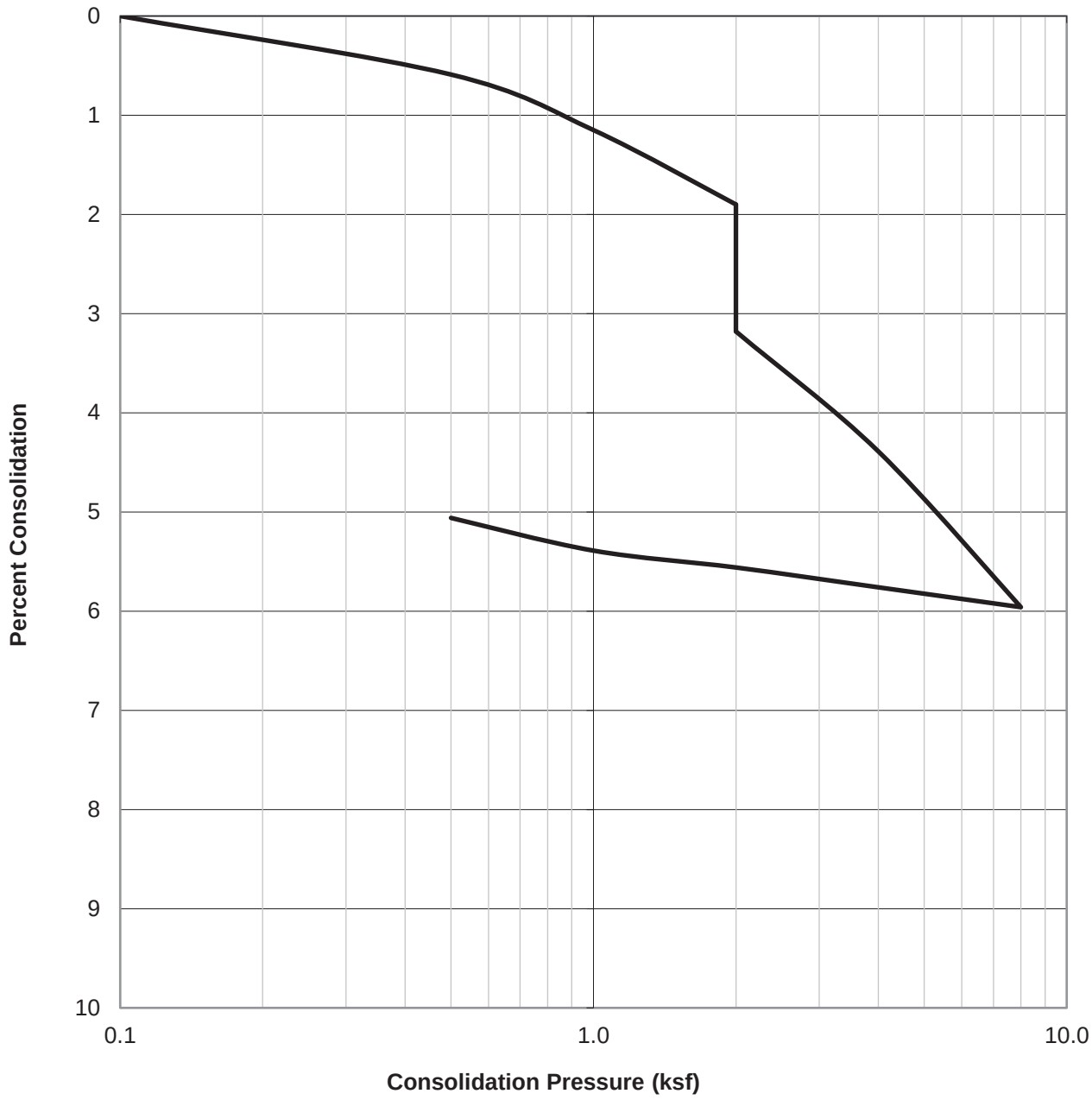
Project No.: T2891-22-01

WAREHOUSE DEVELOPMENT
MEAD VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

