PRELIMINARY GEOTECHNICAL INVESTIGATION

MIXED USE RESIDENTIAL AND COMMERCIAL DEVELOPMENT
VERDEMONT AREA
SAN BERNARDINO, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

STRATA EQUITY GROUP, INC.
SAN DIEGO, CALIFORNIA

APRIL 20, 2015
PROJECT NO. T2616-22-01

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. T2616-22-01
April 20, 2015

Strata Equity Group, Inc.
4370 La Jolla Village Drive, Suite 960
San Diego, California 92122

Attention: Mr. Eric Flodine

Subject: PRELIMINARY GEOTECHNICAL INVESTIGATION
MIXED USE RESIDENTIAL AND COMMERCIAL DEVELOPMENT
VERDEMONT AREA, SAN BERNARDINO, CALIFORNIA

Dear Mr. Flodine:

In accordance with your authorization of Geocon Proposal IE-1333 dated October 27, 2014, and Work Order Authorization dated April 1, 2015, Geocon West, Inc. (Geocon) herein submits the results of our preliminary geotechnical investigation and percolation testing for the subject mixed use residential and commercial development. The accompanying report presents our findings, conclusions and recommendations pertaining to the geotechnical aspects of the proposed development. The study also includes an evaluation of the geologic units and geologic hazards. The recommendations of this study should be reviewed once project plans are developed. Based on the results of this study, it is our opinion the site is considered suitable for the proposed development provided the recommendations of this report are followed.

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

Very truly yours,

GEOCONWEST, INC.

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PRELIMINARY GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for the proposed mixed used residential and commercial development located in the Verdemont area of San Bernardino, California (see *Vicinity Map*, Figure 1). The purpose of the investigation is to evaluate subsurface soil and geologic conditions at the site and, based on the conditions encountered, provide recommendations pertaining to the geotechnical aspects of developing the property. Development plans are not available at this time. The recommendations of this study should be reviewed once project plans are developed.

The scope of our investigation included review of the previous project report by Levine Fricke (LFR), sequential stereoscopic aerial photographs, geologic mapping, subsurface exploration, percolation testing, laboratory testing, engineering analyses, and the preparation of this report. A summary of the information reviewed for this study is presented in the *List of References*.

Our field investigation included excavation of eleven geotechnical test pits, four percolation tests and two deep percolation excavations. Appendix A presents a discussion of the field investigation and logs of the test pits and percolation test results. The approximate locations of the exploratory excavations are presented on the *Site Map* (Figure 2). We performed laboratory tests on soil samples obtained from the exploratory excavations to evaluate pertinent physical and chemical properties for engineering analysis. The results of the laboratory testing are presented in Appendix B. Geotechnical logs and laboratory test data from the previous geotechnical report by LFR are presented in Appendix C.

References to elevations presented in this report are based on readily available topographic information. Geocon does not practice in the field of land surveying and is not responsible for the accuracy of such topographic information.

2. SITE AND PROJECT DESCRIPTION

The site is an irregularly shaped parcel bounded by Little League Drive on the southeast, Red Sky Avenue, Chestnut Avenue and Irvington Avenue to the northeast, and the Platinum Soccer Complex to the northwest. The Cable Creek Channel runs along the northeast side of the site and crosses through the eastern corner of one of the parcels. An electric transmission line on wooden poles crosses the site from southeast to northwest.

The ALTA/ACSM Land Title Survey by DRC Engineering, Inc. indicates that the site is comprised of parcels with APN Designations of 0261-181-01, 0261-181-13, 0261-181-14, 0261-181-15, and 0261-182-10. The site is located at latitude 34.1958 and longitude -117.3652.

We understand that the site will be developed as a mixed use project with commercial and residential structures. We have assumed that the structures will be either concrete tilt-up or wood frame construction with shallow foundations and concrete slab-on-grade floors. The associated utility, roadway, and flatwork improvements will also be constructed. Infiltration basins are proposed in the southeastern and northwestern portions of the site at depths of five to eight feet below existing grades. Grading or site design documents were not available at the time of this report, however, based on existing grades we anticipate cuts and fills to be on the order of five feet or less.

The site is generally vacant and cleared of vegetation. Our aerial photograph review indicates that the site has been periodically plowed and cleared of vegetation. The original alignment of Cable Creek crossed through the southern portion of the site. The creek was realigned to its current location in about the 1940's in conjunction with grading of the roadway that is now Interstate 215.

Site elevations range from approximately 1765 feet above North American Vertical Datum (NAVD) in the northern end of the site to approximately 1730 feet above sea level at the southern end of the site. The locations and descriptions provided herein are based on a site reconnaissance, our field exploration, and project information provided by the client.

3. BACKGROUND

A geotechnical investigation of the site was performed in 2005 by Levine Fricke (LRF, 2005). They excavated five small diameter geotechnical borings and performed laboratory testing. The boring logs and laboratory test results are presented herein in Appendix C. The locations of the explorations are included on our *Site Map*, Figure 2. They performed the following laboratory tests: in-situ moisture and density testing, maximum density/optimum moisture testing, sieve analyses, direct shear strength, collapse, and corrosion screening. They recommended remedial grading including removal and recompaction of the upper five feet of the site soils. They estimated groundwater on the order of 200 feet below the existing ground surface, and considered the potential for site liquefaction to be low. Their laboratory testing indicated a collapse potential of 0.4 to 0.5 percent in the upper 2 to 11.5 feet when saturated at 1,000 pounds per square foot (psf). They provided a foundation bearing capacity of 6,000 psf for combined live and dead loads, but they did not provide an estimate of settlement or detailed foundation and concrete slab-on-grade recommendations. They also did not provide grading recommendations for earthwork at the site. They recommended that further investigation be performed once the project design has been established.

4. GEOLOGIC SETTING

The project site is located in San Bernardino Valley within the Peninsular Ranges Geomorphic Province. The Peninsular Ranges are bounded on the north by the Transverse Ranges (San Gabriel and San Bernardino Mountains) and on the east by the San Andreas fault. The Peninsular Ranges Province extends southward into Mexico and westward past the Channel Islands. Geologic units within the Peninsular Ranges consist of granitic and metamorphic bedrock highlands and deep and broad alluvial valleys. Specifically, the site is located on an alluvial fan emanating from the San Bernardino Mountains. Several hundred feet of sands with variable amounts of gravels, cobbles, and boulders underlie the site.

5. GEOLOGIC MATERIALS

5.1 General

During our field investigation, we encountered Quaternary-age alluvial deposits and localized previously placed fill. The upper portion of the alluvium has been disturbed by previous grading, clearing or agricultural activities. The descriptions of the soil and geologic conditions are shown on the excavation logs located in Appendix A and described herein in order of increasing age.

5.2 Previously Placed Artificial Fill (Qaf)

Previously placed artificial was encountered in the northeast portion of the site, in Test Pit TP-11. It appears that this fill was placed perhaps in association with the adjacent park grading. As encountered, this unit consists of silty sand that is medium dense, moist, brown, and contained some gravel and cobble. The upper portion of this unit will require remedial grading.

5.3 Quaternary Alluvial Deposits (Qal)

Quaternary-age alluvium is present on the remainder of the site and underlies the site at depth. The soils, as encountered within our excavations, consist of sands and gravels with varying amounts of silt and cobbles. The alluvial deposits are generally medium dense and slightly moist. The upper one to two feet of alluvium was disturbed by previous grading, clearing, or agricultural activities and was loose as a result. The alluvium is considered suitable for support of the proposed site improvements. However, the upper portion of this unit will require remedial grading.

6. GROUNDWATER

We did not encounter groundwater during our exploration to the depths explored of 15.5 feet. Groundwater was not encountered during the LFR investigation in 2005 to a depth of 50.5 feet. The LFR report indicates that groundwater is anticipated to be on the order of 200 feet below the ground surface. It is not uncommon for seepage conditions to develop where none previously existed due to the permeability characteristics of the geologic units encountered. During the rainy season, localized perched water conditions may develop above silt and clay layers that may require special consideration during grading operations. Groundwater elevations are dependent on seasonal precipitation, irrigation, and land use, among other factors, and vary as a result.

7. GEOLOGIC HAZARDS

7.1 Seismic Design Parameters

We used the computer program *U.S. Seismic Design Maps*, provided by the USGS. Table 7.1.1 summarizes site-specific design criteria obtained from the 2013 California Building Code (CBC; Based on the 2012 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 should be designed using a Site Class D. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2013 CBC and Table 20.3-1 of ASCE 7-10. The values presented in Table 7.1.1 are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 7.1.1
2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2013 CBC Reference
Site Class	D	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.375g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	1.152g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.5	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.375g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	1.728g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.583g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	1.152g	Section 1613.3.4 (Eqn 16-40)

Table 7.1.2 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 7.1.2
2013 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.916	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.916g	Section 11.8.3 (Eqn 11.8-1)

Conformance to the criteria in Tables 7.1.1 and 7.1.2 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.

7.2 Liquefaction

Liquefaction typically occurs when a site is located in a zone with seismic activity, onsite soils are cohesionless/silt or clay with low plasticity, static groundwater is encountered within 50 feet of the surface, and soil relative densities are 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. Seismically induced settlement may occur whether the potential for liquefaction exists or not. Groundwater depths are anticipated to be on the order of 200 feet below ground surface. However, perched water may develop along the channel during a storm event. This condition was used in our liquefaction analysis as a conservative estimate of the liquefaction potential, even though the probability of the occurrence of the design earthquake during a significant storm event is unlikely.

The current standard of practice, as outlined in the "Recommended Procedures for Implementation of DMG Special Publication 117A, Guidelines for Analyzing and Mitigating Liquefaction in California" requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. Liquefaction typically occurs in areas where the soils below the water table are composed of poorly consolidated, fine to medium-grained, primarily sandy soil. In addition to the requisite soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to induce liquefaction.

Liquefaction analysis of the soils underlying the site was performed using the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (1996). This program utilizes the 1996 NCEER method of analysis. The liquefaction potential evaluation was performed by utilizing groundwater elevation at the bottom of the adjacent channel, a magnitude 7.0 earthquake, and the site class modified peak horizontal acceleration for the site from the 2013 CBC. This semi-empirical method is based on a correlation between values of Standard Penetration Test (SPT) resistance and field performance data. Data from the previous CPT logs was also used to assist in an evaluation of the potential for liquefaction.

Based on the liquefaction analysis, it is anticipated that some of the alluvial soil layers below the level of the high historic groundwater could be prone to settlement during a seismic event. Our analysis indicates that total settlements on the order of up to 2 inches are anticipated with differential settlements on the order of 1 inch over a horizontal distance of 50 feet.

Given the location of the Cable Creek Channel through the planned development, we evaluated the potential for lateral spreading along the side of the channel. Due to the depth of groundwater at the site, the channel slopes would not be subject to lateral spreading unless the design earthquake occurred concurrently with a significant storm event that caused saturation of the soil beneath the Cable Creek Channel side slopes. Given the unlikely possibility of this occurrence, it is our opinion that lateral spreading is not a design consideration.

7.3 Expansive Soil

The geologic units are anticipated to possess a “very low” expansion potential (Expansion Index of 20 or less) when placed at the finish grades beneath the proposed structures. If expansive soils are encountered, these materials can be selectively graded and placed in the deeper fill areas at least three feet below finished grade elevations in order to allow for the placement of the low expansion material at the finish pad grade. Mixing of the silts with the sands during grading will blend the materials and likely result in a reduced overall expansion potential that the original silts.

7.4 Landslides

There are no hillsides on or adjacent to the site. The San Bernardino Mountains are located approximately 1.1 miles northwest of the site. Therefore, the landslide hazard to the site is not a design consideration.

7.5 Slope Stability

We understand that the proposed grading at the project site does not include significant cut or fill slopes as part of the proposed development. However, the existing Cable Creek Channel has embankment slopes estimated to be on the order of 5 to 15 feet in height. In general, it is our opinion that permanent, graded fill slopes constructed of on-site soils with gradients of 2:1 (horizontal to vertical) or flatter and vertical heights of 15 feet or less will possess Factors of Safety of 1.5 or greater (Figure 3). We should re-evaluate the stability of planned slopes once detailed grading plans are available including topographic information for the Cable Creek Channel. Planned cuts into the existing fill or alluvial materials should be over-excavated and reconstructed with compacted fill. Grading of cut and fill slopes should be designed in accordance with the requirements of the local building codes of the City of San Bernardino and the 2013 California Building Code (CBC).

7.6 Tsunamis and Seiches

A tsunami is a series of long period waves generated in the ocean by a sudden displacement of large volumes of water. Causes of tsunamis include underwater earthquakes, volcanic eruptions, or offshore slope failures. The first order driving force for locally generated tsunamis offshore southern California is expected to be tectonic deformation from large earthquakes (Legg, *et al.*, 2002). The site is located 60 miles from the nearest coastline, therefore, the risk associated with tsunamis is not a design consideration.

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement. The site is not located near to or downstream of a body of water. Therefore the potential of seiches affecting the site or flooding is not a design consideration.

8.0 SITE INFILTRATION

8.1 General

Prior to our percolation testing on the site we contacted San Bernardino Flood Control (SBCF) Planning Engineer to inquire as to the required test method to determine infiltration rates for the site. SBCF referred us to San Bernardino Valley Water District where we attempted to contact their engineering department for the preferred infiltration test method. We did not receive a response from San Bernardino Valley Water District. Therefore, we opted to use a percolation test method commonly used in Riverside County and found in Table 1 Infiltration Basin Option 2 of Appendix A of Riverside County – Low Impact Development BMP Design Handbook (Handbook). We choose not to use the double-ring infiltrometer method for the site because the site soils were sandy and the infiltration rate would likely be too rapid for the testing apparatus to accurately measure the rate. We planned to run the tests in accordance with Section 2.3 Shallow Percolation Test Method. This method requires two percolation tests and one deep (extending 10 feet below percolation test elevation) excavation per basin. Infiltration testing was conducted at each of the proposed infiltration basins. Due to the very granular soil conditions encountered at the test locations, the infiltration rates were too rapid for the test method as described above. Therefore, as a means of determining the infiltration rates, timed intervals were taken to determine the rate that five gallons of water infiltrated into the ground through the prepared test holes.

Site geotechnical conditions as encountered in the excavations consist of Quaternary-age alluvium composed primarily of sands and gravels with varying amounts of silt and cobbles.

Historic well data obtained from the California Department of Water Resources Water Data Library and the Western Municipal Water District Cooperative Well Measuring Program indicate that the depth to ground water in the vicinity of the site is greater than 100 feet.

The site location is depicted on the Vicinity Map, Figure 1. The test pit and percolation test locations are depicted on the *Site Map*, Figure 2. Test pit logs and percolation test data are presented in Appendix A. Descriptions of the proposed basins, the testing procedures, and test results are provided below for each basin.

8.2 Southwest Basin:

The current site elevation in the vicinity of the proposed southwest basin is approximately 1727 feet above MSL. Geocon utilized an extend-a-hoe backhoe to excavate the two percolation test holes (P-1 and P-2) and one deep excavation (TR-1 to depth of 15.5 feet below grade) for the proposed basin. Soils encountered within the excavations consisted primarily of sands and gravels with varying amounts of silt and cobbles. No groundwater or evidence of oxidation-reduction mottling was observed within the deep excavation. The percolation test pits were excavated to 4 and 7 feet below existing grades and an 8-inch diameter test hole was hand excavated an additional 12 inches at the bottom of the test pit. Six-inch and 8-inch diameter PVC pipe was placed in the percolation test holes and approximately 2 inches of gravel was placed at the bottom of the PVC pipe. Gravel backfill was placed outside of the pipe within the excavation. The test locations were pre-saturated with five gallons of water. Two trials were conducted for each test to evaluate if the percolations tests should be run with the Sandy Soil Criteria. However, the water percolated into the ground too fast to run the criteria tests.

Due to the very granular soil conditions encountered at the test locations, the infiltration rates were too rapid for the standardized test method as described above. Therefore, as a means of determining the infiltration rates, timed intervals were taken to determine the rate that five gallons of water infiltrated into the ground through the prepared test holes.

Percolation data sheets are presented at the back of this report (Figures A-14 and A-15). Calculations to convert the percolation test rate to infiltration test rate are based on the Porchet Method as outlined in Section 2.3 of the referenced Handbook are presented in the table below. Please note that the Handbook requires a factor of safety of 3 be applied to these values based on the test method used.

Infiltration Test Rates for Southwest Basin

	P-1	P-2
Soil Type	Sandy	Sandy
Change in head over time: ΔH (in)	18.2	18.2
Time Interval (minutes): Δt (min)	0.52	2.80
Radius of test hole: r (in)	4.5	4.5
Average head over time interval: Havg	9.1	9.1
Tested Infiltration Rate (in/hr): It	419	77

8.3 Eastern Basin:

The current site elevation in the vicinity of the proposed eastern basin is approximately 1714 feet above MSL. Geocon utilized an extend-a-hoe backhoe to excavate the two percolation test holes (P-3 and P-4) and one deep excavation (TR-2 to depth of 15.5 feet below grade) for the proposed basin. Soils encountered within the excavations consisted composed primarily of sands and gravels with varying amounts of silt and cobbles. No groundwater or evidence of oxidation-reduction mottling was observed within the deep excavation. The percolation test pits were excavated to 4 and 7 feet below existing grades and an 8-inch diameter test hole was hand excavated an additional 12 inches at the bottom of the test pit. Six-inch and 8-inch diameter PVC pipe was placed in the percolation test holes and approximately 2 inches of gravel was placed at the bottom of the PVC pipe. Gravel backfill was placed outside of the pipe within the excavation. The test locations were pre-saturated with five gallons of water. Two trials were conducted for each test to evaluate if the percolations tests should be run with the Sandy Soil Criteria. However, the water percolated into the ground too fast to run the criteria tests.

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Infiltration Test Rates for Eastern Basin

	P-3	P-4
Soil Type	Sandy	Sandy
Change in head over time: ΔH (in)	10.2	18.2
Time Interval (minutes): Δt (min)	1.72	1.72
Radius of test hole: r (in)	6.0	4.5
Average head over time interval: Havg	5.1	9.1
Tested Infiltration Rate (in/hr): It	132	126

9. CONCLUSIONS AND RECOMMENDATIONS

9.1 General

9.1.1 It is our opinion that soil or geologic conditions were not encountered during the investigation that would preclude the proposed development of the project provided the recommendations presented herein are followed and implemented during construction.

9.1.2 Potential geologic hazards at the site include seismic shaking and regional ground subsidence. Based on our investigation and available geologic information, active, potentially active, or inactive faults are not present underlying or trending toward the site.