Nov 19

Figure B4

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B6@5'	Silty SAND (SM), reddish brown	117.4	4.6	13.3

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

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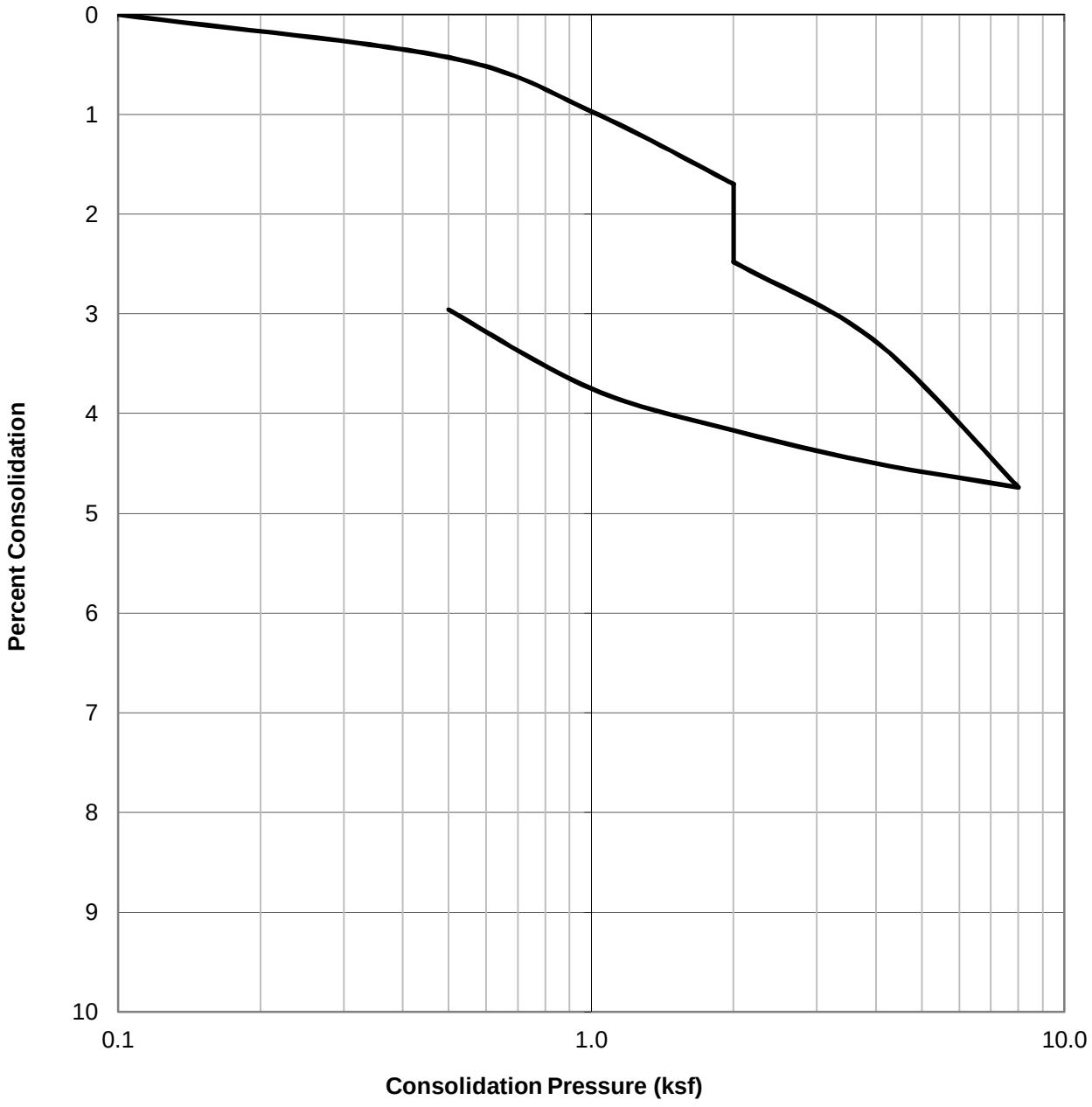
Project No.: T2891-22-01

WAREHOUSE DEVELOPMENT
MEAD VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

Nov 19

Figure B5

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B6@10'	Silty SAND (SM), grayish brown	121.2	11.5	15.4

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

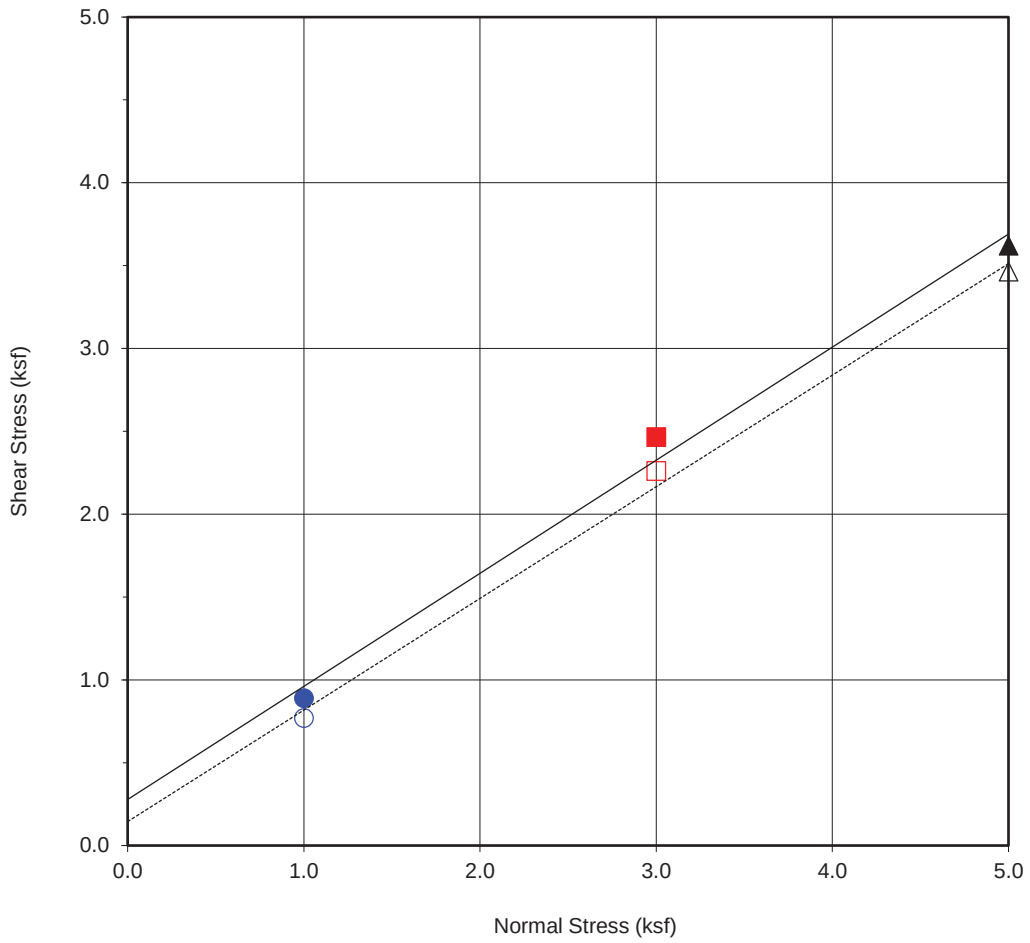
Checked by:

Project No.: T2891-22-01

WAREHOUSE DEVELOPMENT
MEAD VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