9.1.3 The upper four feet of previously placed fill and alluvium are considered unsuitable for the support of compacted fill or settlement-sensitive improvements based on the dry, loose condition observed during our exploration. Remedial grading of the surficial soil will be required as discussed herein. The alluvium and previously placed fill below a depth of four feet are considered suitable to support additional fill and the proposed structures and improvements.

9.1.4 The test pit excavations performed for this study were backfilled by pushing the soil into the excavation. No moisture conditioning or compactive effort were applied during the backfill process. As such, the test pit locations should be re-excavated during grading and replaced with compacted fill as recommended herein.

9.1.5 The site soils should generally be excavatable with conventional earth moving equipment in good working order. However, much of the site soils have little to no cohesion and are prone to caving. The contractor should take precautionary measures to mitigate caving when excavating into the alluvial materials.

9.1.6 We did not encounter groundwater during our subsurface exploration and we do not expect it to be a constraint to project development. Seepage and perched groundwater conditions may be encountered during the grading operations, especially during the rainy seasons or near the Cable Creek Channel.

9.1.7 In general, slopes should possess calculated factors of safety of at least 1.5 when graded at inclinations of 2:1 (horizontal to vertical), or flatter with maximum heights of 15 feet. Buildings should be set back a horizontal distance of at least 15 feet from the top of the Cable Creek Channel to maintain global stability of the channel slopes. Greater setbacks may be needed to mitigate the potential for erosion of the channel walls if the slope is not protected against erosion with a concrete lining or slope protection rock.

9.1.8 Proper drainage should be maintained in order to preserve the engineering properties of the fill in the sheet-graded pads and slope areas. Recommendations for site drainage are provided herein.

9.2 Soil Characteristics

9.2.1 The soil encountered in the field investigation is considered to be "non-expansive" (Expansion Index [EI] less than 20) as defined by 2013 California Building Code (CBC) Section 1803.5.3. Table 9.2.1 presents soil classifications based on the expansion index.

TABLE 9.2.1
SOIL CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	Expansion Classification	2010 CBC Expansion Classification
0 – 20	Very Low	Non-Expansive
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

9.2.2 The existing fill and alluvium possess a "very low" expansion potential (Expansion Index of 20 or less). Additional testing for expansion potential should be performed once final grades are achieved.

9.2.3 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate content tests are presented in Appendix B and indicate that the on-site materials at the locations tested possess a sulfate content of 0.0003% equating to a S0 or negligible sulfate exposure to concrete structures as defined by 2013 CBC Section 1904.3 and ACI 318. Similar sulfate test results (0.002%) were provided by LFR. Table 9.2.3 presents a summary of concrete requirements set forth by 2013 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 9.2.3
REQUIREMENTS FOR CONCRETE
EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate Exposure	Exposure Class	Water-Soluble Sulfate Percent by Weight	Cement Type	Maximum Water to Cement Ratio by Weight	Minimum Compressive Strength (psi)
Not Applicable	S0	0.00-0.10	--	--	2,500
Moderate	S1	0.10-0.20	II	0.50	4,000
Severe	S2	0.20-2.00	V	0.45	4,500
Very Severe	S3	> 2.00	V+ Pozzolan or Slag	0.45	4,500

9.2.4 Laboratory testing indicates the site soils have a pH of 7.6, possess 66 parts per million chloride, and have a minimum resistivity of 14,000 ohm-cm. The LFR report indicated a pH of 7.43, 85 parts per million chloride, and a minimum resistivity of 10,500 ohm-cm. The site would not be classified as corrosive to metal improvements in accordance with the Caltrans Corrosion Guidelines (Caltrans, 2012).

9.2.5 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.

9.3 Grading

9.3.1 Grading should be performed in accordance with the *Recommended Grading Specifications* contained in Appendix D and the City of San Bernardino Grading Ordinance.

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

9.3.3 Site preparation should begin with the removal of deleterious material, debris 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.

9.3.4 Loose and/or dry previously placed fill and alluvium within the limits of grading should be removed to expose competent alluvium. We anticipate these removals will extend four feet below the existing ground surface across the site and could extend deeper in some areas. The overexcavation should extend to a depth of at least two feet below the planned building foundations. In areas that will be cut to achieve finished grades, the upper four feet of soil should be removed, the bottom scarified and moisture conditioned before replacement with compacted fill. The actual depth of removal should be evaluated by the engineering geologist during grading operations. The bottom of the excavations should be scarified to a depth of at least 1 foot, moisture conditioned as necessary, and properly compacted.

9.3.5 We should observe the removal bottoms to check the exposure of the existing fill or older alluvium. Deeper excavations may be required if dry, loose, or soft materials are present at the base of the removals. Removal bottoms should expose soils which are at least 85 percent of maximum density.

9.3.6 The fill placed within 5 feet of proposed foundations should possess a "low" expansion potential (EI of 50 or less), and be free of rock greater than 6-inches in maximum dimension.

9.3.7 The site should be brought to finish grade elevations with fill compacted in layers. Layers of fill should be no thicker than will allow for adequate bonding and compaction. Fill, including backfill and scarified ground surfaces, should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM International (ASTM) D 1557. Fill placed with in 12 inches of finish subgrade elevations in pavement areas should be compacted to 95 percent of the laboratory maximum dry density. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill.

9.3.8 Import fill (if necessary) should consist of granular materials with a "low" expansion potential (EI of 50 or less) generally 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 evaluate its suitability as fill material.

9.3.9 Fill slopes should be overbuilt at least 2 feet and cut back or be compacted by backrolling with a loaded sheepsfoot roller at vertical intervals not to exceed 4 feet to maintain the moisture content of the fill. The slopes should be track-walked at the completion of each slope such that the fill is compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content to the face of the finished slope. Rock greater than 6-inches in maximum dimension should not be placed with three feet of the slope face.

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

9.4 Earthwork Grading Factors

9.4.1 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, the shrinkage of the site soil is anticipated to be approximately 10 to 15 percent. Please note that 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.

9.5 Settlement of Proposed Fill

9.5.1 The post-grading settlement (hydrocompression) could reach up to 1 inch. We expect the settlement will occur over 20 years depending on the influx of rain and irrigation water into the fill and older alluvium. The settlement will likely be linear from the time the fill is placed to the end of the settlement period depending on the permeability of the fill soil. We do not expect the settlement will impact proposed utilities with gradients of 1 percent or greater. In addition, foundation recommendations are provided herein based on the maximum and differential fill thickness to account for potential fill settlement.

9.6 Commercial Structures - Foundation and Concrete Slabs-On-Grade

9.6.1 The proposed commercial structures can be supported on shallow foundation systems bearing on properly compacted fill soils. Foundations for the structures may consist of either continuous strip footings and/or isolated spread footings. Conventionally reinforced continuous footings should be at least 12 inches wide and extend at least 18 inches below lowest adjacent pad grade. Isolated spread footings should have a minimum width of 2 feet and should extend at least 18 inches below lowest adjacent pad grade. Footings should be dimensioned based on an allowable soil bearing pressure of 4,000 psf. This value may be increased by 300 psf for each additional foot in depth and 200 psf for each additional foot of width to a maximum value of 5,000 psf. The allowable bearing pressure value is for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces. Steel reinforcement for continuous footings should consist of at least four No. 5 steel reinforcing bars placed horizontally in the footings, two near the top and two near the bottom. Steel reinforcement for the spread footings should be designed by the project structural engineer.

9.6.2 Figure 4 presents a wall/column footing dimension detail depicting lowest adjacent pad grade.

9.6.3 Footing excavations should be observed by a representative of Geocon prior to placing reinforcing steel or concrete to verify that the excavations are in compliance with recommendations and the soil conditions are as anticipated.

9.6.4 Building interior floor slabs not anticipated to be subjected to forklift loads should be at least 4 inches thick and be reinforced with No. 3 reinforcing bars placed 24 inches on center, in both directions. The reinforcing bars should be placed on chairs at the slab mid-point.

9.6.5 The minimum reinforcement recommendations are based on soil characteristics only and is not intended to replace reinforcement required for structural considerations.

9.6.6 In accordance with the American Concrete Institute's (ACI) Slab Design Manual, concrete slabs-on-grade may be placed directly above a 15 mil Stego or equivalent liner to control vertical vapor transmission. This method should be considered in lieu of a conventional sand-barrier-sand and/or ¾ rock layering system in order to simplify construction and reduce overall cost. More conservative vapor retardant systems may be warranted beneath slabs where post-grading methane gas testing exceeds regulatory action limits, or, where special moisture-sensitive floor coverings are to be used or moisture-sensitive materials are to be stored. If installed, the vapor retarder design should be consistent with the guidelines presented in the American Concrete Institute’s (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06). In addition, the membrane should be installed in accordance with manufacturer’s recommendations and ASTM requirements and installed in a manner that prevents puncture. The vapor retarder used should be specified by the project architect or developer based on the type of floor covering that will be installed and if the structure will possess a humidity-controlled environment.

9.6.7 If employed, sub-slab bedding sand thickness should be determined by the project foundation engineer, architect, and/or developer. However, we should be contacted to provide recommendations if the bedding sand is thicker than 6 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. We suggest that the foundation engineer present concrete mix design and proper curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plans.

9.6.8 We estimate the total settlements under the imposed allowable loads to be about 1 inch with differential settlements on the order of ½ inch over a horizontal distance of 40 feet.

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

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

- Building footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.
- Although other improvements, which are relatively rigid or brittle, such as concrete flatwork or masonry walls, may experience some distress if located near the top of a slope, it is generally not economical to mitigate this potential. It may be possible, however, to incorporate design measures that would permit some lateral soil movement without causing extensive distress. Geocon should be consulted for specific recommendations.

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

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

9.7 Residential Structures - Foundation and Concrete Slabs-On-Grade

9.7.1 The foundation recommendations presented herein are for proposed residential structures. We separated the foundation recommendations into two categories based on either the maximum and differential fill thickness or Expansion Index. We anticipate the majority of structures will be Category I due to the low expansion potential and anticipated geometry of the underlying fill and alluvial materials. However, the category may be increased to Category II where expansion potential or fill geometry dictates. The foundation category criteria for the anticipated conditions are presented in Table 9.7.1. Geocon should provide additional recommendations if site conditions warrant Foundation Category III. Final foundation categories will be evaluated once site grading has been completed.

TABLE 9.7.1
FOUNDATION CATEGORY CRITERIA

Foundation Category	Maximum Fill Thickness, T (Feet)	Differential Fill Thickness, D (Feet)	Expansion Index (EI)
I	T<20	D<10	EI≤50
II	20≤T<50	10≤D<20	50<EI≤90

9.7.2 The proposed residential structures can be supported on shallow foundation systems bearing on properly compacted fill soils. Foundations for the structures may consist of either continuous strip footings and/or isolated spread footings. Conventionally reinforced continuous footings should be at least 12 inches wide, and isolated spread footings should have a minimum width of 2 feet. Footings should extend at least 12 inches below lowest adjacent pad grade for Category I foundations and at least 18 inches below lowest adjacent pad grade for Category II foundations.

9.7.3 Table 9.7.3 presents minimum foundation and interior concrete slab design criteria for conventional foundation systems.

TABLE 9.7.3
CONVENTIONAL FOUNDATION RECOMMENDATIONS BY CATEGORY

Foundation Category	Minimum Footing Embedment Depth (inches)	Continuous Footing Reinforcement	Interior Slab Reinforcement
I	12	Two No. 4 bars, one top and one bottom	6 x 6 - 10/10 welded wire mesh at slab mid-point
II	18	Four No. 4 bars, two top and two bottom	No. 3 bars at 24 inches on center, both directions

9.7.4 As an alternative to the conventional foundation recommendations, consideration should be given to the use of post-tensioned concrete slab and foundation systems for the support of the proposed structures. The post-tensioned systems should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI), Third Edition, as required by the 2013 California Building Code (CBC Section 1808.6). Although this procedure was developed for expansive soil conditions, we understand it can also be used to reduce the potential for foundation distress due to differential fill settlement. The post-tensioned design should incorporate the geotechnical parameters presented on Table 9.7.4 for the particular Foundation Category designated. The parameters presented in Table 9.7.4 are based on the guidelines presented in the PTI, Third Edition design manual. The foundations for the post-tensioned slabs should be embedded in accordance with the recommendations of the structural engineer.

TABLE 9.7.4
POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS

Post-Tensioning Institute (PTI) Third Edition Design Parameters	Foundation Category	
	I	II
Thornthwaite Index	-20	-20
Equilibrium Suction	3.9	3.9
Edge Lift Moisture Variation Distance, e_M (feet)	5.3	5.1
Edge Lift, y_M (inches)	0.61	1.10
Center Lift Moisture Variation Distance, e_M (feet)	9.0	9.0
Center Lift, y_M (inches)	0.30	0.47

9.7.5 The foundations for the post-tensioned slabs should be embedded in accordance with the recommendations of the structural engineer. A wall/column footing dimension detail is provided on Figure 4. If a post-tensioned mat foundation system is planned, the slab should possess a thickened edge with a minimum width of 12 inches and extend below the clean sand or crushed rock layer.

9.7.6 If the structural engineer proposes a post-tensioned foundation design method other than the 2013 CBC:

- The deflection criteria presented in Table 9.7.4 are still applicable.
- Interior stiffener beams should be used for Foundation Category II.
- The width of the perimeter foundations should be at least 12 inches.

- The perimeter footing embedment depths should be at least 12 inches and 18 inches for foundation categories I and II, respectively. The embedment depths should be measured from the lowest adjacent pad grade.

9.7.7 Our experience indicates post-tensioned slabs are susceptible to excessive edge lift, regardless of the underlying soil conditions. Placing reinforcing steel at the bottom of the perimeter footings and the interior stiffener beams may mitigate this potential. Because of the placement of the reinforcing tendons in the top of the slab, the resulting eccentricity after tensioning reduces the ability of the system to mitigate edge lift. The structural engineer should design the foundation system to reduce the potential of edge lift occurring for the proposed structures.

9.7.8 During the construction of the foundation system, the concrete should be placed monolithically. Under no circumstances should cold joints form between the footings/grade beams and the slab during the construction of the post-tension foundation system.

9.7.9 Category I, or II foundations may be designed for an allowable soil bearing pressure of 4,000 pounds per square foot (psf) (dead plus live load). This bearing pressure may be increased by one-third for transient loads due to wind or seismic forces. We estimate the total settlements under the imposed allowable loads to be about 1 inch with differential settlements on the order of ½ inch over a horizontal distance of 40 feet.

9.7.10 Isolated footings, if present, should have the minimum embedment depth and width recommended above for a particular foundation category. Where this condition cannot be avoided, the isolated footings should be connected to the building foundation system with grade beams.

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

9.7.12 The bedding sand thickness should be determined by the project foundation engineer, architect, and/or developer. However, we should be contacted to provide recommendations if the bedding sand is thicker than 6 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. We suggest that the foundation engineer present concrete mix design and proper curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plans.

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

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

- Building footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.
- Geocon should be contacted to review the pool plans and the specific site conditions to provide additional recommendations, if necessary.
- Swimming pools located within 7 feet of the top of cut or fill slopes are not recommended. Where such a condition cannot be avoided, the portion of the swimming pool wall within 7 feet of the slope face be designed assuming that the adjacent soil provides no lateral support
- Although other improvements, which are relatively rigid or brittle, such as concrete flatwork or masonry walls, may experience some distress if located near the top of a slope, it is generally not economical to mitigate this potential. It may be possible, however, to incorporate design measures that would permit some lateral soil movement without causing extensive distress. Geocon should be consulted for specific recommendations.

9.7.15 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.

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

9.8 Exterior Concrete Flatwork

9.8.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations herein assuming the subgrade materials possess an Expansion Index of 50 or less. Subgrade soils should be compacted to 90 percent relative compaction. Slab panels should be a minimum of 4 inches thick and when in excess of 8 feet square should be reinforced with 6x6-W2.9/W2.9 (6x6-6/6) welded wire mesh or No. 3 reinforcing bars spaced 18 inches center-to-center in both directions to reduce the potential for 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 verified prior to placing concrete. Base materials will not be required below concrete flatwork improvements.

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

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

9.9 Conventional Retaining Walls

9.9.1 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 35 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 55 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 90 or less. For those lots where backfill materials do not conform to the criteria herein, Geocon should be consulted for additional recommendations.

9.9.2 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, an additional uniform pressure of 15H psf should be added to the active soil pressure for walls 10 feet high or less.

9.9.3 The structural engineer should determine the seismic design category for the project. If the project possesses a seismic design category of D, E, or F, the proposed retaining walls should be designed with seismic lateral pressure added to the active pressure. The seismic load exerted on the wall should be a triangular distribution with a pressure of 20H (where H is the height of the wall, in feet, resulting in pounds per square foot [psf]) exerted at the top of the wall and zero at the base of the wall. We used a site modified peak ground acceleration of 0.916g calculated from the 2013 California Building Code and applied a pseudo-static coefficient of 0.33.

9.9.4 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.

9.9.5 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 140 (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. 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 50 or less) with no hydrostatic forces or imposed surcharge load. Figure 5 presents a typical retaining wall drainage detail. If conditions different than those described are expected or if specific drainage details are desired, Geocon should be contacted for additional recommendations.

9.9.6 In general, wall foundations having a minimum depth and width of 1 foot may be designed for an allowable soil bearing pressure of 4,000 psf. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure. Therefore, Geocon should be consulted where such a condition is expected.

9.9.7 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.

9.10 Lateral Loading

9.10.1 To resist lateral loads, a passive pressure exerted by an equivalent fluid weight of 350 pounds per cubic foot (pcf) should be used for the design of footings or shear keys poured neat against formational materials. 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.

9.10.2 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.

9.11 Swimming Pool/Spa

9.11.1 If swimming pools or spas are planned, the proposed swimming pool shell bottom should be designed as a free-standing structure and may derive support in newly placed engineered fill or the competent native alluvium. It is recommended that uniformity be maintained beneath the proposed swimming pools where possible. However, swimming pool foundations may derive support in both engineered fill and undisturbed native alluvium.

9.11.2 Swimming pool foundations and walls may be designed in accordance with the Foundation Design and Retaining Wall Design sections of this report. A hydrostatic relief valve should be considered as part of the swimming pool design unless a gravity drain system can be placed beneath the pool shell.

9.11.3 If a spa is proposed it should be constructed independent of the swimming pool and must not be cantilevered from the swimming pool shell.

9.11.4 If the pool is in proximity to the proposed structure, consideration should be given to construction sequence. If the proposed pool is constructed after building foundation construction, the excavation required for pool construction could remove a component of lateral support from the foundations and would therefore require shoring. Once information regarding the pool location and depth becomes available, this information should be provided to Geocon for review and possible revision of these recommendations.

9.12 Preliminary Pavement Recommendations

9.12.1 The final pavement sections for roadways should be based on the R-Value of the subgrade soils encountered at final subgrade elevation. Streets should be designed in accordance with the City of San Bernardino specifications when final Traffic Indices and R-value test results of subgrade soil are completed. A sample of the site soils exhibited an R-value of 71 when tested in accordance with ASTM D2488. We have used an R-value of 50 for on-site soils and an R-Value of 78 for aggregate base materials for the purposes of this preliminary analysis as Caltrans limits the subgrade R-value to 50. Preliminary flexible pavement sections are presented in Table 9.12.1.

TABLE 9.12.1
PRELIMINARY FLEXIBLE PAVEMENT SECTIONS

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Crushed Aggregate Base (inches)
Roadways servicing light-duty vehicles	5.5	50	3.0	4.0
Roadways servicing heavy truck vehicles	7.0	50	4.0	5.0
Collector	8.0	50	5.0	5.0

9.12.2 The upper 12 inches of the subgrade soil should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content beneath pavement sections.

9.12.3 The crushed aggregated base and asphalt concrete materials should conform to Section 200-2.2 and Section 203-6, respectively, of the *Standard Specifications for Public Works Construction* (Greenbook) and the latest edition of the City of San Bernardino Specifications. Base materials should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content.

Asphalt concrete should be compacted to a density of 95 percent of the laboratory Hveem density in accordance with ASTM D 1561.

9.12.4 A rigid Portland cement concrete (PCC) pavement section should be placed in 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-08 Guide for Design and Construction of Concrete Parking Lots using the parameters presented in Table 9.12.4.

TABLE 9.12.4
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

9.12.5 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 9.12.5.

TABLE 9.12.5
RIGID PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Roadways (TC=C)	6.5
Bus Stops (TC=D)	7.0

9.12.6 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 near to slightly above optimum moisture content. This pavement section is based on a minimum concrete compressive strength of approximately 3,000 psi (pounds per square inch). Base material will not be required beneath concrete improvements.

9.12.7 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., a 9-inch-thick slab would have an 11-inch-thick edge). Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed herein.

9.12.8 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should not exceed 30 times the slab thickness with a maximum spacing of 15 feet for the 7-inch-thick slabs (e.g., a 9-inch-thick slab would have a 15-foot spacing pattern), and should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. The depth of the crack-control joints should be determined by the referenced ACI report.

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

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

9.13 Site Drainage and Moisture Protection

9.13.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2013 CBC 1804.3 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.

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

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

9.13.4 We understand the property may incorporate storm water management devices that promote water storage but not water infiltration. The existing and planned soil conditions are not conducive to water infiltration and infiltration should not be performed. In addition, if water is allowed to infiltrate the soil, seepage may occur through the planned retaining walls and could cause slope instability. Water storage devices can be installed to reduce the velocity and amount of water entering the storm drain system but liners will be required if water in contact with soil.

9.13.5 If not properly constructed, there is a potential for distress to improvements and properties located hydrologically down gradient or adjacent to these devices. 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. Based on our experience with similar clayey soil conditions, infiltration areas are considered infeasible due to the poor percolation and lateral migration characteristics. 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.

9.14 Foundation Plan Review

9.14.1 Geocon should review the structural foundation plans for the project prior to final submittal. Additional analyses may be required after review of the foundation plans.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. 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 materials was not part of the scope of services provided by Geocon.

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

3. 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 are 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.

4. 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.

LIST OF REFERENCES

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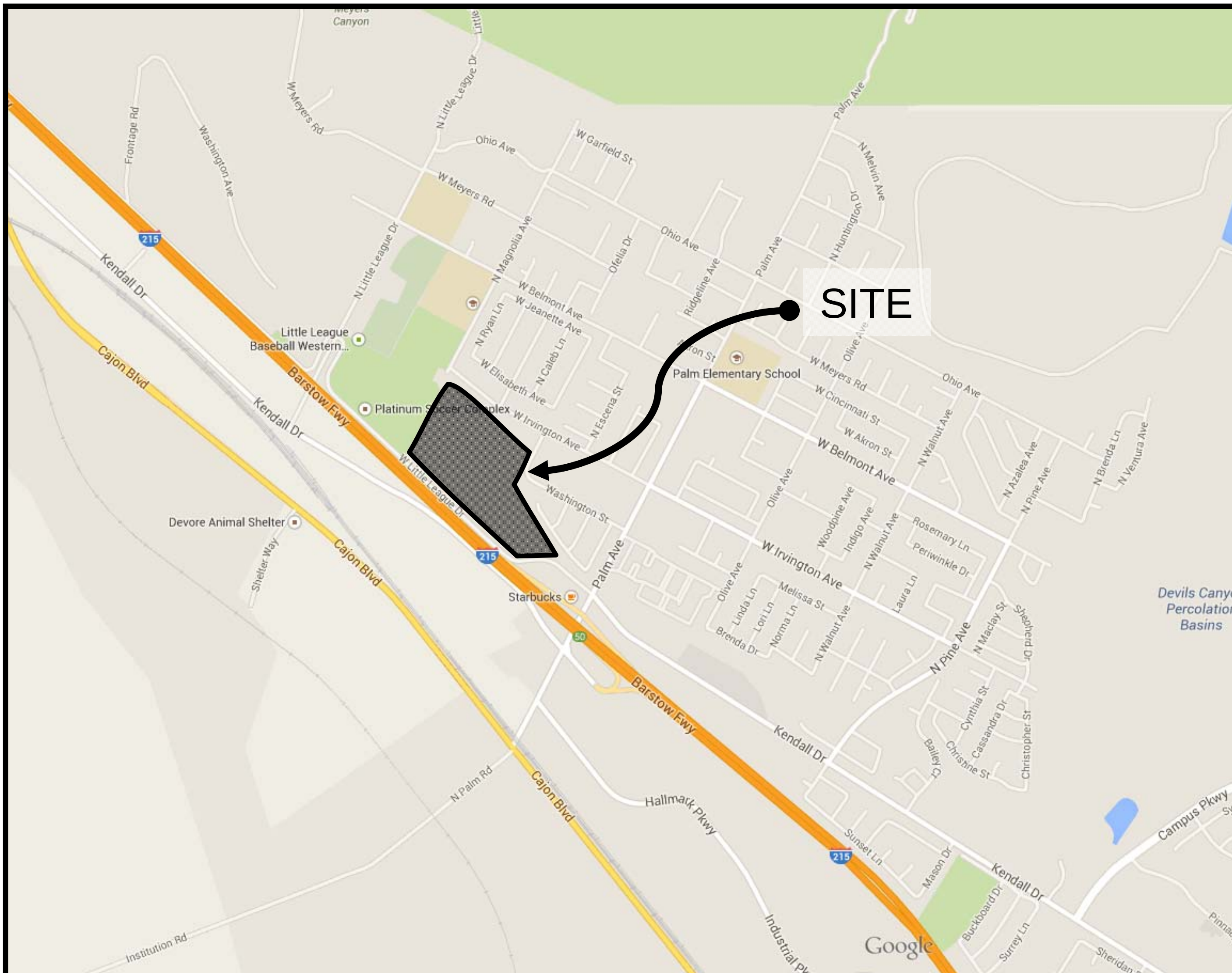

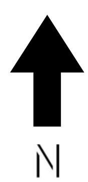

SOURCE: Google Maps, 2014.

1"=2,000'

VICINITY MAP

GEOCON WEST, INC.
GEOTECHNICAL CONSULTANTS
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PHONE 951-304-2300 FAX 951-304-2392

CER		

MIXED USED RESIDENTIAL AND COMMERCIAL DEVELOPMENT
VERDEMONT AREA
SAN BERNARDINO, CALIFORNIA

APRIL, 2015	PROJECT NO. T2616-22-01	FIG. 1

GEOCON LEGEND

TP11 ....APPROX. LOCATION OF GEOCON TEST PIT

B-5 ....APPROX. LOCATION OF LFR BORING (2005)

P-4 ....APPROX. LOCATION OF GEOCON INFILTRATION TEST

TR-2 ....APPROX. LOCATION OF GECON INFILTRATION EXPLORATORY TRENCH

1"=300'

SOURCE: Google Earth, 2014.

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SITE MAP

MIXED USE RESIDENTIAL AND COMMERCIAL DEVELOPMENT
VERDEMONT AREA
SAN BERNARDINO, CALIFORNIA

APRIL , 2015 | PROJECT NO. T2616-22-01 | FIG. 2

ASSUMED CONDITIONS:

SLOPE HEIGHT	H = 15 feet
SLOPE INCLINATION	2.0 : 1.0 (Horizontal : Vertical)
TOTAL UNIT WEIGHT OF SOIL	γ_t = 125 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ = 32 degrees
APPARENT COHESION	C = 150 pounds per square foot
NO SEEPAGE FORCES	

ANALYSIS:

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

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

$\lambda_{c\phi}$	=	7.8	CALCULATED USING EQ. (3-3)
Ncf	=	28	DETERMINED USING FIGURE 10, REFERENCE 2
FS	=	2.2	FACTOR OF SAFETY CALCULATED USING EQ. (3-2)

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SLOPE STABILITY ANALYSIS

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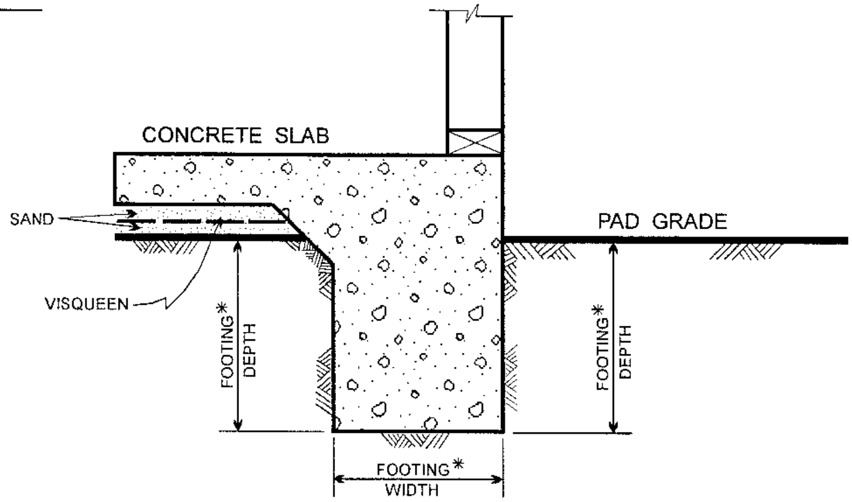
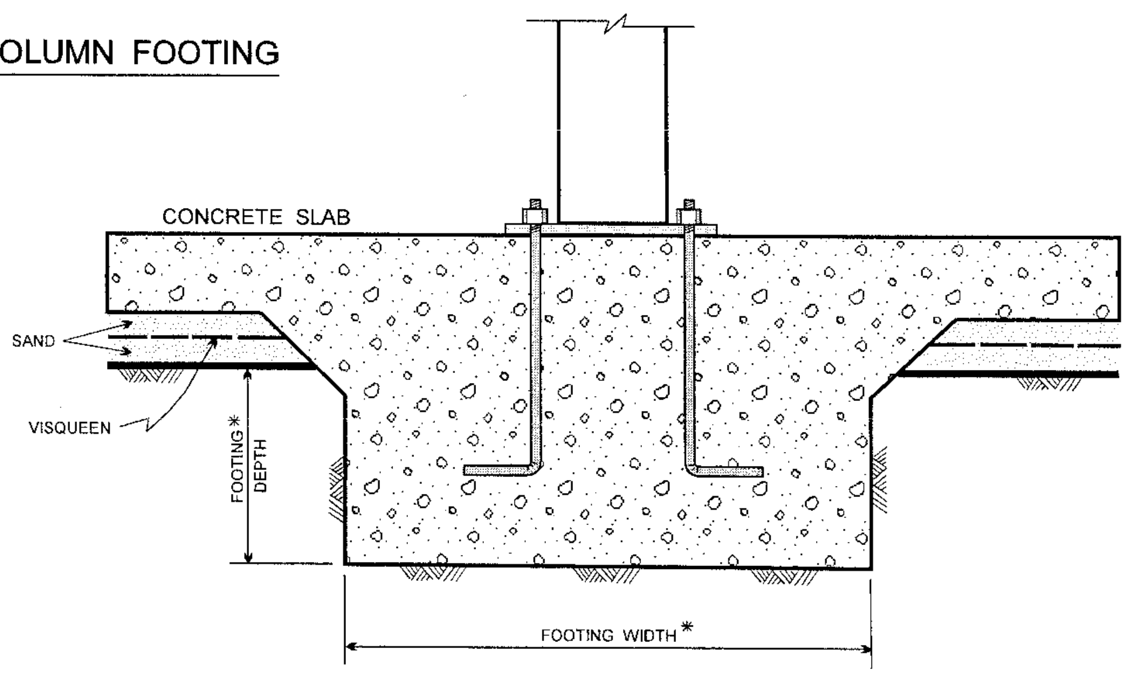
SAN BERNARDINO, CALIFORNIA

APRIL, 2015	PROJECT NO. T2616-22-01	FIG. 3

WALL FOOTING



COLUMN FOOTING



*……SEE REPORT FOR FOUNDATION WIDTH AND DEPTH RECOMMENDATION

NO SCALE

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WALL / COLUMN FOOTING DETAIL

MIXED USE RESIDENTIAL AND
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SAN BERNARDINO, CALIFORNIA

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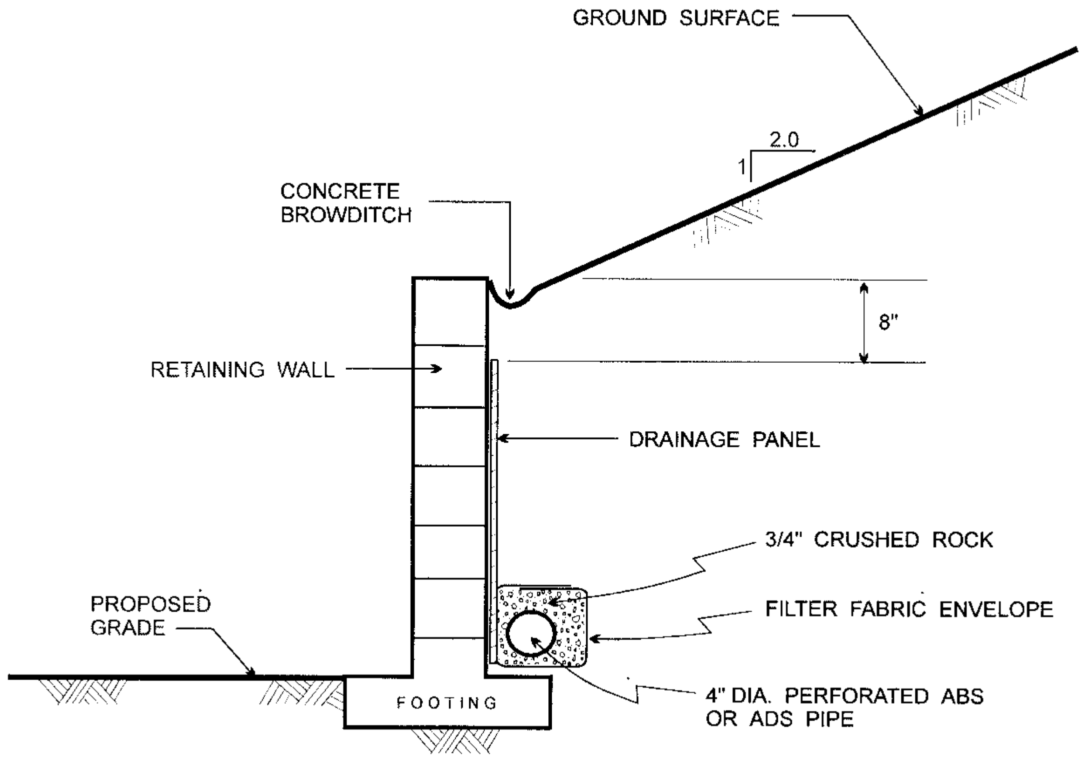
APRIL, 2015	PROJECT NO. T2616-22-01	FIG. 4



NOTES:

1.....WALL DRAINAGE PANELS SHOULD CONSISTS OF MIRADRAIN 6000 OR EQUIVALENT

2......FILTER FABRIC SHOULD CONSIST OF MIRAFI 140N OR APPROVED EQUIVALENT

3......VOLUME OF CRUSHED ROCK SHOULD BE AT LEAST 1 CUBIC FOOT PER FOOT OF PIPE

4......CONCRETE BROWDITCH RECOMMENDED FOR SLOPE HEIGHTS GREATER THAN 6 FEET

NO SCALE

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WALL DRAINAGE DETAL

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VERDEMONT AREA
SAN BERNARDINO, CALIFORNIA

CER			APRIL, 2015	PROJECT NO. T2616-22-01	FIG. 5

APPENDIX A

APPENDIX A

EXPLORATORY EXCAVATIONS

Our subsurface exploration consisted of excavating eleven test pits. We performed the field investigation on November 3, 2014. Percolation testing was performed on April 9, 2015.

The test pits were excavated to depths of up to 15.5 feet to provide exposures of the disturbed surface soil and near surface alluvium. We performed in-situ moisture and density testing of the soils at selected depths with a nuclear moisture/density gauge. We collected representative bag samples of the soils in the test pits. The test pits were loosely backfilled upon completion. These test pit areas should be re-excavated during grading and backfilled with compacted fill. The test pit locations are depicted on the *Site Map*, Figure 2.

We visually examined, classified, and logged the soil conditions encountered in the test pits in general conformance with ASTM International (ASTM) Practice for Description and Identification of Soils (Visual - Manual Procedure D2844). The logs of the test pits are presented on Figures A-1 through A-11 and included herein. The logs depict the various soil types encountered and indicate the depths at which samples were obtained.

Percolation tests were performed within the two proposed basin locations in the southwestern and eastern portions of the site. Tests were performed at depths of five and eight feet in each basin and a 15.5 foot deep excavation was performed to verify no groundwater or impenetrable strata were encountered in the proposed basin areas. Due to the sandy soils at the site we determined that double-ring infiltrometer testing would not be appropriate for the site since the infiltration rate would likely be too high to be measured accurately with the apparatus. Therefore, we planned to perform percolation testing. During pre-saturation, the water infiltrated in less than three minutes. Based on the pre-percolation, the percolation test for sandy soils would not provide sufficient water volume to perform the test. Therefore, we flooded the test locations with five gallons of water and recorded the time for the water to percolate into the soils. Test pit logs and percolation data sheets are presented in Appendix A.