Nov 19

Figure B6



Boring No.	**n/a**
Sample No.	**B2@5'**
Depth (ft)	**5'**
Sample Type:	Ring

Soil Identification:		
Silty SAND (SM), dark reddish brown		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	278	34.3
Ultimate	144	34.0

Normal Strest (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.89	■ 2.47	▲ 3.62
Shear Stress @ End of Test (ksf)	○ 0.77	□ 2.26	△ 3.46
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	8.4	8.1	7.1
Initial Dry Density (pcf)	116.5	118.5	118.8
Initial Degree of Saturation (%)	50.6	51.6	45.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	12.1	12.3	10.5

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

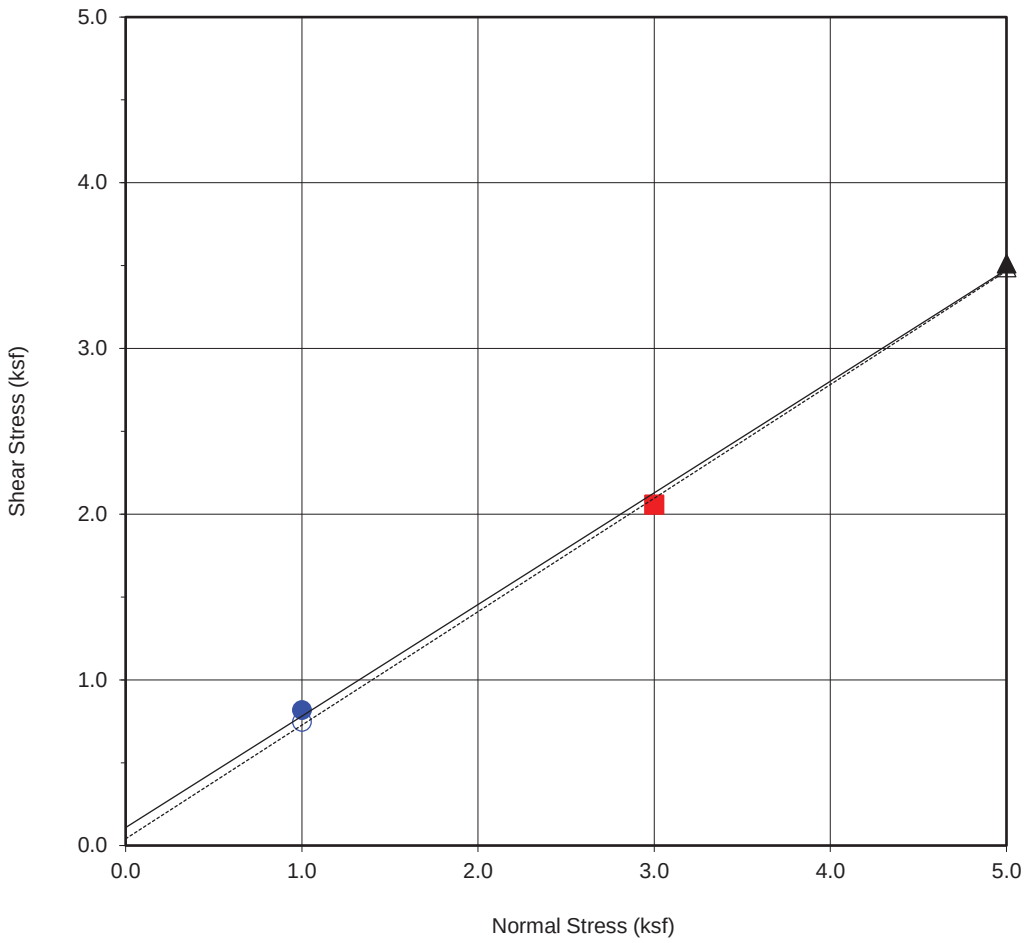
Checked by:

Project No.: T2891-22-01

WAREHOUSE DEVELOPMENT
MEAD VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