TEST PIT TP-1

ELEV. (MSL.) **1765** DATE COMPLETED **11-3-2014**

EQUIPMENT **BACKHOE** BY: **P. THERIAULT**

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	**ALLUVIUM (Qal)** Silty SAND, loose, slightly moist, brown; sand is fine to coarse; trace gravel; upper 18 inches is disturbed; sparse vegetation			
2				SP	SAND with Gravel, medium dense, dry, grayish brown; sand is fine to coarse; slight caving; some cobbles			
4					-Increase in cobbles			
6								
8								
10	TP1@10-11							3.2

Total depth: 11 feet
No groundwater encountered
Caving from 1.5' to 11'
Backfilled with cuttings 11-3-2014

Figure A-1,
Log of Test Pit TP-1, Page 1 of 1

T2616-22-01 APPENDIX A 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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TEST PIT TP-2

ELEV. (MSL.) **1757** DATE COMPLETED **11-3-2014**

EQUIPMENT **BACKHOE** BY: **P. THERIAULT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SP-SM	**ALLUVIUM (Qal)** SAND with Silt, loose, slightly moist, brown; sand is fine to coarse; some gravel; upper 12 inches is disturbed			
2				SP	SAND with Gravel, medium dense, slightly moist, grayish brown; sand is fine to coarse; some cobbles, caving			
4				GP	GRAVEL with Sand, medium dense, slightly moist, grayish brown; sand is fine to coarse; with cobbles			
6				SP	SAND with Gravel, medum dense, slightly moist, grayish brown; sand is fine to coarse			
8								
10								

Total depth: 10.5 feet
No groundwater encountered
Caving from 1' to 10.5'
Backfilled with cuttings 11-3-2014

Figure A-2,
Log of Test Pit TP-2, Page 1 of 1

T2616-22-01 APPENDIX A 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.

PROJECT NO. T2616-22-01

TEST PIT TP-3

ELEV. (MSL.) **1756** DATE COMPLETED **11-3-2014**

EQUIPMENT **BACKHOE** BY: **P. THERIAULT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SP-SM	**ALLUVIUM (Qal)** SAND with Silt and Gravel, loose, slightly moist, grayish brown; sand is fine to coarse; upper 12 inches is disturbed			
2					-Medium dense, caving		96.3	4.6
					-Boulder			
4								
6	TP3@6-7							2.8
8								
10								
12					Total depth: 12 feet No groundwater encountered Caving from 1' to 12' Backfilled with cuttings 11-3-2014			

Figure A-3,
Log of Test Pit TP-3, Page 1 of 1

T2616-22-01 APPENDIX A 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. T2616-22-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TEST PIT TP-4** ELEV. (MSL.) **1752** DATE COMPLETED **11-3-2014** EQUIPMENT **BACKHOE** BY: **P. THERIAULT**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0 – 8				SP	**ALLUVIUM (Qal)** SAND, loose, slightly moist, grayish brown; sand is fine to coarse; some gravel; upper 12 inches is disturbed, trace gravel -With gravel; medium dense -Caving -Cobbles		98.1	3.4
					Total depth: 9 feet No groundwater encountered Caving from 2' to 9' Backfilled with cuttings 11-3-2014			

Figure A-4,
Log of Test Pit TP-4, Page 1 of 1

T2616-22-01 APPENDIX A 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.

TEST PIT TP-5

ELEV. (MSL.) **1755** DATE COMPLETED **11-3-2014**

EQUIPMENT **BACKHOE** BY: **P. THERIAULT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SP-SM	**ALLUVIUM (Qal)** SAND with Silt, loose, slightly moist, brown; sand is fine to coarse; some gravel; some gravel			
2				SP	SAND with Gravel, medium dense, slightly moist, grayish brown; sand is fine to coarse; trace cobbles; caving		98.8	5.3
4								
6	TP5@5-6							2.6
8								
10					Total depth: 10 feet No groundwater encountered Caving from 2' to 10' Backfilled with cuttings 11-3-2014			

Figure A-5,
Log of Test Pit TP-5, Page 1 of 1

T2616-22-01 APPENDIX A 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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TEST PIT TP-6

ELEV. (MSL.) **1756** DATE COMPLETED **11-3-2014**

EQUIPMENT **BACKHOE** BY: **P. THERIAULT**

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	**ALLUVIUM (Qal)** Silty SAND, loose, slightly moist, brown; sand is fine to coarse; micaceous			
2					-Medium dense; gravel channel, heading N45E			
4					-Some gravel		95.7	4.0
6				SP	SAND, medium dense, slightly moist, grayish brown; sand is fine to coarse; some gravel; trace cobbles; slight caving			
8								
10								
12					Total depth: 12 feet No groundwater encountered Caving from 6' to 12' Backfilled with cuttings 11-3-2014			

Figure A-6,
Log of Test Pit TP-6, Page 1 of 1

T2616-22-01 APPENDIX A 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.

PROJECT NO. T2616-22-01

TEST PIT TP-7

ELEV. (MSL.) **1734** DATE COMPLETED **11-3-2014**

EQUIPMENT **BACKHOE** BY: **P. THERIAULT**

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	**ALLUVIUM (Qal)** Silty SAND, loose, slightly moist, light brown; sand is fine to coarse; trace gravel; upper 12 inches is disturbed -Medium dense			
2	TP7@2-3						93.9	3.0
4				SP	SAND with Gravel, medium dense, slightly moist, grayish brown; sand is fine to coarse; trace cobbles; caving			
6								
8								
10								
12					Total depth: 12 feet No groundwater encountered Caving from 4' to 12'<C>> Backfilled with cuttings 11-3-2014			

Figure A-7,
Log of Test Pit TP-7, Page 1 of 1

T2616-22-01 APPENDIX A 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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TEST PIT TP-8

ELEV. (MSL.) **1735** DATE COMPLETED **11-3-2014**

EQUIPMENT **BACKHOE** BY: **P. THERIAULT**

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	**ALLUVIUM (Qal)** Silty SAND, loose, slightly moist, grayish brown; sand is fine to coarse; trace gravel; upper 12 inches is disturbed			
2					-Medium dense, gravel channel about 6 inches thick and 12 inches wide, trends east-west		95.6	3.3
4				SP	SAND with Gravel, medium dense, slightly moist, grayish brown; sand is fine to coarse; caving			
6								
8					-Moist; some cobbles			
10								
12					Total depth: 12 feet No groundwater encountered Caving from 4' to 12' Backfilled with cuttings 11-3-2014			

Figure A-8,
Log of Test Pit TP-8, Page 1 of 1

T2616-22-01 APPENDIX A 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. T2616-22-01

TEST PIT TP-9

ELEV. (MSL.) **1737** DATE COMPLETED **11-3-2014**

EQUIPMENT **BACKHOE** BY: **P. THERIAULT**

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	**ALLUVIUM (Qal)** Silty SAND, loose, slightly moist, grayish brown; sand is fine to coarse; trace gravel -Medium dense		95.7	6.2
4				SP	SAND, medium dense, moist, light brown; sand is fine to coarse; some gravel; trace cobbles; slight caving			
10	TP9@10-11							2.3

Total depth: 11 feet
No groundwater encountered
Caving from 4' to 11'
Backfilled with cuttings 11-3-2014

Figure A-9,
Log of Test Pit TP-9, Page 1 of 1

T2616-22-01 APPENDIX A 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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DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TEST PIT TP-10 ELEV. (MSL.) 1730 DATE COMPLETED 11-3-2014 EQUIPMENT BACKHOE BY: P. THERIAULT	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SP	**ALLUVIUM (Qal)** SAND, loose, slightly moist, grayish brown; sand is fine to coarse -Medium dense; some gravel			
2							90.8	3.2
					-Some silt; sand is fine to medium			
4					-No silt; sand is fine to coarse			
6								
					-Trace cobbles			
8								
10					Total depth: 10 feet No groundwater encountered Caving from 4' to 10' feet Backfilled with cuttings 11-3-2014			

Figure A-10,
Log of Test Pit TP-10, Page 1 of 1

T2616-22-01 APPENDIX A 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.

PROJECT NO. T2616-22-01

TEST PIT TP-11

ELEV. (MSL.) **1762** DATE COMPLETED **11-3-2014**

EQUIPMENT **BACKHOE** BY: **P. THERIAULT**

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	**ARTIFICIAL FILL (Qaf)** Silty SAND with Gravel, medium dense, moist, brown; sand is fine to coarse; trace cobbles; micaceous			
2	TP11@2-3						106.9	4.3
					-Minor debris			
4								
6								
8								
				SP	**ALLUVIUM (Qal)** SAND with Gravel, medium dense, slightly moist, grayish brown; sand is fine to coarse; some cobbles			
10								
12					Total depth: 12 feet No groundwater encountered Backfilled with cuttings 11-3-2014			

Figure A-11,
Log of Test Pit TP-11, Page 1 of 1

T2616-22-01 APPENDIX A 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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TRENCH TR-1** ELEV. (MSL.) **1752** DATE COMPLETED **04-09-2015** EQUIPMENT **BACKHOE** BY: **D. LIND**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					ALLUVIUM (Qal) **Silty SAND, fine to coarse, loose, moist, brown; stratified below distrubed zone in upper 1+/- foot; trace gravel**			
2								
					SAND, fine to coarse, medium dense, moist, grayish brown; stratified; trace gravel; some caving below 3 feet			
4								
6					cobbles present			
8					more abundanced gravel and cobbles (<6" diameter)			
					Sand, fine to coarse, medium dense, brown gray brown; some caving; gravel and cobbles some caving			
10								
12								
14								
					Total depth: 15.5 feet No groundwater mottling encountered Caving from 3' to 15.5' Backfilled with native soil			

Figure A-12,
Log of Trench TR-1, Page 1 of 1

T2616-22-01 APPENDIX A 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.

PROJECT NO. T2616-22-01

TRENCH TR-2

ELEV. (MSL.) **1735** DATE COMPLETED **04-09-2015**

EQUIPMENT **BACKHOE** BY: **D. LIND**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ALLUVIUM (Qal)** Silty SAND, fine to course, loose, slightly moist, grayish brown; upper 12" disturbed			
2					Gravel SAND (channel incision), loose to medium dense, slightly moise, grayish brown			
4					SAND with gravel, fine to coarse, moist, medium dense, brown; stratified			
6					SAND with gravel, fine to coarse, moist, medium dense, brown gray brown; stratified; some cobbles (<6" diameter) present			
8								
10					more cobbles			
12								
14								

Total depth: 15.5 feet
No groundwater or mottling encountered
Caving for 4' to 15.5'
Backfilled with native soil

Figure A-13,
Log of Trench TR-2, Page 1 of 1

T2616-22-01 APPENDIX A 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.

Percolation Test Data

Proj.	Verdemont								
Proj. No.	T2616-22-01								
Date	4/9/2015								
Test No.	P-1								
Depth of Test:	8	feet							
Test Diameter:	9	in							
Test Radius:	4.5	in							
Hole Area:	63.6	in2							
5 gallon bucket tests									
Time	**Time**	**del T**	**ΔT**	**Vol**	**Vol**	**ΔV**	**Head**	**Havg**	**It**
(H:M:S)	**(H:M:S)**	**(H:M:S)**	**(min)**	**(gal)**	**(in3)**	**(in3)**	**(in)**	**(in)**	**(in/hr)**
11:20:00 AM	0:00:00	0:00:00		5	1155	0	18.2		
11:20:29 AM	0:00:29	0:00:29	0.48	0	0	1155	0	9.1	448
11:22:00 AM	0:00:00	0:00:00		5	1155		18.2		
11:22:31 AM	0:00:31	0:00:31	0.52	0	0	1155	0	9.1	419
Proj.	Verdemont								
Proj. No.	T2616-22-01								
Date	4/9/2015								
Test No.	P-2								
Depth of Test:	5	feet							
Test Diameter:	9	in							
Test Radius:	4.5	in							
Hole Area:	63.6	in2							
5 gallon bucket tests									
Time	**Time**	**del T**	**ΔT**	**Vol**	**Vol**	**ΔV**	**Head**	**Havg**	**It**
(H:M:S)	**(H:M:S)**	**(H:M:S)**	**(min)**	**(gal)**	**(in3)**	**(in3)**	**(in)**	**(in)**	**(in/hr)**
10:57:00 AM	0:00:00	0:00:00		5	1155	0	18.2		
10:59:45 AM	0:02:45	0:02:45	2.75	0	0	1155	0	9.1	79
11:07:00 AM	0:00:00	0:00:00		5	1155		18.2		
11:09:48 AM	0:02:48	0:02:48	2.80	0	0	1155	0	9.1	77
Proj.	Verdemont								
Proj. No.	T2616-22-01								
Date	4/9/2015								
Test No.	P-3								
Depth of Test:	5	feet							
Test Diameter:	12	in							
Test Radius:	6	in							
Hole Area:	113.1	in2							
5 gallon bucket tests									
Time	**Time**	**del T**	**ΔT**	**Vol**	**Vol**	**ΔV**	**Head**	**Havg**	**It**
(H:M:S)	**(H:M:S)**	**(H:M:S)**	**(min)**	**(gal)**	**(in3)**	**(in3)**	**(in)**	**(in)**	**(in/hr)**
1:44:00 PM	0:00:00	0:00:00		5	1155	0	10.2		
1:45:40 PM	0:01:40	0:01:40	1.67	0	0	1155	0	5.1	136
1:48:00 PM	0:00:00	0:00:00		5	1155		10.2		
1:49:43 PM	0:01:43	0:01:43	1.72	0	0	1155	0	5.1	132
Proj.	Verdemont								

Percolation Test Data

Proj. No.	T2616-22-01								
Date	4/9/2015								
Test No.	P-4								
Depth of Test:	8	feet							
Test Diameter:	9	in							
Test Radius:	4.5	in							
Hole Area:	63.6	in2							
5 gallon bucket tests									
Time (H:M:S)	**Time (H:M:S)**	**del T (H:M:S)**	**ΔT (min)**	**Vol (gal)**	**Vol (in3)**	**ΔV (in3)**	**Head (in)**	**Havg (in)**	**It (in/hr)**
2:00:00 PM	0:00:00	0:00:00		5	1155	0	18.2		
2:01:40 PM	0:01:40	0:01:40	1.67	0	0	1155	0	9.1	130
2:05:00 PM	0:00:00	0:00:00		5	1155		18.2		
2:06:43 PM	0:01:43	0:01:43	1.72	0	0	1155	0	9.1	126

APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in general accordance with test methods of ASTM International (ASTM), California test (CT) methods or other suggested procedures. Selected samples were tested for direct shear strength, expansion characteristics, moisture density relationships, corrosivity, R-value, and moisture content. The results of the laboratory tests are summarized in Figures B1 through B3.

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

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
TP11 @ 2-3’	Silty Sand	133.0	7.8

SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS ASTM D4829

Sample No.	Moisture Content		Dry Density (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
TP7 @ 2-3’	8.4	14.5	113.5	1

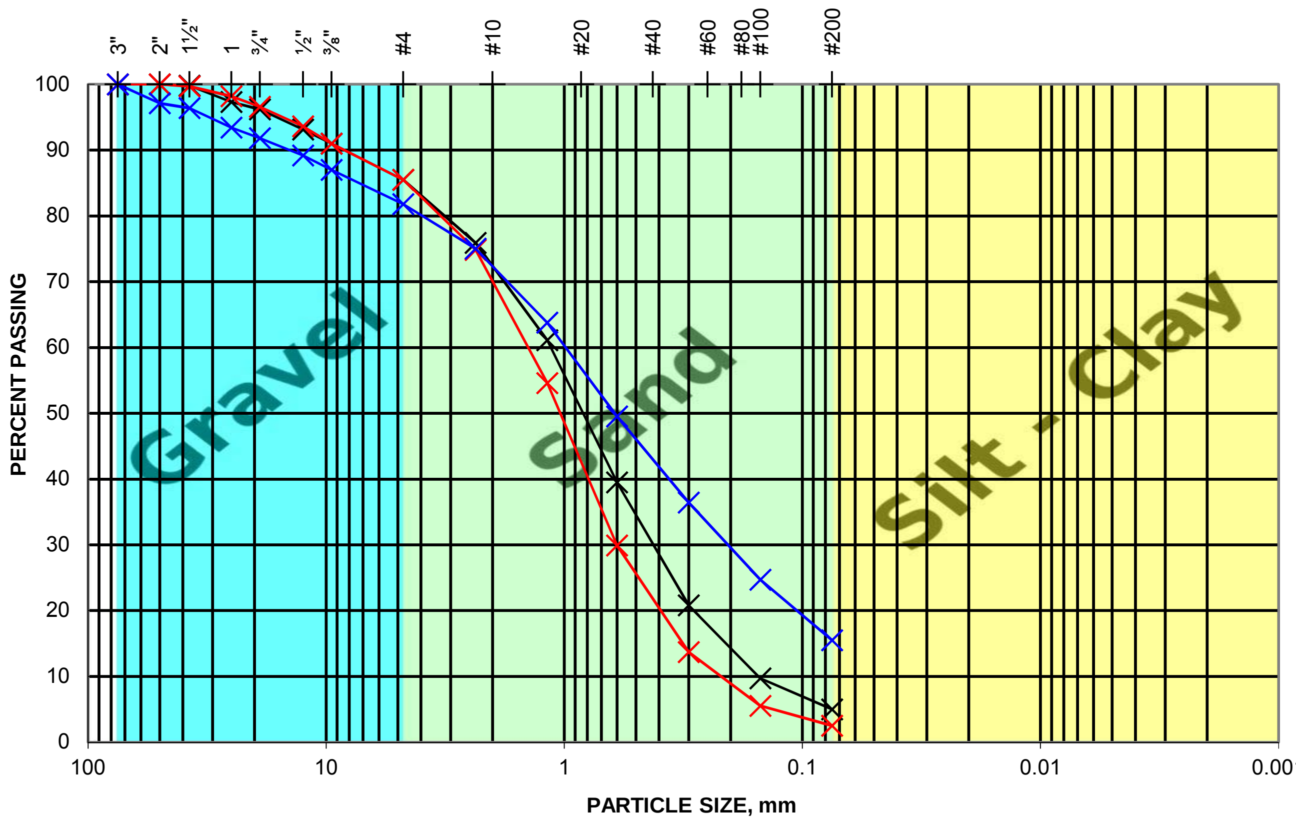
SUMMARY OF CHEMICAL TEST RESULTS

Sample No.	Chloride Content (ppm)	Sulfate Content (%)	pH	Resisitivity (ohm centimeters)
TP7 @ 2-3’	66	0.0003	7.6	14,000

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

SUMMARY OF LABORATORY R-VALUE TEST RESULTS ASTM D2844

Sample No.	R-Value
TP7 @ 2-3’	71



SAMPLE ID	SAMPLE DESCRIPTION
TP3 @ 6-7'	SP-SM SAND with Silt and Gravel
TP5 @ 5-6'	SP - SAND with Gravel
TP11 @ 2-3'	SM - Silty SAND with Gravel

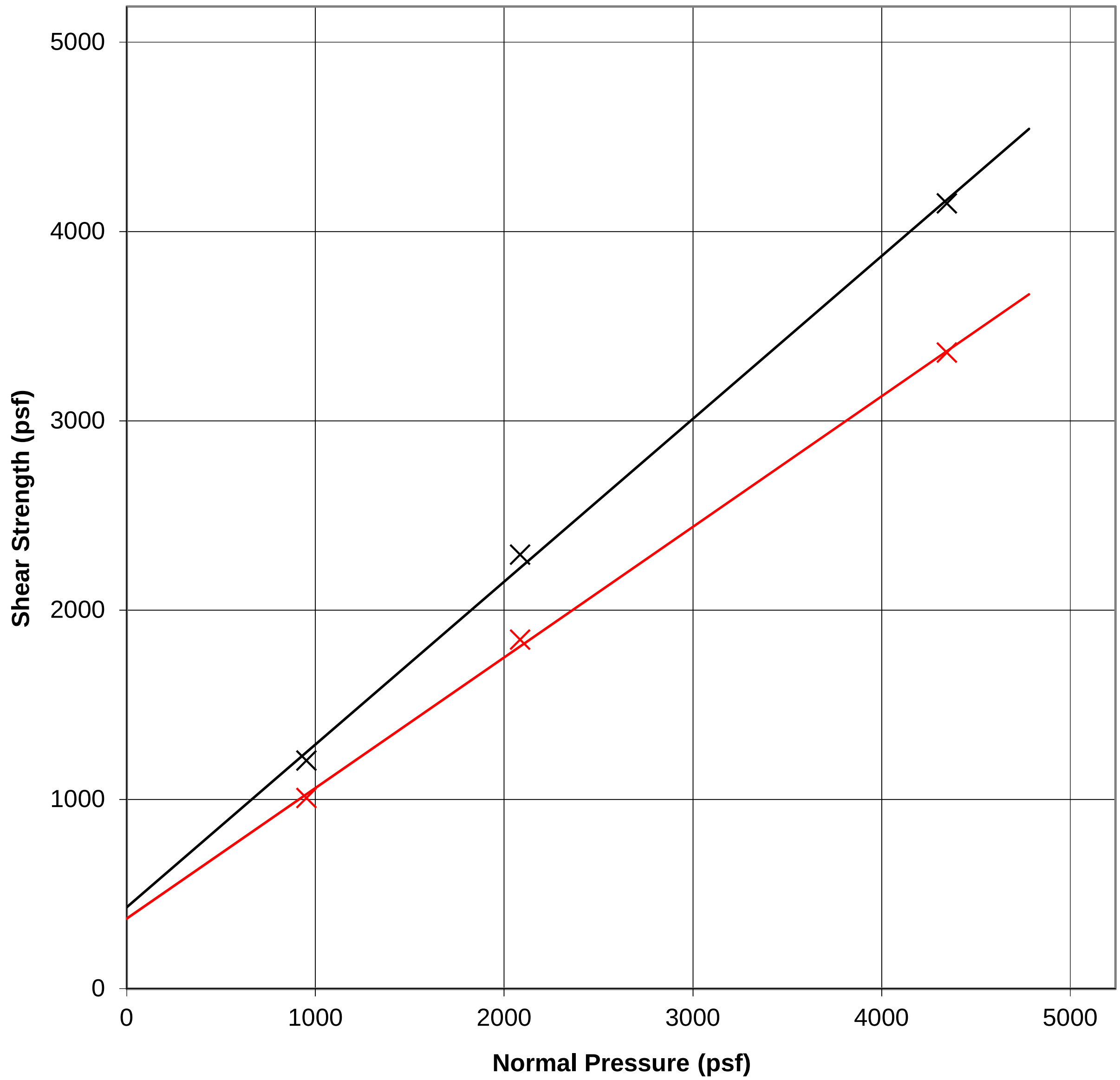
GEOCON
WEST, INC.
GEOTECHNICAL CONSULTANTS
41571 CORNING PLACE SUITE 101 MURRIETA, CA 92562-7065
PHONE 951-304-2300 FAX 951-304-2392

CER		

GRAIN SIZE DISTRIBUTION

MIXED USE RESIDENTIAL AND
COMMERCIAL DEVELOPMENT
VERDEMONT AREA
SAN BERNARDINO, CALIFORNIA

NOVEMBER, 2013	PROJECT NO. T2616-22-01	FIG B2



SAMPLE ID	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)	C (psf)	ϕ (deg)
*TP5 @ 5-6'	SM	120.0	7.4	12.1	430	41
*TP11 @ 2-3'	CL	120.2	7.2	12.3	370	35

*sample remolded to approximately 90% of the maximum dry density

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

CER

DIRECT SHEAR TEST RESULTS

MIXED USE RESIDENTIAL AND
COMMERCIAL DEVELOPMENT
VERDEMONT AREA
SAN BERNARDINO, CALIFORNIA

NOVEMBER, 2013 | PROJECT NO. T2616-22-01 | FIG B3

APPENDIX C

APPENDIX C

BORING LOGS AND LABORATORY TEST DATA
LFR, 2005

FOR

MIXED USE RESIDENTIAL AND COMMERCIAL DEVELOPMENT
VERDEMONT AREA
SAN BERNARDINO, CALIFORNIA

PROJECT NO. T2616-22-01

APPENDIX A

Field Exploration and Boring Logs

FIELD EXPLORATION AND BORING LOGS

Five exploratory borings were drilled for geotechnical purposes at the locations shown on the site plan and boring location map (Figure 2). The borings were approximately 20 to 50 feet deep and were drilled on August 19, 2005, using a hollow-stem-auger drill rig operated by 2R Drilling, Inc., of Corona, California. Mr. Chris Nardi, P.E., G.E., LFR Senior Associate Geotechnical Engineer, observed the drilling and sampling operations and logged the borings in the field.

Relatively undisturbed samples of soils encountered in the borings were obtained using a Modified California drive sampler (2½-inch inner diameter) or Standard Penetration Test sampler. The Modified California drive sampler was lined with thin brass tubes. The sampler was driven into the soil with an automatic hammer with the equivalent energy of a 140-pound hammer falling 30 inches. The samplers were driven 18 inches, in most cases, and the blow counts were recorded for the bottom 12 inches of driving. The resulting blow counts are presented at the corresponding sample location on the boring logs.

When the Modified California drive sampler was withdrawn from the boring, the tubes containing the soil samples were removed and carefully sealed to preserve the natural moisture content of the soil. The samples were then delivered to the laboratory for further examination and testing. Preliminary visual soil classifications were made in the field and were verified by further inspection of the samples in the laboratory and by test results. Boring logs were prepared from the field and laboratory data and are included in this appendix. The borings were backfilled with drill cuttings after completion of the borehole.

The test borings were located in the field by pace and compass methods and should be considered approximate.

LITHOLOGY / SAMPLING DATA / LABORATORY DATA

Depth, feet	Visual Description	ID of Samples (Depth)	Penetration Rate (Blows/ft)	Moisture Content (%)	Dry Density (pcf)	Unconfined Compressive Strength (psf)
0	SILTY SAND (SP-SM), medium dense, brown, fine- to coarse-grained, moist, little fine to coarse gravel to 2 inches, subangular to subrounded	B-1-1 (2-2.5)	26			
4	SILTY SAND (SM), loose, dark brown, fine- to medium-grained, wet, trace gravel					
6	SAND (SP), loose to medium dense, brown, fine- to coarse-grained, moist, trace silt and gravel	B-1-2 (6-6.5)	16	2	105	
10	less gravel	B-1-3 (11-11.5)	21			
14	SILTY SAND (SM), medium dense, brown, fine- to medium-grained, wet	B-1-4 (15-16.5)	14			
20	SILTY SAND (SP-SM), loose to medium dense, dark brown, medium- to coarse-grained, moist, trace gravel	B-1-5 (20-21.5)	20			
26	with granitic gravel to 1-1/2 inches, few cobbles	B-1-6 (26-26.5)	73	3	130	
30	less coarse-grained sand and gravel 30-31 feet	B-1-7 (30-31.5)	30			
34	trace cobbles					

Continued

EXPLANATION

- Clay (CL/CH)
- Silt (ML/MH)
- Sand (SP/SW)
- Gravel (GP/GW)
- Modified California Sampler (3.0-inch O.D. unless noted)
- Standard Penetration Test Split Spoon Sampler

Shaded Areas Indicate Samples Retained for Geotechnical Analysis
Hammer Wt./ Drop: 140 lb /30 inches
Hammer Type: Automatic

LFR Field Staff: C. Nardi
Date Drilled: 8/19/05
Approved by: C. Nardi

Drilling Method: Hollow-Stem Auger (8-in.-OD)
Drilling Company: 2R Drilling, Inc.
∇ Approximate Groundwater Level

LITHOLOGY AND SAMPLE DATA FOR SOIL BORING B-1

LFR LEVINE•FRICKE
Project No. 001-09424-00

HREG / Verdemont Towne Center

9/2/05 C. Nardi/ddi

GEOTECHLFREMV3_DD1 HREGVERD.GPJ 9/5/05

LITHOLOGY | SAMPLING DATA | LABORATORY DATA

Depth, feet	Visual Description	ID of Samples (Depth)	Penetration Rate (Blows/ft)	Moisture Content (%)	Dry Density (pcf)	Unconfined Compressive Strength (psf)
	SILTY SAND (SP-SM), loose to medium dense, dark brown, medium- to coarse-grained, moist, granitic gravel to 1-1/2 inches, trace cobbles (continued)	B-1-8 (35.5-36)	50/4"			
	increase in cobbles 36-38 feet					
40		B-1-9 (40-41)	50/5"			
	increase in coarse-grained sand and cobbles					
	becomes dense to very dense					
45		B-1-10 (45-45.5)	50/3"			
50		B-1-11 (50-50.5)	50/5"			

Bottom of boring = 50.5 feet bgs
No free water encountered
Boring backfilled with compacted cuttings

LFR Field Staff: C. Nardi
Date Drilled: 8/19/05
Approved by: C. Nardi

Drilling Method: Hollow-Stem Auger (8-in.-OD)
Drilling Company: 2R Drilling, Inc.
∇ Approximate Groundwater Level

EXPLANATION

Clay (CL/CH)
Silt (ML/MH)
Sand (SP/SW)
Gravel (GP/GW)

Modified California Sampler (3.0-inch O.D. unless noted)
Standard Penetration Test Split Spoon Sampler
Shaded Areas Indicate Samples Retained for Geotechnical Analysis
Hammer Wt./ Drop: 140 lb /30 inches
Hammer Type: Automatic

LITHOLOGY AND SAMPLE DATA FOR SOIL BORING B-1 (CONTINUED)

Depth, feet	LITHOLOGY: Visual Description	SAMPLING DATA: ID of Samples (Depth)	SAMPLING DATA: Penetration Rate (Blows/ft)	LABORATORY DATA: Moisture Content (%)	LABORATORY DATA: Dry Density (pcf)	LABORATORY DATA: Unconfined Compressive Strength (psf)
0	SILTY SAND (SM), loose, dark brown, fine- to coarse-grained, dry, trace gravel to 1 inch					
3		B-2-1 (3-3.5)	16	3	111	
4	SILTY SAND (SP-SM), medium dense, light brown to brown, fine- to coarse-grained, moist, trace fine to coarse gravel to 2 inches, subangular to subrounded					
6		B-2-2 (6-6.5)	22			
11	less coarse-grained sand and gravel	B-2-3 (11-11.5)	28	3	110	
13	SANDY SILT (ML), medium dense, dark reddish brown, fine-grained, moist to wet, trace clay, micaceous					
16		B-2-4 (16-16.5)	23			
20		B-2-5 (20-21.5)	19			
21	SILTY SAND (SP-SM), medium dense, brown, fine- to coarse-grained, moist, trace gravel					
25		B-2-6 (25-26.5)	12			

Bottom of boring = 26.5 feet bgs
No free water encountered
Boring backfilled with compacted cuttings

LFR Field Staff: C. Nardi
Date Drilled: 8/19/05
Approved by: C. Nardi

Drilling Method: Hollow-Stem Auger (8-in.-OD)
Drilling Company: 2R Drilling, Inc.
∇ Approximate Groundwater Level

EXPLANATION

- Clay (CL/CH)
- Silt (ML/MH)
- Sand (SP/SW)
- Gravel (GP/GW)
- Modified California Sampler (3.0-inch O.D. unless noted)
- Standard Penetration Test Split Spoon Sampler
- Shaded Areas Indicate Samples Retained for Geotechnical Analysis
- Hammer Wt./ Drop: 140 lb /30 inches
- Hammer Type: Automatic

LITHOLOGY AND SAMPLE DATA FOR SOIL BORING B-2

Depth, feet	LITHOLOGY: Visual Description	SAMPLING DATA: ID of Samples (Depth)	SAMPLING DATA: Penetration Rate (Blows/ft)	LABORATORY DATA: Moisture Content (%)	LABORATORY DATA: Dry Density (pcf)	LABORATORY DATA: Unconfined Compressive Strength (psf)
	SILTY SAND (SP-SM), medium dense, light brown, fine- to coarse-grained, dry, trace fine gravel					
		B-3-1 (2-2.5)	33			
5	less coarse-grained sand and gravel, trace cobbles	B-3-2 (5.5-6)	20	4	118	
	SAND (SW), medium dense, light brown to brown, fine- to coarse-grained, dry					
10		B-3-3 (11-11.5)	39			
15		B-3-4 (15-16.5)	21			
20	becomes medium dense to dense	B-3-5 (20-21.5)	36			

Bottom of boring = 21.5 feet bgs
No free water encountered
Boring backfilled with compacted cuttings

LFR Field Staff: C. Nardi
Date Drilled: 8/19/05
Approved by: C. Nardi

Drilling Method: Hollow-Stem Auger (8-in.-OD)
Drilling Company: 2R Drilling, Inc.
▽ Approximate Groundwater Level

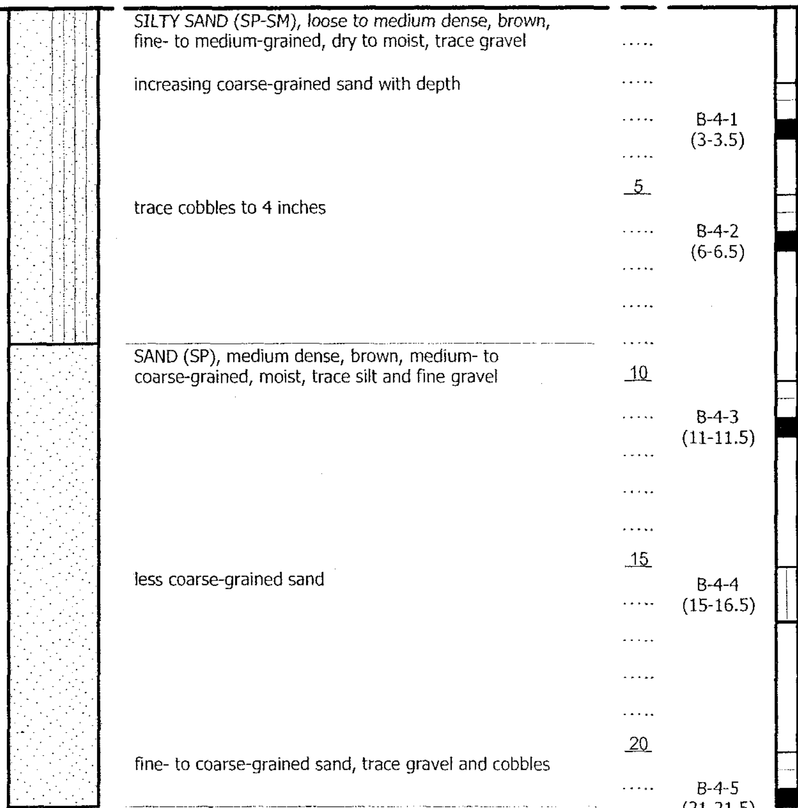
EXPLANATION

Clay (CL/CH)
Silt (ML/MH)
Sand (SP/SW)
Gravel (GP/GW)

Modified California Sampler (3.0-inch O.D. unless noted)
Standard Penetration Test Split Spoon Sampler
Shaded Areas Indicate Samples Retained for Geotechnical Analysis
Hammer Wt./ Drop: 140 lb /30 inches
Hammer Type: Automatic

LITHOLOGY AND SAMPLE DATA FOR SOIL BORING B-3

LITHOLOGY			SAMPLING DATA		LABORATORY DATA		
Depth, feet	Graphic Log	Visual Description	ID of Samples (Depth)	Penetration Rate (Blows/ft)	Moisture Content (%)	Dry Density (pcf)	Unconfined Compressive Strength (psf)
0		SILTY SAND (SP-SM), loose to medium dense, brown, fine- to medium-grained, dry to moist, trace gravel					
		increasing coarse-grained sand with depth					
			B-4-1 (3-3.5)	17	5	102	
5		trace cobbles to 4 inches	B-4-2 (6-6.5)	31			
		SAND (SP), medium dense, brown, medium- to coarse-grained, moist, trace silt and fine gravel					
10			B-4-3 (11-11.5)	31	5	108	
15		less coarse-grained sand	B-4-4 (15-16.5)	18			
20		fine- to coarse-grained sand, trace gravel and cobbles	B-4-5 (21-21.5)	66			
		Bottom of boring = 21.5 feet bgs *No free water encountered* Boring backfilled with compacted cuttings					



EXPLANATION

LFR Field Staff: C. Nardi

Date Drilled: 8/19/05

Approved by: C. Nardi

Drilling Method: Hollow-Stem Auger (8-in.-OD)

Drilling Company: 2R Drilling, Inc.