Nov 19

Figure B7



Boring No.	**B5**
Sample No.	**B5@0-5'**
Depth (ft)	**0-5**
Sample Type:	Ring

Soil Identification:		
Silty SAND (SM), yellowish brown (REMOLDED)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	108	34.0
Ultimate	40	34.4

Normal Strest (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.82	■ 2.06	▲ 3.51
Shear Stress @ End of Test (ksf)	○ 0.75	□ 2.06	△ 3.49
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	7.0	7.0	7.0
Initial Dry Density (pcf)	121.1	121.0	120.9
Initial Degree of Saturation (%)	48.5	47.7	47.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	10.4	10.6	9.4

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by:

Project No.: T2891-22-01

WAREHOUSE DEVELOPMENT
MEAD VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

Nov 19

Figure B8

APPENDIX C

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

WAREHOUSE DEVELOPMENT
NORTHEAST CORNER OF
HARVILL AND PLACENTIA AVENUES
MEAD VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

PROJECT NO. T2891-22-01

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

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

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

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

2. DEFINITIONS

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

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

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

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

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

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

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

3. MATERIALS

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

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

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

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

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

3.3 Materials used for fill, either imported or on-site, shall not contain hazardous materials as defined by the California Code of Regulations, Title 22, Division 4, Chapter 30, Articles 9

and 10; 40CFR; and any other applicable local, state or federal laws. The Consultant shall not be responsible for the identification or analysis of the potential presence of hazardous materials. However, if observations, odors or soil discoloration cause Consultant to suspect the presence of hazardous materials, the Consultant may request from the Owner the termination of grading operations within the affected area. Prior to resuming grading operations, the Owner shall provide a written report to the Consultant indicating that the suspected materials are not hazardous as defined by applicable laws and regulations.

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

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

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

4. CLEARING AND PREPARING AREAS TO BE FILLED

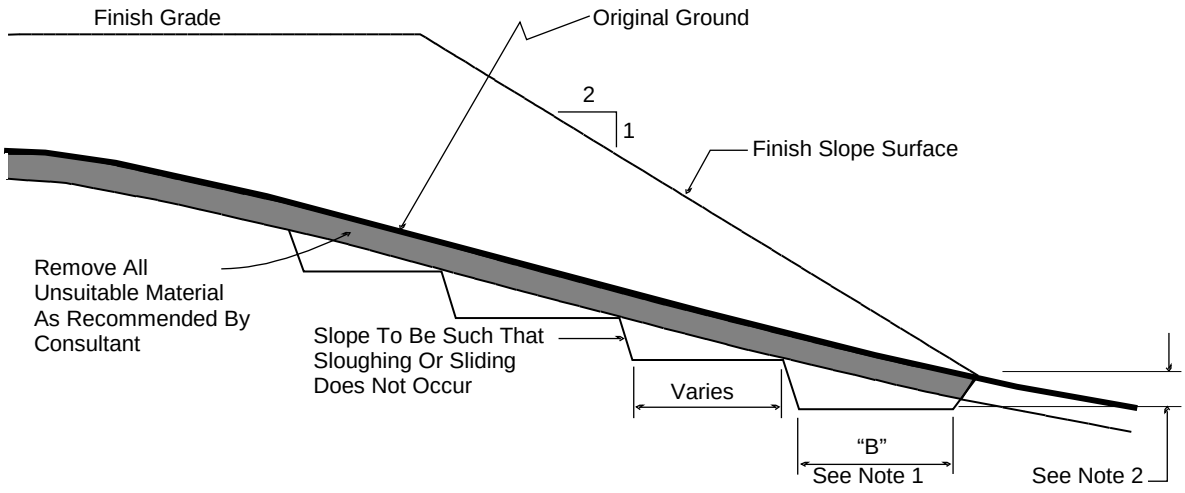
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