∇ Approximate Groundwater Level

Clay (CL/CH)

Silt (ML/MH)

Sand (SP/SW)

Gravel (GP/GW)

Modified California Sampler (3.0-inch O.D. unless noted)

Standard Penetration Test Split Spoon Sampler

Shaded Areas Indicate Samples Retained for Geotechnical Analysis

Hammer Wt./ Drop: 140 lb /30 inches

Hammer Type: Automatic

LITHOLOGY AND SAMPLE DATA FOR SOIL BORING B-4

LFR LEVINE• FRICKE

Project No. 001-09424-00

HREG / Verdemont Towne Center

9/2/05 C. Nardi/ddi

GEOTECHLFREMV3_DDI HREGVERD.GPJ 9/5/05

LITHOLOGY		SAMPLING DATA		LABORATORY DATA		
Depth, feet	Visual Description	ID of Samples (Depth)	Penetration Rate (Blows/ft)	Moisture Content (%)	Dry Density (pcf)	Unconfined Compressive Strength (psf)
0	SILTY SAND (SM), loose, brown, fine-grained, moist to wet, trace coarse-grained sand					
	SANDY SILT (ML) layer, loose, dark brown, moist to wet	B-5-1 (2-2.5)	11			
4	SILTY SAND (SP-SM), loose to medium dense, grayish brown, fine- to coarse-grained, dry to moist, trace gravel, interbedded seams of sandy silt					
6		B-5-2 (6-6.5)	13	7	111	
11		B-5-3 (11-11.5)	30	4	110	
15		B-5-4 (15-16.5)	24			
19	SILTY TO CLAYEY SAND (SM/SC), loose, dark brown, fine- to medium-grained sand, moist to wet, trace to little clay					
20		B-5-5 (20-21.5)	7			
25	cobble	B-5-6 (No Recovery)	50/1"			
27	SILTY SAND (SM), medium dense, brown, fine- to medium-grained, moist, trace gravel and cobbles					
30		B-5-7 (30-31)	50/6"			

Bottom of boring = 31.0 feet bgs
No free water encountered
Boring backfilled with compacted cuttings

EXPLANATION

Clay (CL/CH)
Silt (ML/MH)
Sand (SP/SW)
Gravel (GP/GW)

Modified California Sampler (3.0-inch O.D. unless noted)
Standard Penetration Test Split Spoon Sampler
Shaded Areas Indicate Samples Retained for Geotechnical Analysis
Hammer Wt./ Drop: 140 lb /30 inches
Hammer Type: Automatic

LFR Field Staff: C. Nardi
Date Drilled: 8/19/05
Approved by: C. Nardi

Drilling Method: Hollow-Stem Auger (8-in.-OD)
Drilling Company: 2R Drilling, Inc.
∇ Approximate Groundwater Level

LITHOLOGY AND SAMPLE DATA FOR SOIL BORING B-5

LFR LEVINE·FRICKE
Project No. 001-09424-00

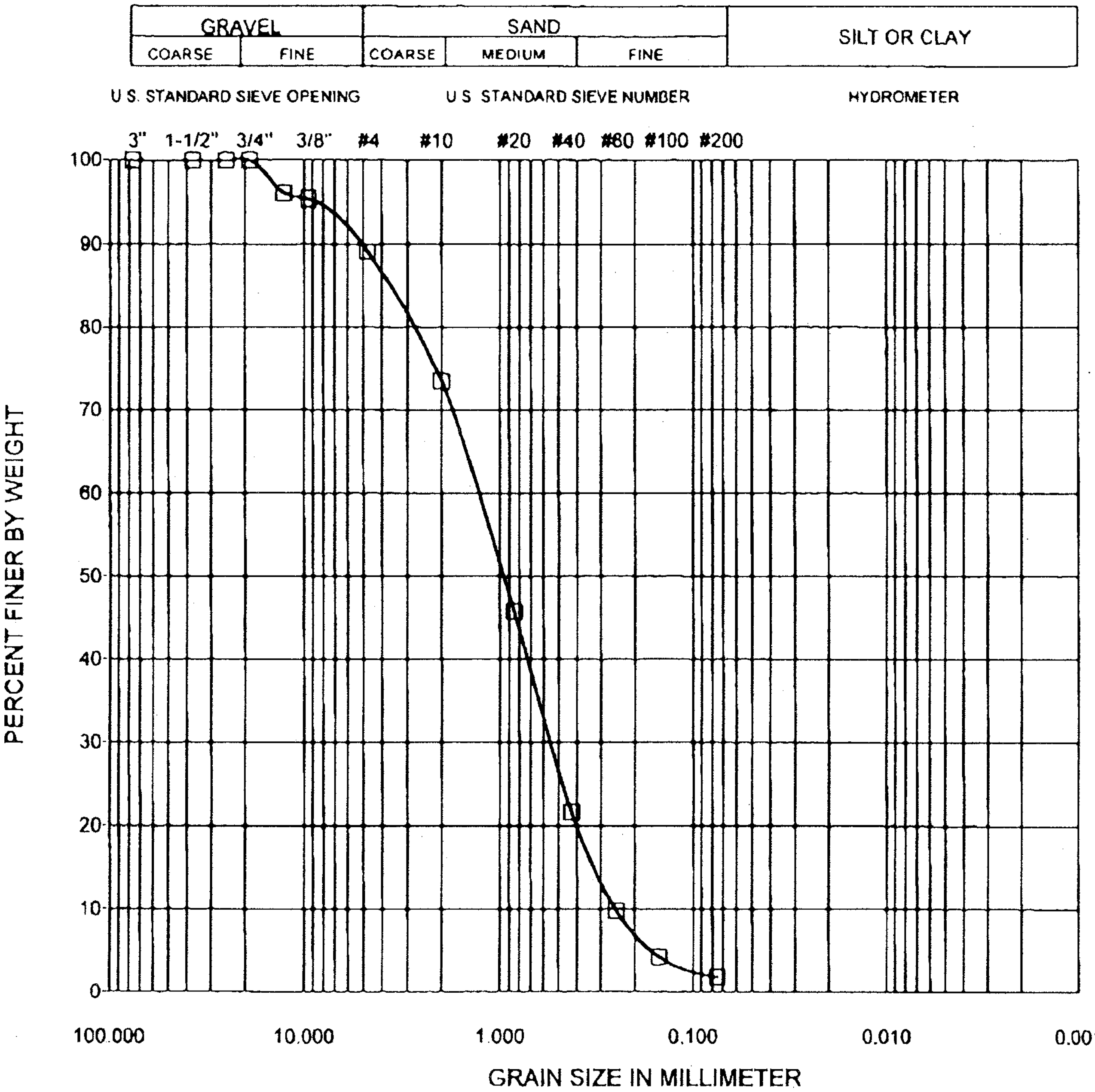
HREG / Verdemont Towne Center

9/2/05 C. Nardi/ddi

GEOTECHLFREMV3_DDI HREGVERD.GPJ 9/5/05

APPENDIX B

Results of Laboratory Testing



SYMBOL	BORING NO	SAMPLE ID	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	LIQUID LIMIT	PLASTICITY INDEX
□	B-1	B-1-2	6-6.5	RING	SP	N/A	N/A

ENVIRONMENTAL
GEOTECHNOLOGY
LABORATORY

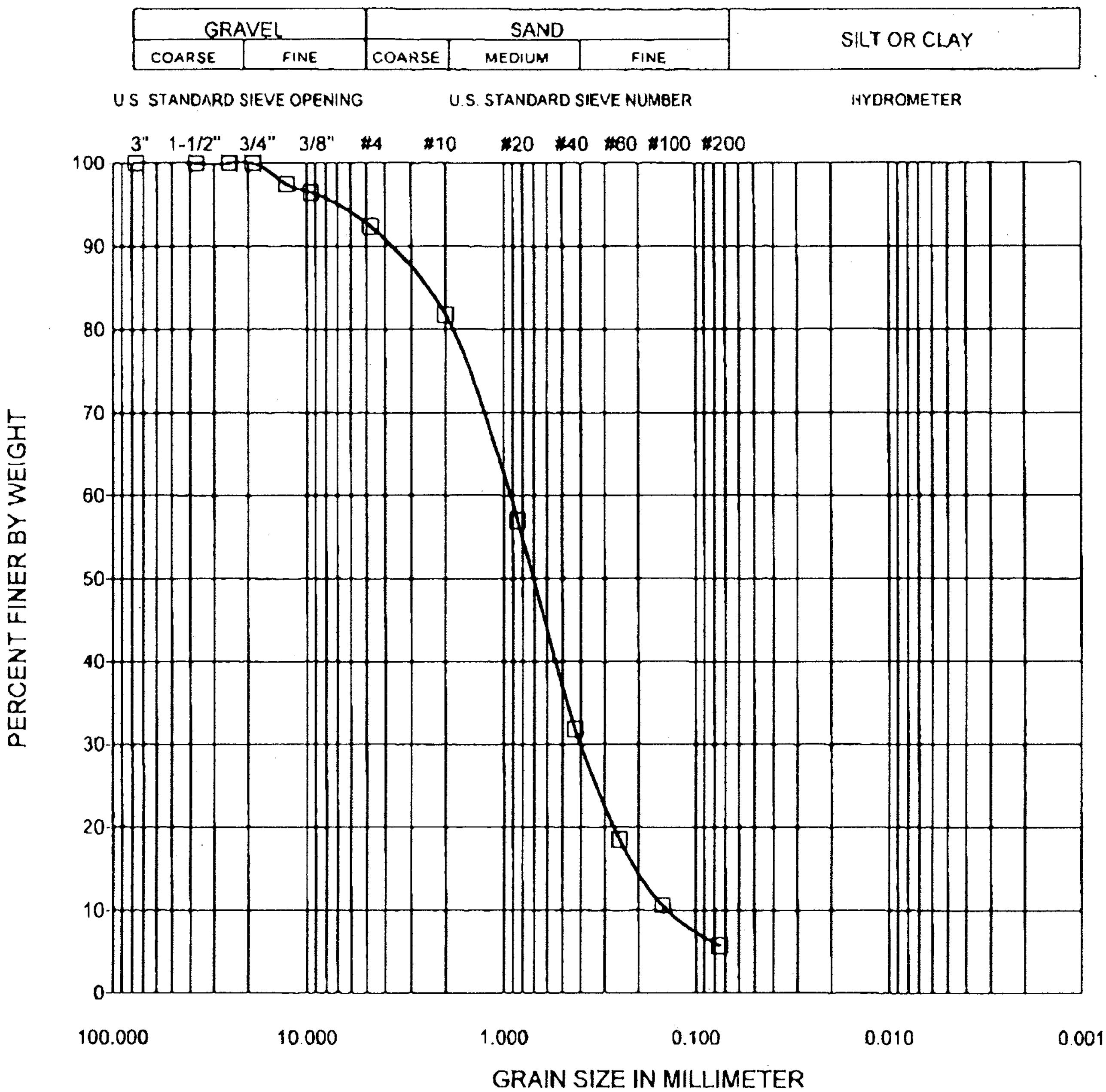
Project Name:
Hreg / Verdemont T-C
Client: LFR
Job No.:001-09424-00
EGL Project No.: 05-264-004

GRAIN SIZE DISTRIBUTION CURVE

(ASTM D422)

08/05

FIGURE



SYMBOL	BORING NO	SAMPLE ID	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	LIQUID LIMIT	PLASTICITY INDEX
□	B-1	B-1-5	20	BAG	SP-SM	N/A	N/A

ENVIRONMENTAL
GEOTECHNOLOGY
LABORATORY

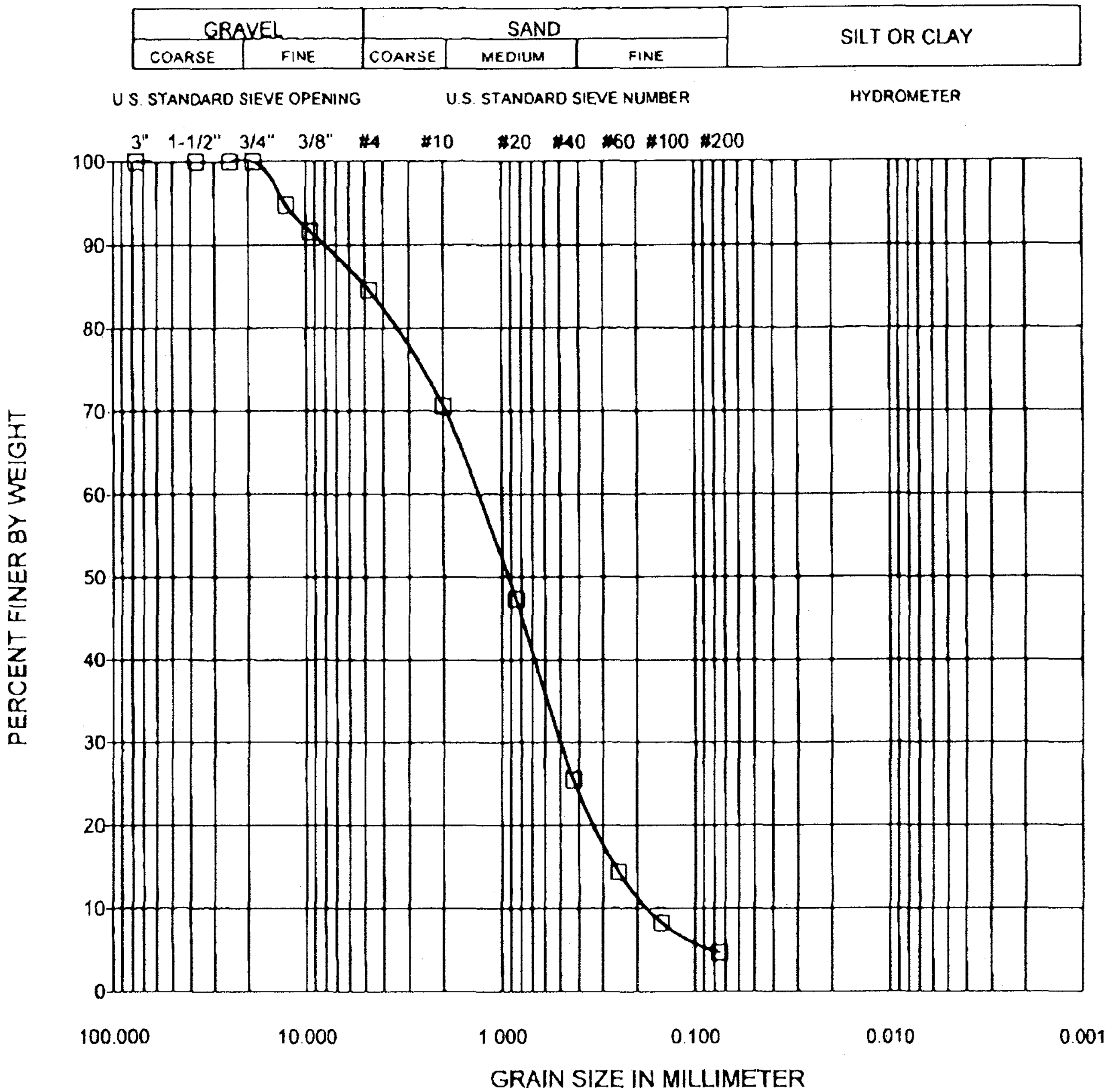
Project Name:
Hreg / Verdemont T-C
Client: LFR
Job No.:001-09424-00
EGL Project No.: 05-264-004

GRAIN SIZE DISTRIBUTION CURVE

(ASTM D422)

08/05

FIGURE



SYMBOL	BORING NO	SAMPLE ID	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	LIQUID LIMIT	PLASTICITY INDEX
□	B-3	B-3-4	15-16.5	BAG	SW	N/A	N/A

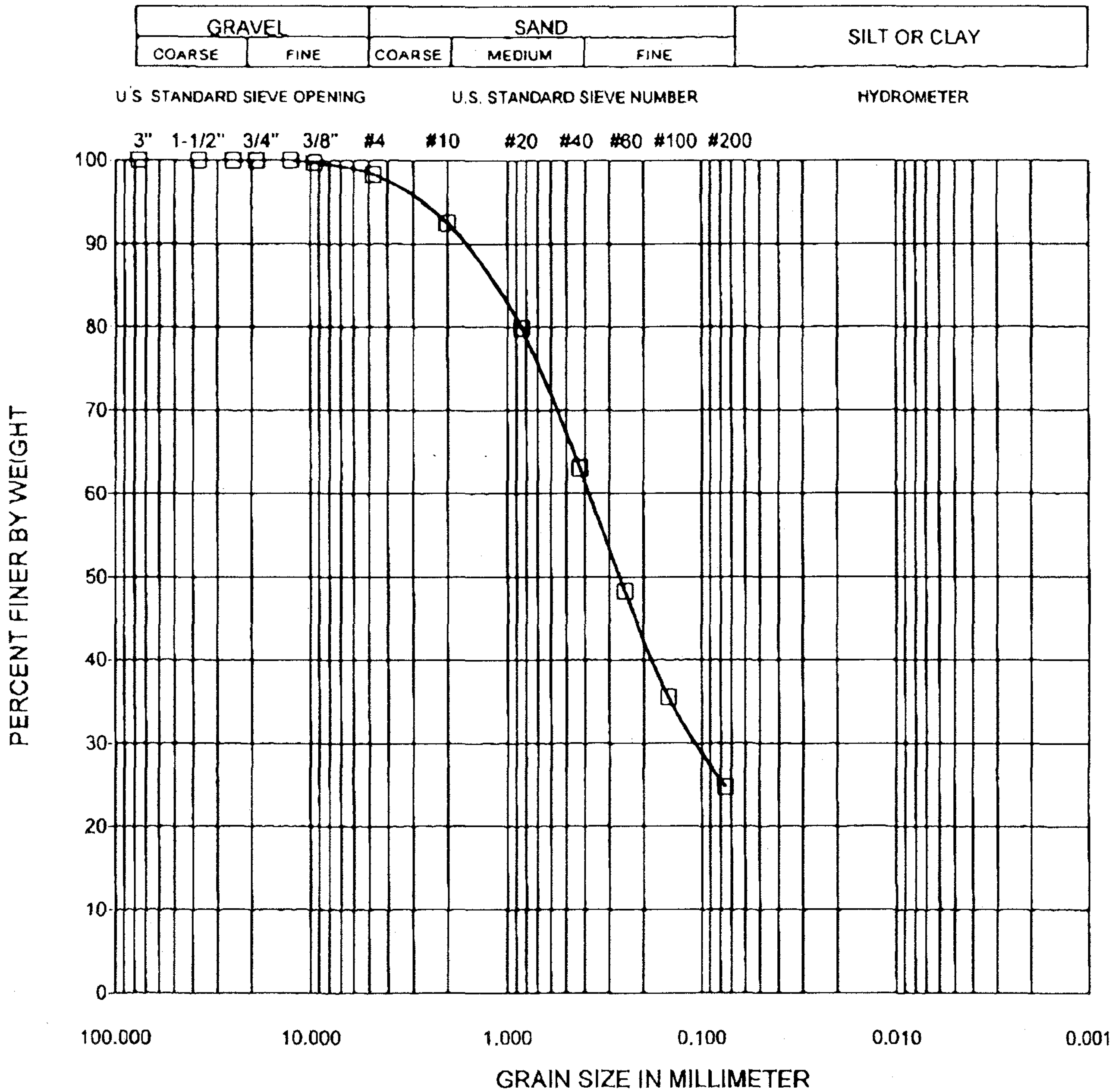
ENVIRONMENTAL GEOTECHNOLOGY LABORATORY

Project Name:
Hreg / Verdemont T-C
Client: LFR
Job No :001-09424-00
EGL Project No.: 05-264-004