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

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

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

TYPICAL BENCHING DETAIL



No Scale

DETAIL NOTES:

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

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

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

5. COMPACTION EQUIPMENT

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

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

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

6.3.3 Plate bearing tests, in accordance with ASTM D 1196, may be performed in both the compacted *soil* fill and in the *rock* fill to aid in determining the required minimum number of passes of the compaction equipment. If performed, a minimum of three plate bearing tests should be performed in the properly compacted *soil* fill (minimum relative compaction of 90 percent). Plate bearing tests shall then be performed on areas of *rock* fill having two passes, four passes and six passes of the compaction equipment, respectively. The number of passes required for the *rock* fill shall be determined by comparing the results of the plate bearing tests for the *soil* fill and the *rock* fill and by evaluating the deflection

variation with number of passes. The required number of passes of the compaction equipment will be performed as necessary until the plate bearing deflections are equal to or less than that determined for the properly compacted *soil* fill. In no case will the required number of passes be less than two.

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

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

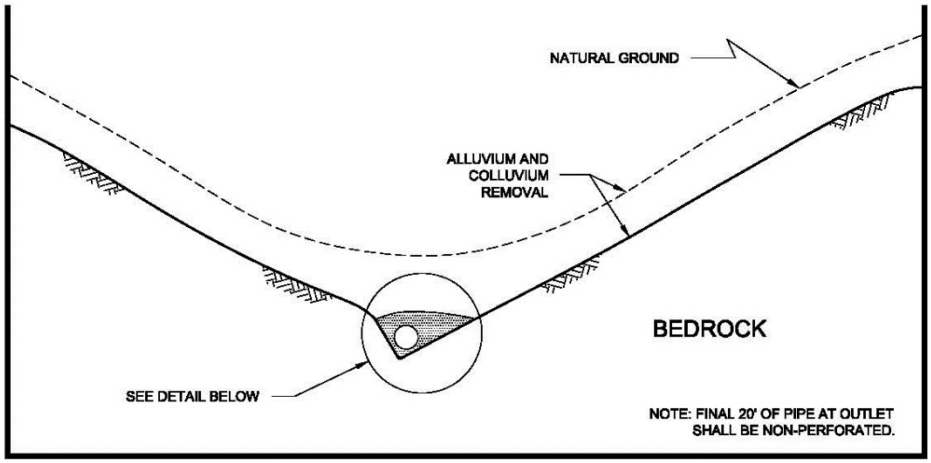
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

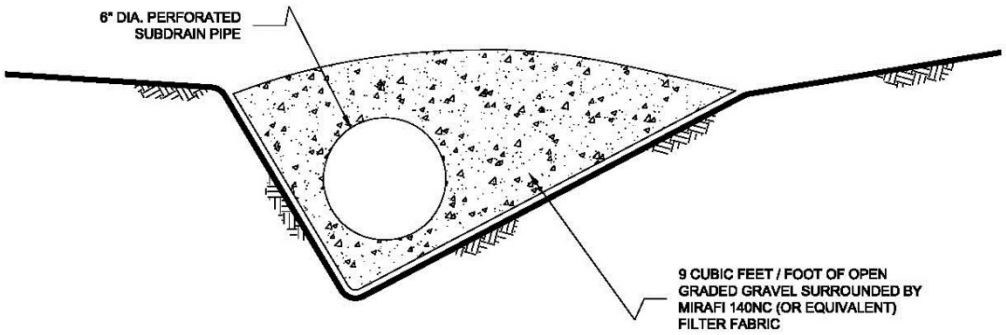
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. SUBDRAINS

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

TYPICAL CANYON DRAIN DETAIL





NOTES:

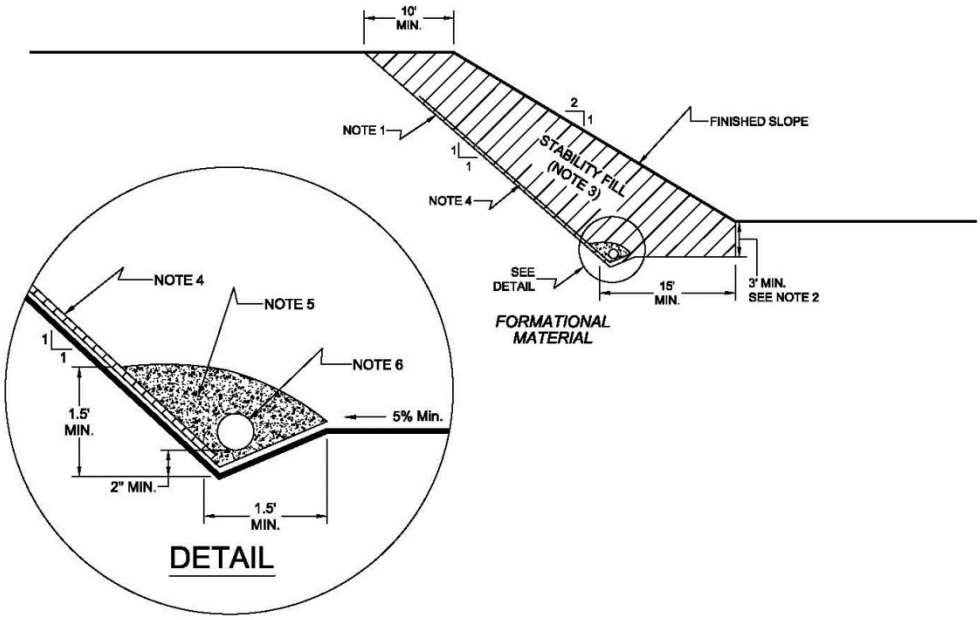
1.....8-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS
IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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 larger) pipes.

TYPICAL STABILITY FILL DETAIL



NOTES:

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

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

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

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

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

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

NO SCALE

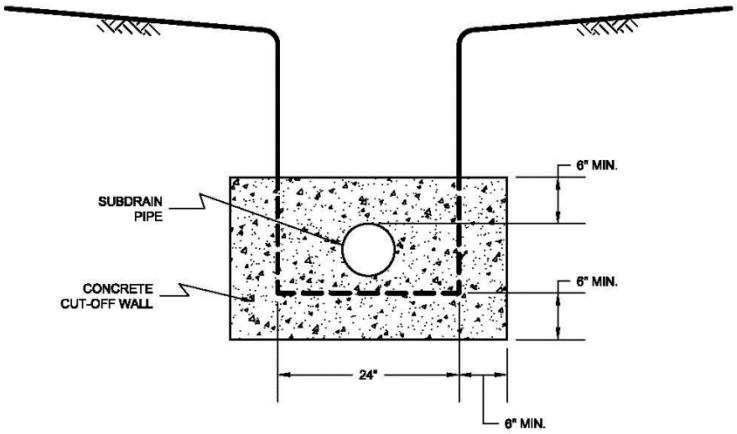
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

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

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

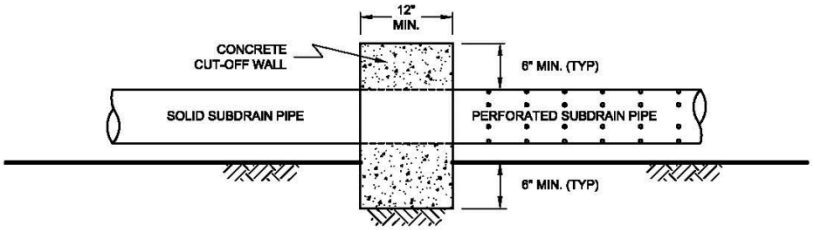
TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