GRAIN SIZE DISTRIBUTION CURVE
(ASTM D422)

08/05

FIGURE



SYMBOL	BORING NO	SAMPLE ID	DEPTH (ft)	SAMPLE TYPE	SOIL TYPE	LIQUID LIMIT	PLASTICITY INDEX
□	B-5	B-5-5	20	BAG	SM	N/A	N/A

ENVIRONMENTAL
GEOTECHNOLOGY
LABORATORY

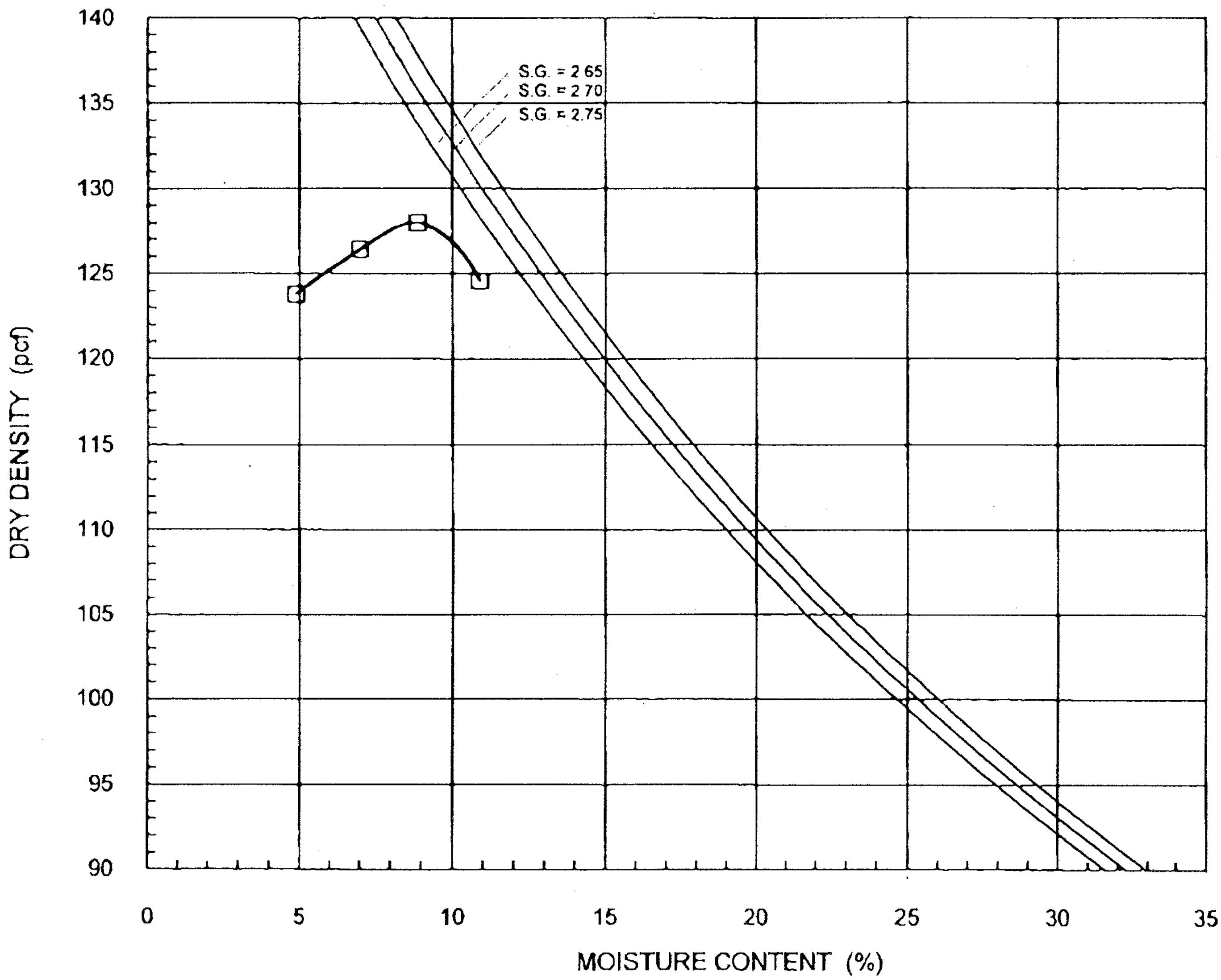
Project Name:
Hreg / Verdemont T-C
Client. LFR
Job No.:001-09424-00
EGL Project No.: 05-264-004

GRAIN SIZE DISTRIBUTION CURVE

(ASTM D422)

08/05

FIGURE



SYMBOL	BORING NO	SAMPLE ID	DEPTH (ft)	SAMPLE TYPE	VISUAL CLASSIFICATION	OPTIMUM MOISTURE(%)	MAX. DRY DENSITY(pcf)
□	B-1, B-2 B-3, B-4 & B-5	COMP 1-5	0-3	BULK	SM	9.0	128.0

Environmental Geotechnology Laboratory

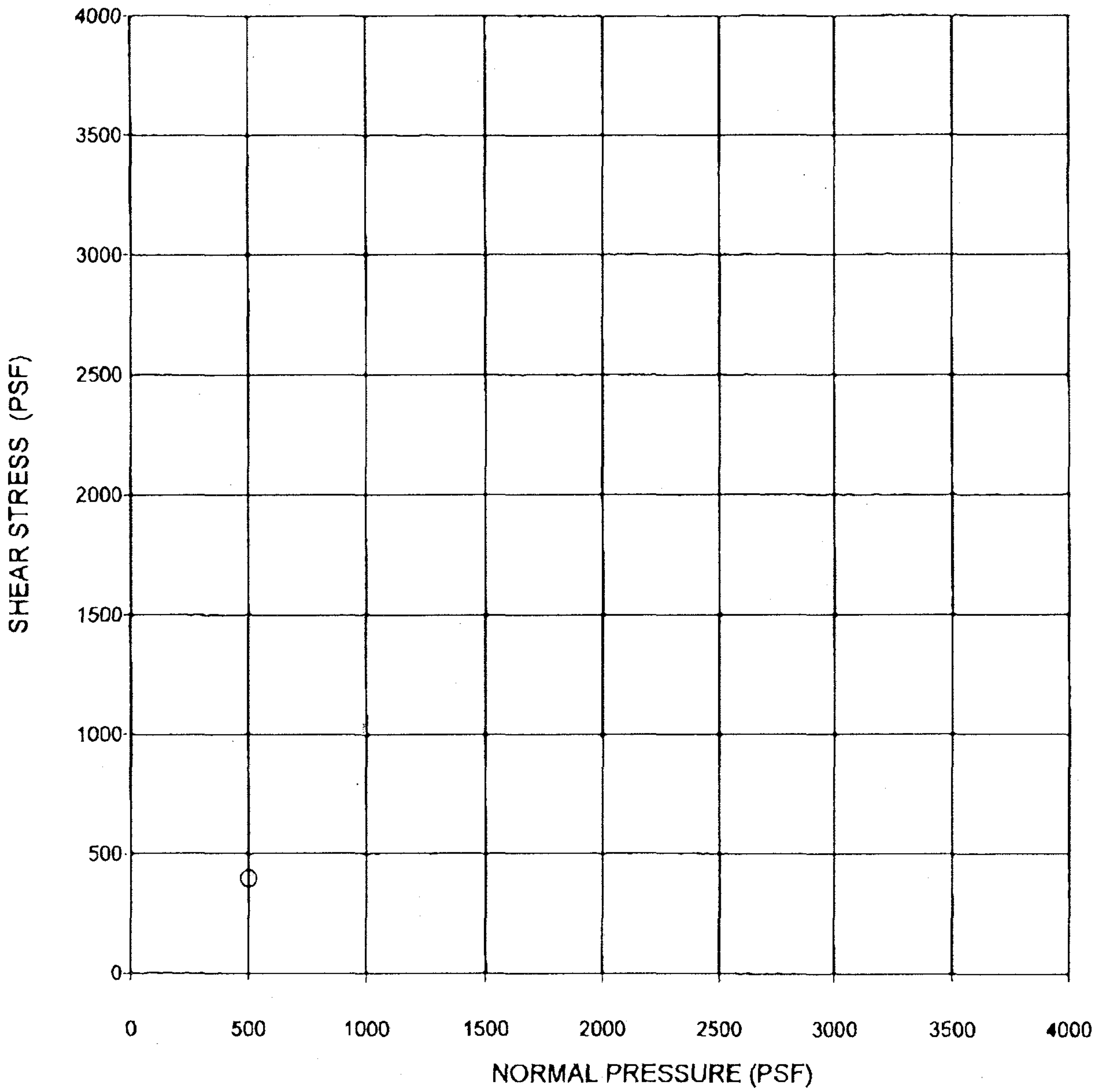
Project Name:
Hrag / Verdemont T-C
Client: L.F.R.
Job No.: 001-09424-00
EGL Project No 05-264-004

MODIFIED PROCTOR

(ASTM D1557)

08/05

FIGURE



SYMBOL	BOREHOLE ID	SAMPLE ID	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	COHESION (PSF)	FRICTION ANGLE (DEG)
○	B-5	B-5-1	2-2.5	RING	SP-SM	---	---

Vertical Loads (PSF)	Moisture Content Before Test (%)	Moisture Content After Test (%)
500	5.2	27.6

Environmental Geotechnology Laboratory

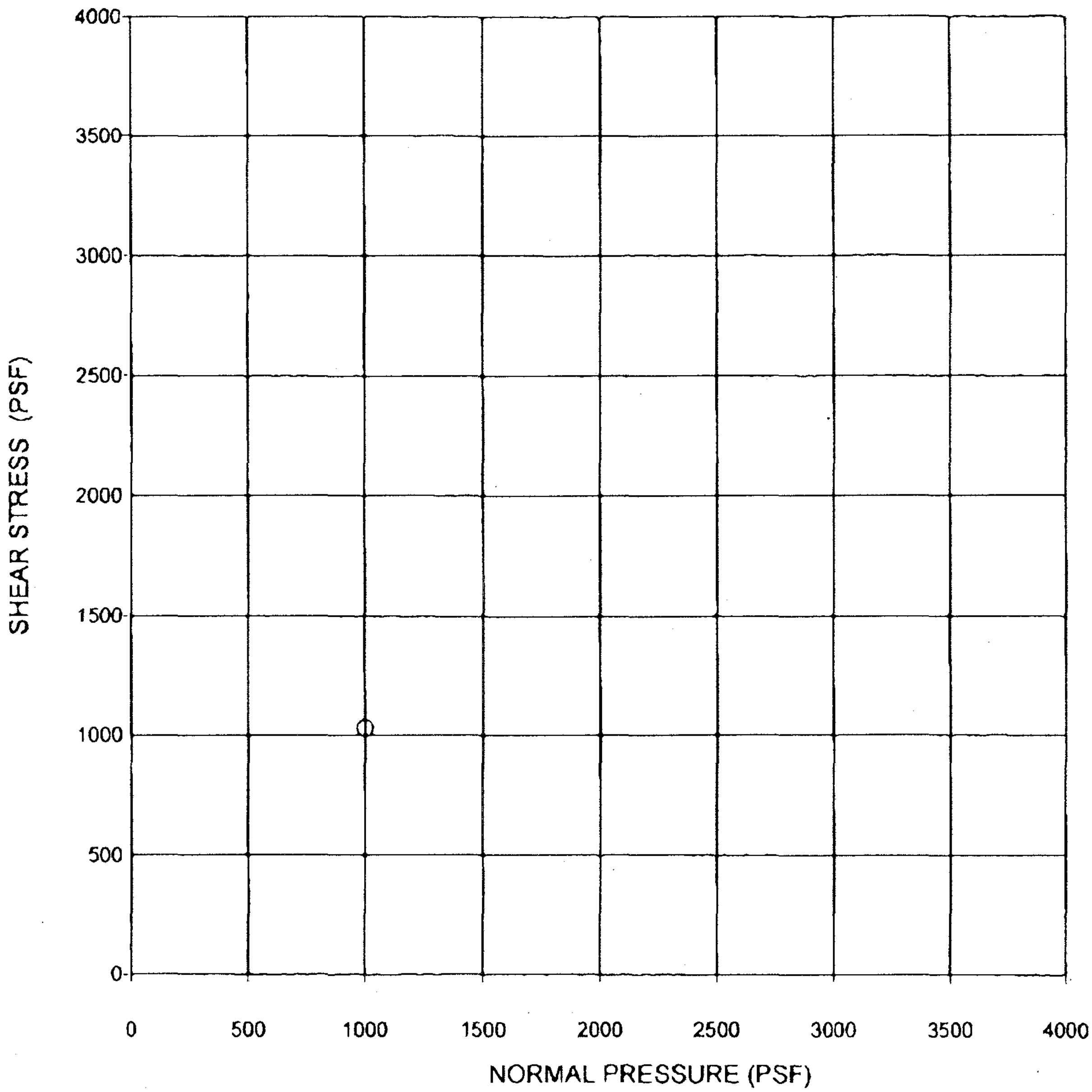
Project Name:
Herg / Verdemont T-C
Client: LFR
Job No.: 001-09424-004
EGL Project No.: 05-264-004

DIRECT SHEAR

(ASTM D3080)

08/05

FIGURE



SYMBOL	BOREHOLE ID	SAMPLE ID	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	COHESION (PSF)	FRICTION ANGLE (DEG)
O	B-1	B-1-2	6-6.5	RING	SP	---	---

Vertical Loads (PSF)	Moisture Content Before Test (%)	Moisture Content After Test (%)
1000	1.9	20.2

Environmental Geotechnology Laboratory

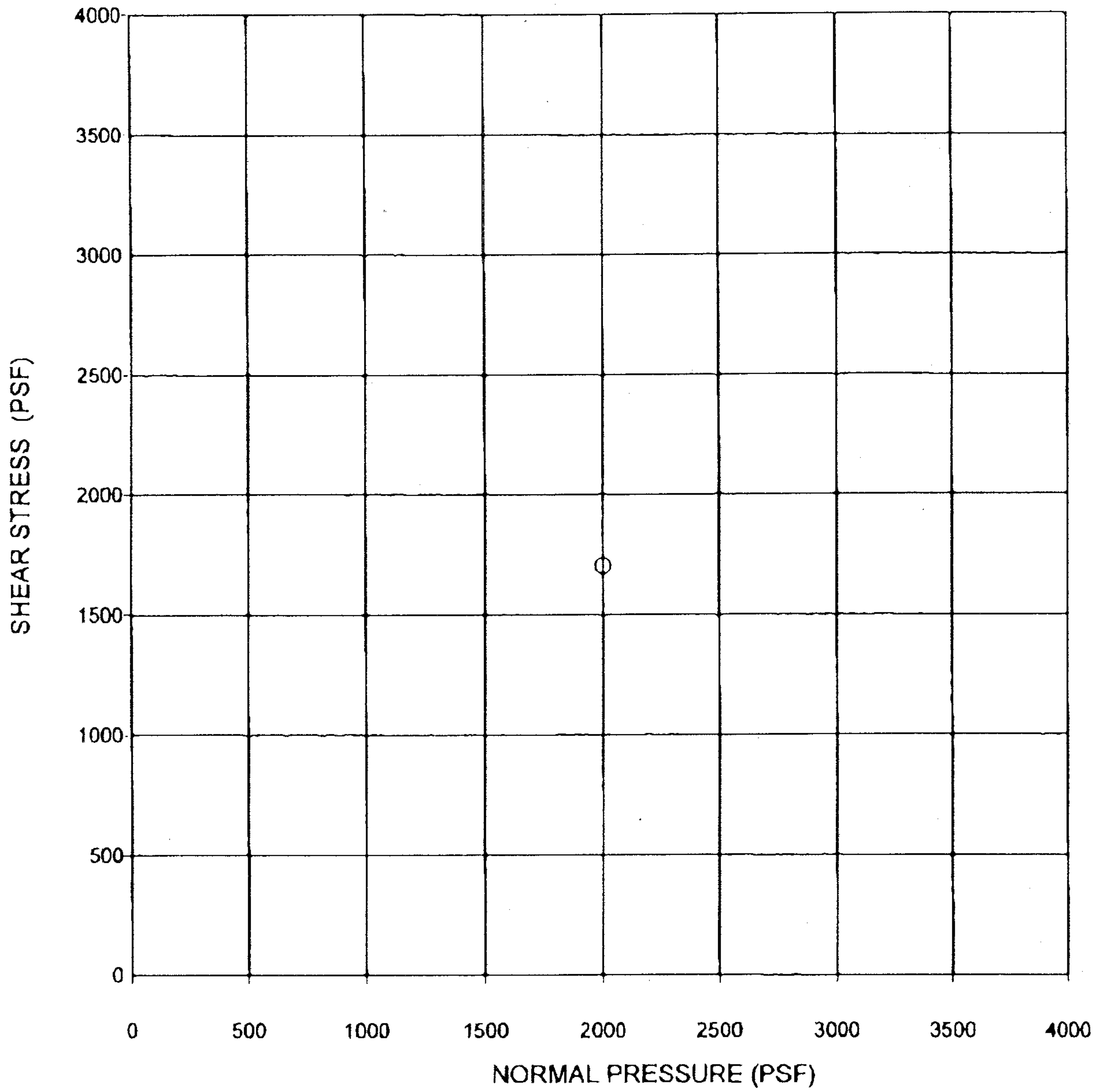
Project Name:
Herg / Verdemont T-C
Client: LFR
Job No.: 001-09424-004
EGL Project No.: 05-264-004

DIRECT SHEAR

(ASTM D3080)

08/05

FIGURE



SYMBOL	BOREHOLE ID	SAMPLE ID	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	COHESION (PSF)	FRICTION ANGLE (DEG)
O	B-2	B-2-3	11-11.5	RING	SP	---	---

Vertical Loads (PSF)	Moisture Content Before Test (%)	Moisture Content After Test (%)
2000	3.2	19.0

Environmental Geotechnology Laboratory

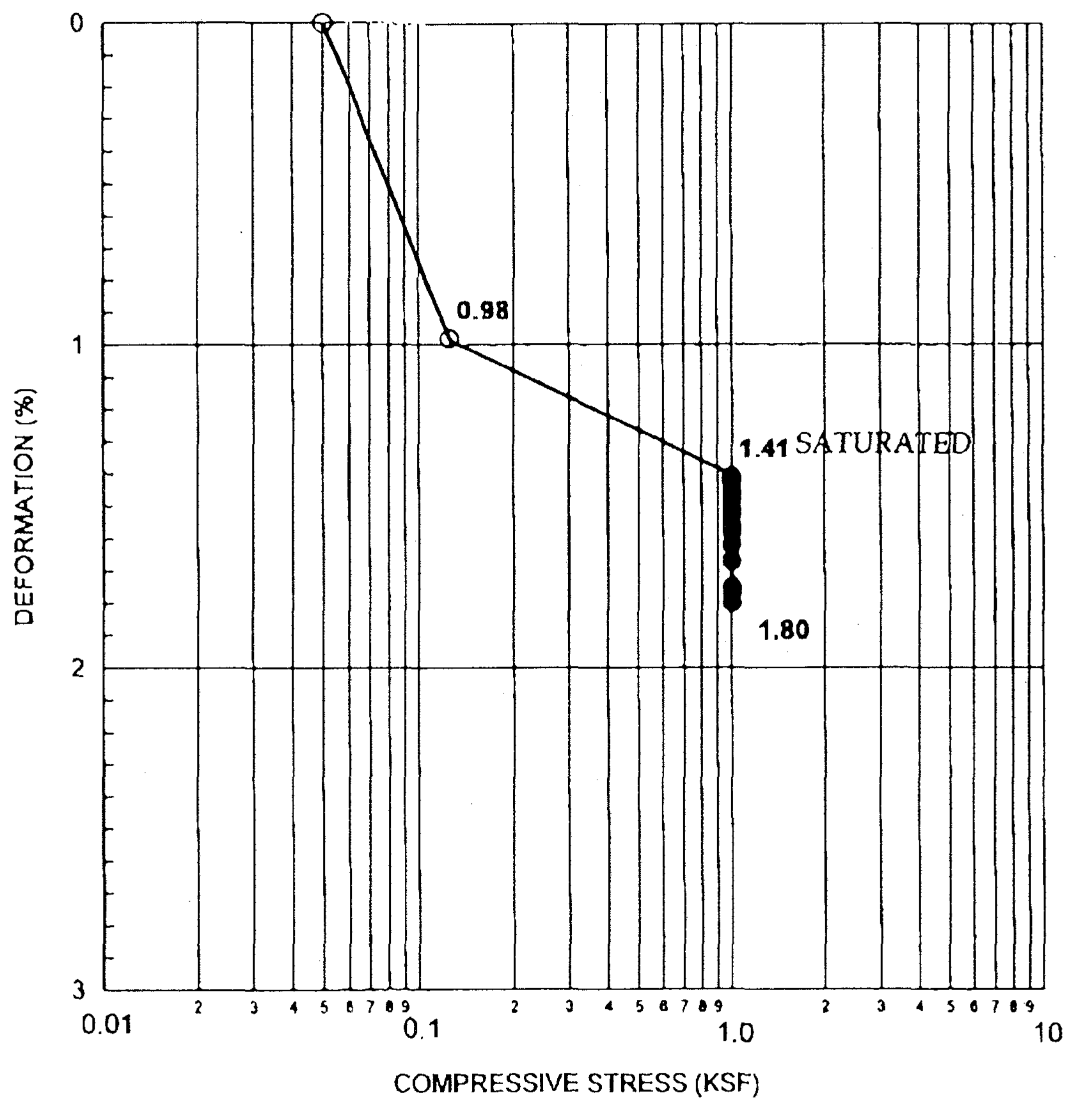
Project Name:
Herg / Verdemont T-C
Client: LFR
Job No.: 001-09424-004
EGL Project No.: 05-264-004

DIRECT SHEAR

(ASTM D3080)

08/05

FIGURE



SYMBOL	BORING NO	SAMPLE ID	DEPTH (FT)	SOIL TYPE	INIT. MOISTURE CONTENT (%)	INIT. DRY DENSITY (PCF)	INIT. VOID RATIO
O	B-5	B-5-1	2-2.5	Ring	5.2	94.8	0.777

Normal stress on sample: 0.5 ksf

ENVIRONMENTAL GEOTECHNOLOGY LABORATORY

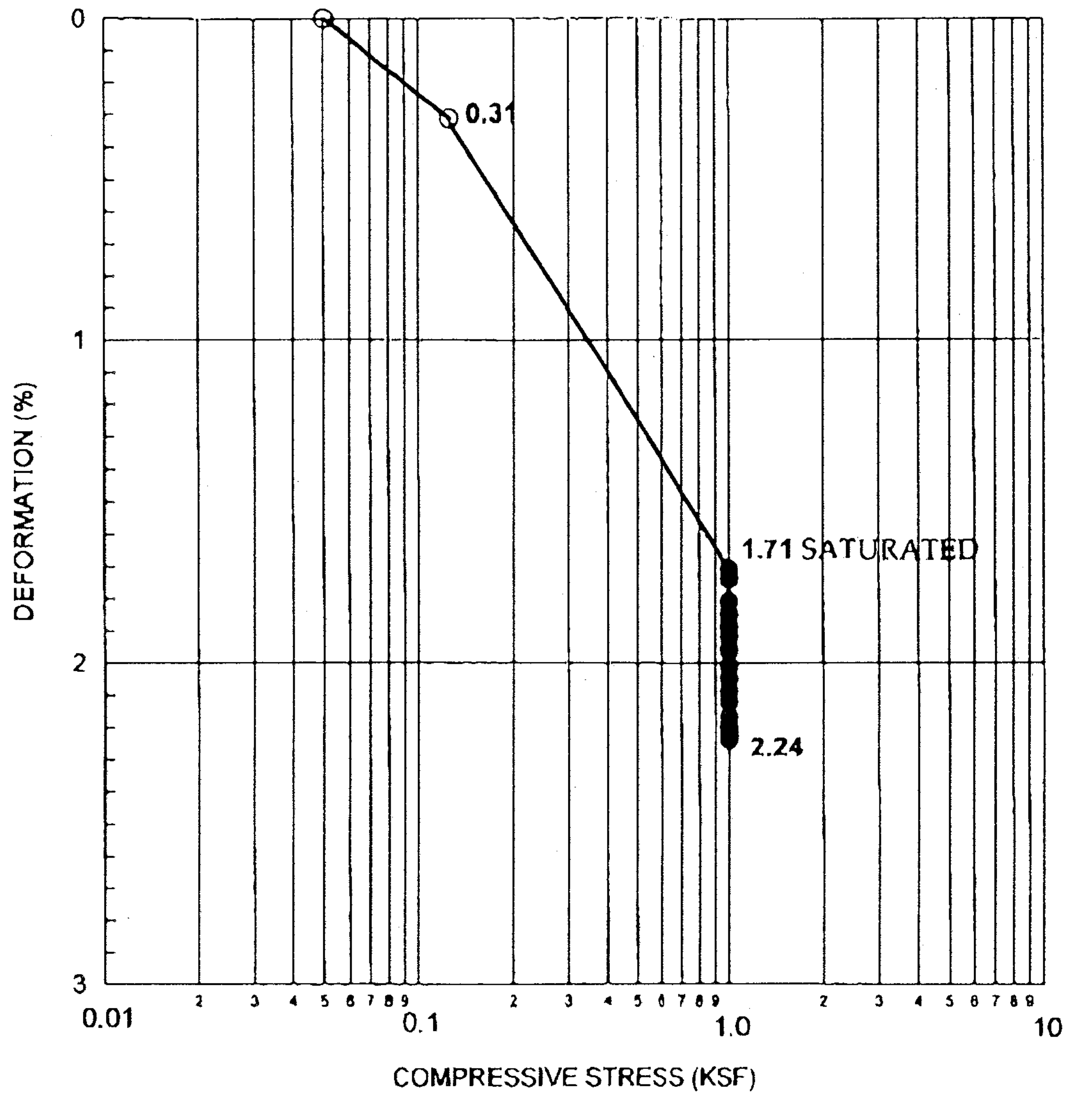
Project Name:
Hreg / Verdemont T-C
Job No.: 001-09424-00
Client: LFR
EGL Project No.: 05-264-004

COLLAPSE POTENTIAL

(ASTM D5333)

08/05

FIGURE



SYMBOL	BORING NO	SAMPLE ID	DEPTH (FT)	SOIL TYPE	INIT. MOISTURE CONTENT (%)	INIT. DRY DENSITY (PCF)	INIT. VOID RATIO
○	B-1	B-1-2	6-6.5	Ring	1 9	105.2	0.601

Normal stress on sample: 1.0 ksf

ENVIRONMENTAL GEOTECHNOLOGY LABORATORY

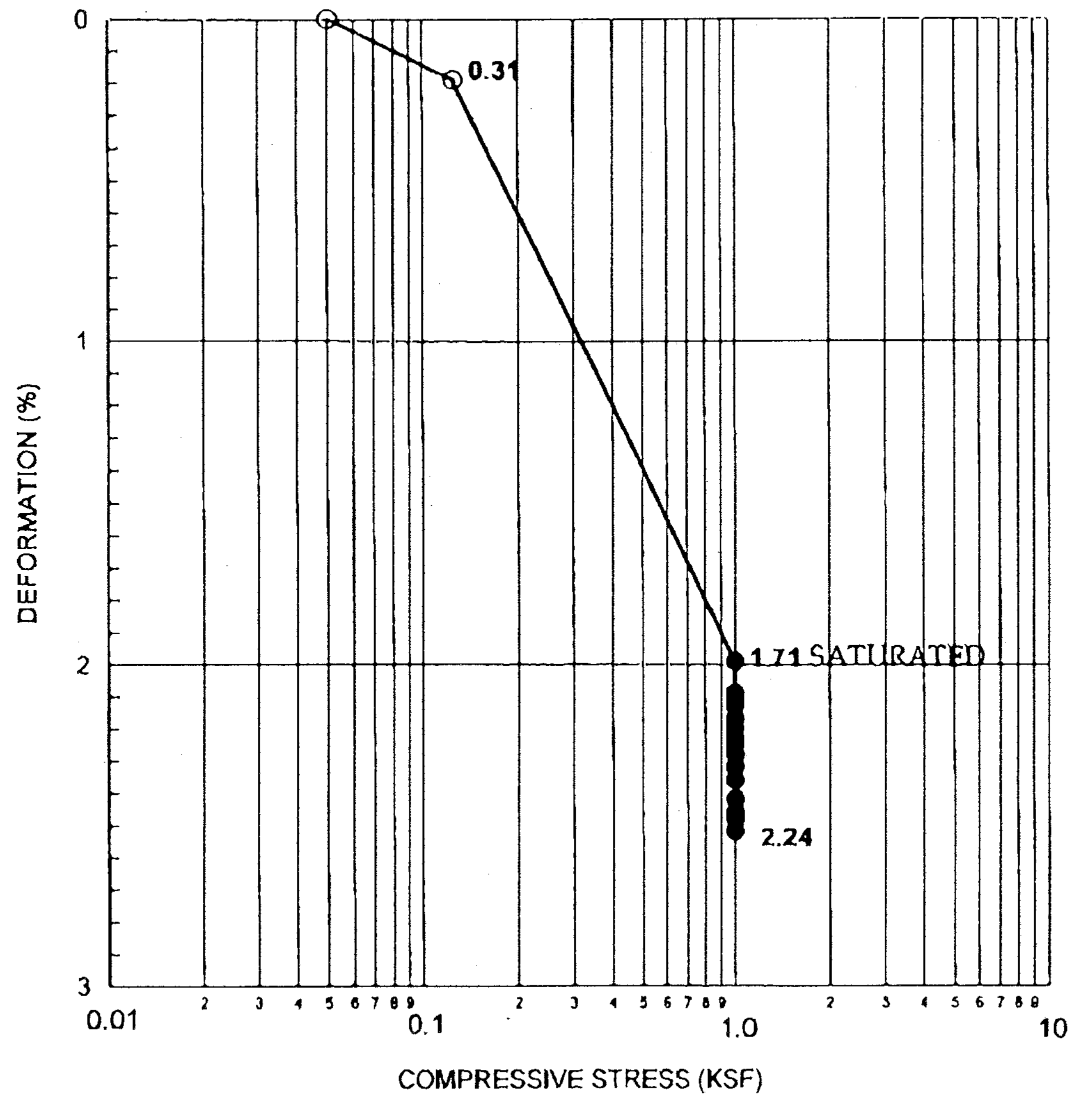
Project Name:
Hreg / Verdemont T-C
Job No.: 001-09424-00
Client: LFR
EGL Project No.: 05-264-004

COLLAPSE POTENTIAL

(ASTM D5333)

08/05

FIGURE



SYMBOL	BORING NO	SAMPLE ID	DEPTH (FT)	SOIL TYPE	INIT. MOISTURE CONTENT (%)	INIT. DRY DENSITY (PCF)	INIT. VOID RATIO
O	B-2	B-2-3	11-11.5	Ring	3.2	109.5	0.538

Normal stress on sample. 2.0 ksf

ENVIRONMENTAL GEOTECHNOLOGY LABORATORY

Project Name:
Hreg / Verdemont T-C
Job No.: 001-09424-00
Client: LFR
EGL Project No.: 05-264-004

COLLAPSE POTENTIAL

(ASTM D5333)

08/05

FIGURE

SUMMARY OF CORROSION TEST RESULTS

PROJECT NAME: Hreg / Verdemont T-C

EGL JOB NO.: 05-264-004

PROJECT NO.: 001-09424-004

CLIENT: LFR

DATE: 08-31-05

SUMMARIZED BY: VW

BORING NO	SAMPLE NO	DEPTH (ft)	pH CALTRANS 643	CHLORIDE CONTENT CALTRANS 422 (ppm)	SULFATE CONTENT CALTRANS 417 (% by weight)	MINIMUM RESISTIVITY CALTRANS 532 (ohm-cm)
N/A	COMP 1-5	0-3	7.43	85	0.002	10500

APPENDIX D

APPENDIX D

RECOMMENDED GRADING SPECIFICATIONS

FOR

MIXED USE RESIDENTIAL AND COMMERCIAL DEVELOPMENT VERDEMONT AREA SAN BERNARDINO, CALIFORNIA

PROJECT NO. T2616-22-01

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 Inland Empire, Incorporated. 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, 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 in the maximum dimension.

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, gradation and chemical 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

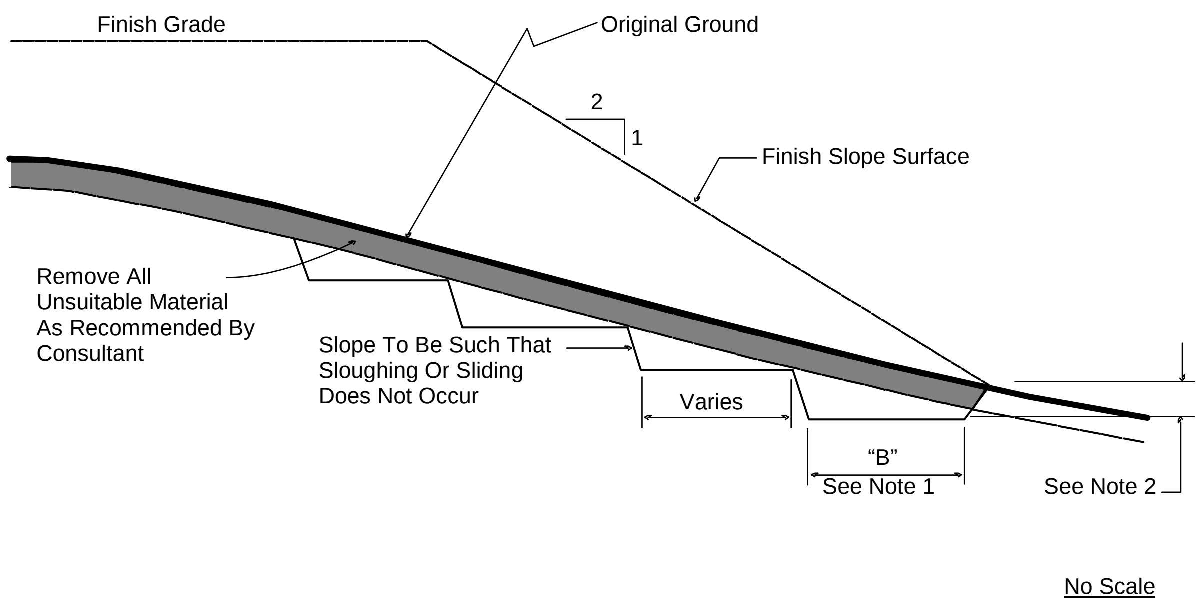
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material 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 Any asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility. Concrete fragments that are free of exposed reinforcing steel may be placed in fills, provided they are placed in accordance with Section 6.2 or 6.3 of this document.

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

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

TYPICAL BENCHING DETAIL



DETAIL NOTES:

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

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

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

5. COMPACTION EQUIPMENT

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

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

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

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

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

6.1.2 In general, the *soil* fill shall be compacted at a moisture content at or above the optimum moisture content as determined by ASTM D 1557-02.

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 1557-02. 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 10 feet below finish grade or 3 feet below the deepest utility, whichever is deeper. In the event that placement of oversized rock is planned less than 10 feet below finish grade, 15 feet behind slope face, or 3 feet below deepest utility, Geocon should be consulted for additional recommendations.

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 1196-93, 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. OBSERVATION AND TESTING

7.1 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.

7.2 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.

7.3 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.

7.4 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.

7.5 The Consultant should observe the placement of subdrains, to verify that the drainage devices have been placed and constructed in substantial conformance with project specifications.

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

7.6.1 Soil and Soil-Rock Fills:

7.6.1.1 Field Density Test, ASTM D 1556-02, *Density of Soil In-Place By the Sand-Cone Method.*

7.6.1.2 Field Density Test, Nuclear Method, ASTM D 2922-01, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth).*

7.6.1.3 Laboratory Compaction Test, ASTM D 1557-02, *Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-Pound Hammer and 18-Inch Drop*.

7.6.1.4. Expansion Index Test, ASTM D 4829-03, *Expansion Index Test*.

7.6.2 Rock Fills

7.6.2.1 Field Plate Bearing Test, ASTM D 1196-93 (Reapproved 1997) *Standard Method for Nonreparative Static Plate Load Tests of Soils and Flexible Pavement Components, For Use in Evaluation and Design of Airport and Highway Pavements.*

8. PROTECTION OF WORK

8.1 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.

8.2 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.

9. CERTIFICATIONS AND FINAL REPORTS

9.1 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.

9.2 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.