NO SCALE

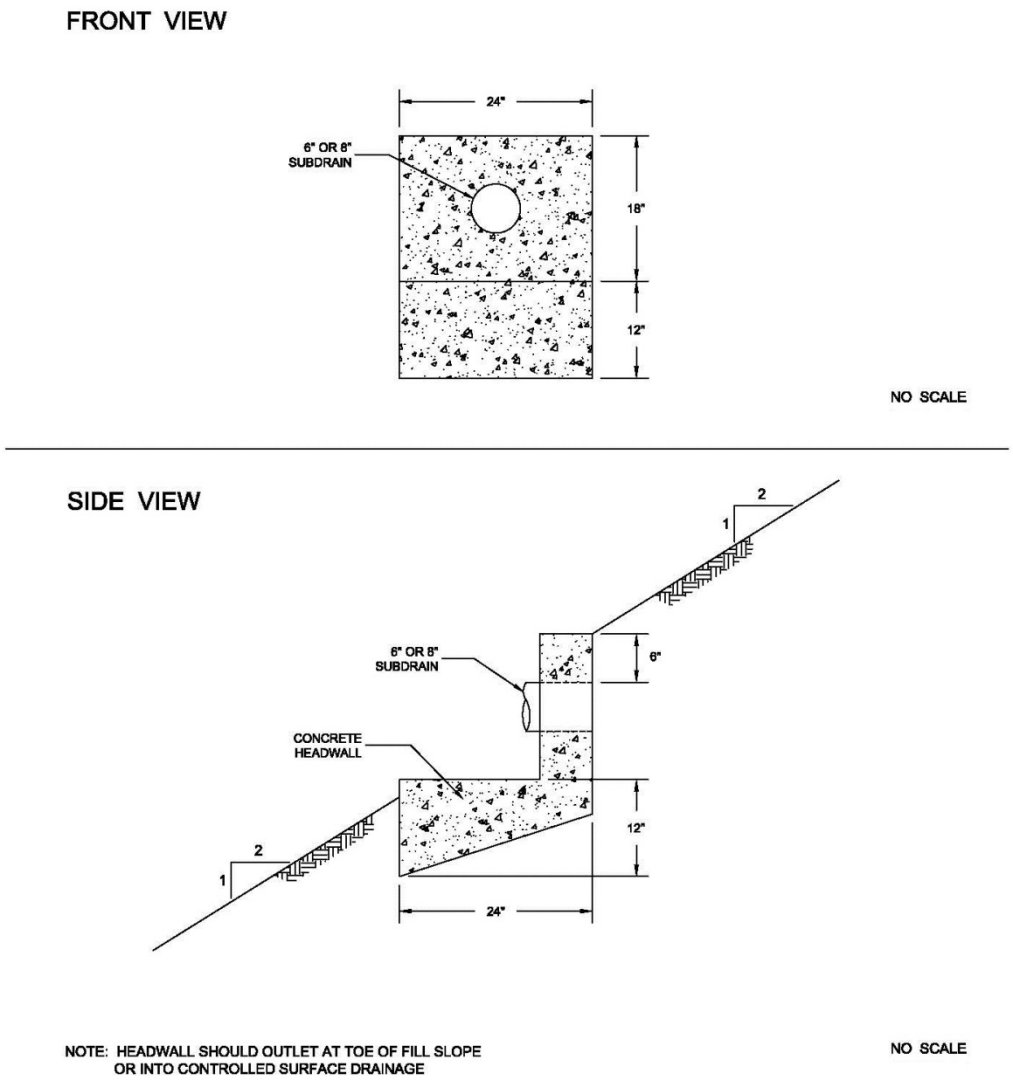
SIDE VIEW



NO SCALE

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

TYPICAL HEADWALL DETAIL



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

8. OBSERVATION AND TESTING

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

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

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

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

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

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

8.6.1 Soil and Soil-Rock Fills:

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

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

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

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

9. PROTECTION OF WORK

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

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

10. CERTIFICATIONS AND FINAL REPORTS

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

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